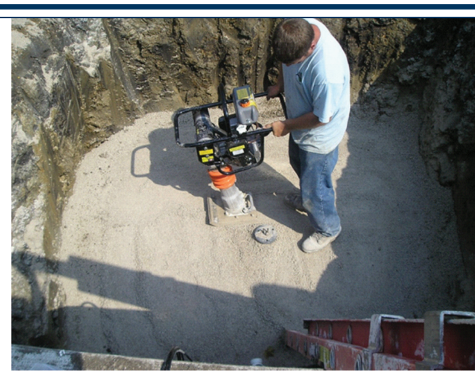
Investigation of Improved Utility Cut Repair Techniques to Reduce Settlement in Reapaired Areas, Phase II

Iowa SUDAS
STATEWIDE URBAN DESIGN AND SPECIFICATIONS

Final Report
December 2010

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16. Abstract

Trench maintenance problems are caused by improper backfill placement and construction procedures. This report is part of a multiphase research project that aims to improve long-term performance of utility cut restoration trenches. The goal of this research is to improve pavement patch life and reduce the maintenance of the repaired areas. The objectives were to use field-testing data, laboratory-testing data, and long-term monitoring (elevation survey and falling weight deflectometer testing) to suggest and modify recommendations from Phase I and to identify the principles of trench subsurface settlement and load distribution in utility cut restoration areas by using instrumented trenches. The objectives were accomplished by monitoring local agency utility construction from Phase I, constructing and monitoring the recommended trenches from Phase I, and instrumenting trenches to monitor changes in temperature, pressure, moisture content, and settlement as a function of time to determine the influences of seasonal changes on the utility cut performance.

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UTILITY CUT REPAIR TECHNIQUES—INVESTIGATION OF IMPROVED UTILITY CUT REPAIR TECHNIQUES TO REDUCE SETTLEMENT IN REPAIRED AREAS, PHASE II

Final Report
December 2010

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EXECUTIVE SUMMARY

The common procedure of installing utilities, such as gas, water, telecommunications, and sanitary and storm sewers, requires excavation to install the pipes or lines. Utility cut restoration has a significant effect on pavement performance. It is often observed that the pavement within and around utility cuts fails prematurely, increasing maintenance costs.

For example, early distress in a pavement may result in the formation of cracks where water can enter the base course, in turn leading to deterioration of the pavement. The resulting effect has a direct influence on the pavement integrity, life, and aesthetic value, as well as driver safety.

The magnitude of the effect depends upon the pavement patching procedures, backfill material condition, climate, traffic, and pavement condition at the time of patching. While new pavement should last between 15 and 20 years, once a cut is made, the pavement life is reduced to about eight years. Furthermore, several cuts in a roadway can lower the road life by 50%.

Poor performance of pavements around utility trenches on local streets and state highway systems often causes continual maintenance due to improper backfill placement, such as improper backfill material, under compacted, too dry, too wet, and so forth. The cost of repairing poorly-constructed pavements can be reduced with an understanding of proper material selection and construction practices. Current utility cut and backfill practices vary widely across Iowa, which results in a range of maintenance issues.

Phase I survey results indicated that many restored utility cut restorations fail in less than two years. Field and laboratory tests of backfill indicated inadequate compaction, moisture content, and density of the backfill are factors that contribute to utility cut trench restoration failures. Falling weight deflectometer (FWD) tests indicated weakened subgrade soil around the utility cut trench restorations. This weakened soil is known as the zone of influence.

Based on the monitoring of the trenches constructed during Phase I, the six recommended trenches during Phase II, and the three instrumented trenches constructed during Phase II, the conclusions and recommendations of this research follow.

Material Selection

- Relative density test (i.e., vibrating table compaction) is recommended for any potential granular backfill.
- Simple column test is recommended to determine the collapse potential (i.e., reduction in volume or settlement) of different backfill materials.
- The range of moisture content around the smallest density and highest collapse potential is known as the bulking moisture content. Backfill materials installed at moisture content higher than the bulking moisture content and at relative density of medium to dense show good performance in the field.
- Lift thicknesses should be limited to less than 1 foot.

- Using the 3/8 inch minus backfill material installed with proper construction practices showed minimum settlement during spring/summer and a heave comparable to surrounding pavement, avoiding the formation of a bump at the utility cut location.
- Avoid using backfill soils that have high silt content, which are susceptible to frost heave.
- One-inch clean limestone or other clean backfill with limited fines do not experience collapse and are least susceptible to frost heave. The use of 1 inch clean limestone improves the performance of the trenches. It stiffens the response of the trench in FWD testing, and the settlement within the trench is less.

Construction Practices

- The use of a concrete patch with dowels improved the performance at the utility cut location.
- Remove at least 2 feet of pavement around the perimeter and compact the soil if a T-section is not constructed.
- When comparing trenches constructed with 1 inch clean, the trench constructed with highest relative density showed the smallest settlement and highest uplift movement. The trench with the lowest relative density showed the highest settlement.
- When comparing FWD test results, the trenches with T-sections showed reduced measured deflections within the zone of influence. However, the T-section may have caused a shift of the zone of influence.
- Smaller settlement was measured in trenches with higher relative densities, and smaller FWD deflections corresponded to higher California Bearing Ratio (CBR) values estimated from the dynamic cone penetration (DCP) test.
- The T-section could be modified to use walls that are beveled outward to facilitate compaction of backfill. Beveled edges may reduce the amount of disturbance to the surrounding soil and eliminate the vertical excavation; however, it may make compacting the backfill at the edges difficult. This is expected to prevent the zone of influence from migrating outside of the T-section.
- Construction equipment should be kept away from the edges of the open trench to reduce its effects on the zone of influence. FWD testing showed that damage caused by equipment during construction had a long-term impact on trench performance.
- The use of geogrid in the trenches did not improve the performance of the trenches compared to the trenches constructed without the geogrid for the trenches using 3/8 inch minus limestone.

Future Research Needs

- Continue FWD testing on the trenches.
- Evaluate pavement surface roughness in utility cut areas using the International Roughness Index (IRI) data to assess the ride quality and determine the effect on pavement maintenance and methods to improvement smoothness in these areas.

CHAPTER 1. INTRODUCTION

Background

The common procedure of installing utilities, such as gas, water, telecommunications, and sanitary and storm sewers, requires an excavation to install the pipes or lines. Utility cut restoration has a significant effect on pavement performance. It is often observed that the pavement within and around utility cuts fails prematurely, increasing maintenance costs. For instance, early distress in a pavement may result in the formation of cracks where water can enter the base course, in turn leading to deterioration of the pavement (Peters 2002). The resulting effect has a direct influence on the pavement integrity, life, and aesthetic value, and as well as drivers' safety (Arudi et al. 2000). The magnitude of the effect depends upon the pavement patching procedures, backfill material condition, climate, traffic, and pavement condition at the time of patching. Bodosci et al. (1995) noted that new pavement should last between 15 and 20 years; however, once a cut is made, the pavement life is reduced to about 8 years. Furthermore, Tiewater (1997) indicates that several cuts in a roadway can lower the road life by 50%.

Poor performance of pavements around utility trenches on local streets and state highway systems often causes maintenance due to improper backfill placement (i.e., improper backfill, under-compacted, too dry, too wet, etc.). The cost of repairing poorly constructed pavements can be reduced with an understanding of proper material selection and construction practices. Current utility cut and backfill practices vary widely across Iowa, which results in a range of maintenance issues.

To address these concerns, the Institute for Transportation (InTrans, formerly the Center for Transportation Research and Education) at Iowa State University (ISU), along with the Iowa Department of Transportation (Iowa DOT), began a multiyear investigation into utility cut restoration failures. The Iowa Highway Research Board funded two phases of this investigation. "Investigation of Improved Utility Cut Repair Techniques to Reduce Settlement in Repaired Area" (Phase I) was an initial investigation into utility cut restoration failures to document the occurrence and frequency of failures and to determine the failure mechanisms. Phase II continued the Phase I investigations of failure mechanisms and the monitoring of utility cut restorations at various locations around Iowa. In addition, Phase II implemented the Phase I recommendations for construction and monitoring of several new utility cut trenches and investigated trench settlement using instrumentation in trenches.

Three activities took place during Phase I to evaluate the construction and performance of utility trench restorations: (1) a survey was conducted to document construction practices used in utility trench restorations in several cities in Iowa; (2) laboratory tests were performed on backfill to determine its engineering properties; and (3) trench restoration performance was monitored using falling weight deflectometer (FWD) testing. Survey results indicated that many restored utility cut restorations fail in less than two years. Field and laboratory tests of backfill indicated that inadequate compaction, moisture content, and density of backfill are factors that contribute to utility cut trench restoration failures. Falling weight deflectometer tests indicated weakened subgrade soil around the utility cut trench restorations. This weakened soil is known as the “zone of influence.”

Based on the results of Phase I, a three-part research project, “Utility Cut Repair Techniques—Investigation of Improved Utility Cut Repair Techniques to Reduce Settlement in Repaired Areas, Phase II,” was initiated to further investigate the influences of compaction, moisture content and density of the backfill, and the zone of influence on the performance of utility cut trench restorations. These research areas include: (1) continued monitoring of the utility cut restorations constructed during Phase I for two additional years; (2) the construction of new trenches using six recommended practices; and (3) instrumentation of three new trenches to understand the mechanisms of trench backfill settlement and load distribution.

The objectives of Phase II were to

- Correlate the long-term performance of trench restorations with the in-situ properties of the backfill during construction and the engineering properties in laboratory testing
- Continue the monitoring of the utility cut restorations constructed during Phase I
- Construct recommended trenches and monitor their long-term performance
- Research and identify the principles of trench subsurface settlement and load distribution in utility cut restoration areas using the three instrumented trenches
- Update the recommendations made during Phase I and recommend the best practices for utility cut restoration repair techniques for the Statewide Urban Design and Specification (SUDAS) Program

Research Plan

To achieve the objectives of the project, four tasks were outlined. First, monitoring of trenches constructed during Phase I was continued. Second, six trenches were constructed using different practices. Third, three instrumented trenches were constructed to compare the performance trenches constructed using different construction practices. Fourth, data collected from the first three tasks were evaluated and summarized.

Task 1: Continued Monitoring of Trenches Documented during Phase I

During Phase I, the construction practices of four utility cut restorations were documented across Iowa. While the trenches were being constructed, the top lift was tested using nuclear density tests and dynamic cone penetration (DCP). After the restorations were completed, elevation surveys were performed; however, only three of the trenches were tested using FWD testing during Phase I. These three trenches were monitored during Phase II with elevation survey and FWD testing.

Task 2: Construct the Five Remaining Trenches Proposed in Phase I and Monitor All Six Trenches Using FWD

One of the six recommended trenches was constructed in Phase I; the remaining five utility cut restorations were constructed during Phase II. These utility cut trench restorations, along with the one constructed during Phase I, were monitored using FWD tests.

The engineering properties of the soil removed from the trenches and the backfill materials were measured in the field and in laboratory testing. Laboratory testing included sieve analysis, hydrometer, standard proctor tests, and relative minimum and maximum density tests. During the construction of the proposed trenches field tests, nuclear density, DCP, and/or Clegg Hammer tests were used to evaluate the in-situ properties of the backfill material. The results from the field and laboratory investigations were used to evaluate how the in-situ properties of the backfill material affect the long-term performance of the utility cut restoration.

Task 3: Instrument and Monitor Three Utility Cut Trenches

To better understand the principles of settlement and load distribution in and around the utility cut restoration area, three trenches were instrumented. The trenches, located on Kellogg Avenue in Ames, Iowa, were constructed using

- A shallow trench (less than eight feet deep) using the City of Ames current construction practices; the current construction practices included lifts of two feet, granular backfill with minimum moisture, and density control (Trench AI)
- A shallow vertical walled trench with granular backfill, lift thickness less than 12 inches, moisture control, and relative density of 65% or more (Trench BI)
- A shallow T-section trench with granular backfill, lift thickness less than 12 inches, moisture control, and relative density of 65% or more (Trench CI)

The instrumentation monitored settlement with extensometers, overburden pressure with pressure cells, moisture content of the backfill material with moisture sensors, and temperature using temperature sensors. These trenches were monitored from fall 2007 to spring 2009 (20 months).

Task 4: Data Evaluation

Data collected from the tasks above were analyzed. The field- and laboratory-testing data from Tasks 1 and 2 were compared with the settlement and FWD measurements. From this comparison, the goal was to determine if the recommended construction practices improved the performance of the utility cuts. The data output from the instrumentation in Task 3 was compared to determine the specific mechanisms that cause deterioration of the utility cut patches.

Recommendations and Conclusions

Based on the monitoring of the trenches constructed during Phase I, the six recommended trenches during Phase II, and the three instrumented trenches constructed during Phase II, the following conclusions and recommendations can be made:

Material Selection

- Three-eighths-inch minus backfill is not an acceptable backfill material, because it exhibits collapse behavior when wetted, as seen when water infiltrated around the temporary patch into instrumented Trench AI. It is also frost susceptible and undergoes heave during freezing conditions, as shown in the instrumented trenches placed with moisture control and proper compaction techniques.
- Soils containing silt-sized particles are most susceptible to frost heave.
- One-inch clean limestone or other clean backfill with limited fines do not experience collapse and are least susceptible to frost-heave. The use of 1-inch clean limestone improved the performance of the trenches. It stiffened the response of the trench in FWD testing, and the settlement within the trench was less than in trenches constructed with 3/8-inch minus limestone.
- Soils excavated from the trenches could be mixed with granular backfill if laboratory tests indicated the range of moisture content and densities that the material needed to be placed at and appropriate quality control measures were used.

Construction Practices

- The use of a concrete patch with dowels performed the best over the long term. This was documented with the utility cut constructed in Des Moines.
- It is recommended that at least two feet of pavement around the perimeter be removed and the soil compacted if a T-section is not constructed. This is supported by the results in Trench A.
- When comparing trenches constructed with 1-inch clean (Trenches D, E, and F), the trench constructed with highest relative density showed the smallest settlement and highest uplift movement. The trench with the lowest relative density showed the highest settlement.
- When comparing the FWD tests performed on Trenches D, E, and F, the trenches with T-sections (Trenches E and F) showed reduced measured deflections within the zone of influence. The T-section may have caused a shift of the zone of influence.
- Instrumented trenches showed higher settlement in the trench constructed using the City of Ames specifications (i.e., lower relative density), which was constructed with lift thicknesses of two feet with no moisture or compaction control.
- Instrumented trenches with construction control showed lower settlement and higher uplift movement when compared to the city practice trench. However, trenches with construction control showed uplift movement closer to the surrounding soil than the city practice trench.

- Trench BI showed higher deflections than Trenches AI and CI, measured using the FWD test results. This trench corresponded to the California Bearing Ratio (CBR) values estimated from DCP test results (i.e., higher CBR corresponded to smaller FWD deflections).
- Monitoring recommended and instrumented trenches showed that smaller settlement was measured in trenches with higher relative densities, and smaller FWD deflections corresponded to higher CBR values estimated from the DCP test.
- The T-section could be modified to use walls that are beveled outward to facilitate compaction of backfill. Beveled edges may reduce the amount of disturbance to the surrounding soil and eliminate the vertical excavation; however, it may make compacting the backfill at the edges difficult. This is expected to prevent the zone of influence from migrating outside of the T-section.
- Construction equipment should be kept away from the edges of the trench. Falling weight deflectometer testing on the Cedar Rapids trench showed that damage caused by equipment during construction had a long-term impact on the performance of the trench.
- The use of geogrid in the trenches did not improve the performance of the trenches compared to the trenches constructed without the geogrid for the trenches using 3/8-inch minus limestone. For trenches constructed with 1-inch clean backfill material, Trench F (T-section with geogrid) showed smaller settlement than Trench E (T-section without geogrid), but Trench F also was compacted at higher relative density. The geogrid appears to have stiffened the response of the trench based on FWD testing.
- In 2008, temperature profiles in Trenches AI and BI showed freezing temperatures as deep as 2.7 feet in the city practice trench (AI) and 1.7 feet in the controlled construction trench. Trench CI (with T-section) showed a deeper freezing temperature, which could be the effect of the native soil around the T-section. The freezing temperature in the native soul was about 2.5 feet.
- In 2008, pressures corresponded to the depth of freezing. Temperatures showed a reduction during winter, which could be attributed to heaving of the layer above.
- In 2008, settlement profiles in different trenches showed that most movement happened in the top layer, except the trenches with T-sections, which could be related to the deeper temperature change observed in this trench.

Quality Management

- Quality control measures should be implemented in the field to ensure that compaction requirements are met. This includes achieving at least medium-to-dense relative density with moisture contents above the bulking moisture content for cohesionless soils and above 95% of Standard Proctor and +/- 2% of optimum moisture content for cohesive soils.
- An educational program should be established to educate city maintenance crews on the importance of proper construction practices. Based on the experience with the city of Ames, a program including demonstrations will help solidify the importance of moisture control during the construction of trenches.

Future Research Needs

- Reduce work associated with T-section and improve the zone of influence, construct trenches with a beveled cross section at the top to facilitate the compaction of the backfill at the perimeter of the trench.
- Continue FWD testing on the trenches.
- Continue to monitor the settlement of the trenches.
- Continue to monitor the instrumented trenches.
- Pavement surface roughness using the International Roughness Index (IRI) data to assess the ride quality in utility cut areas needs to be studied and investigated across the state of Iowa to help in maintenance and improve ride quality in utility cut.

CHAPTER 2. LITERATURE REVIEW

Summary of Phase I Results

Pavement settlement at utility cuts is a common concern that requires municipal resources for additional maintenance. The Phase I report pointed out that utility cut restoration failures increased maintenance costs.

During Phase I, a survey of several Iowa cities supplemented with site visits identified factors that contributed to the settlement of utility cut restorations in pavement sections throughout Iowa. The construction practices, backfills, compaction effort, and moisture content used in utility cut restorations were then documented, evaluated, and analyzed (Schaefer et al. 2005).

The results from the Phase I survey showed the following:

- The likely months for water main breaks were December and January. Frost loading increased the vertical loads on the water mains.
- The majority of responding cities indicated that utility cut restorations showed signs of failure within two years.
- The specifications for compaction were based on the Standard Proctor test for all soil types.
- In the field, quality management of the backfilling operation was minimal.

The Phase I field investigation documented problems associated with utility cut trench restoration performance, construction, and backfilling. Problems identified during the field and laboratory investigations of Phase I (see Schaefer et al. 2005) were

- The use of large construction equipment
- Uneven compaction and differential settlement, especially along the edge of the trench, caused by large equipment unable to maneuver within the trench and compact the backfill
- Damage of pavement around the perimeter of the trench caused by large compaction equipment too near the edge of the utility cut (see Figure 2.1)
- The placement of backfill in lift thicknesses greater than 12 inches (in some cities lift thicknesses exceeded 3 feet), as shown in Figure 2.2
- The placement of granular backfill within the bulking moisture content range, increasing the collapse potential of the backfill when subjected to changes in moisture content (see Table 2.1)
- Low relative densities and loosely compacted backfill caused by inadequate compaction effort and minimal moisture content control in the field (see Figure 2.3)
- Soil surrounding the utility cut weakened during excavation because of the loss of lateral support (affected region around trench known as the zone of influence—during Phase I, it was determined that the FWD tests can detect the weakened zones of soil; see Figure 2.4)

Figure 2.1. Cracking pavement surrounding the utility cut area because of construction equipment getting too close to the edge of open cut

Figure 2.2. Large lift thickness used in utility cut trench backfilling

Table 2.1. Measured moisture contents of granular backfill materials during installation compared to bulking moisture content

Backfill material	Classification	γ_{Max} * (lb/ft^3)	W% (bulking)	*W%* (field)
Ames	SM	140	4.0 to 8.0	4.3 to 5.4
Cedar Rapids	SC	130	7.0 to 10.0	5 to 7
Davenport	GC	140	4.0 to 8.0	6.3 to 7.8
Des Moines	SW-SM	138	7.0 to 11.0	5.4 to 11.7

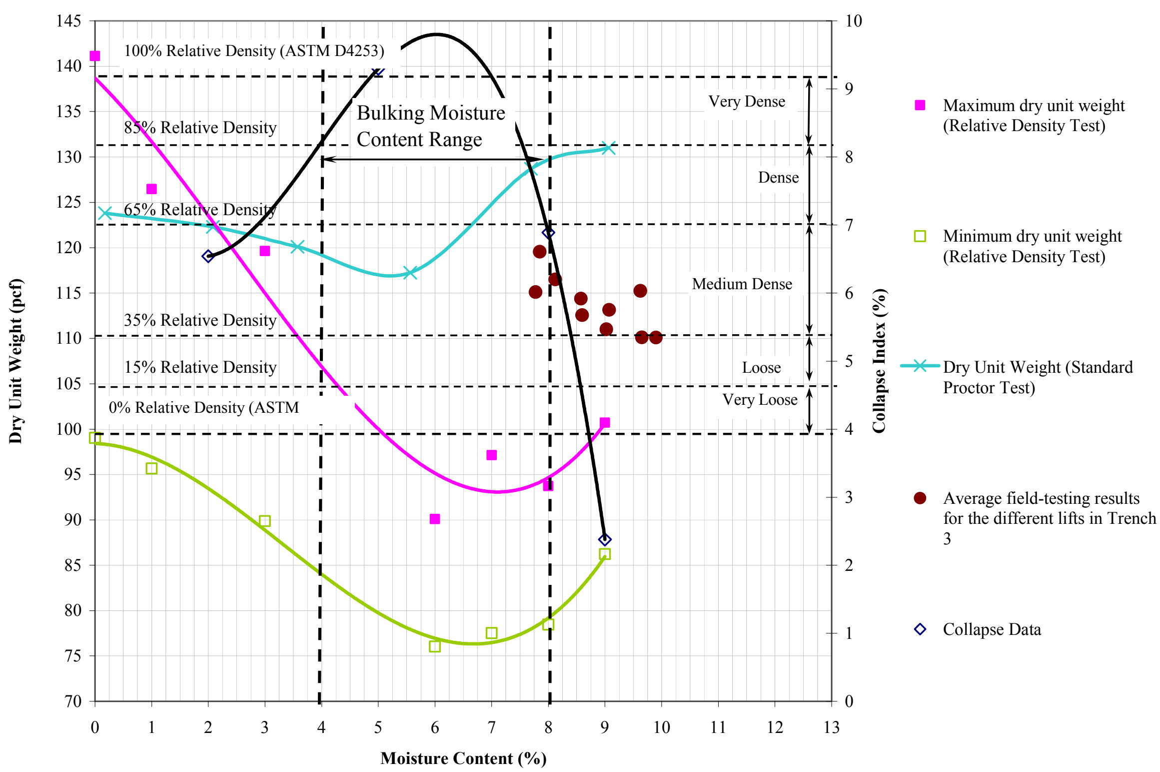


Figure 2.3. Measured densities compared with maximum and minimum densities, showing loose backfill material after compaction

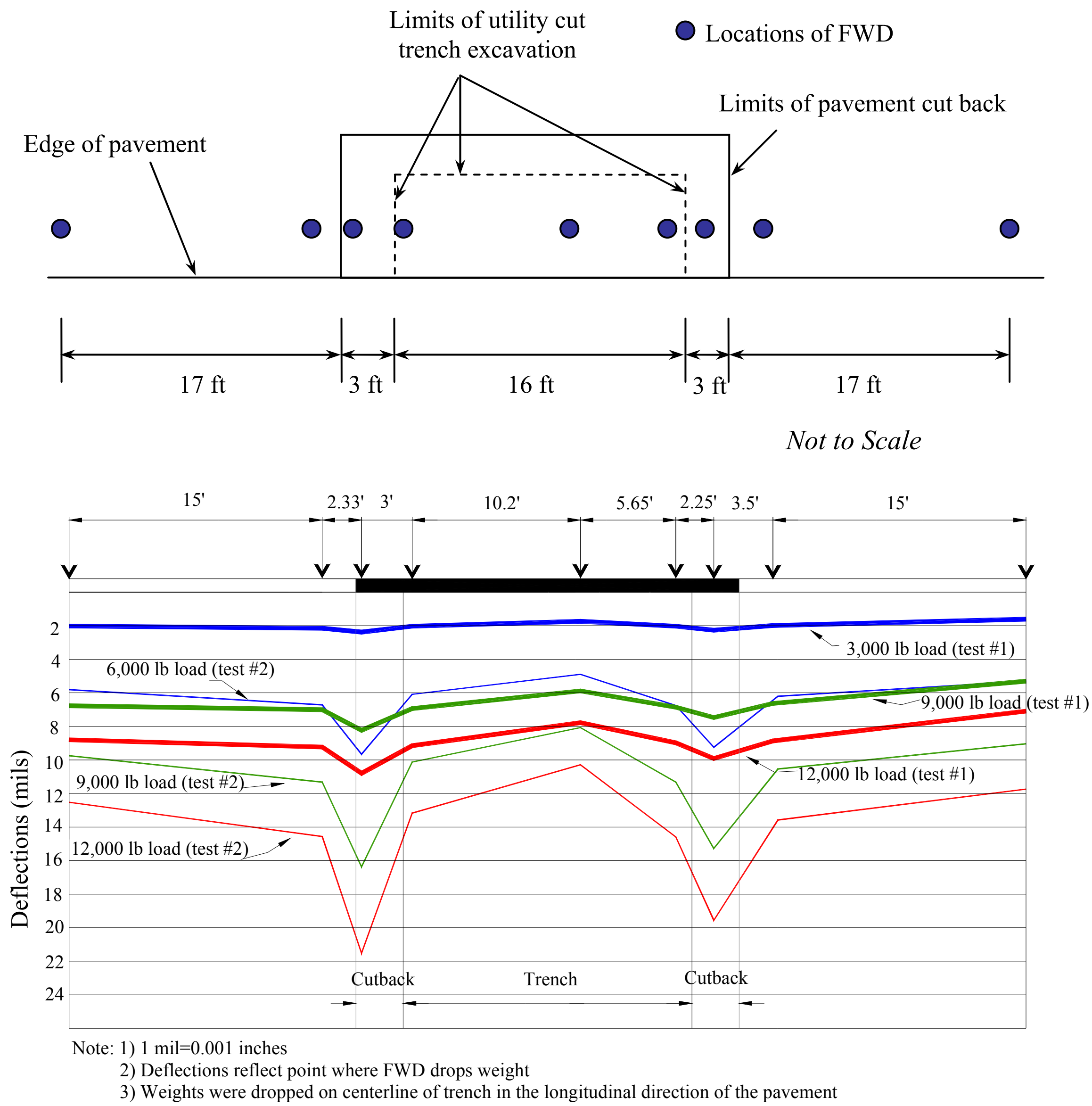


Figure 2.4. Locations and results of FWD tests performed at a utility cut location in Ames, Iowa, showing deflection within the zone of influence

Long-Term Performance of Trench Restoration Literature Review

The Phase I report documented that utility cut restorations influence the performance of pavement systems and reviewed literature before 2005. This section summarizes the literature review from Phase I and the literature related to long-term monitoring and freezing and thawing of different material.

Types of Failures

The Phase I report established that in Iowa there are several causes for utility cut restoration failure. The types of failures documented were: (1) settlement of the utility cut restoration; (2) a "bump" forming over the restoration; and (3) weakening of the surrounding soils.

The first failure type, the settlement of the restoration, was caused by two main factors. The first factor was poor compaction of the trench backfill. Poor compaction effort was the result of a combination of large lift thickness and the equipment used. Second, wet and frozen conditions also increased the settlement caused by poor compaction effort.

The second failure type was a "bump" failure of a utility cut restoration. The bump was a result of uplift or settlement of surrounding soil. The uplift was caused by frost action resulting in pavement heaving. Frost heave can occur in unsaturated soils. The third type of restoration failure was the weakening of the surrounding native soils. The weakening of the native soil caused cracking in the surrounding pavement. When a trench was excavated, the stress state of the surrounding soil changed. This caused a zone of weakened soil around the trench. In the Phase I report, this was referred to as the zone of influence (see Figure 2.5). Phase I also determined the zone of influence can be detected with FWD testing.

The magnification of these failures in utility cuts depended on the geometry of the trench. Trenches parallel to the curb were more susceptible to settlement along the edges than in the center of the trench. Settlement in trench cuts that were perpendicular to the curb occurred in the wheel paths along the center of the trench. Most of the settlement occurred within the first two years of construction.

Humphrey and Parker (1998) used finite element analysis to evaluate the failure mechanism of utility cuts up to depths of 5 feet. To model a utility cut to a depth of 5 feet and capture stress and displacements in the surrounding soil, the total model was extended 3 feet below the bottom of the cut and 4.9 feet beyond the edge of the cut. The total model dimensions, including the utility cut, were 7.5 feet in width and 8 feet in height. The model consisted of

- Asphalt concrete—4 inches
- Granular base—8 inches
- Granular fill/subgrade—8 feet

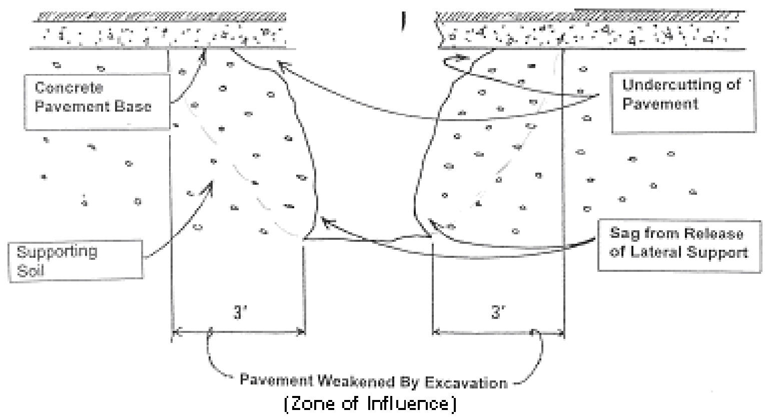


Figure 2.5. Overstressing of the pavement and natural materials adjacent to the trench

A step analysis was used to simulate the process of excavating a trench to a depth of 5 feet. The base and the fill/subgrade of the trench was modeled with well-graded gravel to sandy gravel.

The following results were documented from the model:

- After the utility cut was made, the unsupported face of the excavation displaced into the trench. The region of soil beyond the face of the cut that was affected by the displacements at the face extended up to 3.5 feet beyond the face of the cut.
- The displacements, because of the "stretching" in the models, were a function of the soil cohesion, angle of internal friction, density, and cut depth. The asphalt was in tension after the utility cut was made and restricted the movement of the subbase. Tension cracking was likely between the utility cut and the active failure plane; however, the asphalt concrete pavement stabilized the soil and prevented cracking.
- The region of soil with the highest stress caused by the displacements at the face was the granular pavement subbase, which was located between the asphalt pavement and native soils that were being excavated.

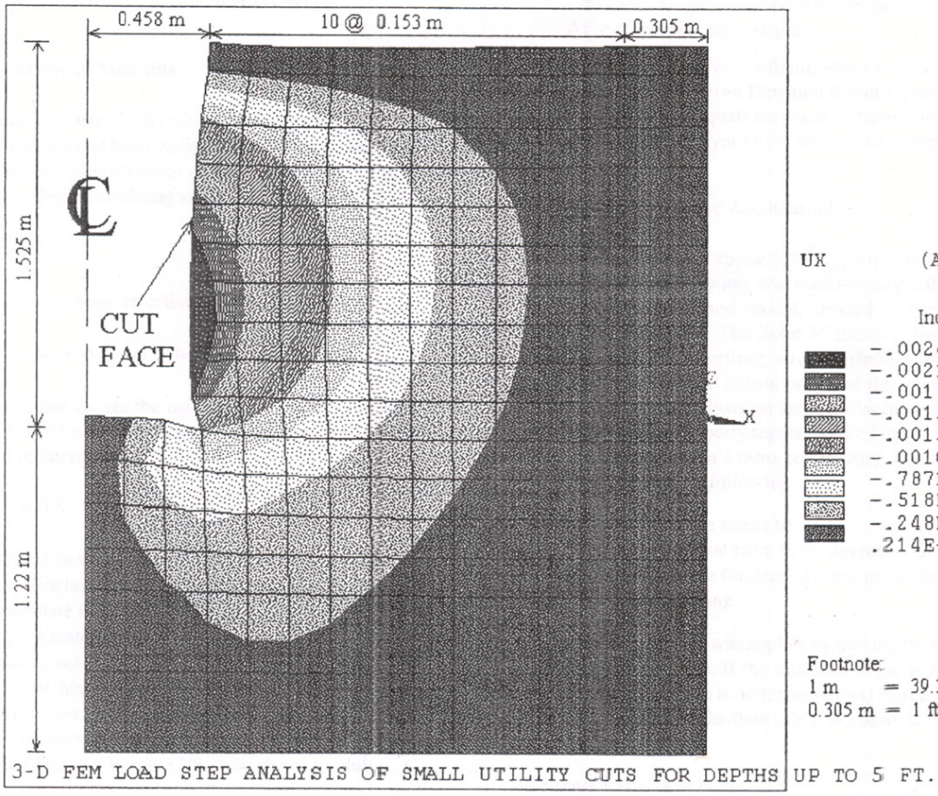


Figure 2.6. Finite element analysis from Humphrey and Parker (1998)

Properties of Backfill Material

Classification of Backfill

Backfill was classified using the American Association of State Highway and Transportation Officials (AASHTO) and Unified Soil Classification System (USCS) "Standard Classification of Soils for Engineering Purposes" (Unified Soil Classification Systems 2000). The AASHTO classification system (AASHTO 2000), used by the Iowa DOT for cohesionless materials, is presented in Table 2.2. Table 2.3 shows the AASHTO classification for cohesive materials. The USCS classification system is shown in Table 2.4.

To compare measured field parameters, published maximum dry unit weights, CBR values, and optimum moisture content for various compacted materials from the Naval Facility Engineering Command (1986), Rollings and Rollings (1996), and Sowers (1979) are presented in Table 2.5. In Table 2.6, the relative density standards developed by Budhu (2000) are presented.

Table 2.2. AASHTO M145-91 for cohesionless materials (modified from AASHTO M145-91, AASHTO 2000)

General classification	Granular materials (35% or less passing sieve #200)						
Group classification	A-1		A-3	A-2			
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7
Sieve analysis, percent passing							
No. 10 (2.00 mm)	50 max	--	--	--	--	--	--
No. 40 (0.425 mm)	30 max	50 max	51 min	--	--	--	--
No. 200 (75 μm))	15 max	25 max	10 max	35 max	35 max	35 max	35 max
Characteristics of fraction passing 0.425 mm (no. 40)							
Liquid limit	--		--	40 max	41 min	40 max	41 min
Plasticity index	6 max		NP	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine Sand	Silty or clayey gravel and sand			
General rating as subgrade material	Excellent to Good						

Table 2.3. AASHTO classification for cohesive materials (modified from AASHTO M145-91, AASHTO 1991 [AASHTO 2000])

General classification	Silt-clay materials (more than 35 percent passing the no. 200 sieve)				
Group classification	A-4	A-5	A-6	A-7	
				A-7-5	A-7-6
Sieve analysis, percent passing					
No. 10 (2.00 mm)	--	--	--	--	--
No. 40 (0.425 mm)	--	--	--	--	--
No. 200 (75 µm)	36 min	36 min	36 min	35 max	35 max
Characteristics of fraction passing 0.425 mm (no. 40)					
Liquid limit	40 max	41 min	40 min	41 max	41 min
Plasticity index	10 max	10 max	11 min	Equal to or less than LL-30	Equal to or greater than LL-30
Usual types of significant constituent materials	Silty Soils		Clayey Soils		
General rating as subgrade material	Fair to Poor				

Table 2.4. USCS (modified from ASTM D 2487 2000)

<table>
<tr><th colspan="4" rowspan="2">Criteria for assigning group symbols and group names using laboratory tests</th><th colspan="2">Soil classification</th></tr>
<tr><th>Group symbol</th><th>Group name</th></tr>
<tr><td rowspan="8">Coarse-Grained Soils More than 50% retained on the No. 200 Sieve</td><td rowspan="4">Gravels More than 50% of coarse fraction retained on No. 4 sieve</td><td rowspan="2">Clean Gravels Less than 5% fines</td><td>$C_u \geq 4$ and $1 \leq C_c \leq 3$</td><td>GW</td><td>Well-graded gravel</td></tr>
<tr><td>$C_u < 4$ and/or $1 > C_c > 3$</td><td>GP</td><td>Poorly graded gravel</td></tr>
<tr><td rowspan="2">Gravels with Fines More than 12% fines</td><td>Fines classified as ML or MH</td><td>GM</td><td>Silty gravel</td></tr>
<tr><td>Fines classified as CL or CH</td><td>GC</td><td>Clayey gravel</td></tr>
<tr><td rowspan="4">Sands 50% or more of course fraction passes No. 4 sieve</td><td rowspan="2">Clean Sands Less than 5% fines</td><td>$C_u \geq 6$ and $1 \leq C_c \leq 3$</td><td>SW</td><td>Well-graded sand</td></tr>
<tr><td>$C_u < 6$ and/or $1> C_c > 3$</td><td>SP</td><td>Poorly graded sand</td></tr>
<tr><td rowspan="2">Sands with Fines more than 12% fines</td><td>Fines classify as ML or MH</td><td>SM</td><td>Silty sand</td></tr>
<tr><td>Fines classify as CL or CH</td><td>SC</td><td>Clayey sand</td></tr>
<tr><td rowspan="8">Fine-Grained Soils 50% or more passes the No. 200 Sieve</td><td rowspan="4">Silt and Clays Liquid limit less than 50</td><td rowspan="2">Inorganic</td><td>PI > 7 and plots on or above the “A” line</td><td>Cl</td><td>Lean clay</td></tr>
<tr><td>PI < 4 or plots below “A” line</td><td>ML</td><td>Silt</td></tr>
<tr><td rowspan="2">Organic</td><td>Liquid limit – Oven dry < 0.75</td><td rowspan="2">OL</td><td>Organic clay</td></tr>
<tr><td>Liquid limit – Not dried</td><td>Organic silt</td></tr>
<tr><td rowspan="4">Silt and Clays Liquid limit 50 or more</td><td rowspan="2">Inorganic</td><td>PI plots on or above the “A” line</td><td>CH</td><td>Fat clay</td></tr>
<tr><td>PI plots below “A” line</td><td>MH</td><td>Elastic silt</td></tr>
<tr><td rowspan="2">Organic</td><td>Liquid limit – Oven dried</td><td rowspan="2">< 0.75</td><td>Organic clay</td></tr>
<tr><td>Liquid limit – Not oven dried</td><td>Organic silt</td></tr>
<tr><td colspan="2">Highly organic soils</td><td colspan="2">Primarily organic matter, dark in color and organic odor</td><td>PT</td><td>Peat</td></tr>
</table>

Table 2.5. Properties of compacted materials

Group Symbol	Typical properties of compacted backfill materials (Modified) (NAVFAC 1986)			Maximum dry unit weight for Standard Proctor test (Sowers 1979) (pcf)	Range of CBR values (%) (Rollings and Rollings Jr. 1996)
	Range of maximum dry unit weight (pcf) for the Standard Proctor test	Range of optimum moisture (%) for the Standard Proctor test	Range of CBR values (%)		
GW: Well graded gravels	125-135	11-8	40-80	125 - 135	60 to 80
GP: Poorly graded gravels	115-125	14-11	30-60	115 - 125	35 to 60
GM: Silty gravels	120-135	12-8	20-60	120 - 135	40 to 80
GC: Clayey gravels	115-130	14-9	20-40	115 - 130	20 to 40
SW: Well graded sands	110-130	16-9	20-40	107 - 130	20 to 50
SP: Poorly graded sands	100-120	21-12	10-40	100 - 120	10 to 25
SM: Silty sands	110-125	16-11	10-40	110 - 125	20 to 40
SM-SC: Sand-silt clay	110-125	15-11	5-30		---
SC: Clayey sands	105-125	19-11	5-20	105 - 125	10 to 20
ML: Silts	95-120	24-12	15 or less	95 - 120	5 to 15
ML-CL: Inorganic silt and clay	100-120	22-12	---	---	---
CL: Lean clay	95-120	24-12	15 or less	95 - 120	5 to 15
OL: Organic silts and silt-clays,	80-100	33-21	5 or less	80 - 100	4 to 8
MH: Elastic silts	70-95	40-24	10 or less	75 - 100	4 to 8:
CH: Fat clay	75-105	36-19	15 or less	80 - 105	3 to 5
OH: Organic clays and silty clays	65-100	45-21	5 or less	70 – 100	3 to 5

Table 2.6. Relative density and compaction (Budhu 2000)

Compaction	Relative density
Very Loose	0 to 15
Loose	15 to 35
Medium Dense	35 to 65
Dense	65 to 85
Very Dense	85 to 100

Moisture Content

The dry unit weight obtained during compaction was a function of the material's moisture content. When water was added so granular materials were moistened, water surrounded the particles and formed a meniscus between the particles (see Figure 2.7). The meniscus caused high capillary tension forces between the particles. The tension force between the particles prevented the particles from rearranging into denser alignments. These forces between particles are difficult to overcome.

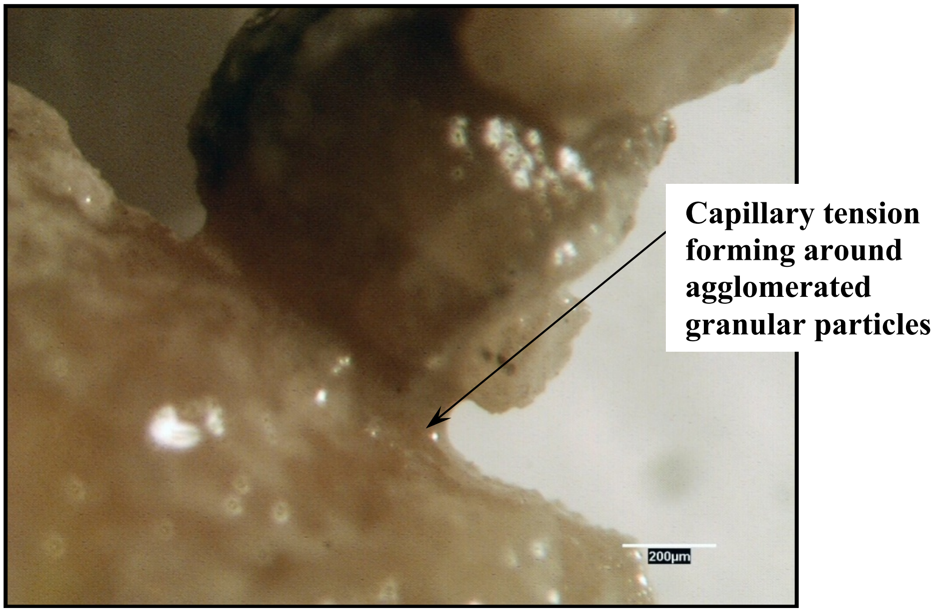


Figure 2.7. Meniscus between two granular particles

When additional water was introduced, the voids began to fill up and the meniscus between the particles decreased, while the tension forces were released. This caused the granular particles to slide over each other, forming a denser arrangement. This rearrangement of particles is referred to as collapse of granular materials. Collapse behavior of a soil aggregate can be determined in the field or laboratory.

Houston et al. (1996) explored the differences between field and laboratory collapse test methods. The field test was a plate load test and the laboratory test was a response-to-wetness test. The advantages of the field tests were: (1) there was minimal sample disturbance; and (2) the degree of wetting to cause collapse was similar to actual conditions. The disadvantages of the field tests included nonuniform stress state of the soil contributing to settlement and also difficulty in determining the stress-strain relationship.

The advantage of the laboratory tests were that a uniform stress-strain curve could be found. The disadvantages of the laboratory tests were that the sample disturbance and the saturation required to cause collapse in the laboratory were higher than in the field. The bases for judging which test is appropriate depends on the application, field conditions, and avalibility.

Based on the findings of Houston et al. (1996), the bulking moisture contents were defined based on the moisture content for the maximum collapse potential +/- 2%.

Iowa Backfill Standards

Acceptable Gradations

The Iowa DOT specifications allowed for a wide variety of backfill materials. SUDAS specifications also included several different types of trench backfill materials that were based on ASTM D 2321-00 (XXXX). SUDAS Class I material is a cohesionless material with fines limited to 5% and is classified as nonplastic. SUDAS Class I material was primarily used for pipe bedding but was also recommended for trench backfill in areas under pavement.

The Iowa DOT Specifications, Section 4120, contained several backfill gradations. Table 2.7 presents the Iowa DOT and the SUDAS specifications gradations.

Table 2.7. Backfill material gradation standards (modified from Schaefer et al. 2005)

Sieve size	Acceptable Iowa DOT gradations								SUDAS Class I bedding and backfill	
	No. 10		No. 11		No. 16		No. 32			
	Upper limit	Lower limit	Upper limit	Lower limit	Upper limit	Lower limit	Upper limit	Lower limit	Upper limit	Lower limit
1.5"					100		100% passing the 3" sieve		100	100
1.0"			100						100	95
3/4"	100		100	95						
0.500"			90	70	50	0			60	25
4	80	50	55	35	10	0	100	20	10	0
8	60	25	40	11						
200			16	6			0	10		

Backfill Placement

Lift thicknesses in Iowa often exceeds 24 inches. The SUDAS specifications stated that lift thicknesses should be a maximum of 6 inches in the primary and secondary backfill areas. Other lifts should be placed in loose lifts with a thickness less than 12 inches.

Compaction Equipment

To properly select compaction equipment for a given backfill, three considerations should be made: type of material, lift thickness, and application (Hilf 1991). Because of the confined area of a trench, the choice of equipment is limited. Table 2.8 shows two different equipment types and the requirements to meet 95% to 100% Standard Proctor maximum density.

Table 2.8. Lightweight compaction equipment (modified from Hilf 1991)

Equipment type	Applicability	Requirements for compaction of 95 to 100 percent Standard Proctor maximum density			Possible variations in equipment
		Compacted lift thickness, inches	Passes or coverage	Dimensions and weight of equipment	
Vibrating base plate compactors	For coarse-grained soils with less than 12 percent passing No. 200 sieve. Best suited for materials with 4 to 8 percent passing the No. 200 sieve placed thoroughly wet.	8 to 10 inches	3 passes	Single pads or plates should weigh no more than 200 pounds. May be used in tandem where working space is available. For clean coarse-grained soils, vibration frequencies should be no less than 1,600 cycles per minute.	Vibrating pads or plates are available, hand-propelled or self-propelled, single or gangs, with width coverage from 1½ to 15 feet. Various types of vibrating-drums equipment should be considered for larger areas.
Power tamper or rammer	For difficult areas, trench backfill. Suitable for all inorganic soils.	4 to 6 inches for silt or clay, 6 inches for coarse grained soils.	2 passes	30 pounds minimum weight. Considerable range is tolerable, depending on materials and conditions.	Weight up to 250 pounds; foot diameters 4 to 10 inches

As lift thickness increased beyond the recommended thickness for the equipment type, the effectiveness of the equipment decreased. With lightweight equipment, such as the equipment used in trenches, the effective compaction depth was shallow. Vibrations do not compact material deeper than 4 to 10 inches into the lift (Table 2.8). Conversely, for heavyweight equipment, the effective depth of compaction was deeper; however, material near the surface remained relatively unaffected by the vibrations being applied to the surface.

Another consideration when selecting compaction equipment was the effect of equipment on moisture content and dry unit weight. According to Winterkorn and Pamukcu (1991), as the size of the backfill tamper equipment increased, the dry unit weight of the compacted soil decreased and the moisture content increased. In addition, they found base plate compactors in the field yielded higher optimum moisture content than found in laboratory testing.

Field Quality Management

During the Phase I project, it was concluded that backfill placement quality management on a project was critical to the performance of the utility cut restoration. To monitor the placement of backfill, three different tests were used: nuclear density, DCP, and Clegg Hammer.

Nuclear Density Test

The Nuclear Density test, Standard Test Method for In-Place Density and Moisture Content of Soil and Soil-Aggregate by Nuclear Methods (ASTM D 6948-08 2008), provided the moisture content and dry unit weight for in-situ soils. The gauge works by emitting two types of radiation. The neutron radiation measures the moisture content of the material, and the gamma ray radiation measures the dry unit weight of the material. The probe can be inserted up to 12 inches into the material being tested. When operating in a confined area (i.e., a trench), the nuclear density gauge must be calibrated.

Because of the radioactive material in the gauge, users are required to become certified to operate the equipment. Phase I found that some state and local governments are limiting the use of the nuclear density gauge because of the concerns about radioactive material.

DCP Test

The DCP test is an in-situ test. This test provides a profile of the CBR values for each test location. The test was performed by driving a cone-tipped rod into the ground with a 17.6-pound hammer. A vertical profile of millimeters per blow for the material was developed, and then the DCP Index (DCPI) was calculated using Equation 2.1 (Sawangsuriya and Edil 2004):

$$DCPI_{wtavg} = \frac{1}{H}\sum_{i}^{N}\left[(DCPI)_i \times (z)_i\right] \tag{2.1}$$

Where: H = total penetration depth
z = lift thickness
$DCPI_{wage}$ = penetration index for z

The DCPI was then correlated to the CBR. Several CBR correlations are available; however, to maintain consistency between the Phase I report and Phase II report, the same correlation was used (Equation 2):

$$CBR = \frac{292}{DCPI^{1.12}} \tag{2.2}$$

The ASTM standard for the DCP test is ASTM D 6951 (2003), Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.

The following are advantages of the DCP test:

- Minimal training is needed for operation.
- Equipment is inexpensive.
- Immediate results can be used on different soil types.

The following are disadvantages of the DCP test:

- Field results require detailed calculations to obtain CBR values.
- There can be inconsistencies for well-graded materials and granular materials.
- Test results are inconsistent if less than 10 millimeters are penetrated for a given blow count.

Clegg Hammer Test

The third test used to evaluate the placement of the backfill material was the Clegg Hammer test. The ASTM standard for the Clegg Hammer was ASTM Standard D 5874 (2002), Standard Test Method for Determination of the Impact Value (IV) of a Soil. The Clegg Hammer test dropped a 9.9-pound hammer 18 inches. The hammer was dropped four times. The deceleration of the hammer was measured for each drop. The fourth drop was the IV reading. The IV reading was correlated to a CBR value by Equation 2.3 developed by Clegg (1986):

$$CBR = \left(0.24\left(IV\right)+1\right)^2 \tag{2.3}$$

This equation is for general use. For other materials (including granular), laboratory and field-testing data need to be correlate for each individual material. According to Dr. Baden Clegg:

> "However, since CBR is particularly subject to high variability, even within one organization, one soil type, etc., correlations from individual sources may vary from the general equation. To avoid a change of standards it is appropriate therefore that each organization (sic) should consider establishing its own relationship for specific materials and conditions…"

For this project, no correlation was developed between field and laboratory results. Therefore, there may be inconsistencies with CBR values found from the Clegg Hammer test.

Post-Construction Monitoring

Falling weight deflectometer testing can be used to monitor the long-term performance of pavements. The FWD test applies various point loads to the pavement and measures the deflections of the pavement under the load, 12 inches in front of the load, and behind the load at 8, 12, 18, 24, 36, 48, and 60 inches. The deflections measured in the pavement are related to the strength/stiffness of the pavement. During Phase II, the FWD test was performed at a series of locations across each trench. The locations that were chosen can be generalized as follows: outside the zone of influence, 1 to 2 feet away from the edge of the trench, 1 to 2 feet inside the edge of the trench, and at the center of the trench. This testing plan was modified for each trench, depending on the trench geometry. By plotting the deflections under the load at each test point versus distance, weakened zones in the pavement structure were detected.

Al-Suhaibano et al. (1992) used FWD testing to study utility cut restoration failures and the impact on the overall pavement performance. Seventy-five utility cut restorations were randomly selected across Riyadh, Saudi Arabia, on roads wider than 25 feet. The utility cuts were monitored with FWD testing at four points on the trenches (center, inner edge of the cut, outer edge of the cut, and on original uncut pavement). This is similar to the FWD testing plan used in this project.

Al-Suhaibano et al. (1992) concluded: (1) utility cuts increased the deterioration of the pavement compared to undisturbed pavements; and (2) the geometry of the trench affected the performance of the trench. As the width of the trench increased, the deflection of the center of the trench decreased in FWD testing.

They recommended: (1) increasing the thickness of the pavement over the utility cut (this report did not provide a recommended thickness); 2) removing the pavement adjacent to the trench and extending the patch over the existing subbase (this paper recommended researching the distance beyond the cut that the pavement should be removed); (3) increasing quality management for the materials used and the construction practices; and (4) increasing the width of the utility cut to allow for better compaction of the backfill material. Determining the appropriate width of a utility cut "warranted further study."

The research outlined in this report will help further refine the recommendations of Al-Suhaibano et al. (1992) concerning the removal and replacement of pavement arround the trench, and it will evaluate the performance of two different utility cut methods.

To investigate the behavior of the pavement system with utility cuts using FWD, FWD testing should be performed during the spring/early summer and fall. Testing at these times of the year will help monitor the overall long-term performance of the trenches, but it will also provide evidence as to the seasonal effects on the stiffness of the backfill and soils surrounding the trench. Andersland and Landanyi (2004) explained that the stiffness of a pavement is related to ambient temperature (see Figure 2.8). Using multiple tests per year for several years, the deterioration of the ulitilty cut and the surrounding pavements can be quantified.

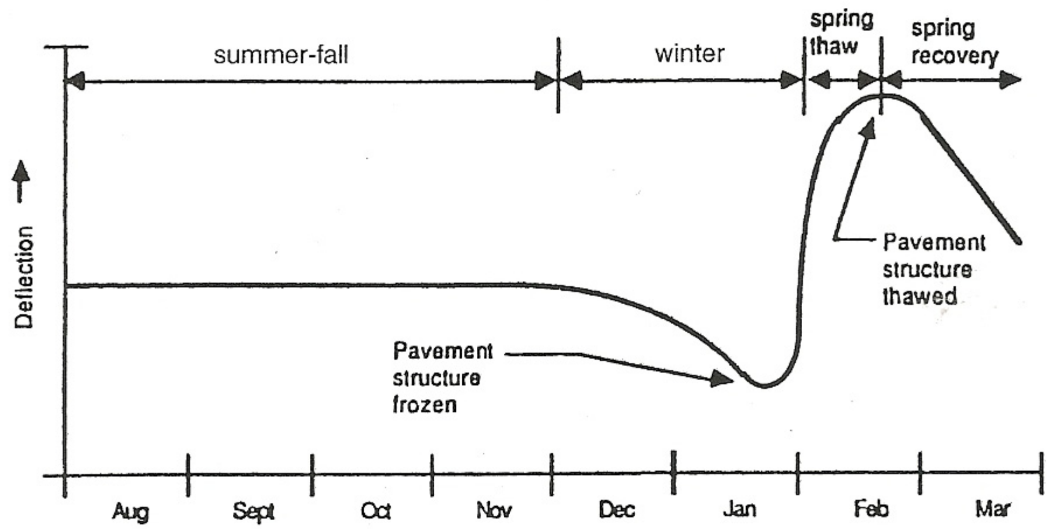
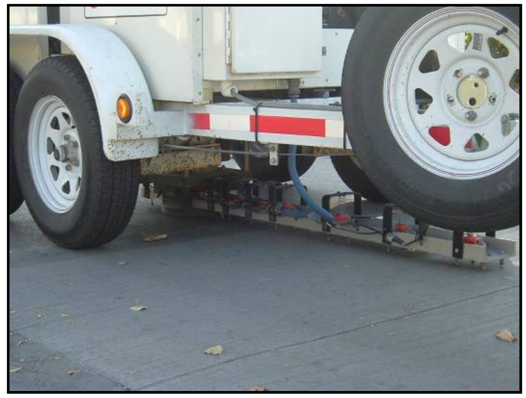


Figure 2.8. Seasonal pavement surface deflections illustrating the large decrease in strength (stiffness) during spring thaw (Andersland and Landanyi 2004)

The FWD equipment used in this project was owned and operated by the Iowa DOT (see Figure 2.9). The deflection of the pavement was measured at the load application point and at ten additional points. The loads applied to the pavement were 6,000 pounds, 9,000 pounds, 12,000 pounds, and 15,000 pounds. These loads were chosen to detect the response of the surface pavement layer and the effect of subgrade on the pavement system response.



(a) (b)

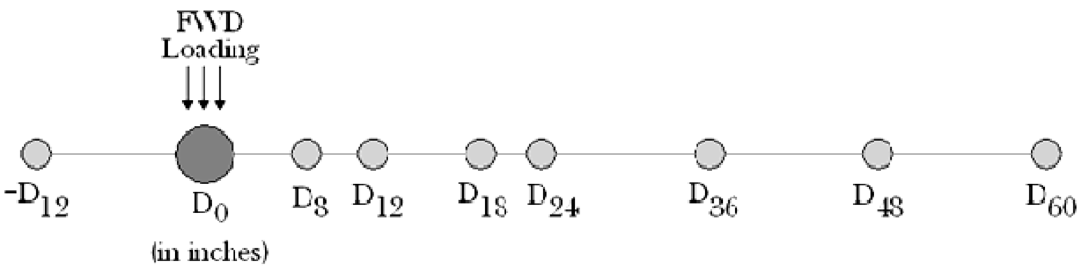


Figure 2.9. (a) Iowa DOT FWD equipment, (b) Iowa DOT FWD sensor configuration

Seasonal Effects

One of the major impacts of trench performance is seasonal effects. Frost can cause two problems according to Anderson et al. (1984). First, frost changes the stiffness of the soil structure during freeze and thaw cycles, as shown in Figure 2.10. As frost forms, the pavement structure stiffens. When frost thaws, the increase in water content in the soil causes the pavement structure to weaken. Second, displacements are caused by the formation of ice lens and the pressure on related structures, which are normal to the growth of the lens (see Figure 2.10).

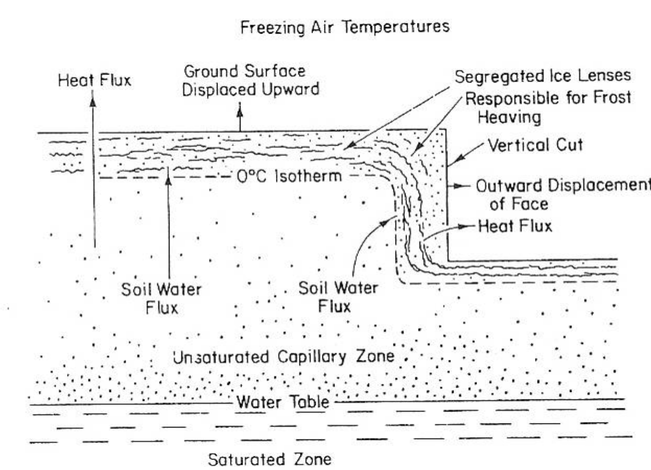


Figure 2.10. Schematic illustration of frost heave (Anderson et al. 1984)

Frost formation is affected by the soil type and its thermal/hydrostatic conductivities, the temperature gradient, and available moisture. Heat flux between the soil and adjacent medium (air or soil) flows perpendicular to the interface between the mediums, as illustrated in Figure 2.10. Frost formation affects pressures within a soil body and movements (both vertical and horizontal).

As frost forms in a soil, a frozen fringe develops (see Figure 2.11). This fringe develops at freezing temperatures and extends downward. The frozen fringe forms below where an active lens is forming. As freezing temperatures penetrate deeper in a soil, the frozen fringe and zone of active lens formation also migrate downward. The active ice lens layer in the soil is also a boundary where the permeability of the soil decreases (Andersland and Landanyi 2004) because the pores begin filling with ice. This boundary prevents water from traveling upward beyond the active lens zone. Because of this, no additional ice lens will form above the active ice lens zone. The downward movement of the frozen fringe affects the size of the ice lens. When the front advances rapidly through a soil, the lens will be thin; however, when the frozen fringe remains at a stationary point because of the heat flow balance, a larger lens will form.

As ice lenses form, they exert an outward pressure on the pore (Andersland and Landanyi 2004). When the pressures are greater than the overburden pressures, the soil will heave. The heave (expansion of soil) occurs at the frost line, which is assumed to be at freezing. Frozen soils above the frost line do not expand because there is no influx of moisture (Andersland and Landanyi 2004).

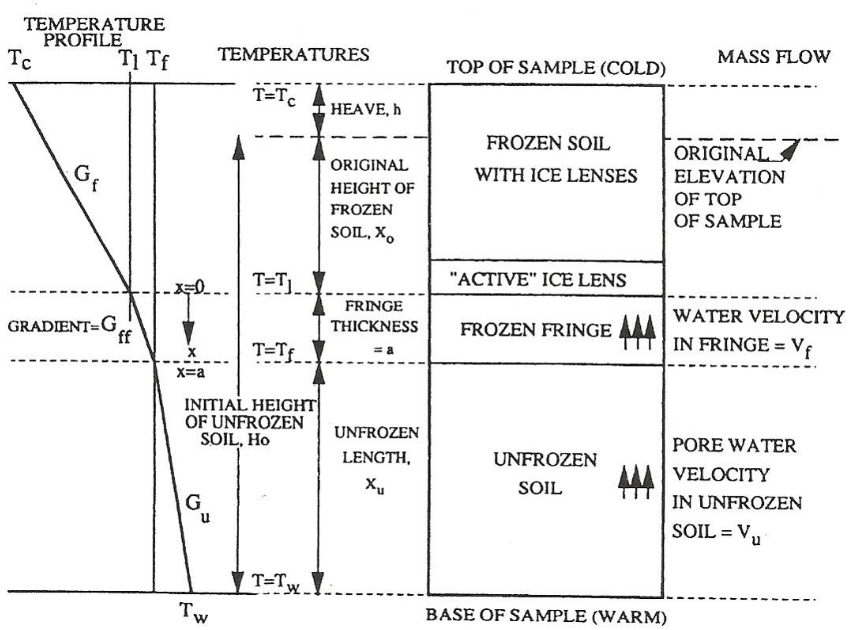


Figure 2.11. Frost heave in an idealized one-dimensional soil column (Andersland and Landanyi 2004)

According to Taber (1929), soils do not need to be saturated to experience frost formation. Conversely, when a soil is at freezing temperatures, not all of the water is frozen. This is shown in Figure 2.12, where the amount of frozen water in a soil varies by the type of soil.

When frost forms in soils, the heave causes pressures on adjacent structures. Figure 2.13 shows the linear relation between temperature and frost heave pressures. Heaving pressure only occurs at the frozen fringe and when the ice lens at the frozen fringe displaces a sufficient number of soil particles that exceed the overburden pressures.

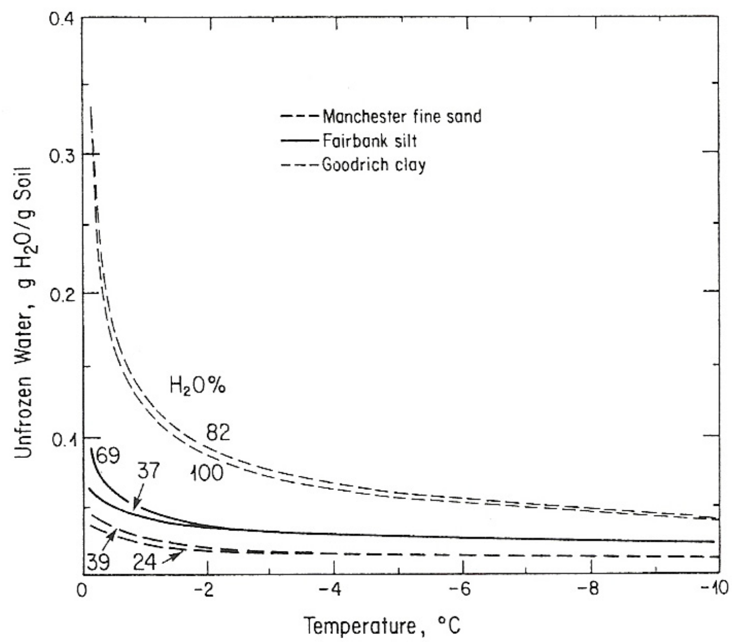


Figure 2.12. Phase comparison of representative soils (Anderson et al. 1984)

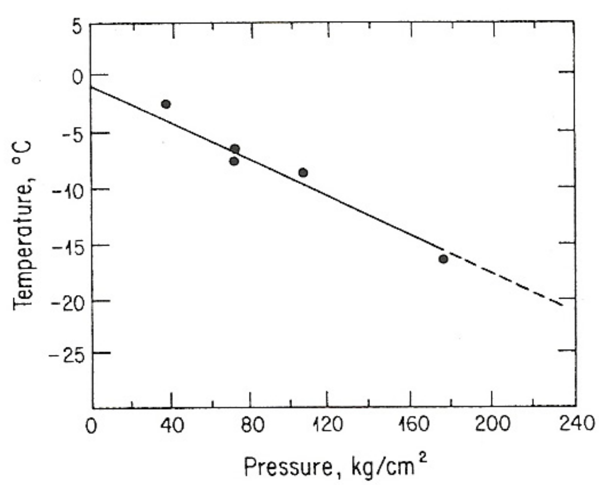


Figure 2.13. Relationship between temperature and maximum observed heaving pressure in montmorolonite clay (Anderson et al. 1984)

Frost Susceptibility Classifications for Soils

Several methods have been developed to determine the frost susceptibility of a soil. All of the classifications are based on the size and quantity of fine-sized particles in any soil. According to Anderson et al. (1984), the frost susceptibility of a soil is a function of particle and void size. Gravels have large voids and large particles. This allows water to flow freely through the soil. When water does freeze in gravel, the void size is large enough so that as the frozen fringe passes through, the lens cannot grow large enough to displace the particles. Clays, at the other end of the gradation chart, have very small particles and small voids. Less force is required to displace smaller soil particles than larger particles, such as gravels. This make soils with high percentages of fines more susceptible to frost. However, the low permeability and higher thermal conductivity, according to Anderson et al. (1984), result in the frozen front moving quickly through a soil before water is able to migrate to the frozen front. Anderson et al. (1984) state that silt-size particles are more susceptible to frost formations and vertical movements than gravels or clays.

The Army Corps of Engineers has a classification system based on percentage of particle weight finer than 0.02 mm. Table 2.9 presents the classification system with the design group classification, and Figure 2.14 presents the classification system graphically with the design group classification. The design group classification is used for design of structures in frozen soils. For this report, all frost heave classification will be based on the U.S. Army Corps of Engineers system.

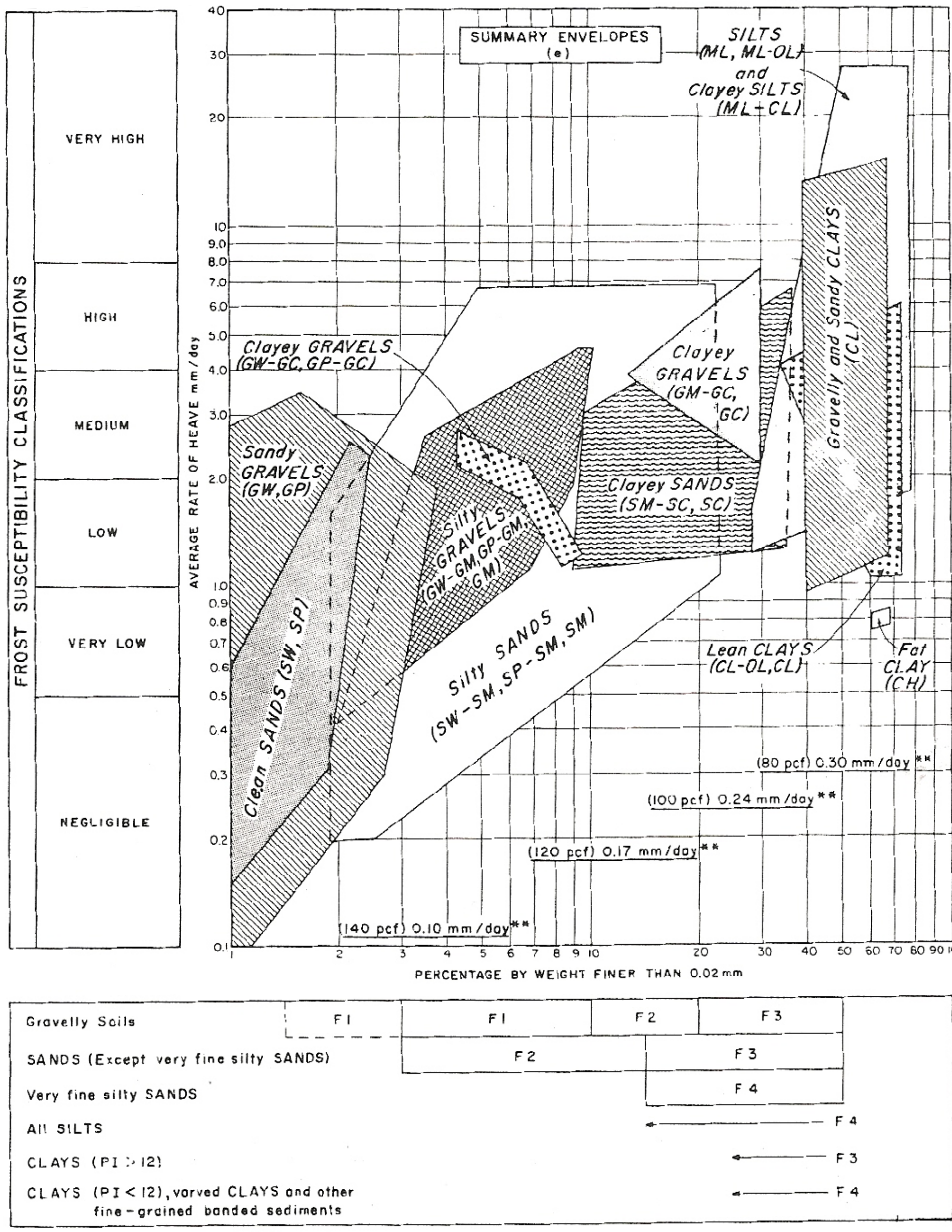
The Alaska frost susceptibility is based on percent of particles by weight finer than 0.074 mm and the depth below pavement (see Figure 2.15). Alaska classifications account for the affect of overburden pressure. As overburden pressures increase, the ability of frost heave decreases (Andersland and Landanyi 2004).

The New Brunswick frost susceptibility criteria is based on limiting the total fines smaller than 0.074 mm to less than 7% and, using Figure 2.16, determining what percent of the fines are clay, sand, or silt.

Table 2.9. U.S. Army Corps of Engineers Frost Design Classification System (Anderson et al. 1984)

Frost susceptibility[a]	Frost group	Kind of soil	Amount finer than 0.02 mm (wt %)	Typical soil type under USCS[b]
Negligible to low	NFS[c]	a. Gravels	0–1.5	GW, GP
		b. Sands	0–3	SW, SP
Possibly	PFS[d]	a. Gravels	1.5–3	GW, GP
		b. Sands	3–10	SW, SP
Low to medium	S1	Gravels	3–6	GW, GP, GW-GM, GP-GM
Very low to high	S2	Sands	3–6	SW, SP, SW-SM, SP-SM
Very low to high	F1	Gravels	6–10	GM, GW-GM, GP-GM
Medium to high	F2	a. Gravels	10–20	GM, GM-GC, GW-GM, GP-GM
Very low to very high		b. Sands	6–15	SM, SW-SM, SP-SM
Medium to high	F3	a. Gravels	>20	GM, GC
Low to high		b. Sands except very fine silty sands	>15	SM, SC
Very low to very high		c. Clays, $I_p > 12$	—	CL, CH
Low to very high	F4	a. All silts	—	ML, MH
Very low to high		b. Very fine silty sands	>15	SM
Low to very high		c. Clays, $I_p > 12$	—	CL, CL-ML
Very low to very high		d. Varved clays and other fine-grained banded sediments	—	CL and ML; CL, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

[a] Based on laboratory frost-heave tests.
[b] G, gravel; S, sand; M, silt; C, clay; W, well graded; H, high plasticity; L, low plasticity.
[c] Non-frost susceptible
[d] Requires laboratory frost-heave test to determine frost susceptibility.
Source: Johnson et al. 1986.



NOTES: Standard tests performed by Cold Regions Research and Engineering Laboratory; specimens 6 in. dia. by 6 in. high, frozen at penetration rate of approximately 0.25 in. per day, with free water at 38°F continuously available at base of specimen. Specimens compacted to 95% or better of applicable standard, except undisturbed clays. Saturations before freezing generally 85% or greater.

* Undisturbed specimen

** Indicated heave rate due to expansion in volume, if all original water in 100% saturated specimen were frozen, with rate of frost penetration 0.25 inch per day.

Figure 2.14. Degree of frost susceptibility of soils to the U.S. Army Corps of Engineers (Anderson et al. 1984)

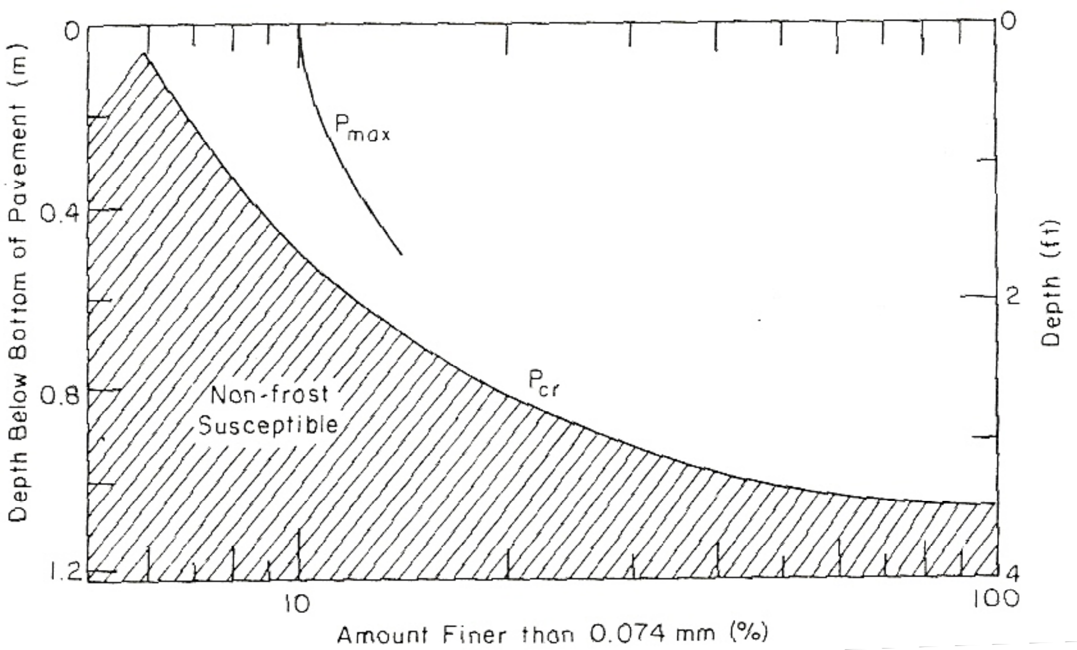


Figure 2.15. State of Alaska frost susceptibility criteria (Anderson et al. 1984)

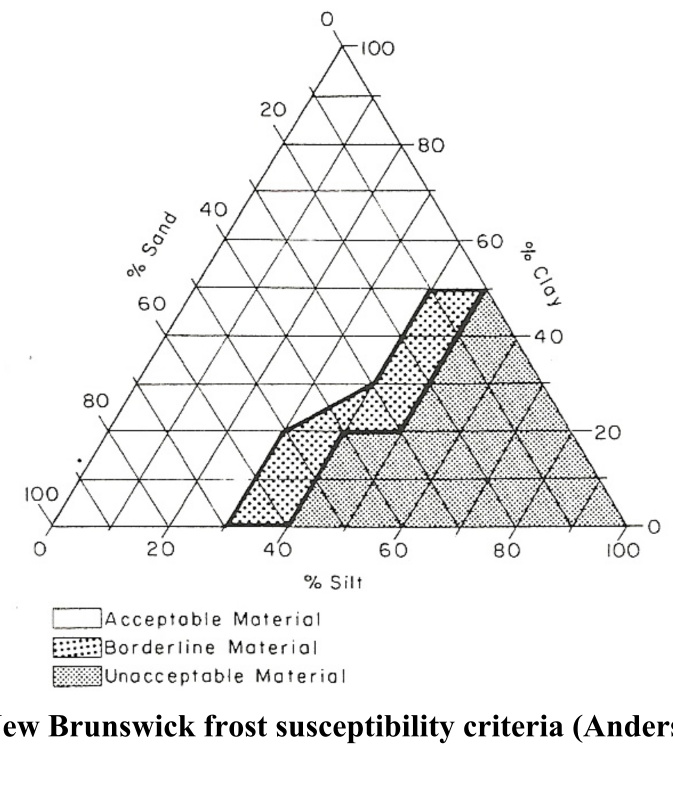


Figure 2.16. New Brunswick frost susceptibility criteria (Anderson et al. 1984)

CHAPTER 3. CONTINUED MONITORING OF PHASE I UTILITY CUT RESTORATIONS

Part of the Phase II project was to continue monitoring the performance of utility cuts that were observed in Phase I. The trenches selected for continued monitoring from Phase I had field tests on the backfill performed during the construction of the trench. Information and data from before May 2007 in this chapter was reported in the Final Phase I report (see Schaefer et al. 2005). .

Location and Summary of Local Agency Utility Cut Restorations

During Phase I, construction practices for utility cut restorations used in several cities in Iowa were documented at sites in Ames, Cedar Rapids, Council Bluffs, Davenport, Des Moines, Dubuque, and Waterloo (see Figure 3.1).

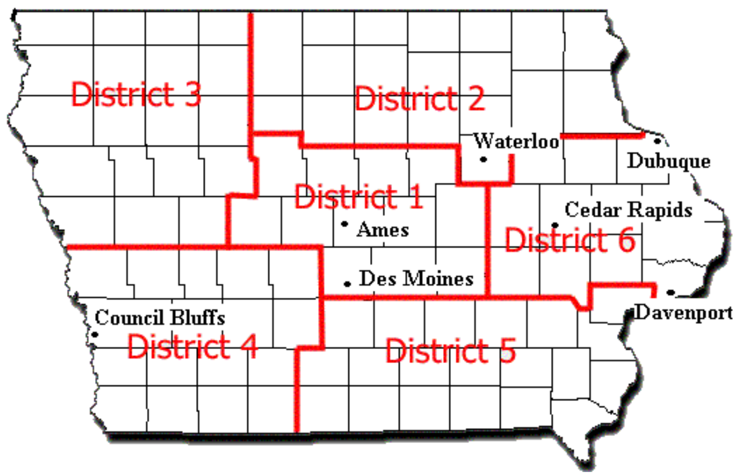


Figure 3.1. District map of Iowa showing the locations of cities where utility cut restoration practices were documented

A wide variety of construction practices and backfill materials was used in utility cut trench restorations in Iowa. The selection of backfill mainly depended on what was available in the area. Backfill placed with lifts greater than 3 feet resulted in low dry unit weights, increased settlement, and distressed pavement in the utility cut areas. To understand the effects of various backfills and lift thickness, four sites were selected for field testing and monitoring. Figure 3.2 shows the location of the trench sites:

- Ames, 20th Street and Hayes Avenue
- Cedar Rapids, Miami Drive and Sherman Avenue
- Davenport, Iowa Street and East 4th Street
- Des Moines, East Grand Avenue and East 28th Street

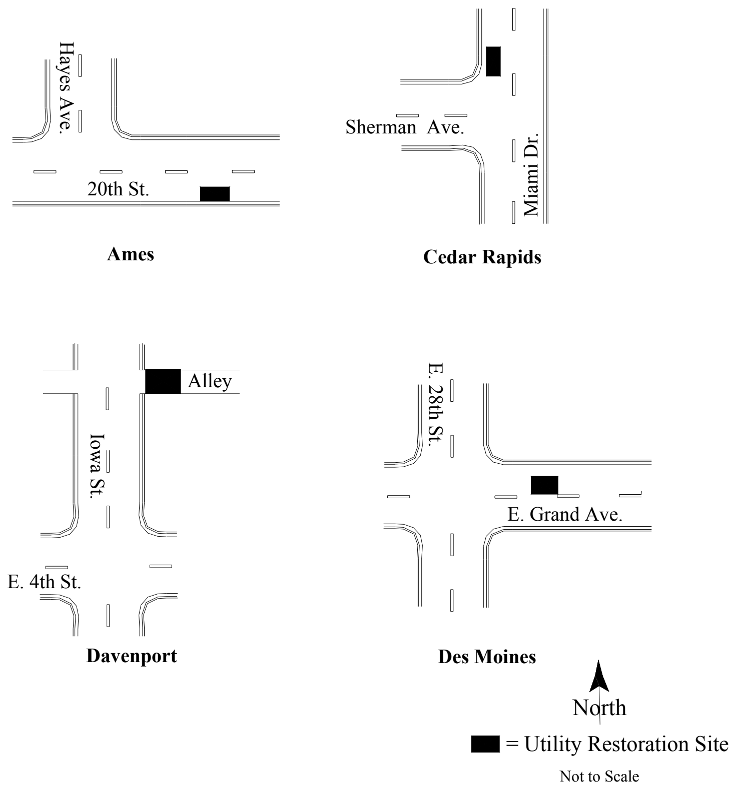


Figure 3.2. Local agency utility cut restoration field-testing site locations

Utility cut restoration sites in Council Bluffs, Dubuque, and Waterloo, were visited, and the construction practices (materials, lift thickness, and methods) were observed and documented by the Iowa State research team; however, field tests were not performed on these sites because of time constraints.

A description of construction procedures, field testing, and laboratory test results for the four sites selected for field testing was summarized in the Phase I report. This report provides a brief summary of the construction, field testing, and laboratory test results for the sites in Ames, Cedar Rapids, and Des Moines. On these three sites, surveying and FWD testing was continued during Phase II. The Davenport site was not monitored because the utility cut spanned the length of the alley. Because of its length, it was not possible to conduct FWD testing. The portion of the trench where field testing was performed is shown in Figure 3.2.

Ames, 20th Street and Hayes Avenue

On October 18, 2004, a water main break required construction of a utility trench and its restoration at 20th Street and Hayes Avenue in Ames. The site was located in a high-traffic area. Traffic loads consisted of traffic from the local high school and the CyRide bus system. The City of Ames performed the excavation, construction, and restoration of the trench. The construction procedure can be summarized as follows:

- Asphalt pavement was removed from the trench area. The trench was approximately 16 feet long, 6 feet wide, and 10 feet deep.
- Saturated soil from the sides of the trench fell into the trench during the excavation.
- One-inch limestone was placed as bedding material under the pipe and was loosely placed as backfill around the pipe until the crown of the pipe was covered by 2 feet of this material.
- A vibratory plate compactor attached to a backhoe was used for compacting the bedding and backfill.
- After the 1-inch limestone was compacted, 3/8-inch minus limestone material (a by-product produced by Martin Marietta Quarry in Ames, Iowa) was placed in loose 2-foot lifts.
- After placing the backfill in the trench to the level of the pavement, the trench remained open to traffic for about two weeks to further compact the backfill material.
- After two weeks, 2.5 feet of additional pavement surrounding the trench was removed, the upper portion of the trench backfill material was removed, and the pavement patch material was placed over the trench and surrounding area.

Results from Field and Laboratory Testing

The laboratory testing in Phase I classified the 3/8-inch minus backfill as SP-SM (silty sands, poorly graded sand, and silty mix). The maximum dry unit weight was 140.0 pcf from the relative density test, and the bulking moisture content ranged from 4% to 8% based on the collapse potential testing.

During the Phase I field work, only one nuclear density test was performed due to time constraints (see Field Investigation section of Phase I report, page 61). The one nuclear density test performed on the top lift of backfill yielded a dry unit weight of 115.6 pcf at a moisture content of 6.3%. The backfill was compacted to a relative density of 47%, which according to Table 2.6 is medium dense. Figure 3.3 shows the results from the laboratory tests and the average field-testing results.

The CBR values from the Clegg Hammer ranged from 5.8% to 7.6%, with an average CBR value of 6.7%. The reported CBR values from the DCP test ranged from 3.7% to 17.4%, with an average CBR value of 11.3%. The CBR values from the DCP and Clegg Hammer tests were below the typical CBR values of 20% to 40% indicated by NAVFAC in Table 2.5.

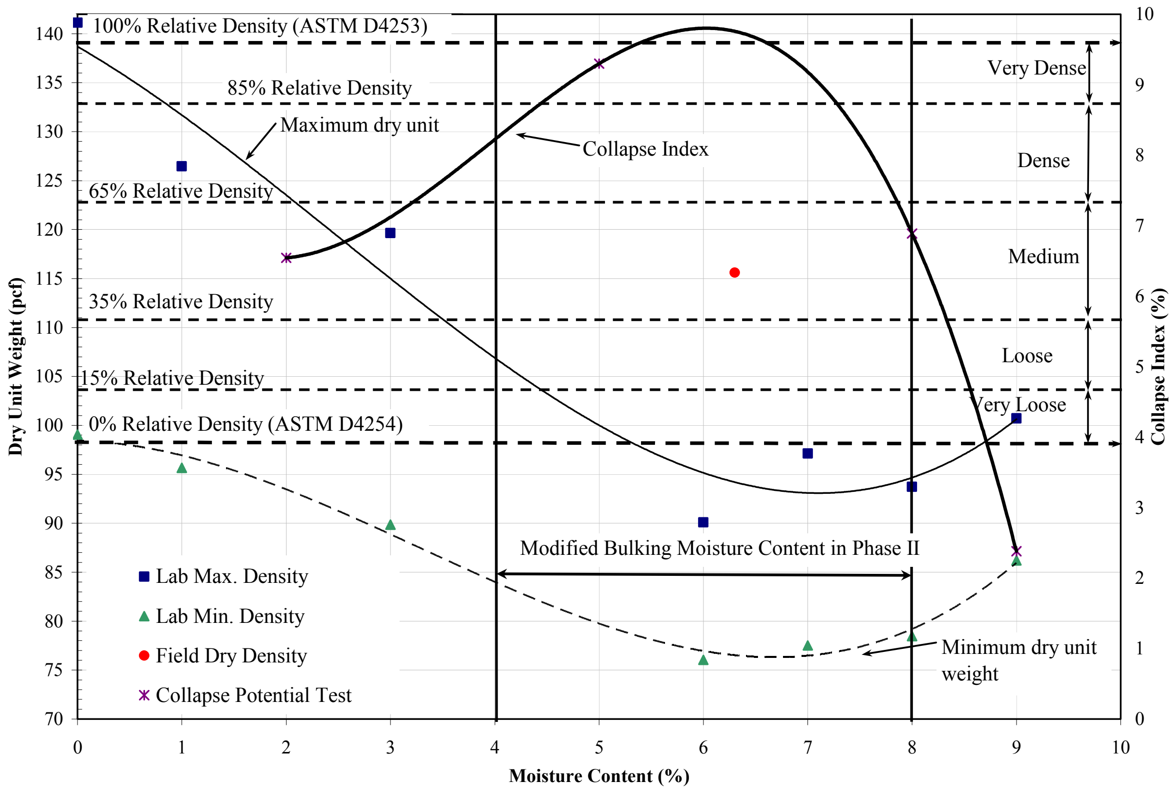


Figure 3.3. Relative density and average field-testing results for the trench backfill material for the Ames site

Continued Monitoring

The trench was surveyed three times (December 17, 2004; May 10, 2005; and May 11, 2007). Figure 3.4 shows the centerline profiles for the trench on these dates. The maximum settlement along the centerline of the trench was 0.96 inches at survey point 17 (middle of the trench). The maximum settlement, which was along the edge of the trench, was 1.20 inches at survey points 24 and 26. The maximum settlement along the centerline of the trench occurred where the patch was initially placed at a higher elevation than the surrounding patch. A line was drawn between the two survey points located outside the perimeter of the trench. This line shows the elevation of the patch if it was initially installed level with the road. The west side of the patch was placed below the level patch line. The patch experienced the majority of the total settlement between the December 17, 2004, survey and the May 10, 2005, survey, approximately within six months after construction of the utility trench. Settlement of the surrounding pavement also occurred.

Settlements as a function of time are shown on Figure 3.5. This shows that survey points within the trench (15, 16, 17, 19, 20, and 21) have continued to settle with time. The rate of settlement, which is the slope of the line between two survey points, was greater in the first six months after construction (0.09 to 0.13 inches per month) than the rate of settlement (-0.005 [uplift] to 0.05 inches per month) 6 to 18 months after construction. Survey points 18 and 19 experienced uplift from 6 to 18 months.

For the survey points outside of the trench (29, 30, 34, and 35), the settlement in the first six months after construction was similar to settlements measured within the trench. The settlement rate for the first six months ranged from 0.10 to 0.16 inches per month. Six to 18 months after construction, the survey points outside the trench uplifted. The rate of uplift was 0.03 to 0.06 inches per month. Because of this uplift, the area outside the trench was at similar elevations to when the trench was first constructed. The uplift at the survey points outside the trench resulted in larger differential movements between the areas outside the trench and inside the trench.

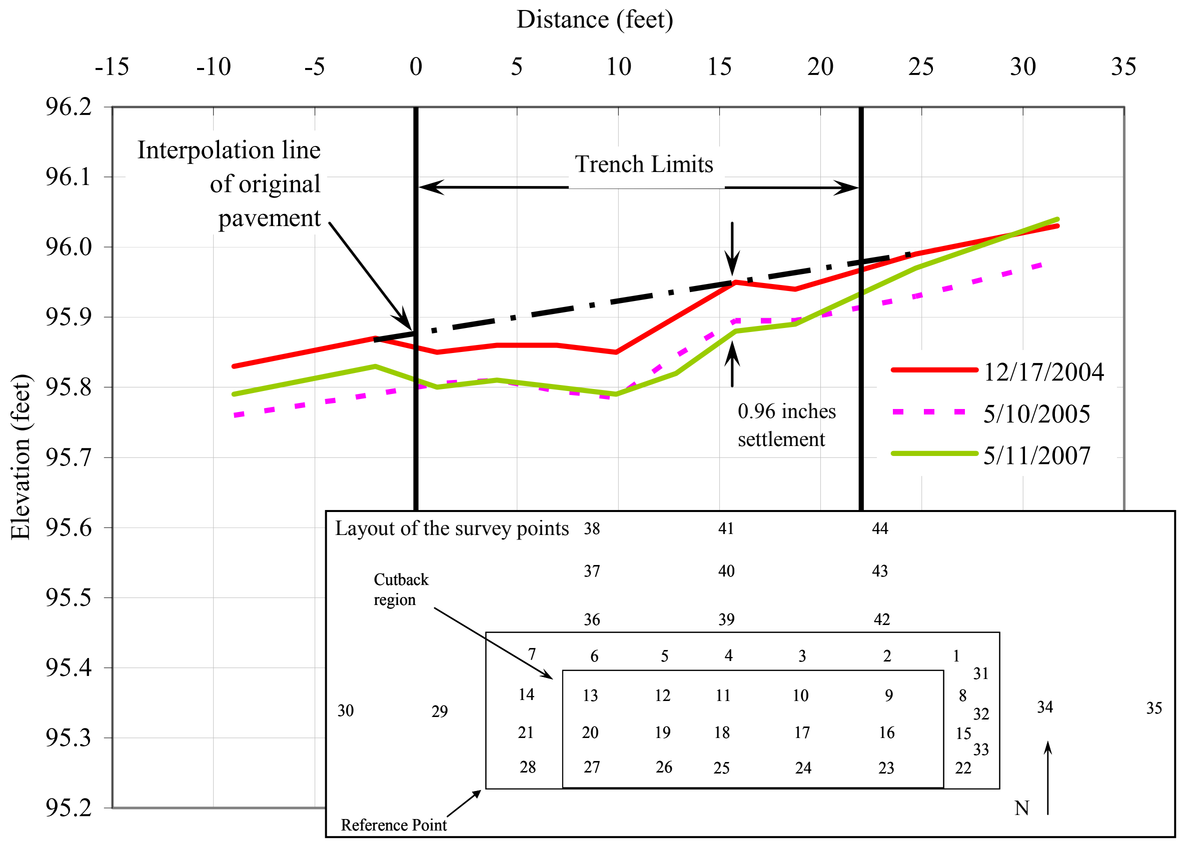


Figure 3.4. Profile of the Ames trench along the centerline of the trench

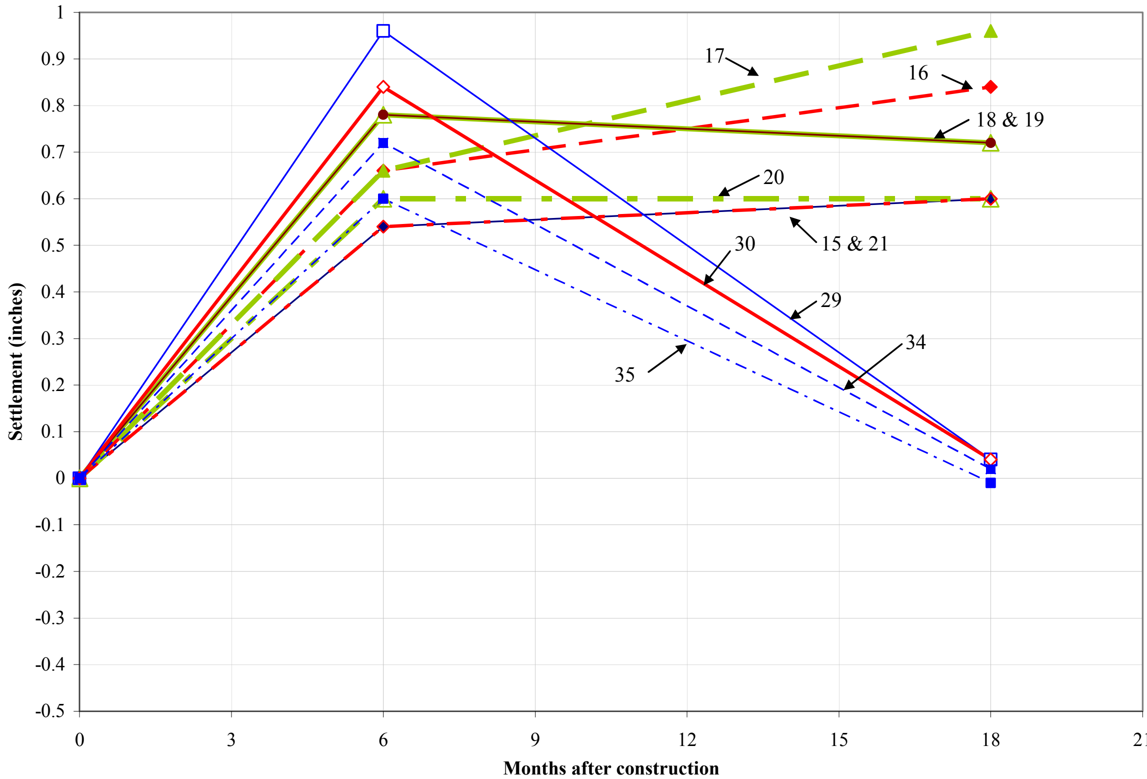


Figure 3.5. Settlement as a function of time for survey points along the centerline of the trench

Falling weight deflectometer testing was performed on November 22, 2004; November 05, 2007; June 25, 2008; and March 25, 2009. Figure 3.6 shows the locations of the FWD tests. The results of FWD tests at all the dates mentioned above are shown in Figures 3.7–3.10.

The results of FWD tests from November 5, 2007, and March 25, 2009, were similar in shape for the 15 kip load as shown in Figure 3.11. The results from FWD tests in November 2004 and 2007 were also similar in response. This was the result of high-moisture content in the subgrade. In the spring, the subgrade had a higher moisture content, causing the deflections to increase because of softer subgrade. Then in the fall, after the warm summer weather, the moisture under the patch decreased, and the stiffness of the subgrade increased.

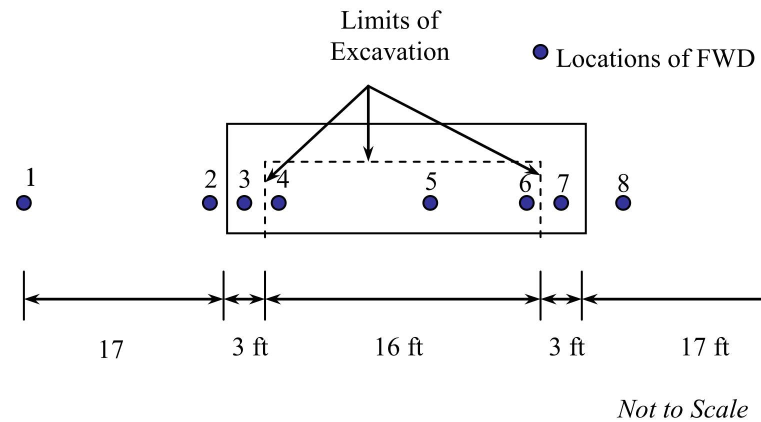


Figure 3.6. FWD testing locations

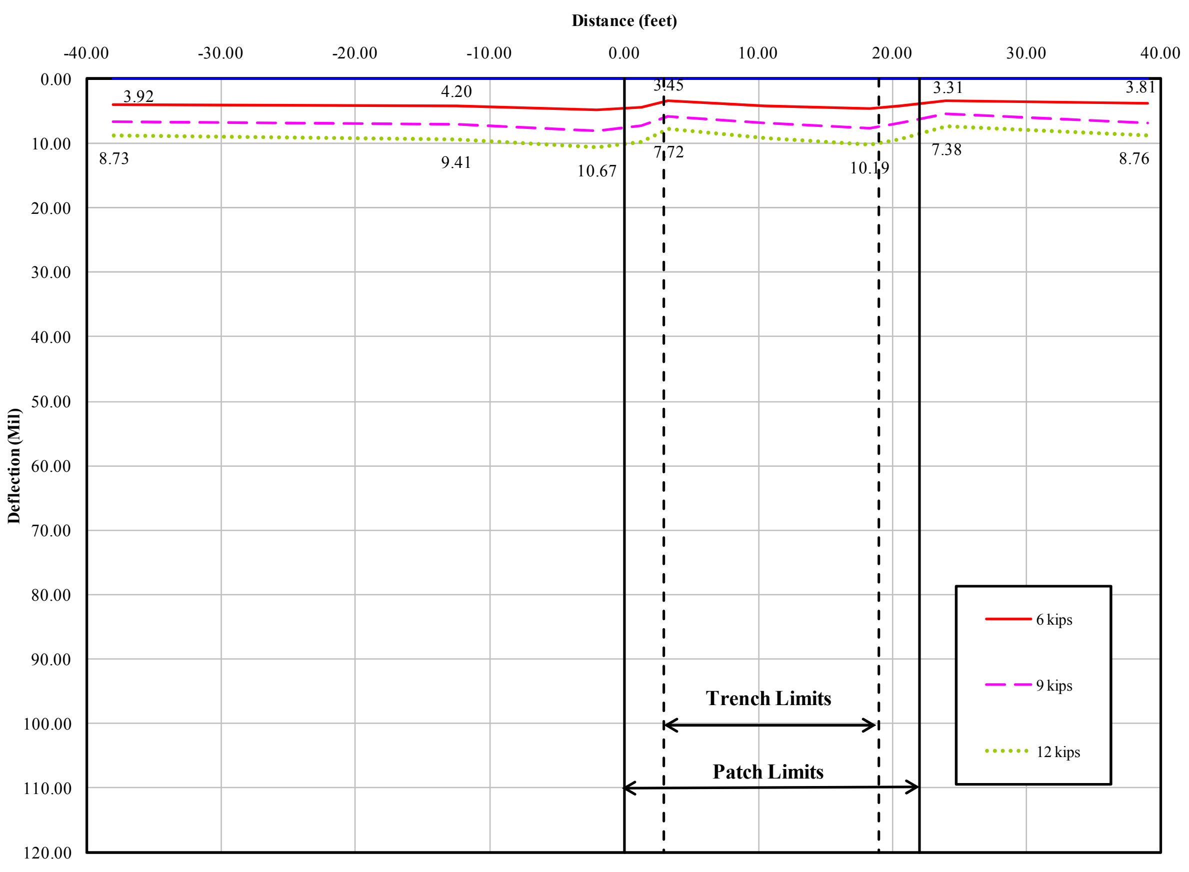


Figure 3.7. FWD testing results for the trench in Ames conducted on November 22, 2004

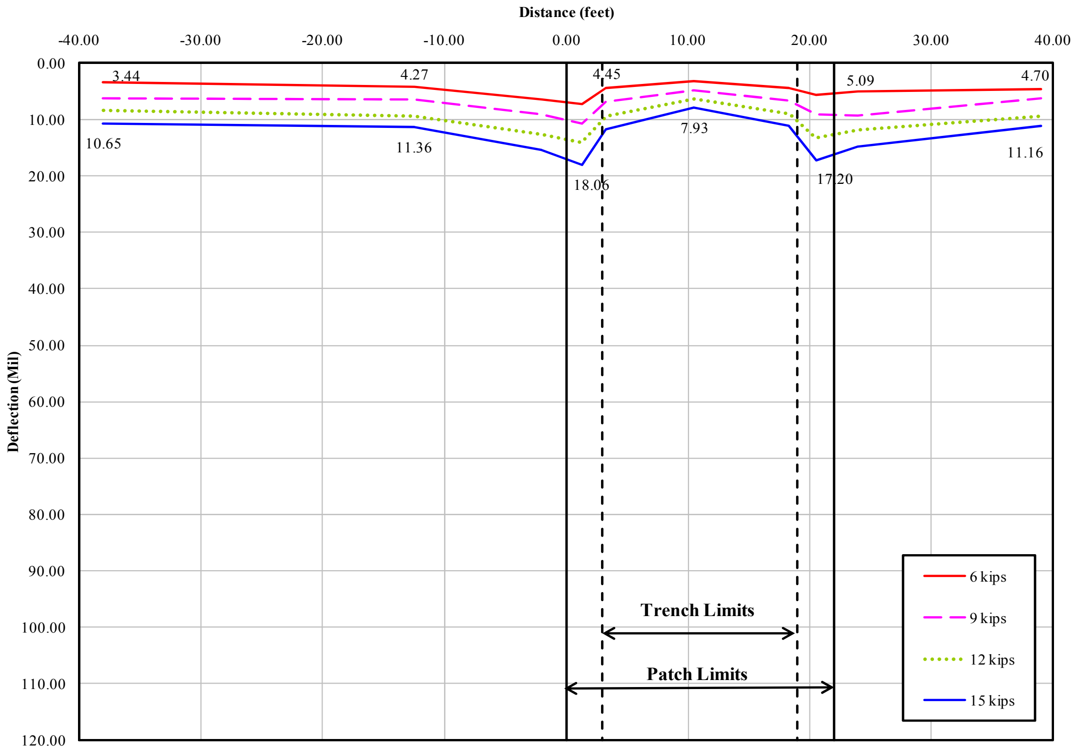


Figure 3.8. FWD testing results for the trench in Ames conducted on November 5, 2007

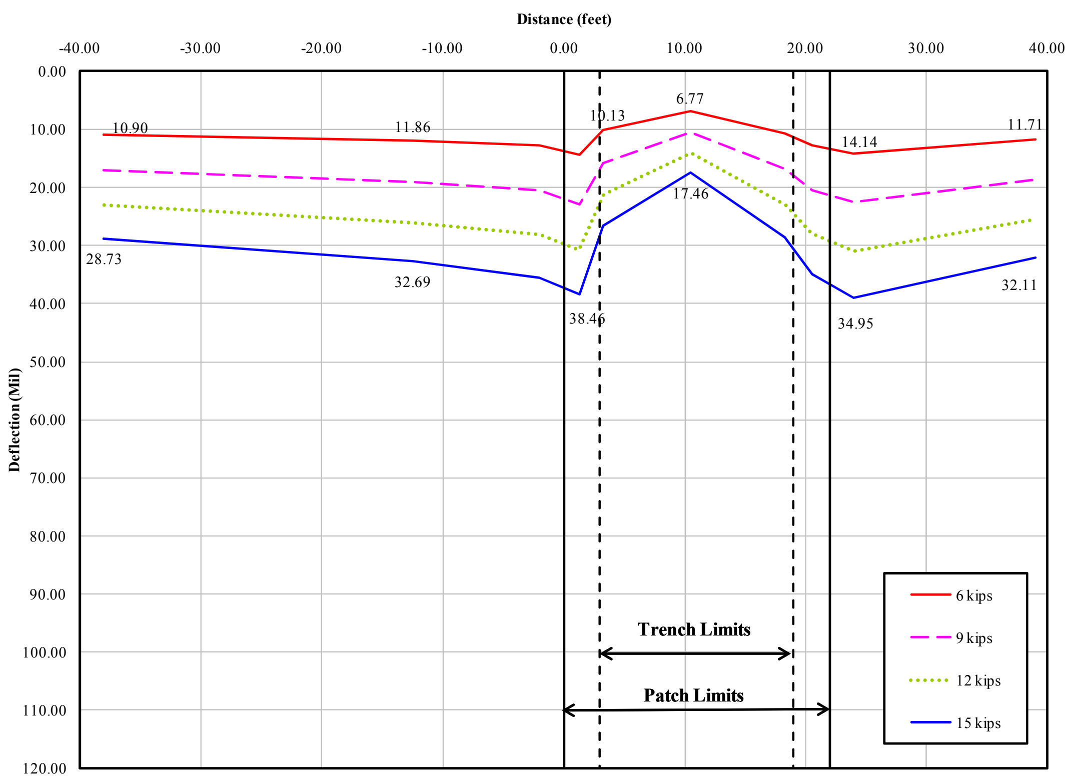


Figure 3.9. FWD testing results for the trench in Ames conducted on June 25, 2008

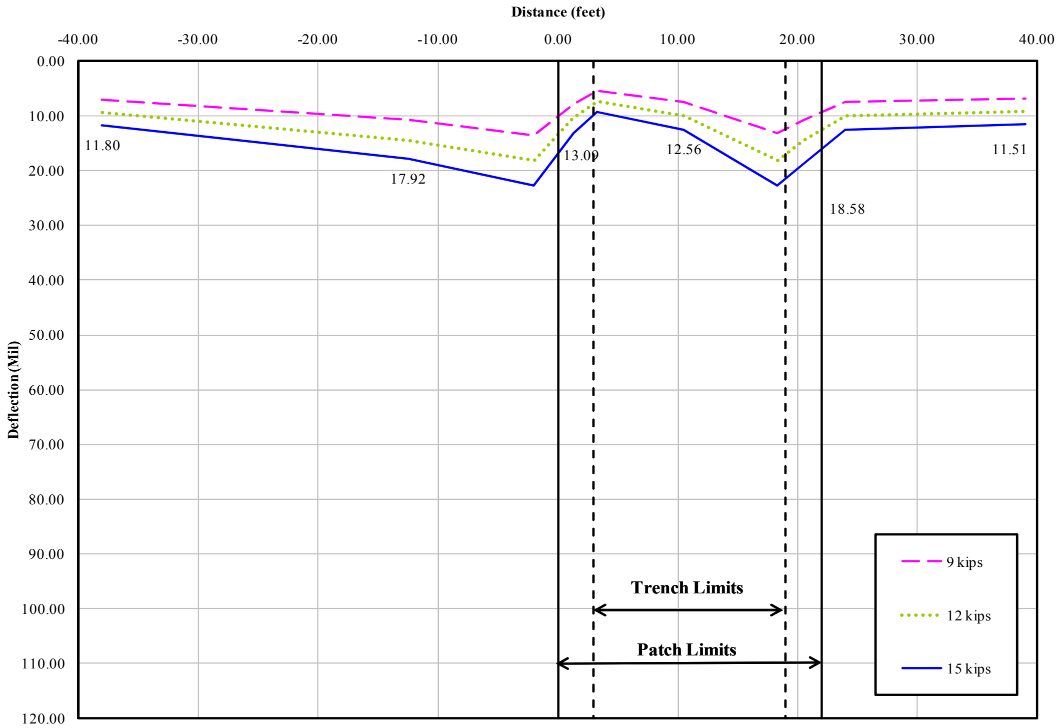


Figure 3.10. FWD testing results for the trench in Ames conducted on March 25, 2009

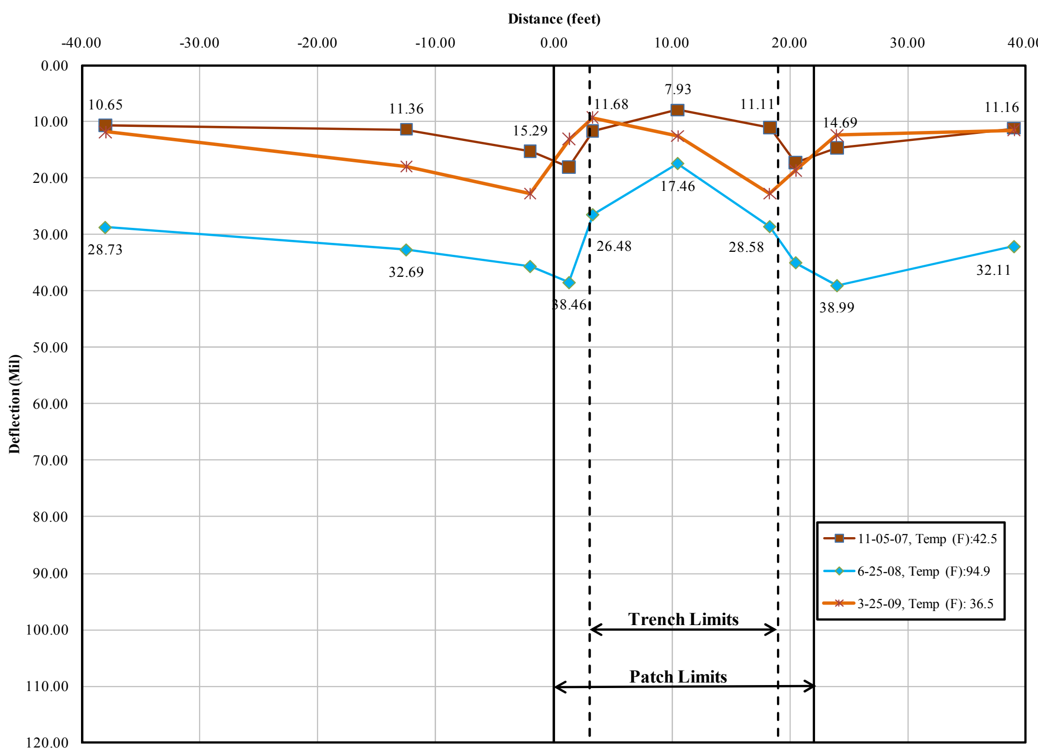


Figure 3.11. Comparison of deflections from the 15 kip load for the trench in Ames conducted on test dates

Cedar Rapids, Miami Drive and Sherman Avenue

On July 14, 2004, the excavation, construction, and restoration of a utility trench took place in Cedar Rapids at the corner of Miami Drive and Sherman Avenue to replace a leaking water main valve. The utility restoration site carried bus traffic from the bus depot. The completed trench was 8 feet wide, 12 feet long, and 10 feet deep. The work was completed by the City of Cedar Rapids Water and Street Department. Important construction elements were the following:

- The existing pavement was a composite of 6 inches of concrete with 2 inches of asphalt overlay. The pavement was removed to the perimeter of the utility trench.
- The pipe bedding material consisted of 1-inch clean stone. Thickness of the bedding material was not documented in Phase I.
- Above the bedding material, the remaining lifts were constructed using recycled crushed concrete. The particle size was ¾ inch or less.
- The remaining lifts were 3 to 4 feet thick. A backhoe-mounted vibratory plate compactor was used to compact the backfill.
- The pavement area was patched, but it was not enlarged beyond the trench perimeter.
- The pavement outside of the patch area was cracked during construction because a loaded truck came too close to the edge of the open trench.

Results from Field and Laboratory Testing

Phase I laboratory testing classified the ¾-inch minus crushed concrete backfill as an SC (clayey sand, poorly graded sand, and clay mix). Laboratory testing determined the backfill had a maximum dry unit weight of 130.0 pcf from the relative density test. The bulking moisture content range was estimated to be from 4% to 8%. The backfill in the field was compacted to an average dry unit weight of 122.9 pcf and an average moisture content of 5.7%. Figure 3.12 shows the results from the laboratory tests and the average field-testing results. For the nine test points, the backfill was compacted to a dense-to-very-dense state (72% to 95% relative density). The complete field-testing results for this trench can be found on Table 14 in the Phase I report.

The CBR values obtained from the Clegg Hammer test ranged from 8.2% to 12.9%, with an average CBR value of 12.9%. The CBR values from the DCP tests ranged from 4.9% to 25.0%, with an average CBR value of 13.3%. These values were "just above to below the typical CBR values of 10% to 20%." Complete CBR data is in Table 15 of the Phase I report.

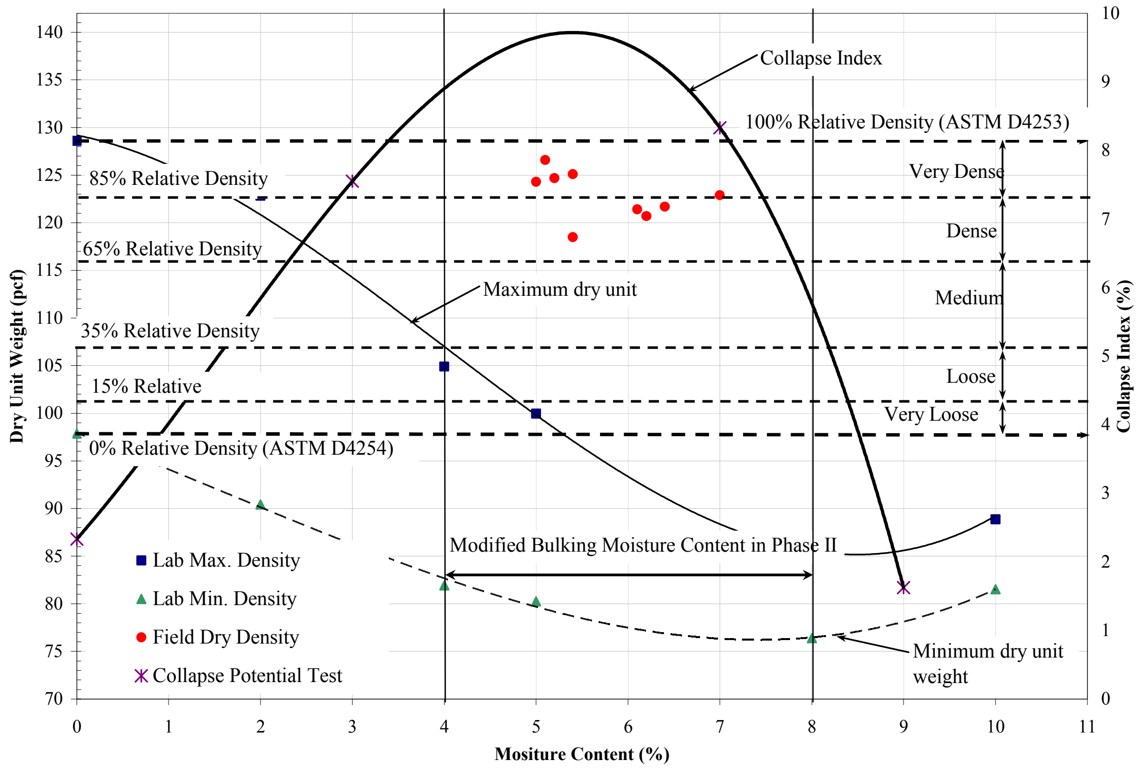


Figure 3.12. Relative density and average field-testing results for the trench backfill material for the Cedar Rapids site

Continued Monitoring

The trench was surveyed five times (July 21, 2004; October 29, 2004; April 20, 2005; May 22, 2007; and July 31, 2008). Figure 3.13 shows the points used for surveying and centerline profile. Figure 3.14(a) shows the centerline profiles for the trench on the various test dates. The maximum settlement along the centerline of the trench was 0.48 inches at survey points 4, 6, and 10, as well as survey point 14 outside the trench. This was also the maximum total settlement for the trench. Settlement of the surrounding pavement was observed up to a distance of 3 feet from the trench.

Settlement as a function of time is shown in Figure 3.14(b) for survey points along the center of the trench. The figure shows that the survey points within the trench (4, 6, 8, 10, and 12) have continued to settle based on the positive slope of the line between the survey points, except for survey point 12. The rate of settlement, which is the slope of the line, ranged from 0 to 0.04 inches per month for the first three months after construction. The rate of settlement increased for all the survey points within the trench during the time period from three to six months after construction. The rate of settlements ranged from 0.02 to 0.1 inches per month. The rates of settlement for 9 to 19 months after construction ranged from -0.004 to 0.02 inches per month. Survey point 12 uplifted during this time interval. The survey points outside of the trench (1, 2, 14, and 15) settled to differing degrees. Survey point 15 has shown no movement since monitoring of the utility cut began, while survey point 14 (closer to the trench) has had the same settlements as survey point 10, which is within the trench. Survey points 1 and 2 have shown the opposite behavior. Survey point 1, which is furthest from the trench, has continued to settle, while survey point 2 has uplifted. The rate of settlement for the survey points outside the trench for the first three months ranged from 0 to 0.04 inches per month. From three to nine months after construction, the rate of settlement ranged from 0 to 0.10 inches per month. From 9 to 19 months after construction, the rate of settlement for the test points outside the trench ranged from -0.03 to 0.001 inches per month. The upper limit of these settlement rates was controlled by survey point 14.

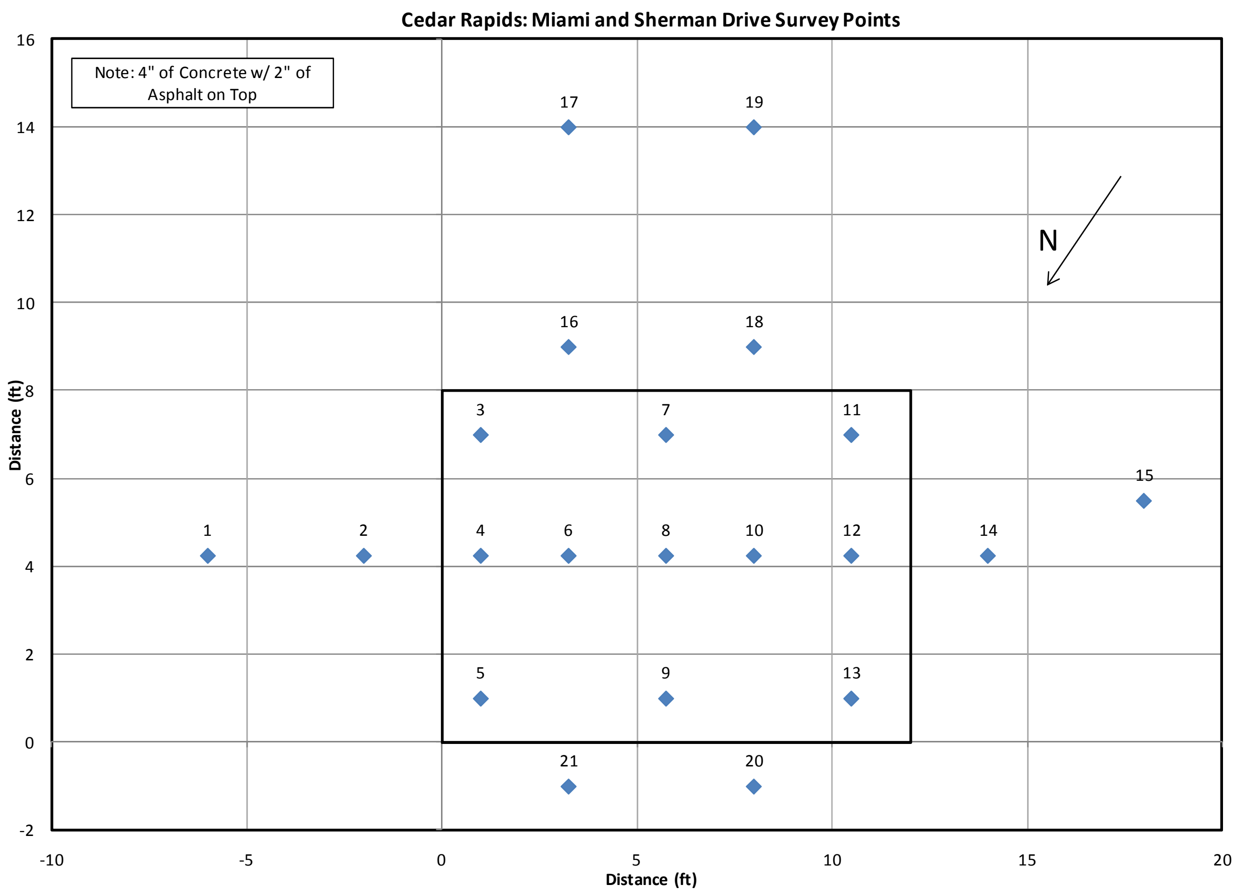


Figure 3.13. Survey points for Cedar Rapids trench; displacements used for comparison are along the center line (i.e., points 2, 4, 6, 8, 10, 12, 14)

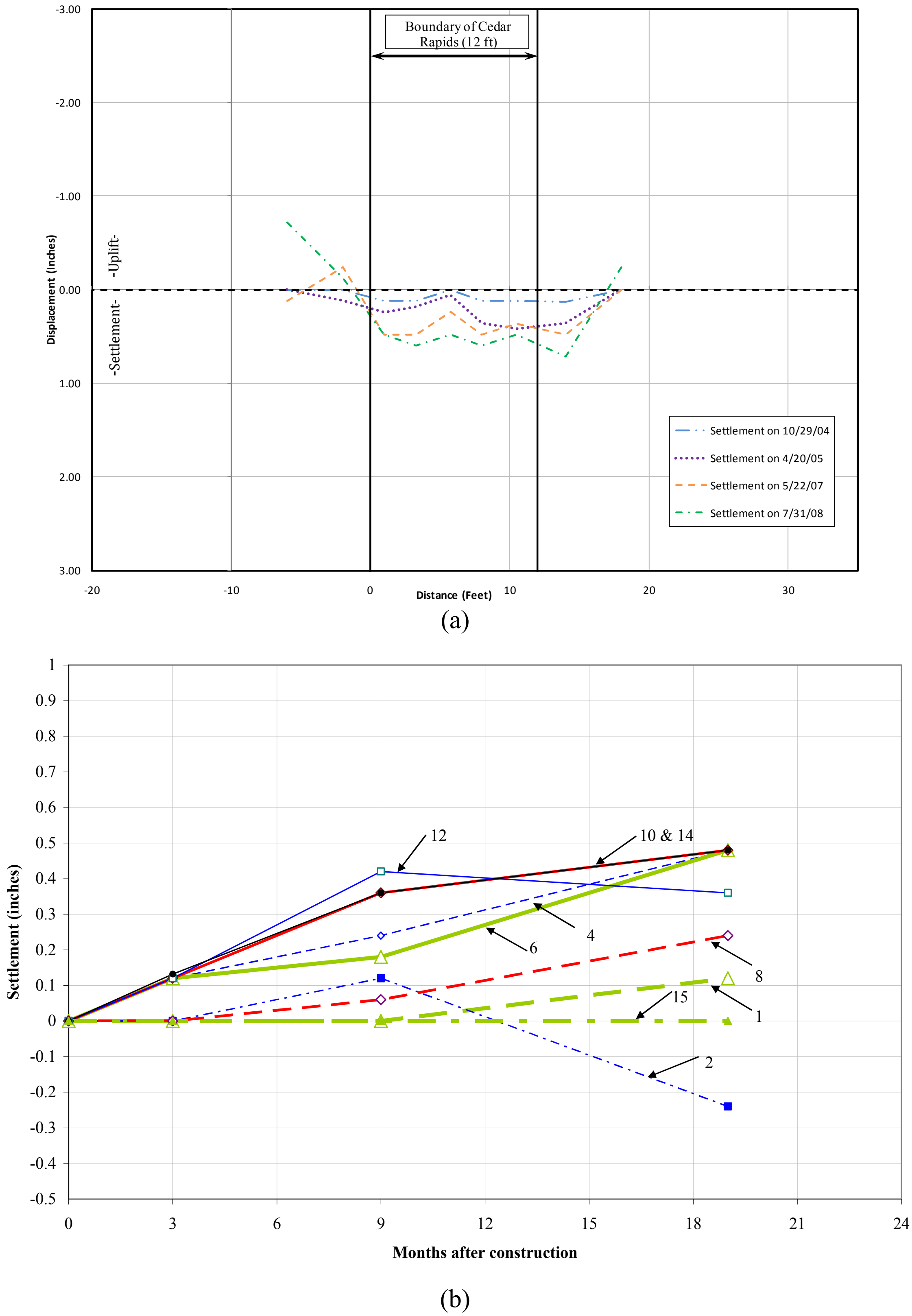


Figure 3.14. Settlement of the trench and surrounding area (a) trench centerline, and (b) settlement as a function of time.

On October 25, 2004; April 20, 2005; June 13, 2007; June 26, 2008; and March 26, 2009, the Iowa DOT performed FWD testing on the Cedar Rapids trench. The locations of these FWD tests are shown in Figure 3.15. Figures 3.16 through 3.20 show the results from the FWD testing. Four loads (6, 9, 12, and 15 kips) were used each time. The FWD testing from the spring/early summer was similar in shape to the 15-kip load. This can be seen in Figure 3.16. In the spring, the subgrade increased in moisture content. The increased moisture content caused the deflections to increase due to softer subgrade. Then in the fall after the warm summer weather, the moisture under the patch decreased and the subgrade stiffened. This trend was more evident in this trench than in the other trenches.

Figure 3.21 shows three dates at 15-kip load. This graph shows a weakened zone at the edge of the trench. This weakened zone was present in earlier testing and was the result of the construction equipment being near the edge of the trench.

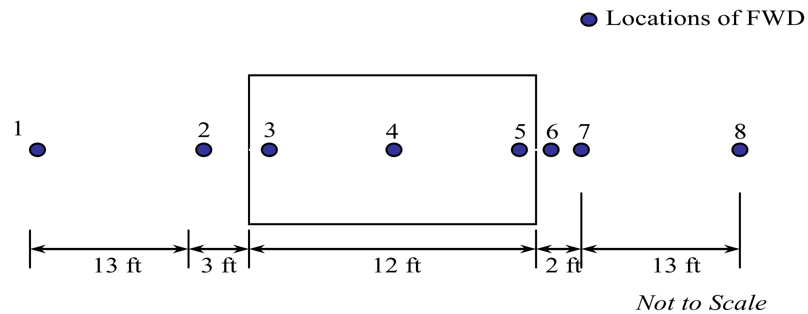


Figure 3.15. Locations of FWD testing points for Cedar Rapids trench

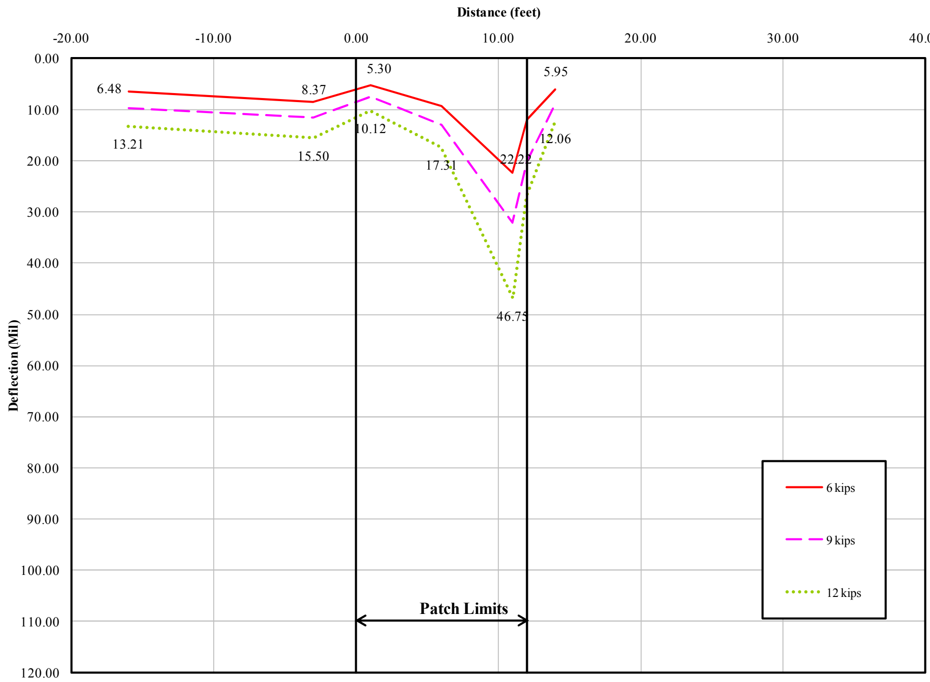


Figure 3.16. Falling weight deflectometer testing results for the trench in Cedar Rapids for October 25, 2004

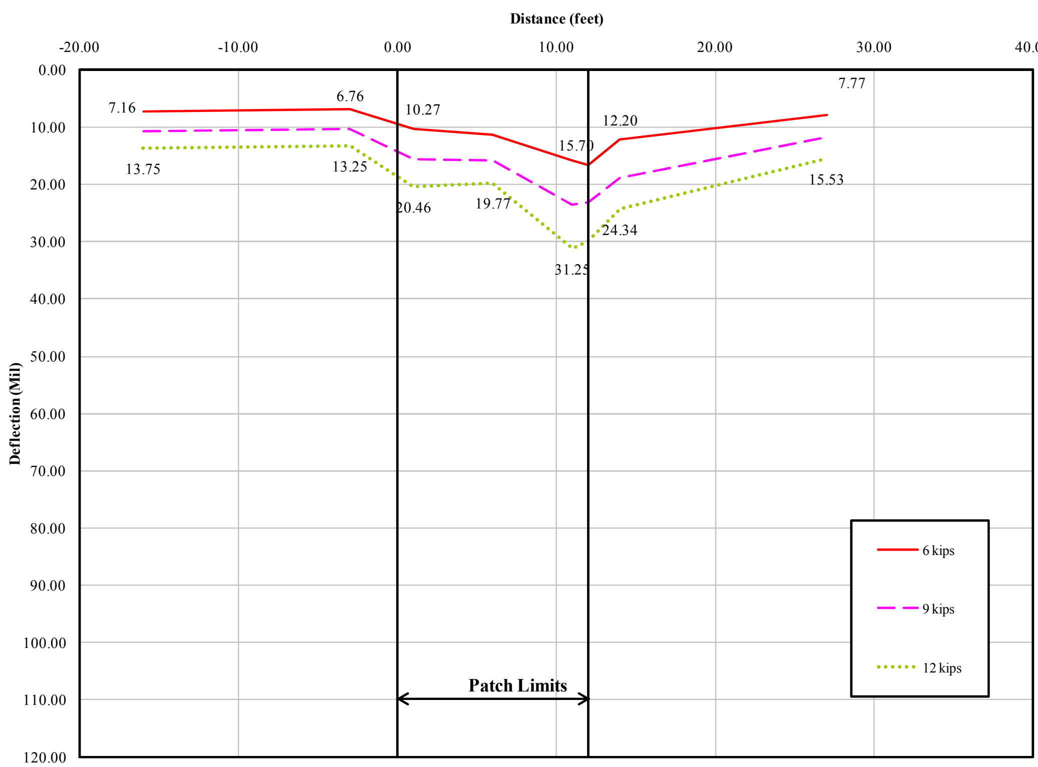


Figure 3.17. Falling weight deflectometer testing results for the trench in Cedar Rapids for April 20, 2005

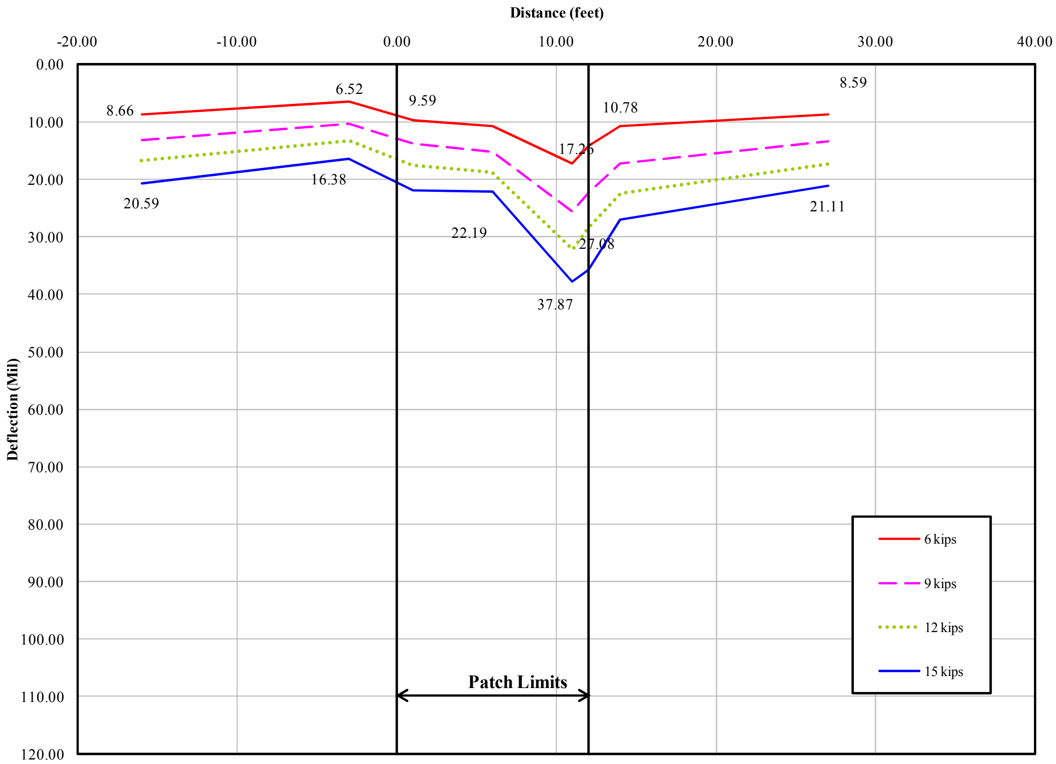


Figure 3.18. Falling weight deflectometer testing results for the trench in Cedar Rapids for June 13, 2007

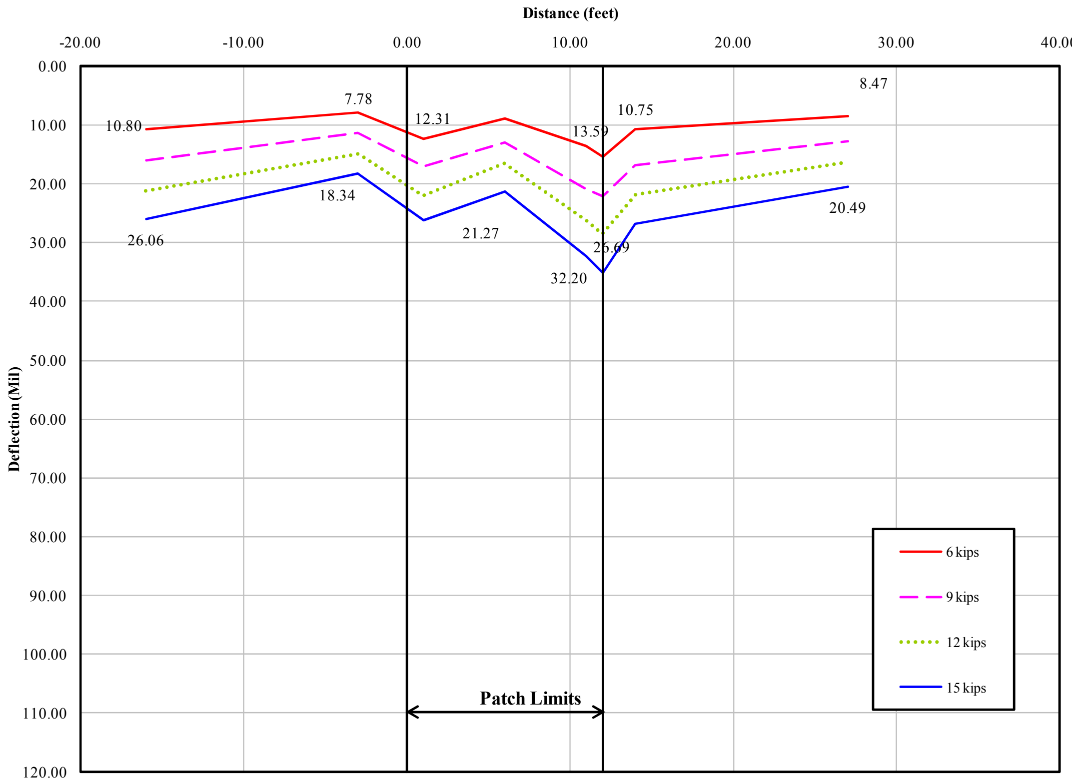


Figure 3.19. Falling weight deflectometer testing results for the trench in Cedar Rapids for June 26, 2008

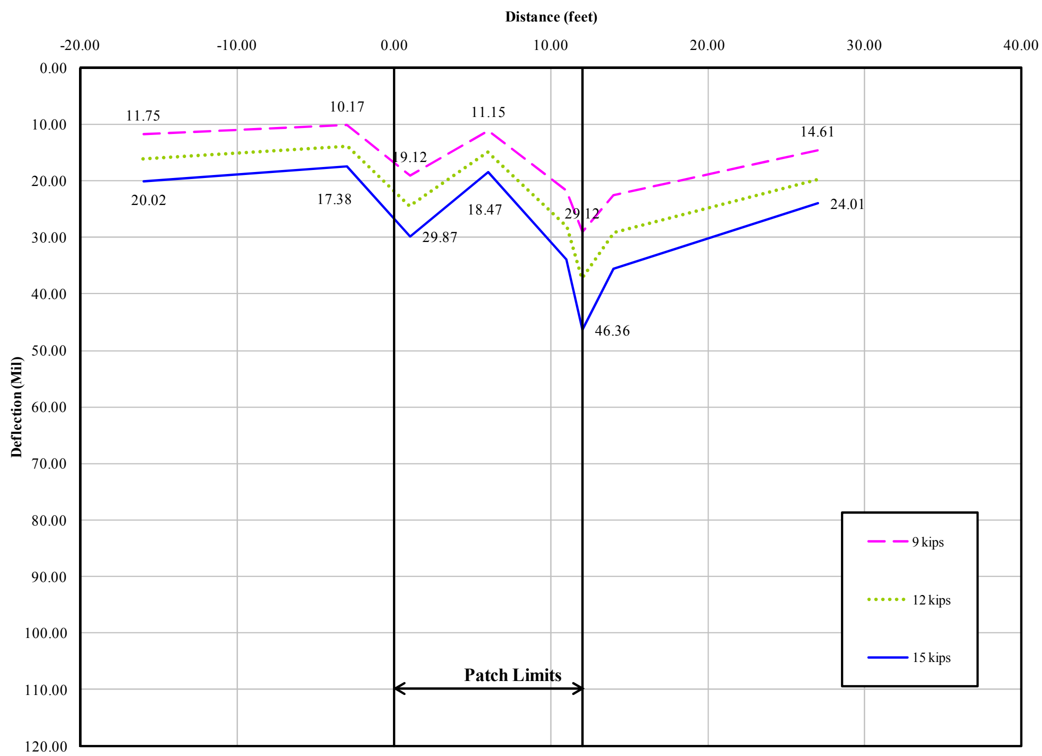


Figure 3.20. Falling weight deflectometer testing results for the trench in Cedar Rapids for March 26, 2009

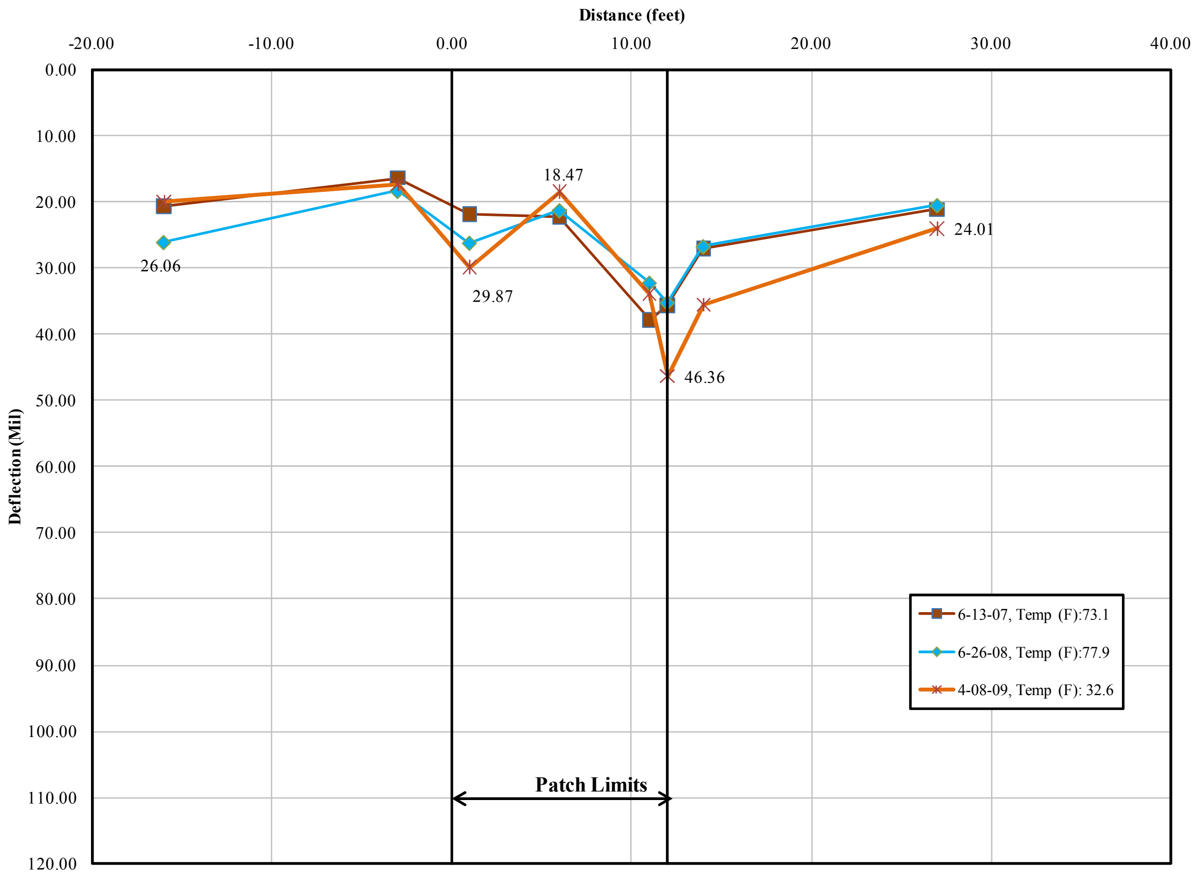


Figure 3.21. Comparison of deflections from the 15-kip load for the trench in Cedar Rapids conducted on three test dates

Des Moines, East 28th Street and Grand Avenue

On June 30, 2004, a sewer main break was repaired in the city of Des Moines at East 28th Street and Grand Avenue. A private contractor completed the work. The Iowa State research team did not observe the repair of the utility but was present to test the backfill and the placement of the patch. Important construction features on the utility cut were the following:

- The backfill material was sand-size particles referred to as manufactured sand.
- The existing pavement was 8-inch thick concrete. The concrete was removed from around the trench. The size of the patch was not documented in the Phase I report.
- To tie the patch into the existing concrete road surface, dowel bars were used as a mechanical connection in the longitudinal and transverse directions. After the concrete was allowed to cure, a joint was cut in the patch to match the surrounding joint spacing.

Results from Field and Laboratory Testing

The backfill of manufactured sand was classified in Phase I as SW-SM (well graded sand with silt). Laboratory testing found the backfill to have a maximum dry unit weight of 135.0 pcf from the relative density test. The bulking moisture content ranged from 5% to 8%. The nuclear density tests yielded an average moisture content of 7.6%, and the average dry unit weight was 105.9 pcf for the 16 test points. The backfill was placed at an average 29.3% relative density, which corresponded to a medium-dense state. The moisture content of the backfill was at the same bulking moisture content found in the laboratory testing. The complete field-testing results for this trench can be found in Table 14 in the Phase I report. Figure 3.22 shows the results from the laboratory tests and the average field-testing results.

The CBR values from the Clegg Hammer ranged from 4.6% to 15.1%, with an average CBR value of 8.6%. The CBR values from the DCP test ranged from 2.7% to 34.9%, with an average CBR value of 12.5%. Complete CBR data is in Table 15 of the Phase I report.

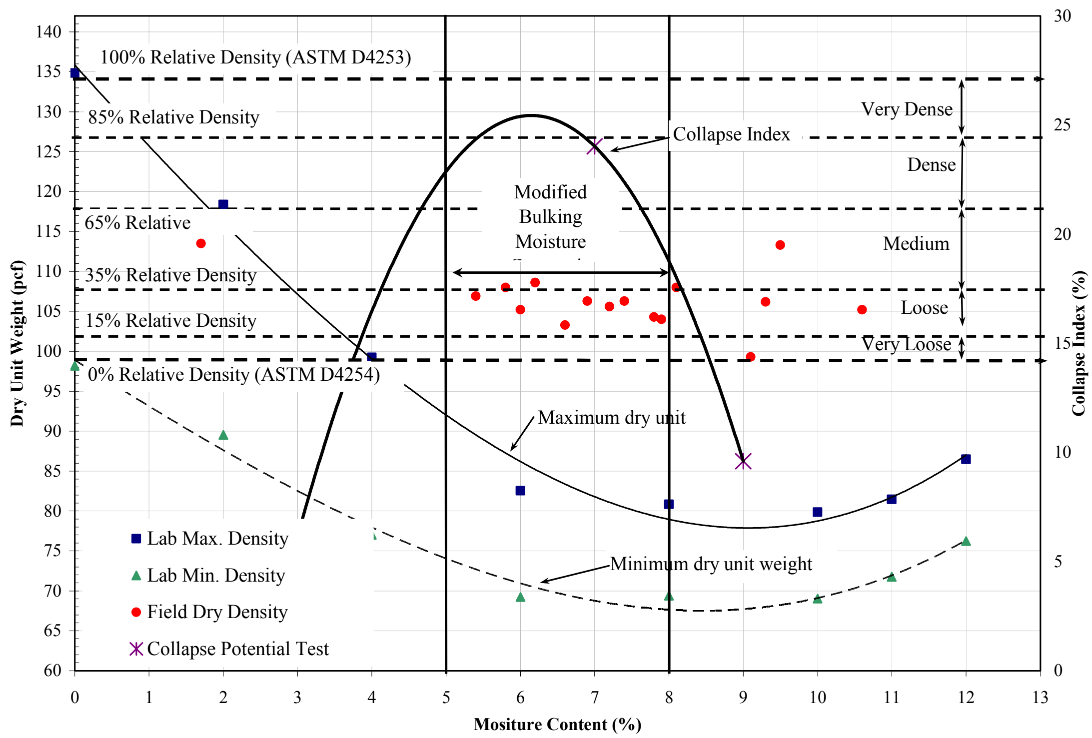


Figure 3.22. Relative density and average field-testing results for the trench backfill material for the Des Moines site

Continued Monitoring

The trench was surveyed six times (July 17, 2004; October 29, 2004; April 16, 2005; May 14, 2007; July 31, 2008; and March 26, 2009). Figure 3.23 shows the survey points, while Figure 3.24 (a) shows the displacement according to date. The maximum settlement along the centerline of the trench was 0.2 inches at survey points 2, 4, and 10. The maximum settlement of the trench, which was at the edge of the trench, was 0.36 inches. When the patch was initially installed, the edge of the patch was higher than the center of the patch. As the trench settled over the last two years, the lowest point in the patch did not settle; however, the surrounding pavement in the patch settled to the same elevation.

Settlements versus time were plotted in Figure 3.24 (b). This figure shows that settlements as a function of time were less than in the other trenches. The maximum rate of settlement was 0.03 inches per month, occurring three to six months after construction for survey points 2, 4, 10, 15, and 16. The maximum rate of uplift movement was 0.01 inches per month.

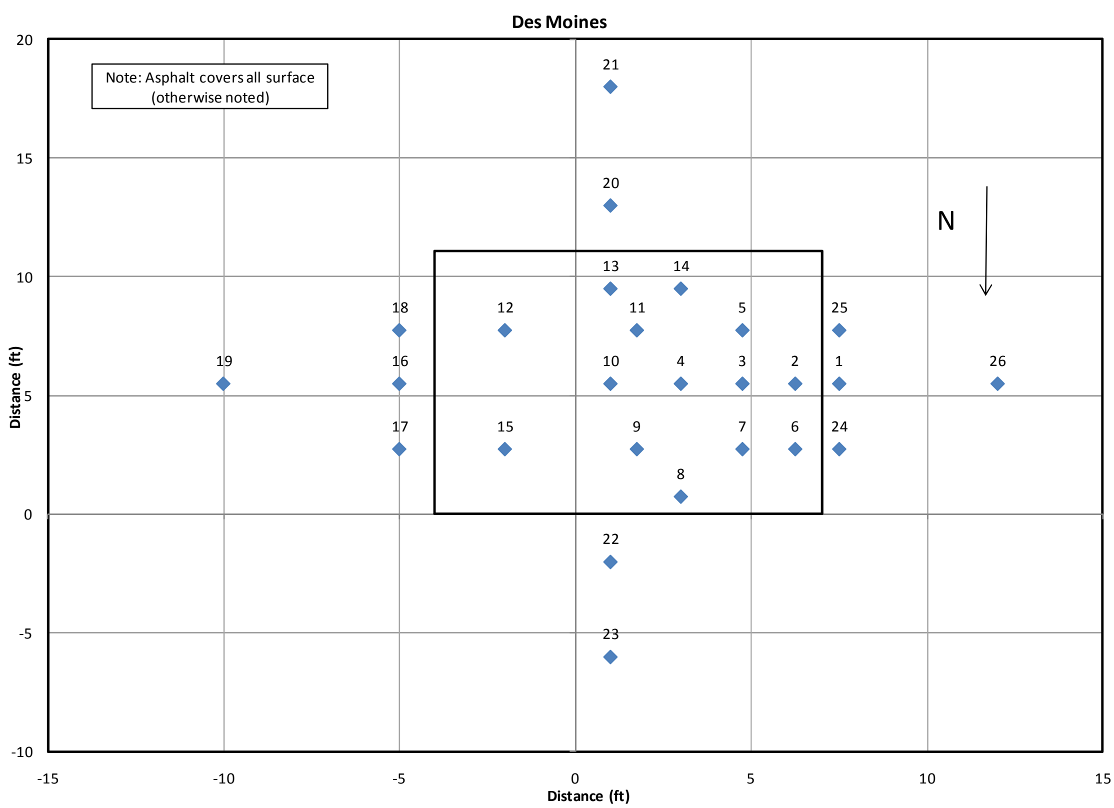
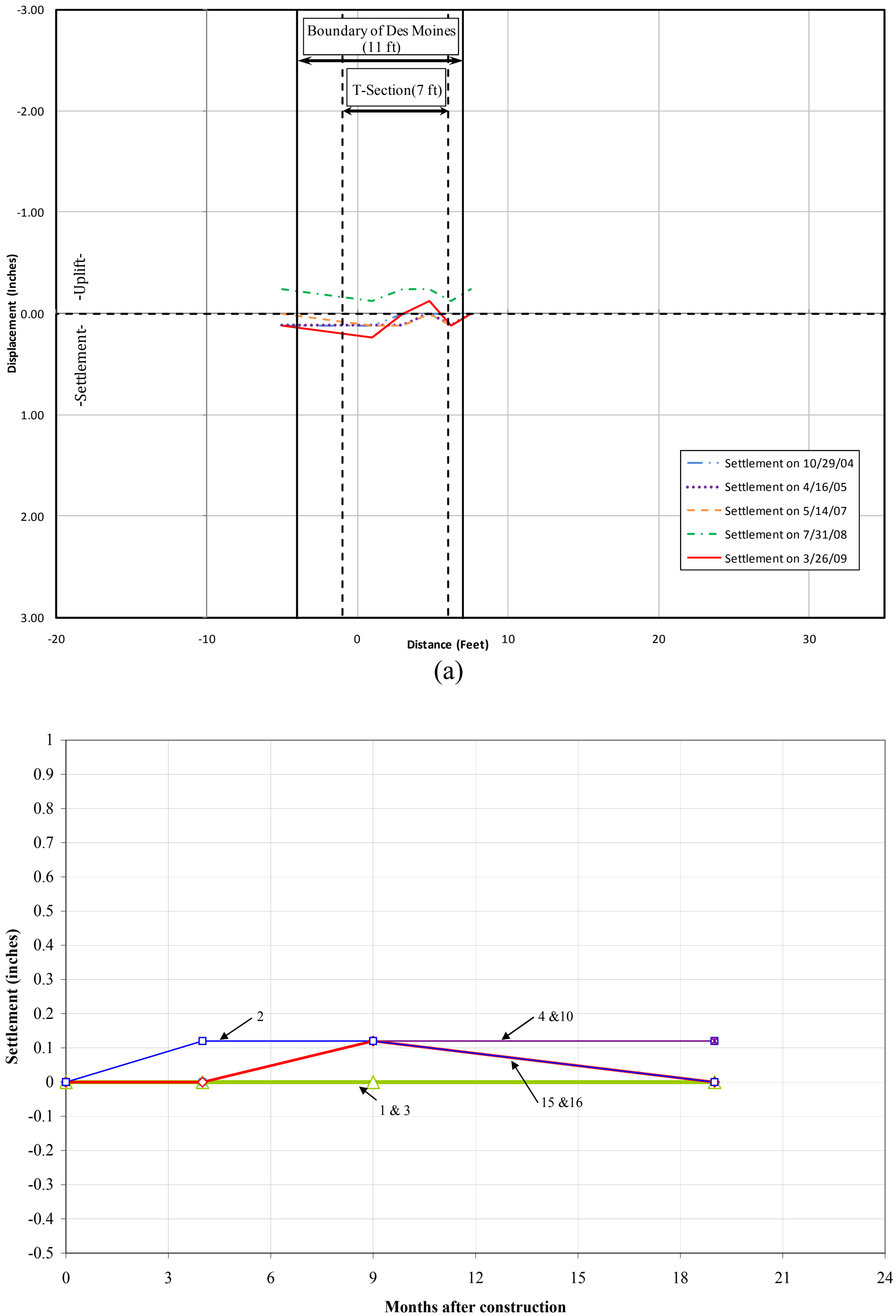


Figure 3.23. Survey points for Des Moines trench; the displacement provided below uses center line, point number 19, 16, 10, 4, 3, 2, 1, 26



(a)

(b)

Figure 3.24. (a) Displacement for Des Moines trench, (b) Settlement versus time for Des Moines trench

Falling weight deflectometer testing was performed on October 25, 2004; April 13, 2005; June 13, 2007; June 30, 2008; and March 26, 2009. Four loadings (6, 9, 12, and 15 kips) were used each time. The locations of these FWD tests are shown in Figure 3.25. Figures 3.26 through 3.30 show the results from the FWD testing. Figure 3.31 shows the deflections for the 15-kip load from three different dates.

The FWD testing on this trench does not show the same seasonal effects as the other trenches, which had higher deflections in the spring and early summer and smaller deflections in the fall FWD tests. This was confirmed with plotting the test results from the 6-kip load in Figure 3.27 and the 15-kip load in Figure 3.28. The response of the trench did not vary with seasonal effects. These results may be attributed to being a concrete patch doweled with surrounding pavement.

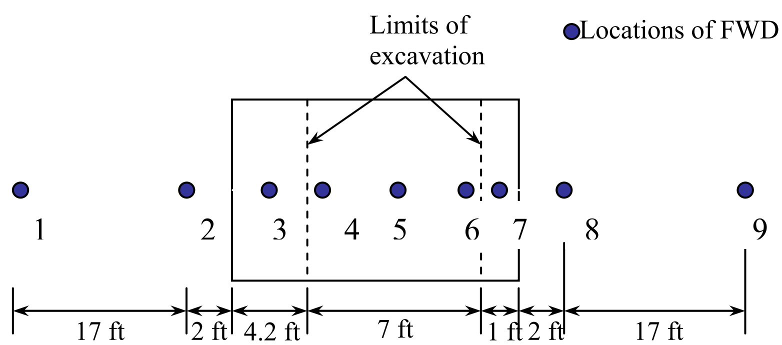


Figure 3.25. Locations of FWD testing points for Des Moines trench

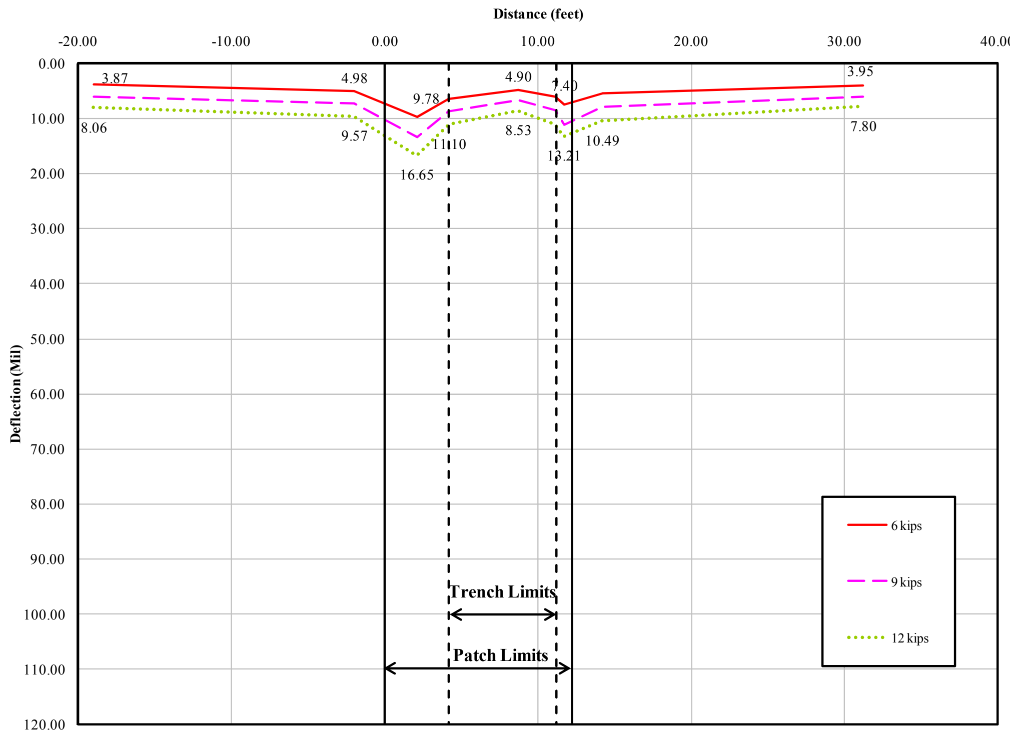


Figure 3.26. Falling weight deflectometer testing for the trench in Des Moines conducted on October 25, 2004

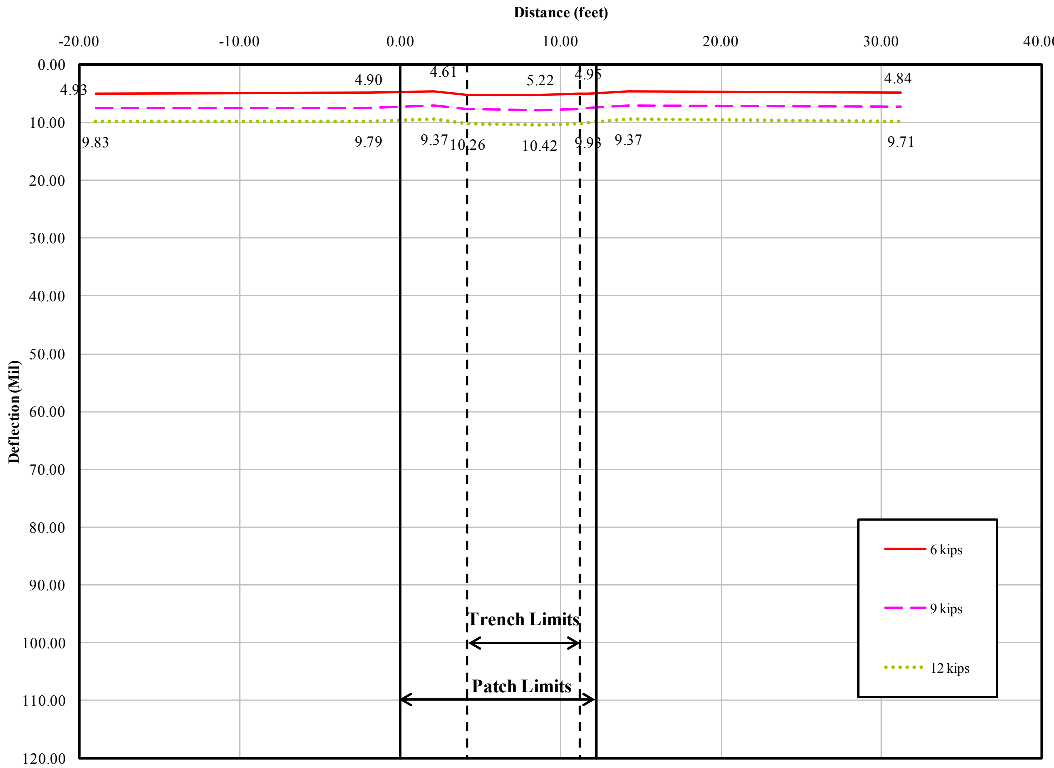


Figure 3.27. Falling weight deflectometer testing for the trench in Des Moines conducted on April 13, 2005

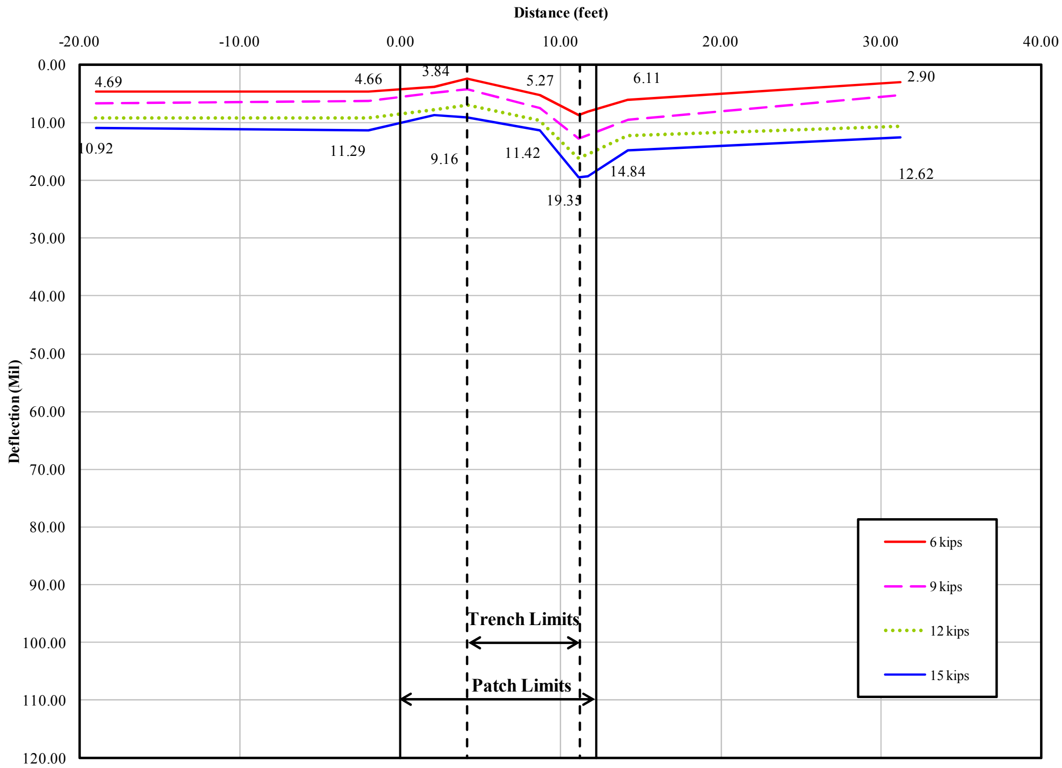


Figure 3.28. Falling weight deflectometer testing for the trench in Des Moines conducted on June 13, 2007

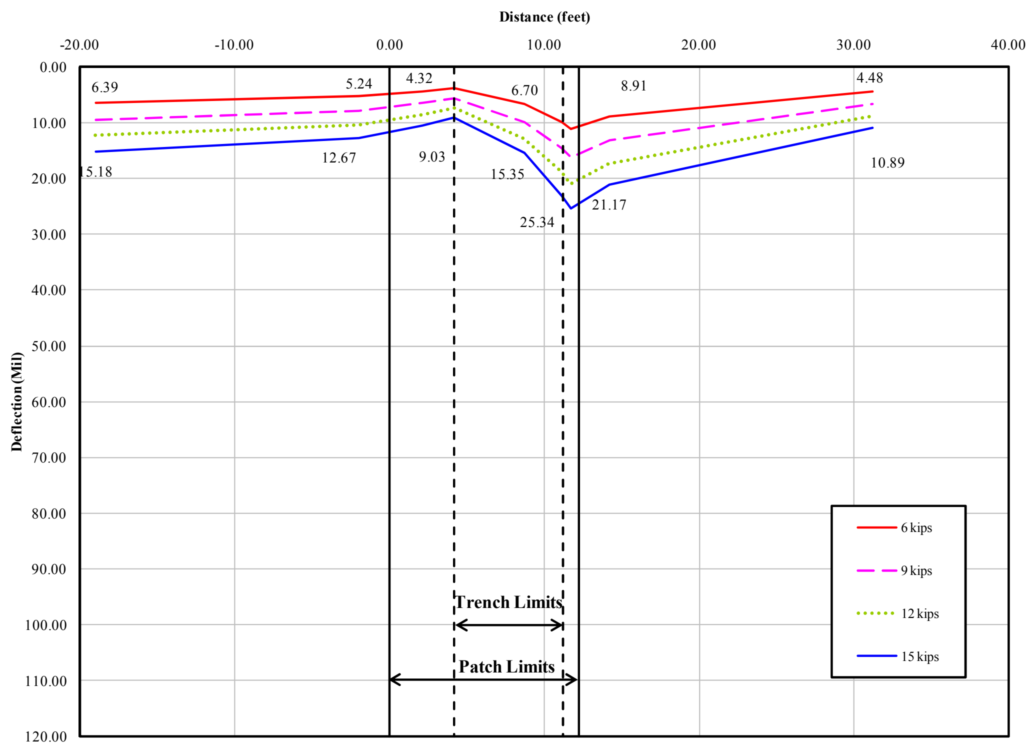


Figure 3.29. Falling weight deflectometer testing for the trench in Des Moines conducted on June 30, 2008

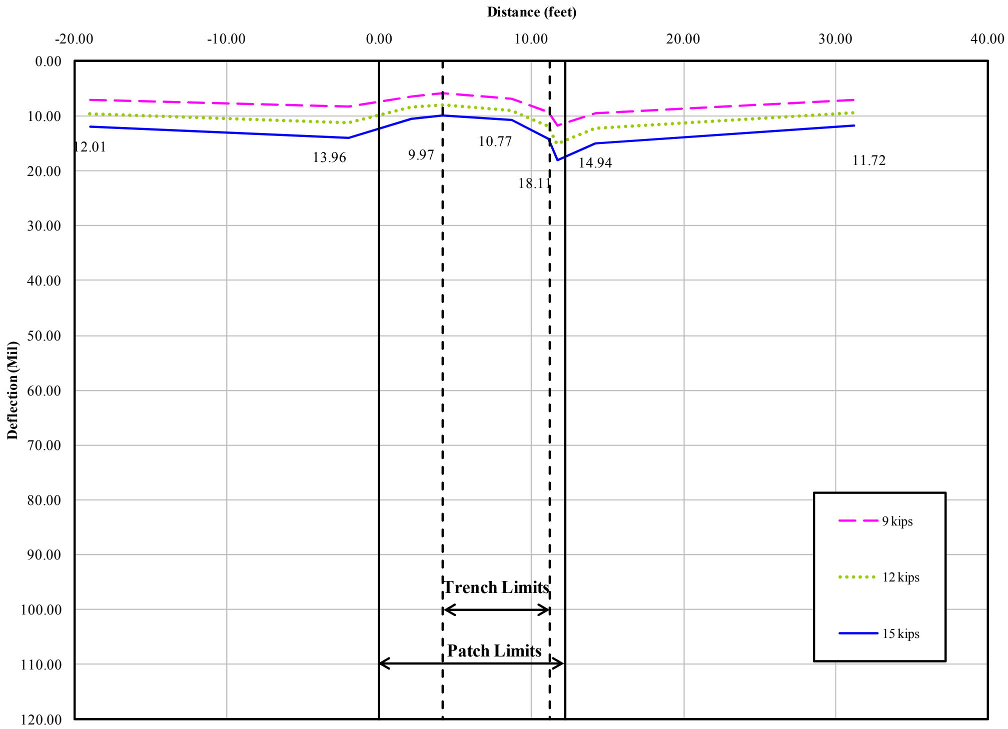


Figure 3.30. Falling weight deflectometer testing for the trench in Des Moines conducted on March 26, 2009

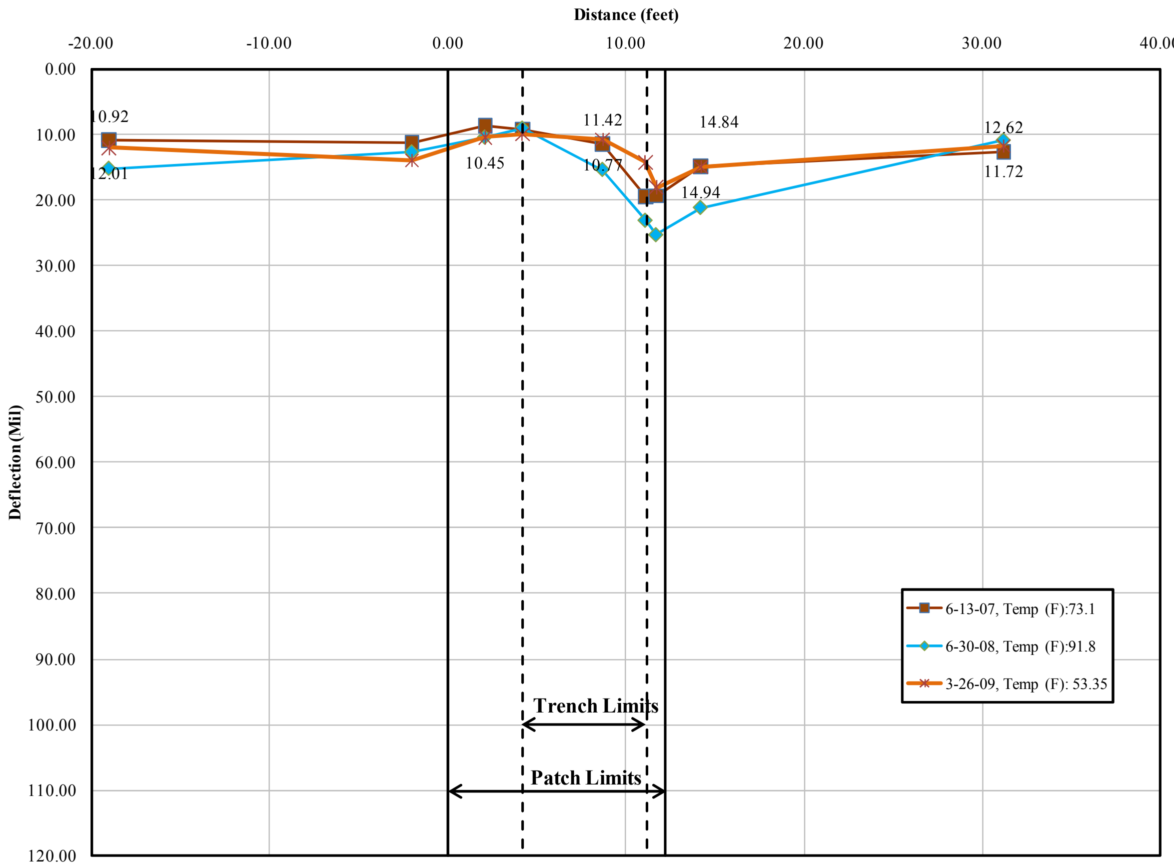


Figure 3.31. Comparison of deflections from the 15-kip load for the trench in Des Moines conducted on three test dates

Comparison of Trenches

Table 3.1 compares the settlement of the three trenches with the field-testing results. This shows that in all the trenches, the backfill was placed within its bulking moisture content. This will result in all the trenches being susceptible to collapse. When comparing the Ames and Cedar Rapids trenches, which were constructed with an asphalt patch, the patch in Ames has settled more over time than the patch in Cedar Rapids. The backfill in the trench in Ames was placed at a lower relative density than the backfill in the Cedar Rapids trench. This accounts for the difference in the settlement between the two trenches. The Des Moines patch has had the smallest settlements; however, this trench was constructed using dowel bars between the existing concrete and the concrete patch. The dowels allow for the patch to bridge over the utility cut where backfill settlement is expected. The settlement of the Des Moines patch is not necessarily the settlement of the backfill in the trench because of this bridging.

To compare the performance of the three trenches monitored in Phase I, the 15-kip FWD test results from summer 2008 (Figure 3.32) were plotted and the spring 2009 test results were used (see Figure 3.33). These figures show that the Des Moines trench had the lowest deflections from FWD testing in June. The deflection patterns across the trenches were similar on both sides of the trenches, showing that the loss of moisture over the summer resulted in the response of the trench becoming more uniform. This is important because when an area of the sub-base is softer than the surrounding area, it places additional stress on the surrounding pavement. This effect on load distribution is also seen with beams supporting continuous elastic foundations and pile design. The Cedar Rapids trench did not show improved performance on both sides of the trench as did the Des Moines and Ames trenches. The side of the highest deflections had equipment located on it during construction. The damage caused by the equipment to the sub-base during construction was not minimized over time.

Table 3.1. Comparison of field-testing results and settlements in the trenches

Trench	Average relative density (%)	Revised bulking moisture content (%)	Average moisture content	Average CBR values from Clegg Hammer/DCP	Average settlement after one winter (inches)	Average settlement after two winters (inches)
Ames	18	4 to 8	6.3	6.7/8.5	0.70	0.56
Cedar Rapids	85	4 to 8	5.2	12.9/13.3	0.22	0.30
Des Moines	28	5 to 9	7.6	8.6/12.5	0.08	0.05

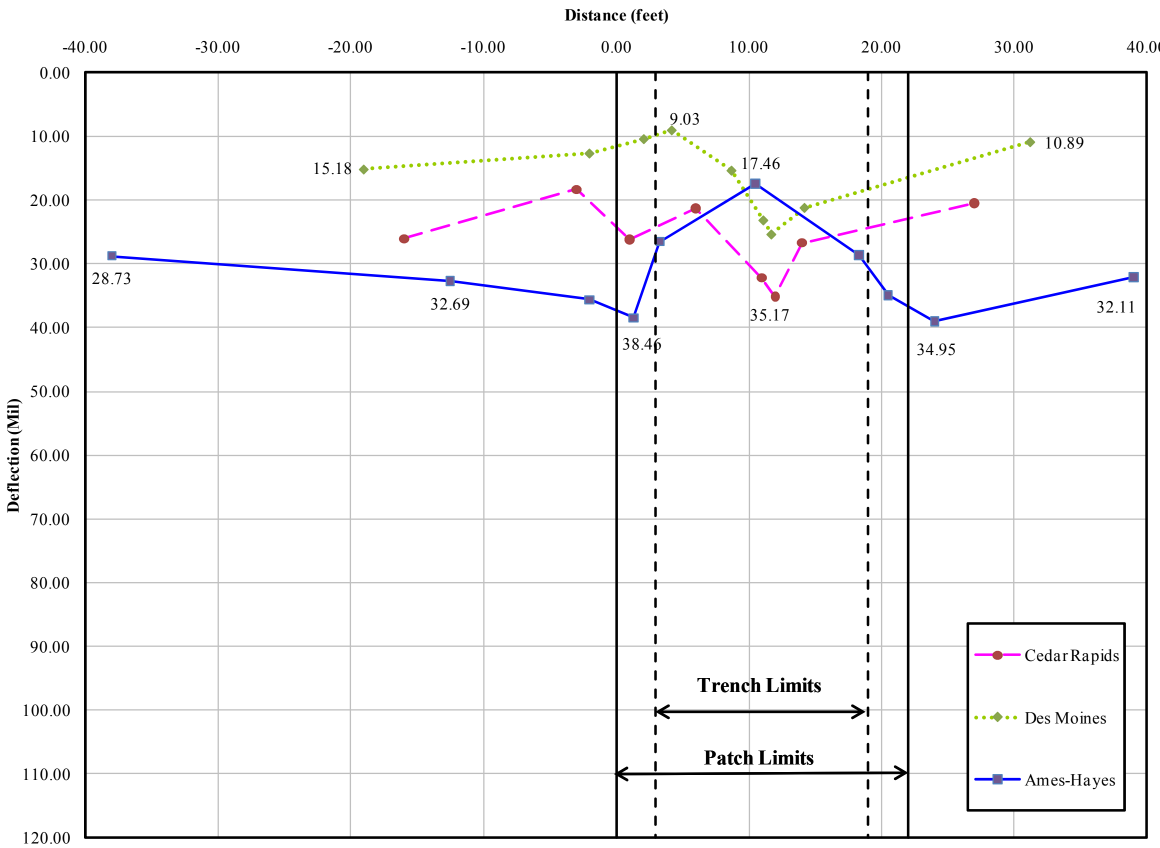


Figure 3.32. Comparison of the 15-kip FWD test results for summer 2008

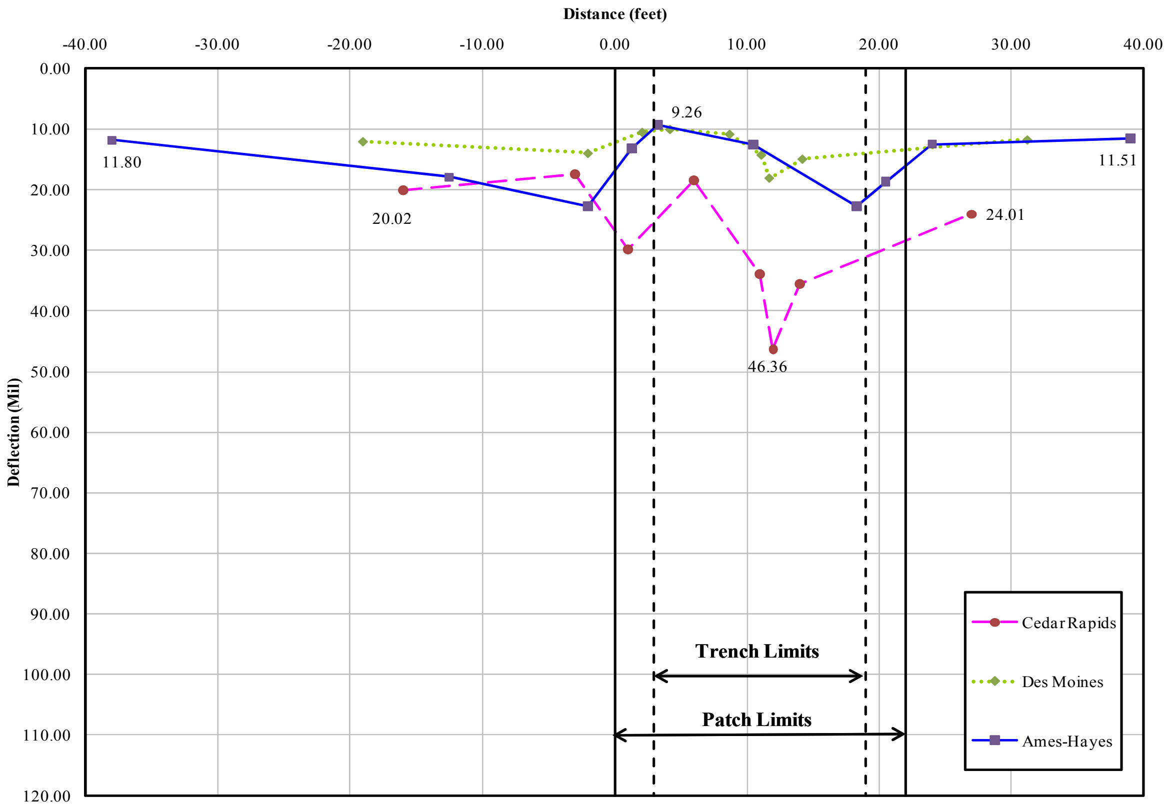


Figure 3.33. Comparison of the 15-kip FWD test in spring 2009

Summary of Monitoring

The following was concluded from the continued monitoring of the utility cut restorations documented in Phase I:

- The patches that were originally installed were not even with the existing pavement.
- The trenches have continued to settle, with maximum settlements ranging from 0.03 inches per month for the trench in Des Moines to 0.13 inches per month for the trench in Ames. The maximum rate of settlement was measured over the winter months (except point 2 in the Des Moines trench).
- The Des Moines trench patch performed better based on lowest total settlements and uplifts for the patch and the surrounding pavement, and it did not experience the same seasonal softening effects measured with FWD testing, as did the trenches in Ames and Cedar Rapids. This was the result of the concrete being doweled into the surrounding pavement.
- The FWD test showed a similar response of the trenches based on the season of the tests. According to the literature reviewed, this was the result of differences in the moisture content in the soil during the spring and fall.
- The FWD testing on the Cedar Rapids trench showed that damage caused by equipment during construction had a long-term impact on the performance of the trench.

CHAPTER 4. RECOMMENDED TRENCH CONSTRUCTION PRACTICES

Recommended Practices from Phase I

Based on field observations, measurements, and laboratory testing during Phase I, the following practices are recommended:

- When specifying granular soils as backfill, relative density criteria should be used. A minimum relative density of 65% was recommended.
- When using granular backfill for utility cut restorations, the moisture content should be greater than the bulking moisture content. This reduces the collapse potential of the soil that can occur with changes in moisture content.
- Quality management practices for backfill placement should be implemented in the field; however, specific tests were not specified.

In most states, the use of the nuclear density gauge to monitor dry unit weight and moisture content in the field is becoming increasingly difficult because of regulatory concerns. The DCP test can provide an alternative method for monitoring the quality of compacted backfills; however, each specific backfill requires different DCP correlations.

During Phase I, an area around the utility cut known as the zone of influence was found to be a factor in the degradation of a utility cut restoration. The zone of influence resulted from the loss of lateral support in the trench walls during excavation. As a result of Phase I, a cut of 2 to 3 feet beyond the boundaries of the utility trench was proposed to mitigate the effects of the zone of influence. Backfill placed in the pavement cut area and the excavation area would be compacted.

During Phase II, the six recommended trenches were constructed and monitored for about 22 months.

Recommended Phase I Trench Designs

At the conclusion of the Phase I project, three trench restoration designs and two types of backfill were proposed to minimize settlement and the effects of the zone of influence. The three trench restoration designs, shown in Figure 4.1, included the following:

- Trenches A and D consisted of a trench with vertical walls extending from the bottom of the trench to the overlying pavement with up to 3 feet of pavement removal around the perimeter of the utility trench. After the pavement removal, the exposed subgrade soil was compacted in place (see Figure 4.1, Trenches A and D).
- Trenches B and E consisted of a trench with T-section vertical walls extending to the base of the trench, with the upper 2 feet of the trench being horizontally outward 2 to 3 feet beyond the perimeter of the normal trench walls and pavement removal to the limits of the T-section. The excavated soil from the T-section was then placed across

the trench in 1-foot lifts and compacted. When there was insufficient soil removed from the trench to complete the trench restoration, available granular soil was used (see Figure 4.1, Trenches B and E).

- Trenches C and F consisted of a T-section trench constructed the same as Trenches B and E, with a structural geogrid placed on the bottom of the excavated T-section area and across the trench (see Figure 4.1, Trenches C and F).

The two proposed backfills were 3/8-inch minus granular backfill (Trenches A, B, and C) and SUDAS Class I gradation granular backfill (Trenches D, E, and F). Figure 4.1 shows these trenches, indicating the two types of backfill.

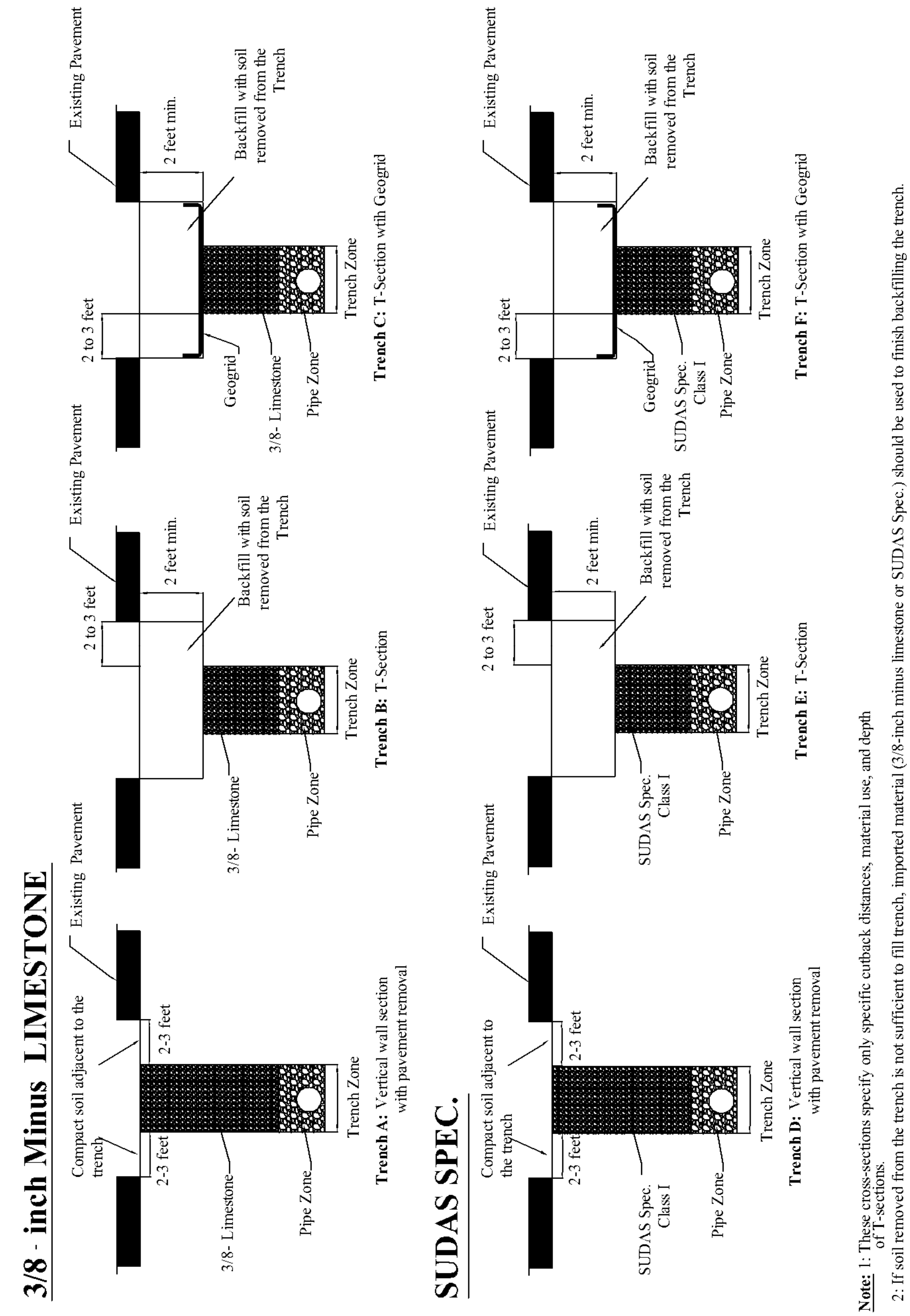


Figure 4.1. Phase I recommended utility cut trench restorations (modified from Schaefer et al. 2005)

Construction of the Recommended Trench Designs

Trench C, as shown in Figure 4.1, was constructed during Phase I, and the other five trenches (A, B, D, E, and F) were constructed during Phase II. Table 4.1 summarizes the location of each trench and the key design feature. Figure 4.2 shows the locations of the six trenches in the city of Ames.

Table 4.1. Summary of recommended trench construction

Trench	Date of construction	Location*	Backfill	Total depth	Design features
Trench A	08/07/2007	1413 McKinley Drive	3/8-inch minus limestone	6.5 ft	Vertical trench walls
Trench B	07/12/2007	9th Street and Carroll Avenue	3/8-inch minus limestone	9.0 ft	T-section
Trench C	05/16/2005	Fillmore Avenue and McKinley Drive	3/8-inch minus limestone	7.0 ft	T-section with geogrid
Trench D	07/23/2007	2201 Ferndale Avenue	SUDAS Class I 1-inch clean limestone	8.5 ft	Vertical trench walls
Trench E	07/11/2007	7th Street and Carroll Avenue	SUDAS Class I 1-inch clean limestone	6.5 ft	T-section
Trench F	07/11/2007	6th Street and Carroll Avenue	SUDAS Class I 1-inch clean limestone	5.5 ft	T-section with geogrid

*All trenches located in Ames, Iowa

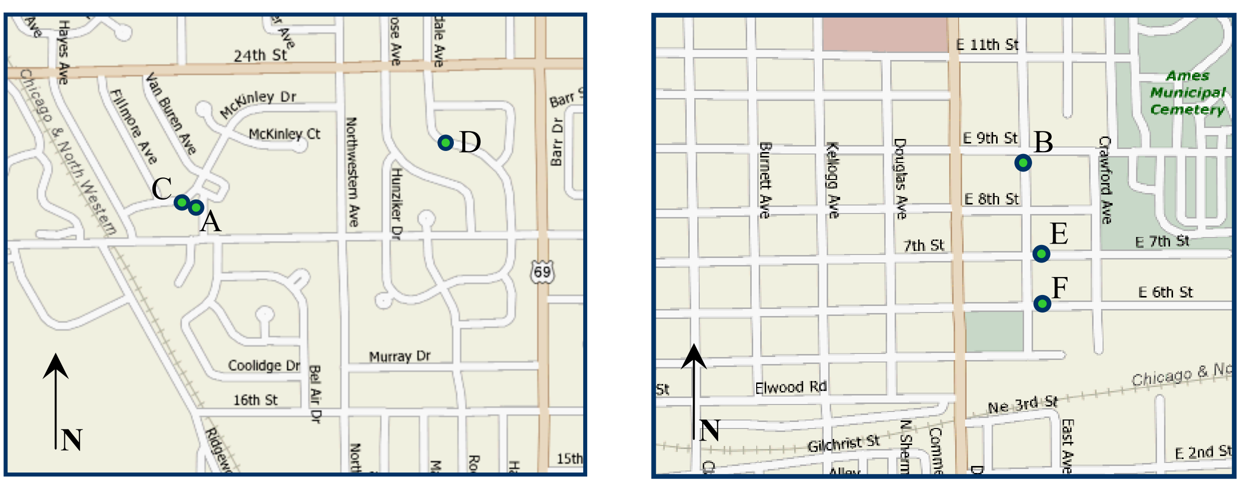


Figure 4.2. Locations of the recommended trenches in Ames, Iowa

To measure the field properties of the backfill at the sites, nuclear density, DCP, and/or Clegg Hammer tests were performed. The results from these field tests are summarized in the sections below. From these sites, bulk samples of the backfill and the soils removed from the excavation were collected for laboratory tests. The results of the laboratory testing are summarized below.

Recommended Trench A

Trench A was constructed on August 7, 2007, on the south side of the street at 1413 McKinley Drive. The utility trench cut was made to replace a water main valve. The excavation, construction, and restoration of the trench were completed by the City of Ames.

The excavated trench was 26.5 feet long, 9.5 feet wide, and 6.5 feet deep. The base of the trench cut consisted of clay. Water from a water main break had saturated the clay. Sand from a previous utility trench cut restoration was encountered on the south and west sides of the trench. During the excavation of the trench, the backhoe was located near the southwest edge of the trench (see Figure 4.3). The sand sloughed from the trench sides into the excavation. The trench was backfilled with one lift of 1½-inch limestone mixed with the saturated clay to form a base and five lifts of 3/8-inch minus limestone. A vibratory plate compactor attached to a backhoe was used to compact each lift. The backhoe was located on the northeast side of the trench, while on the east side of the trench the truck unloaded at the edge of the trench (see Figure 4.4). After backfilling, the trench was left open for three days to allow traffic to compact the backfill further.

On August 10, 2007, the City of Ames removed additional pavement from around the trench: 2 feet to the east, between 2 and 3 feet to the north, and 1 foot to the south. A small vibratory compactor was used to compact the surrounding soil after the pavement was removed. Four inches of asphalt were then used to patch the trench. The completed patch was 30 feet long and 12 feet wide according to the plan view. Figure 4.5 presents a cross-section of Trench A.

Figure 4.3. Backhoe operating on the southeast edge of Trench A

Figure 4.4. Truck backing into Trench A from the east while a backhoe with an attached vibratory plate compactor operates on the northwest side of the trench

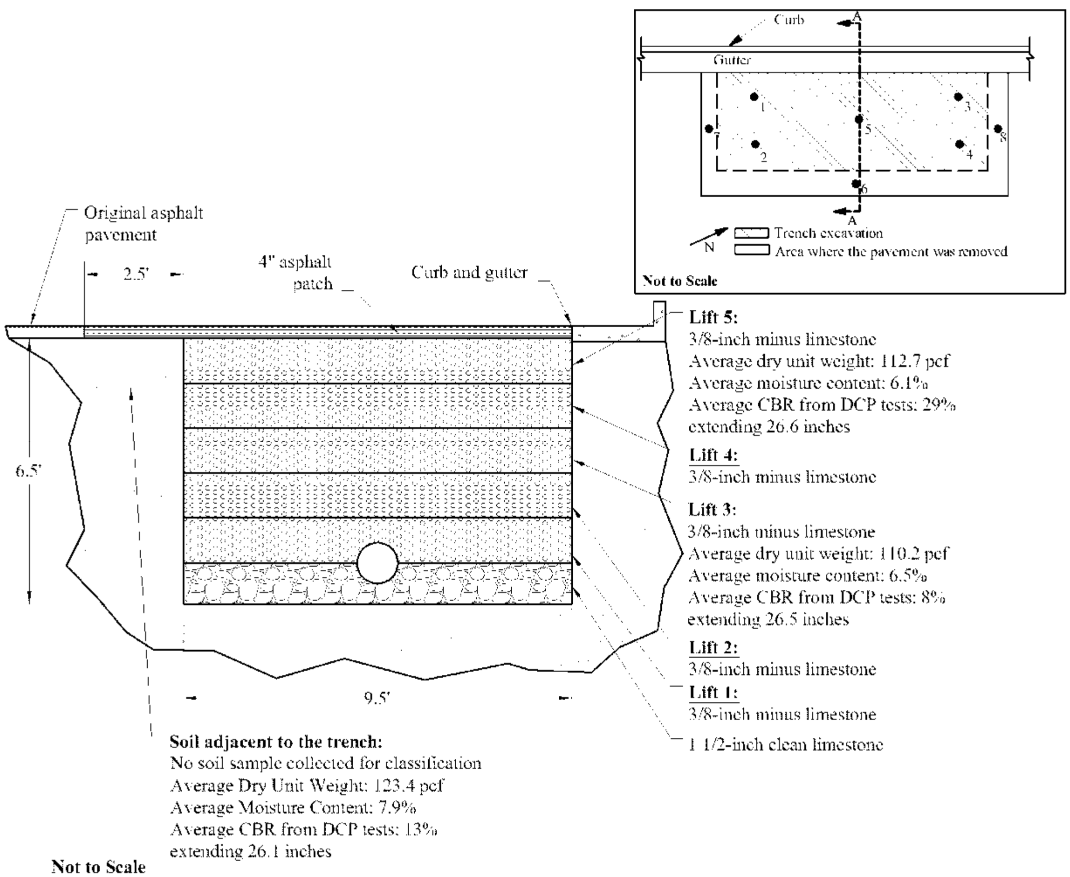


Figure 4.5. North-south cross-section of Trench A; a) plan view, b) cross-section (Note: Testing locations 6, 7, and 8 were located in the soil adjacent to the trench)

Recommended Trench B

Trench B was constructed on July 12, 2007, on Carroll Avenue just south of 9th Street in Ames. The utility cut was constructed to replace a water main valve. The excavation, construction, and restoration of the trench were completed by the City of Ames.

The excavated trench was 10 feet long, 9 feet wide, and 9 feet deep. The trench was backfilled using 1½-inch limestone for bedding and five lifts of 3/8-inch minus limestone above the pipe. The soil was tested in the lab and was classified as a gravel and sand with LL of 39 and PL of 19. A vibratory plate compactor attached to a backhoe was used to compact each lift. After backfilling the trench, it was left open to traffic for five days.

Pavement around the trench was cut and removed on July 17, 2007. The total area of pavement removal was 15 feet wide by 33 feet long. This large area of pavement was removed because the original pavement had cracked during construction. On July 18, 2007, 2 to 3 feet of in-place soil around the trench was excavated.

During compaction, the backhoe drove into the cut area because it could not access the entire area (see Figure 4.6). The truck also backed into the open cut to reach the excavated area as shown in Figure 4.7.

Figure 4.6. Backhoe operating where pavement was removed on the northeast side of Trench B

Figure 4.7. Truck operating where pavement was removed on the northeast edge of Trench B

Supplemental backfill used in the T-section consisted of two different soils. The first soil was a mixture of organics, bottom ashes (cinders) from the coal-burning power plant in Ames, and 3/8-inch minus limestone (Backfill No. 1). The second backfill contained cinders from the City of Ames Power Plant mixed with the 3/8-inch minus limestone backfill (Backfill No. 2). Backfill No.1 was placed below Backfill No. 2. Soil removed during excavation was not used because the City of Ames did not store the soil.

The trench was left open overnight. During the night, the City of Ames received 1½ inches of rain. The next day water was standing in the trench (see Figure 4.8). The trench was left open to dry for seven days. By July 25, 2007, the surface of the trench had mostly dried. The city then placed a thin lift of 3/8-inch minus limestone on the south side of the cut area where standing water remained. The 3/8-inch minus limestone was compacted using a small, hand vibratory compactor as shown in Figure 4.9. The trench was then patched with 6 inches of asphalt.

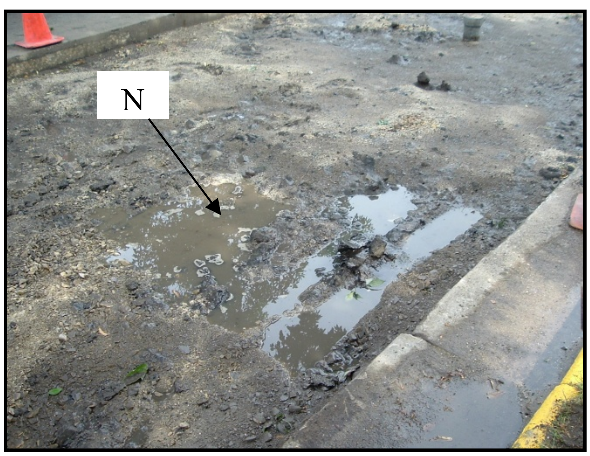


Figure 4.8. Standing water in Trench B after it rained

Field tests were performed on lifts 3 and 5 on July 17, 2007 (see Figure 4.10). When the T-section was constructed on July 18, 2007, lift 5 was removed to allow the placement of the replaced fifth (top) lift. On July 18 and 25, 2007 (see Figures 4.11 and 4.12), field tests were performed on the final lift constructed in the T-section at four points within the trench and four points in the area where the pavement was removed. Detailed results from this testing can be found in the next section; however, Figures 4.11 and 4.12 indicate rain reduced the CBR calculated using the DCP of the top soil layer from 15% to 7% (more than 50%).

Figure 4.9. Small vibratory compactor used to compact the backfill in Trench B

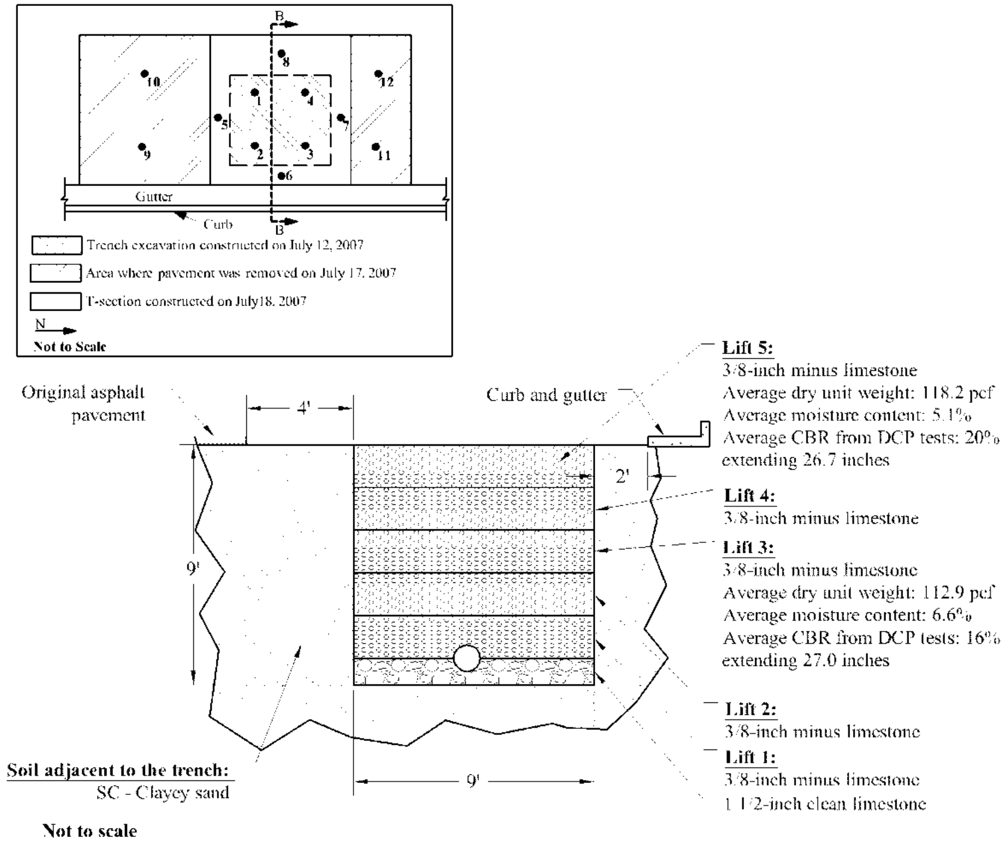


Figure 4.10. East-west cross-section of Trench B (before construction of the T-section) on July 17, 2007; (a) plan view, (b) cross-section

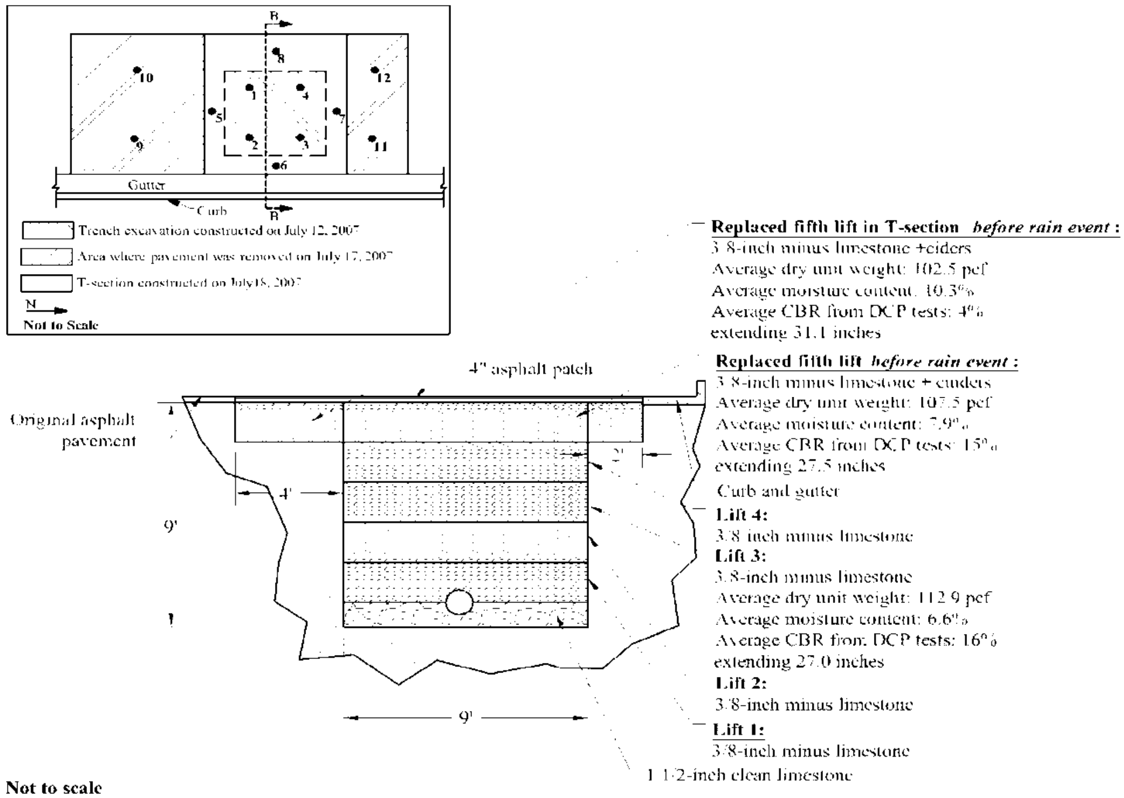


Figure 4.11. East-west cross-section of completed Trench B on July 18, 2007; (a) plan view, (b) cross-section

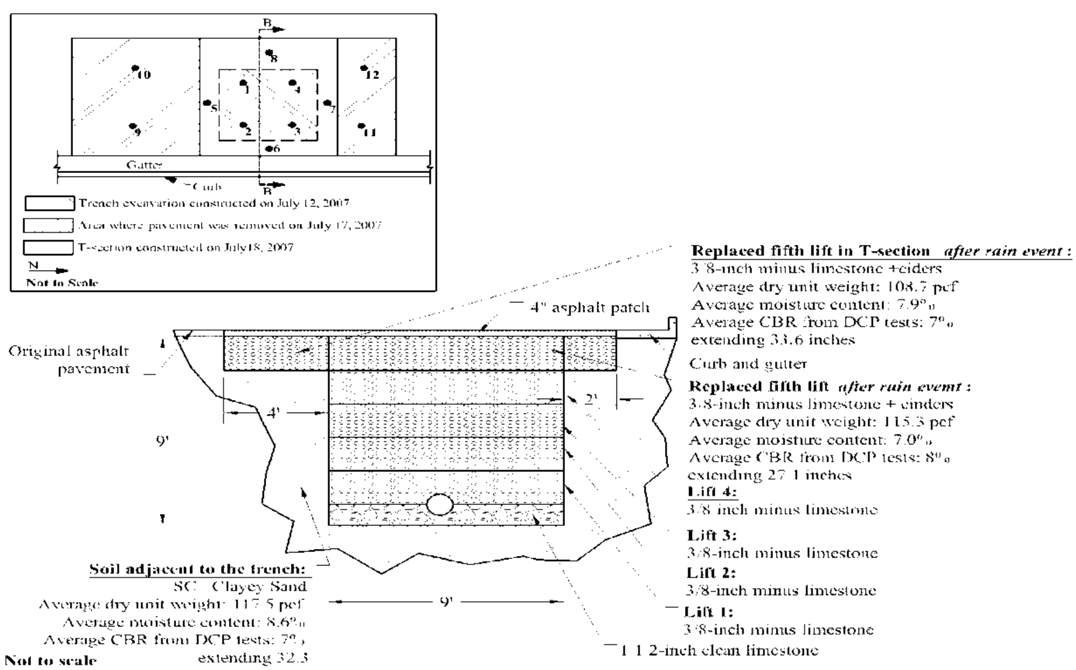


Figure 4.12. East-west cross-section of completed Trench B on July 25, 2007, after being left open for six days; (a) plan view, (b) cross-section

Recommended Trench C

Trench C was constructed on May 16, 2005, at McKinley Drive and Fillmore Avenue. The excavation, construction, and restoration of the trench were completed by the City of Ames.

The completed trench (including the T-section) was 24.7 feet along the curb and 13.6 feet wide perpendicular to the curb. Beyond the limits of the trench, the T-section was 3 feet wide and 2 feet deep. The backfill for the lower lifts was 3/8-inch minus limestone. The Iowa State research team did not perform any field testing below the geogrid. As a result, no comparisons between the field-testing results and laboratory-testing results can be made for the lower portion of the trench.

After the T-section was excavated around the trench, BX 1100 geogrid was placed at the bottom of the T-section (i.e., about 2 feet below the ground surface). The Iowa State research team monitored the placement of the geogrid (see Figure 4.13) and the last two lifts to complete the trench.

The final two lifts of the trench used mixtures of 3/8-inch minus backfill and soil excavated from the trench. Two 1-foot lifts were used to complete the trench. Field-testing consisted of nuclear density and DCP tests. Figure 4.14 illustrates the cross-section of Trench C with a summary of soil properties.

Figure 4.13. Geogrid being placed in Trench C

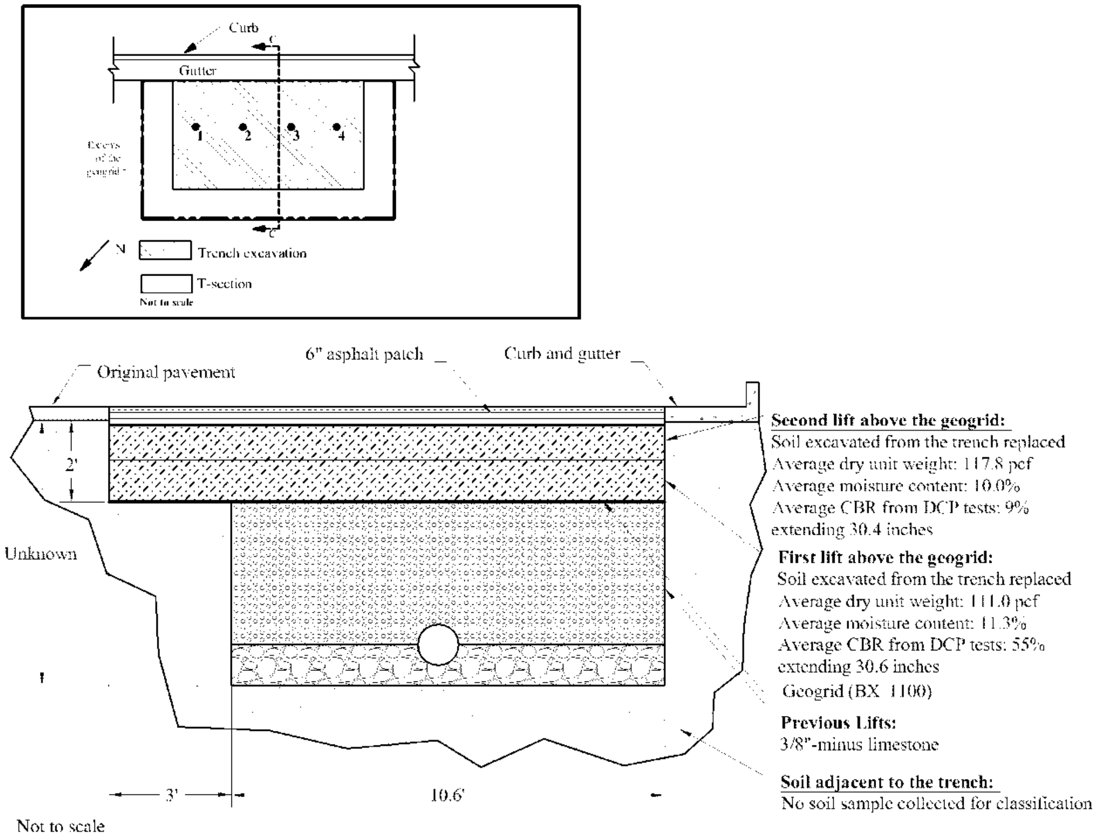


Figure 4.14. Cross-section of Trench C; (a) plan view, (b) cross-section

Recommended Trench D

Recommended Trench D was constructed on July 23, 2007, at 2201 Ferndale Avenue. The utility cut was constructed to replace a sanitary sewer. The excavation, construction, and restoration of the trench were completed by the City of Ames.

The trench was 7 feet wide, 11 feet long, and 8.5 feet deep. The sewer was repaired with PVC truss pipe, pipe bands, and cement. One and a half-inch clean limestone was placed as bedding for the pipe and around the pipe. The trench was backfilled using six 1-foot lifts of 1-inch limestone (SUDAS Specifications Class I). A vibratory plate attached to a backhoe was used to compact the soil. During the construction of the trench, construction equipment was located near the east and west edges of the trench (see Figures 4.15 and 4.16). After the final lift, the backhoe was driven over the trench to further compact the backfill. The trench was left open for two days.

On July 25, 2007, the City of Ames removed additional pavement from the perimeter of the trench for a distance of about 1 foot. A small vibratory compactor was used to compact the surrounding soil where the pavement was removed. Four inches of asphalt were used to patch the trench. Figure 4.17 displays a cross-section of Trench D with the field-testing results.

Figure 4.15. Backhoe operating on the east side of Trench D, with a dump truck operating on the north side

Figure 4.16. Front end loader operating on the southwest side of Trench D

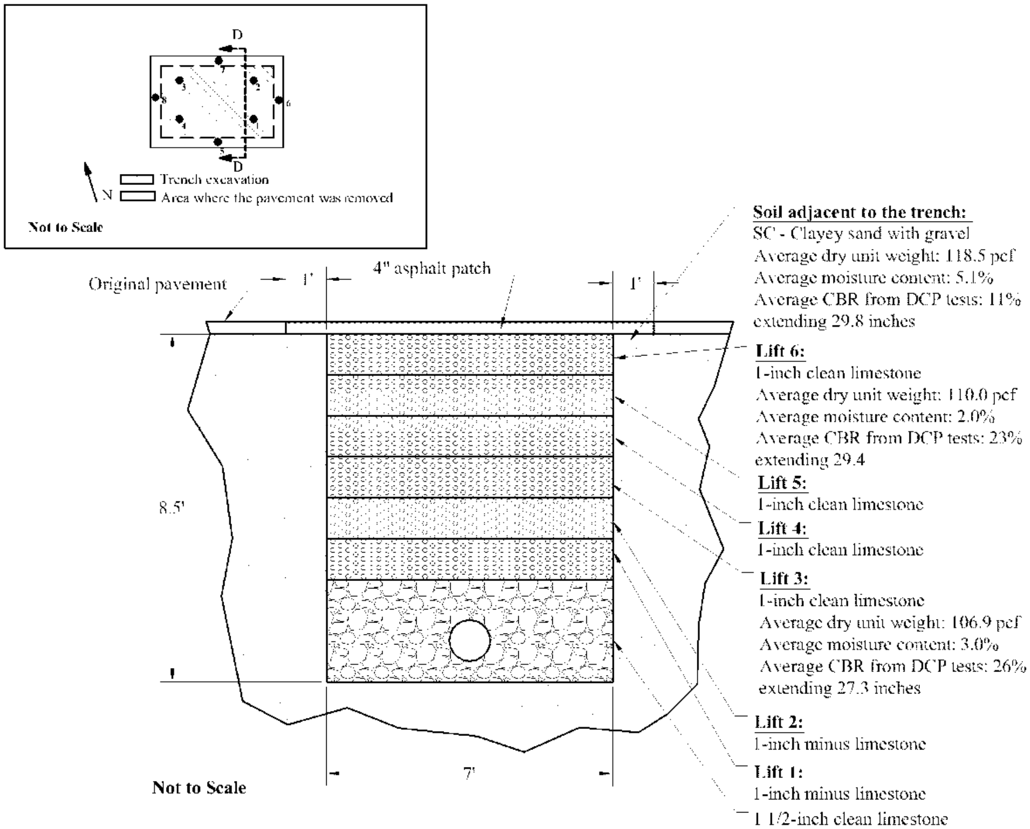


Figure 4.17. Cross-section D-D of Trench D; (a) plan view, (b) cross-section

Recommended Trench E

Trench E was constructed on July 11 and 12, 2007, on 7th Street east of Carroll Street. The utility cut was constructed to replace a water main valve. The excavation, construction, and restoration of the trench were completed by the City of Ames.

The excavation took place on July 11, 2007. The completed trench was 10 feet long, 7.5 feet wide, and 6.5 feet deep, excluding the T-section. The valve was replaced on July 12, 2007. The first lift consisted of 1½-inch clean limestone to a thickness of about 1½ feet above the pipe. For the remaining five lifts, SUDAS Class I limestone, which was placed at a thickness of 1 foot, was used. A vibratory plate attached to a backhoe was used for compaction. The trench was filled until it was level with the top of the existing pavement. In addition to the vibratory plate compaction, the backhoe was driven over the trench. The trench was left open to traffic for five days to further compact the backfill.

On July 17, 2007, the pavement around the trench was removed. On July 18, 2007, the T-section was excavated to a depth of about 2 feet. The T-section was backfilled with a mixture of 1-inch clean limestone and soil from the City of Ames soil supply piles. During the placement of the backfill, two different ratios of 1-inch clean limestone and soil from the City of Ames supply piles were used. Two bulk soil samples were collected of this backfill: Additional Backfill No. 1 and Additional Backfill No. 2. The origin of the soil in the supply piles was not known. During construction, the equipment was close to the edge of the trench (see Figures 4.18 and 4.19). The T-section was backfilled until it was at the elevation of the bottom of the existing pavement. A thin lift of 3/8-inch minus limestone was loosely placed as a base for the pavement patch. The asphalt patch was 6 inches thick.

Figure 4.20 shows a cross-section of Trench E on July 17, 2007, and Figure 4.21 shows the cross-section of Trench E on July 18, 2007.

Figure 4.18. Soil previously removed being placed back in Trench E by a truck to the south, while a backhoe operates on the east edge of the pavement

Figure 4.19. Backhoe support on the east edge of the pavement by Trench E

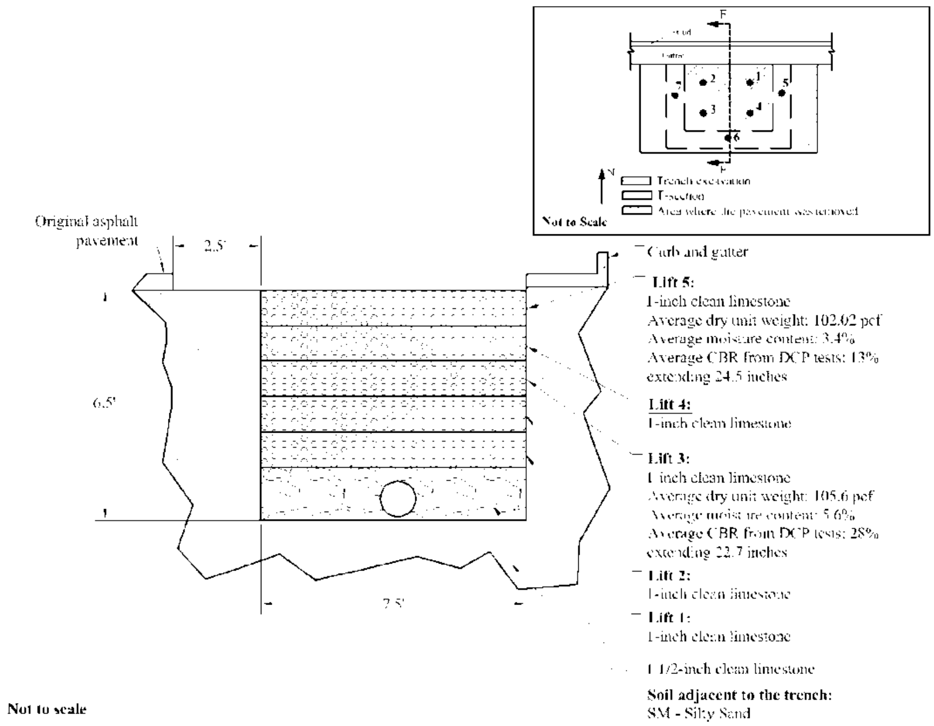


Figure 4.20. Cross-section of Trench E on July 17, 2008; (a) plan view, (b) cross-section

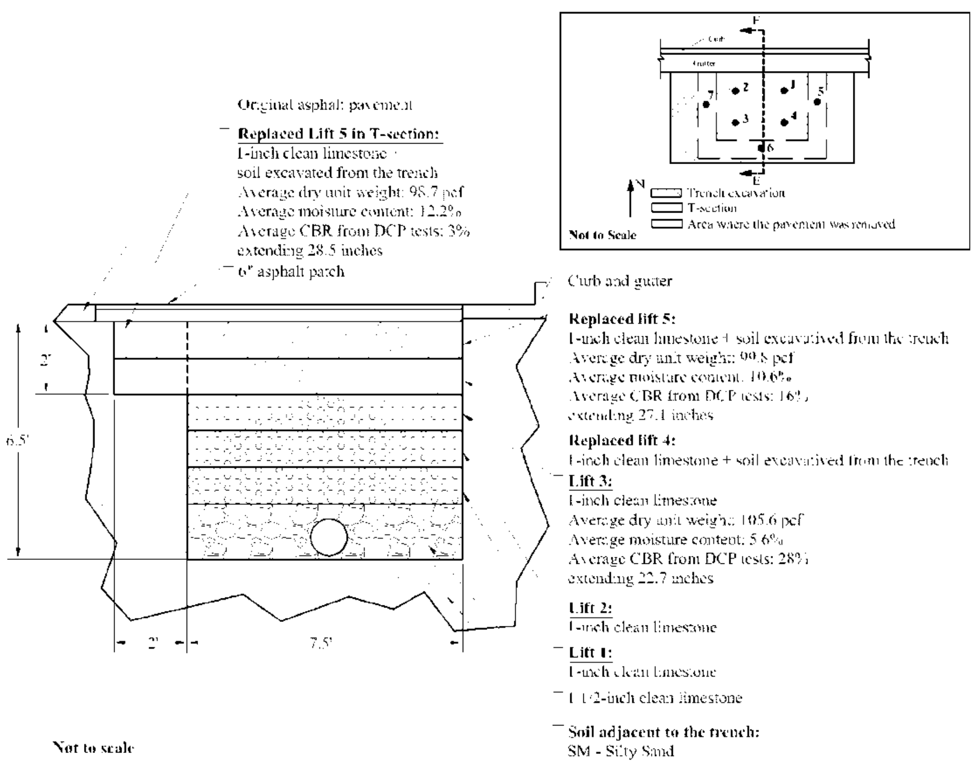


Figure 4.21. Cross-section of Trench E on July 18, 2007; (a) plan view, (b) cross-section

Recommended Trench F

Trench F was constructed on July 11 and 12, 2007, on 6th Street just east of Carroll Street. The utility cut was conducted to replace a water main valve. The excavation, construction of the trench, and restoration was completed by the City of Ames.

The excavation of the trench took place on July 11, 2007. The completed excavation was 9 feet long, 7 feet wide, and 5.5 feet deep, excluding the T-section. On July 12, 2007, the water value was replaced (valve and pipe were replaced and connected using pipe sleeves). Excess water from the water main spilled into the trench and was pumped out, as seen in Figure 4.22.

For the first lift, the soil consisted of about 1½ feet of 1½-inch limestone. To complete the trench, three additional lifts of clean limestone were constructed for a total of four lifts (one lift of 1½-inch limestone and three lifts of 1-inch clean limestone). The second and third lifts were about 1 foot thick and the fourth lift was 2 feet thick, which was removed to construct the T-section. During the construction of the trench, construction equipment operated on the east and west edges of the trench (see Figures 4.23 and 4.24). All compaction was completed with a vibratory plate compactor attached to a backhoe. The backhoe drove over the trench to further compact the backfill. The backfill was leveled with the road surface. The trench was left unpaved for five days.

On July 17, 2007, the City of Ames returned to the site and removed pavement surrounding the trench to construct the T-section. On July 18, 2007, the T-section was excavated 2 feet deep; the fourth lift was removed. Geogrid was placed in the excavated area (see Figure 4.25). The trench was then backfilled with soil excavated from the trench on July 11, 2007, and placed in a 2-foot lift. The soil was tested in the lab and was classified as clayey soil with LL of 40 and PL of 23. A vibratory plate compactor on a backhoe was used for compaction. Six inches of asphalt was installed for the permanent patch on the trench. Figure 4.26 illustrates the cross-section of Trench F on July 12, 2007, and Figure 4.27 shows the cross-section of Trench F on July 18, 2007.

Figure 4.22. Water being pumped out of Trench F

Figure 4.23. Truck being backed up to the east edge of Trench F

Figure 4.24. Truck being backed up to the west edge of Trench F

Figure 4.25. Geogrid being placed in Trench F after the fourth lift and T-section were excavated

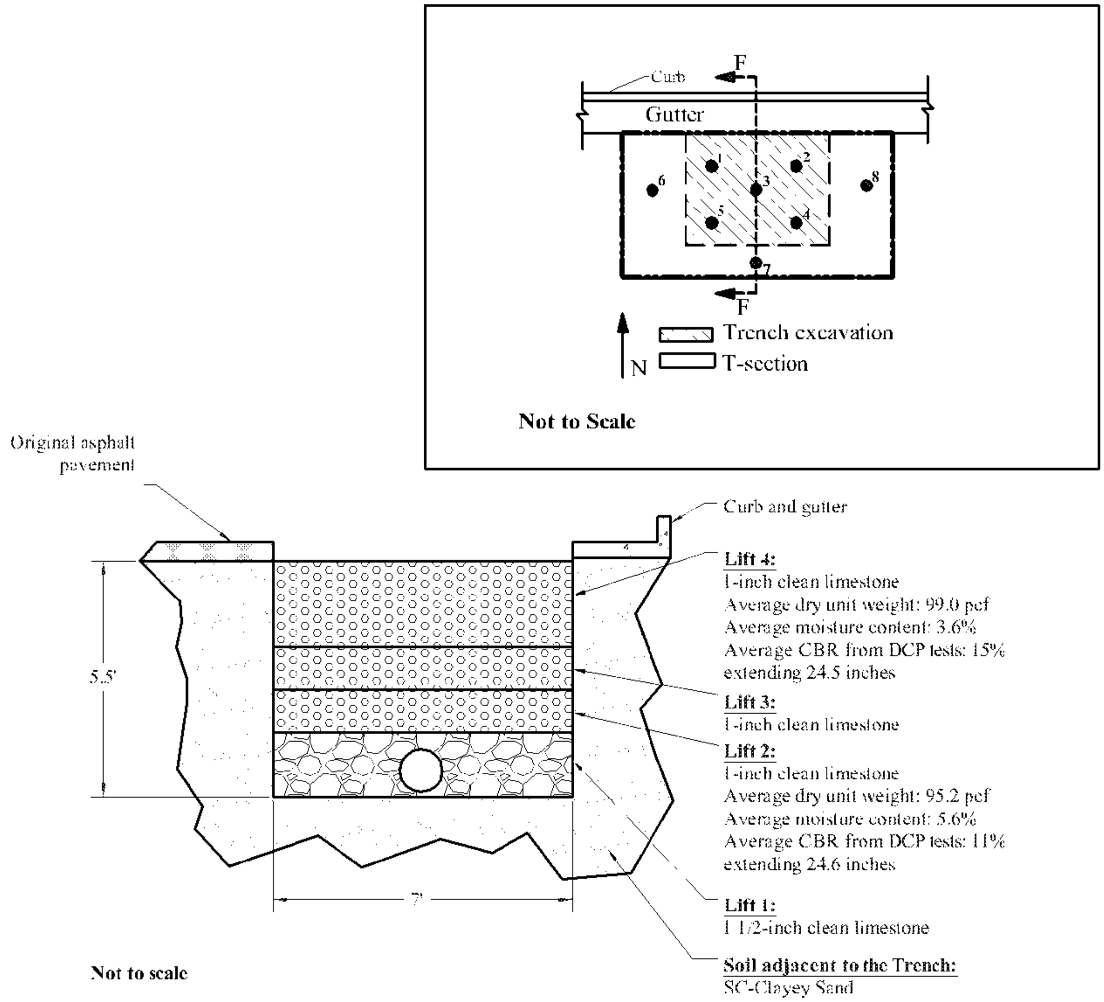


Figure 4.26. Cross-section of Trench F on July 12, 2007; (a) plan view, (b) cross-section

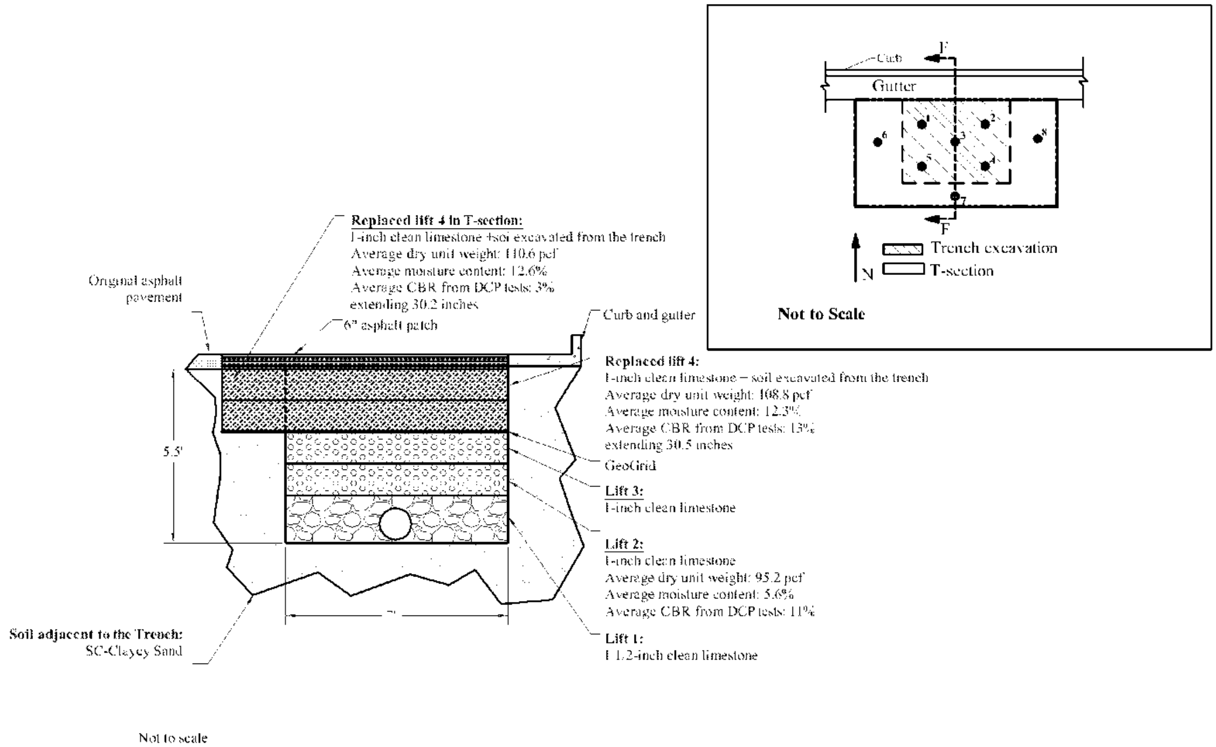


Figure 4.27. Cross-section of Trench F on July 18, 2007; (a) plan view, (b) cross-section

Laboratory Test Results and Discussion

Laboratory tests performed according to the corresponding ASTM standards were conducted on the excavated soil and backfill used in the six recommended trenches. These tests included particle size distribution with sieve and hydrometer analyses, Atterberg Limits, water content, Standard Proctor, and minimum and maximum relative density. These laboratory tests were performed to determine soil properties and classify the soils used in the field as well as to compliment the field data (see Table 4.2).

Table 4.2. Standard tests used in the laboratory

Test	ASTM
Particle size distribution	ASTM D 422-63 (2007), "Standard Test Method for Particle-Size Analysis of Soils"
Atterberg Limits	ASTM D 4318-95a (1995), "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils"
Standard Proctor	ASTM 698-91 (1991), "Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort"
Maximum dry unit weight	ASTM D 4253 2000 (2003), "Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table"
Minimum dry unit weight	ASTM D 4254-2000 (2003), "Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density"

All soils were classified using the USCS and AASHTO. The AASHTO classification is used by the Iowa DOT for determining the appropriate use of soils on a construction project.

Soil Gradation

Samples from both the soil excavated during the construction of the utility trench cut and the backfill were sieved. The 3/8-inch minus limestone backfill classification information is presented in Table 4.3. The classification information for the 1-inch clean limestone backfills is presented in Table 4.4. The classification information of the secondary backfills used in the T-sections of Trenches B, C, E, and F is presented in Table 4.5. The classification information of the soils excavated from the trench is presented in Table 4.6.

The gradation curves were constructed from the gradation tables and the hydrometer results. The gradation curves for the limestone backfills are shown in Figure 4.28. The gradation curves for the soils excavated from the trenches are presented in Figure 4.29. The backfill soils used in Trench B consisted of cinders from the Ames Power Plant. Because of the elongation, there was a bump in the gradation graphs between the sieve and hydrometer readings. The gradation curves for samples from Trench B have been smoothed out.

The 3/8-inch minus limestone used in trenches A, B, and C meets the Iowa DOT specifications standards for granular backfill. The 3/8-inch minus limestone was used on Trenches A, B, and C. Because the 3/8-inch minus backfills were from the same soil supply piles in the City of Ames, the results from their particle gradations were averaged, except for the 3/8-inch minus gradation from Trench C, constructed in 2005. The gradation of the 3/8-inch minus soil from 2005 is also presented in Table 4.3.

In Trench A, the 3/8-inch minus limestone was the only backfill used. In Trenches B and C, the 3/8-inch minus limestone was used to backfill the trenches to about 2 feet below the surface. The remaining 2 feet of the trenches were filled with a mixture of 3/8-inch minus limestone and various cohesive materials. The soil mixture used in the top two 2 feet of Trench B was a secondary backfill. In Trench C, no bulk samples of the mixed soils were collected. Therefore, there is not laboratory data for the top 2 feet of Trench C.

The 1-inch clean limestone meets the SUDAS specification standards. This aggregate was used in Trenches D, E, and F. In Trench D, 1-inch limestone was the only backfill used in the trench. In Trenches E and F, this aggregate was used to backfill the trench to about 2 feet of the surface. The remaining 2 feet of Trenches E and F were filled with a mixture of 1-inch clean limestone and cohesive soils.

Table 4.3. Classification of the limestone backfills

Sample	Soil classification						AASHTO subgrade rating
	C_U/C_C	% Passing No.4/No. 200	USCS		AASHTO		
Trenches A and B Ames summer 2007; 3/8-inch minus limestone	22.69/6.17	53.21/8.83	SP-SM	Poorly graded sand with silt and gravel	A-1-a	Stone fragments, gravel, and sand	Good to excellent
Trench C Ames summer 2005; 3/8-inch minus limestone	75.0/1.3	97.9/0.8	SM	Sand/silt	A-1-a	Stone fragments, gravel, and sand	Good to excellent
Trench D 1-inch clean limestone from 9th and Carroll	224.5/42.7	23.6/0.6	GP	Poorly graded gravel	A-1-a	Stone fragments, gravel, and sand	Good to excellent
Trenched E and F 1-inch clean limestone from 6th/7th Carroll	487.5/18.0	31.7/0.6	GP	Poorly graded gravel	A-1-a	Stone fragments, gravel, and sand	Good to excellent

Table 4.4. Supplemental backfills used in the top 2 feet of Trenches B, E, and F

Sample	Soil classification							AASHTO subgrade rating
	C_U/C_C	Liquid limit/ Plastic index	% Passing No.4/No. 200	USCS		AASHTO		
Trench B Backfill No. 1	220/42.6	39/21	79.0/16.0	SC	Clayey sand	A-2-6	Silty and clayey gravel and sand	Good to excellent
Trench B Backfill No. 2	39.28/7.6	48/28	86.67/20.29	SC	Clayey sand	A-2-7	Silty and clayey gravel and sand	Good to excellent
Trench E Additional Backfill No. 1	487.5/18.0	36/17	73.1/20.2	SC	Clayey sand	A-2-6	Silty and clayey gravel and sand	Good to excellent
Trench E Additional Backfill No. 2	224.5/42.7	40/23	79.2/16.0	SC	Clayey sand	A-2-6	Silty and clayey gravel and sand	Good to excellent
Trench F Final Backfill	187.5/13.3	26/11	86.7/20.3	SC	Clayey sand	A-2-6	Silty and clayey gravel and sand	Good to excellent

Table 4.5. Soil classifications and laboratory results for soil excavated from the trench cuts

Sample	Soil classification							AASHTO sub-grade rating
	C_U/C_C	Liquid limit/ Plasticity index	% Passing No.4/ No. 200	USCS		AASHTO		
Trench A Sand from previous cut	12.34/ 1.05	---	100/ 2.9	SW	Well-graded sand	A-3	Fine sand	Good to excellent
Trench B	11.68/ 1.95	39/19	65.4/ 4.4	SW-SC	Well-graded sand with gravel	A-1-B	Stone fragments, gravel and sand	Poor to fair
Trench D	3.88/ 161	---	98.5/ 49.1	SC	Clayey sand with gravel	A-6	Clayey soil	Poor to fair
Trench E	366/ 104.6	26/10	98.5/ 49.1	SM	Silty sand	A-2-4	Silty and clayey gravel and sand	Good to excellent
Trench F	---	40/23	99.8/ 42.9	CL	Sandy lean clay	A-6	Clayey soil	Poor to fair

Table 4.6. Standard Proctor test results

Sample	Classification	Maximum dry unit weight and optimum moisture content from the Standard Proctor Test		Range of dry unit weights and optimum moisture contents from NAVFAC	
		γ_{Max} (pcf)	Optimum moisture content (%)	γ_{Max} (pcf)	Optimum moisture content (%)
Trenches A and B Ames summer 2007; 3/8-inch minus limestone	SP-SM	131.0	9.0*	110 to 125	11 to 16
Trench C Ames summer 2005; 3/8-inch minus limestone	SM	127.3	11.1*	110 to 125	11 to 16
Trenches D, E, and F 1-inch clean limestone	GP	N/A	N/A	115 to 125	11 to 14
Trench B Average Backfill	SC	116.9	13.1	105 to 125	11 to 19
Trench E Additional Average Backfill	SC	106.9	18.3	105 to 125	11 to 19
Trench F Final Backfill	SC	119.1	13.2	105 to 125	11 to 19
Trench B	SW	128.8	9.1	110 to 130	9 to 16
Trench D	SC	120.2	12.1	105 to 125	11 to 19
Trench E	SC	123.7	10.9	105 to 125	11 to 19
Trench F	CL	122.7	11.9	95 to 120	12 to 24

*The maximum moisture content tested was reported because at higher moisture contents the water accumulated in the bottom of the container and was not held between the particles.

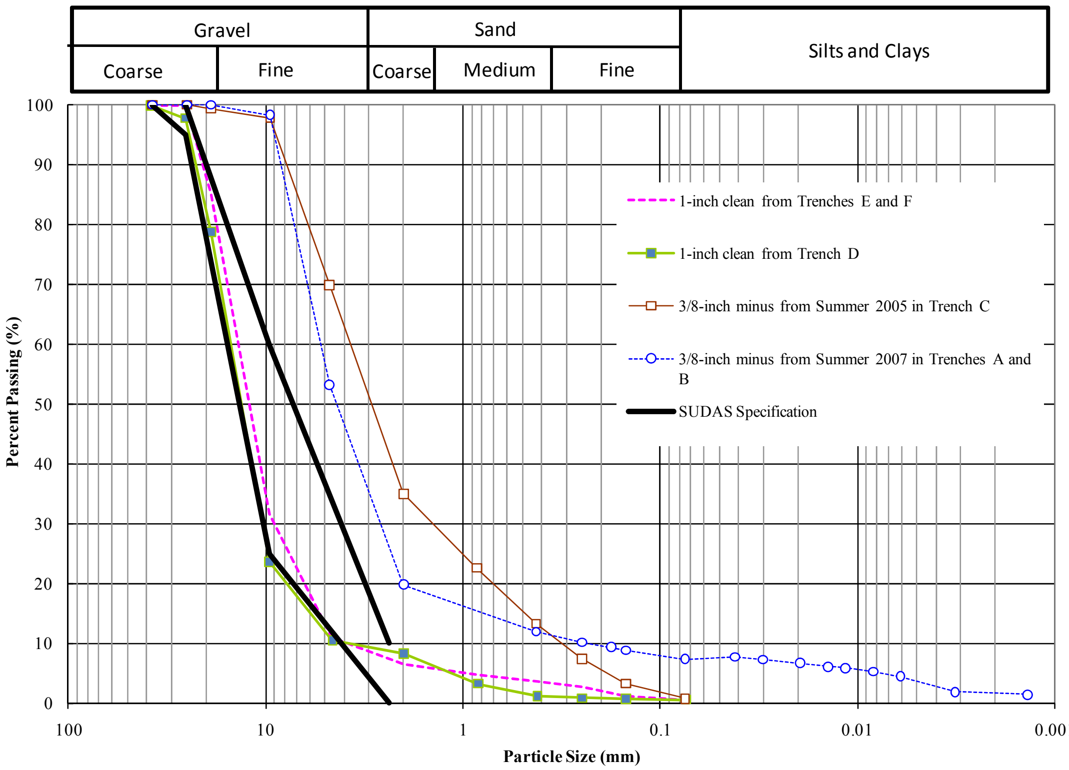


Figure 4.28. Gradation of backfill materials with the SUDAS specification and the Iowa DOT specification

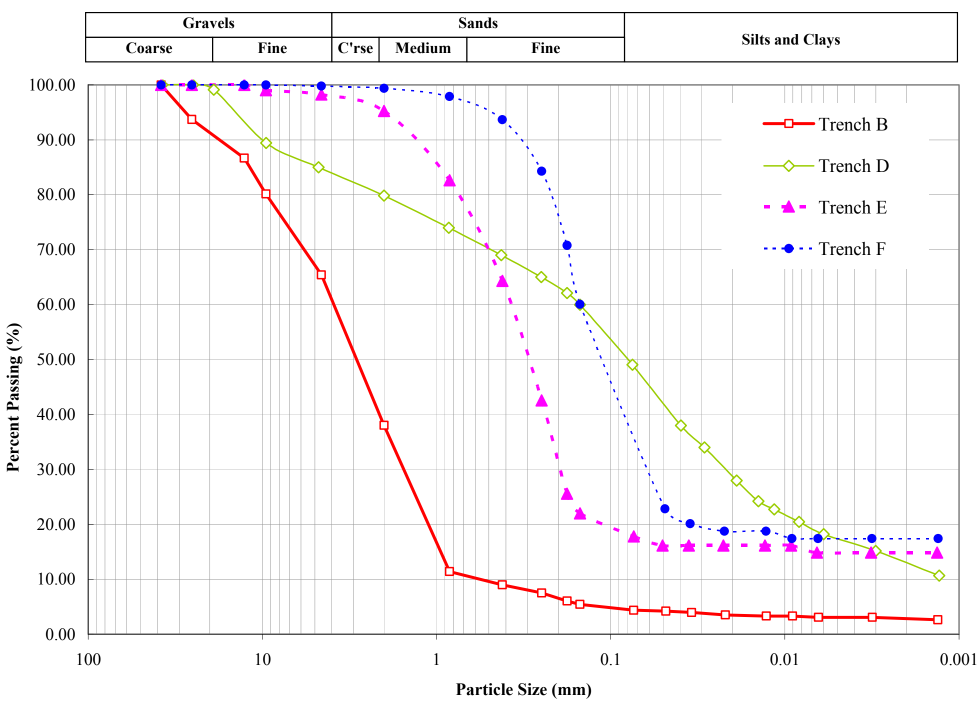


Figure 4.29. Gradation of soils excavated from different trenches

Classification of Backfills

Table 4.3 summarizes the backfill classifications for the limestone backfills, and Table 4.4 summarizes the supplemental backfills, which were mixtures of limestone backfill and other soils.

The 3/8-inch minus backfill was classified as SP-SM (poorly graded sand with silt). The AASHTO classification was A-1-b (stone fragments, gravel, and sand). AASHTO rated subgrade suitability of this soil as good to excellent in Tables 2.2 and 2.3.

The 3/8-inch minus backfill used in Trench C for the lifts below the geogrid, constructed in the summer of 2005, had a USCS classification of SM (sand with silt) in the Phase I report. The AASHTO classification was A-1-a (stone fragments, gravel, and sand). AASTHO rated this soil as good to excellent for subgrade suitability in Table 4.3.

The 1 inch clean backfill used in Trenches D, C, and E had a USCS classification of GP (poorly graded gravel). The AASHTO classification of this soil was A-1-a (stone fragments, gravel, and sand). AASHTO rated this backfill as good to excellent for subgrade suitability.

Classification of Soils Excavated from the Trenches

Table 4.5 summarizes the soil classification test results for the soils removed during the trench excavation.

Soil from around Trench A was not sampled. However, during the excavation of the trench, sand from the previous utility cut restorations was encountered. This sand was classified as SW (well-graded sand). Soil removed from Trench B was classified as SW (clayey well-graded sand) by USCS. The AASHTO classification was A-1-b (stone fragments, gravel, and sand). The AASHTO in Tables 1.2 and 1.3 rated this soil a good to excellent subgrade material. Soil removed from Trench C was not tested as part of the Phase II report. The Phase I report did not record data for the soil removed from the trench. Soil removed from Trench D was soil classified as SC (clayey sand with gravel) by USCS. The AASHTO classification was A-6 (clayey soil). The AASHTO classification in Table 4.5 rated Trench D soil as poor to fair for subgrade soil. Soil removed from Trench E was classified as SC (clayey sand). The AASHTO classification was A-2-4 (silty and clayey gravel and sand). AASHTO rated this soil as good to excellent for subgrade suitability. Soil removed from Trench F was classified as CL (sandy lean clay). The AASHTO classifications in Table 4.5 were A-6 (clayey soil). AASHTO rated this soil as poor to fair for subgrade suitability in Table 4.5.

Standard Proctor Test Results

The Standard Proctor test was performed on the following soil samples: soils excavated from Trenches B, C, D, and E; supplement backfills for Trenches B, C, E, and F; and the 3/8-inch minus limestone from the City of Ames in summer 2007. Table 4.6 shows the results from the

Standard Proctor tests for the various soils excavated from the trenches and the suggested values given by NAVFAC (1988). This table shows that for all backfills, the maximum dry unit weights and optimum moisture contents fall within the range of typical values except for 3/8-inch minus backfill. In the laboratory, the Standard Proctor tests were performed for several moisture contents.

To compare the field-testing data to the laboratory data for 3/8-inch minus limestone, the Standard Proctor test results from 2007 were plotted on the relative density test results from 2005.

Relative Density Test Results

Relative density tests were performed on the following samples: 3/8-inch minus limestone, 1-inch minus limestone, and Trench A sand from previous cut, which was the backfill material the City of Ames used in the 1990s. Table 4.7 shows the relative density test results.

The 1-inch clean limestone results from the relative density tests were not considered accurate for three reasons. First, there were limited fines in the samples. Therefore, when more water was added to the sample, it did not affect the particle interaction. Second, the aggregate was too large for the mold available in the laboratory. Finally, the particles were angular and interlocked when the test was performed, preventing further compact. However, when the particles were removed from the mold it was possible to rearrange the particles in the mold to a denser configuration. During Phase I, SUDAS 1-inch clean material was tested. The results from Phase I were used to evaluate the performance of the trenches.

Table 4.7. Relative density test results for granular backfills

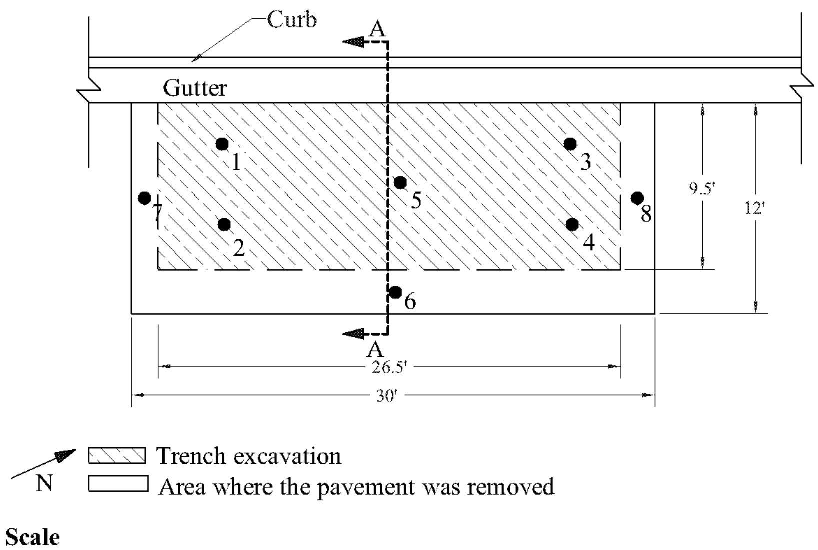
Sample	Maximum/Minimum dry unit weight and bulking moisture content from relative density testing		Range of maximum dry unit weights and optimum moisture contents from NAVFAC	
	$\gamma_{Max}/\gamma_{Min}$ (pcf)	Bulking moisture content (%)	γ_{Max} (pcf)	Optimum moisture content (%)
Trench A Sand from previous cut	104.7/100.0	2 to 6	110 to 130	9 to 16
Trench C Ames 3/8-inch minus limestone from summer 2005	140/99.0	4.0 to 8.0	110 to 125	11 to 16
Trenches D, E, and F 1-inch clean limestone	132.3/85.2	---	115 to 125	11 to 14

Field Test Results and Discussion for the Recommended Trenches

The field tests performed on the trenches were nuclear density and DCP tests.

Recommended Trench A

The DCP and nuclear density tests were performed during construction at five different test points for the third and fifth (top) lifts on August 8, 2007. On August 10, 2007 (when the trench was patched), DCP and nuclear density tests were also performed on the same five points within the trench and at three additional points where the pavement was removed (see Figure 4.30 for test point locations). For the test points within the trench, additional tests were performed August 8 and 10, 2007, to document changes in the backfill properties because the trench was left open for two days.



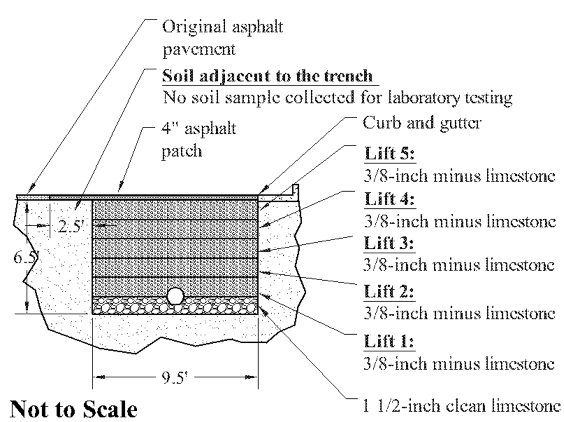


Figure 4.30. Location of test points in Trench A; cross-section of the trench

Nuclear Density Test Results

Table 4.8 summarizes the dry unit weights measured using the nuclear density test, and Table 4.9 summarizes the moisture content results from the nuclear density test. The probe depth was 6 inches.

The third and fifth lifts were constructed with 3/8-inch minus limestone. The limestone backfill was classified as SP-SM (poorly graded sand and silt). On August 8, 2007, the third lift had an average dry unit weight from the nuclear density test for the five points within the trench of 110.2 pcf and a moisture content of 6.5%. Based on laboratory tests, the backfill was placed at a relative density of 34%. This corresponded to loose compaction according to Table 1.6. However, moisture content at placement of the material was within the bulking moisture content range, increasing the collapse potential of the backfill.

On August 8, 2007, the fifth (top) lift had an average dry unit weight for the five points within the trench of 120.7 pcf and a moisture content of 6.0%. The fifth (top) lift was placed at 60% relative density, which corresponds to a medium dense compaction state according to Table 1.6. The moisture content of the backfill at the placement was within the bulking moisture content range of the backfill (i.e., 4.0% to 8.0%).

On August 10, 2007, before the patch was placed, the fifth (top) lift had an average dry unit weight for the five points within the trench of 122.3 pcf and a moisture content of 6.1%. Leaving the trench open for two days resulted in an increase of 1.4% in the dry unit weight. Based on the laboratory results, the backfill was placed at a relative density of 65%. This corresponded to a dense compaction. The moisture content of backfill after leaving the trench open for two days was within the bulking moisture content range, increasing its collapse potential.

The soil adjacent to the trench on August 10, 2007, had an average dry unit weight, from the nuclear density test, for the three points (6, 7, and 8) of 123.4 pcf and a moisture content of 5.0%.

Table 4.8. Dry unit weight results from the nuclear density tests on Trench A, with backfill having γmax of 139 pcf and γmin of 98 pcf

Location	Number of test points	Average dry unit weight (pcf)	Min/Max dry unit weight from field-testing (pcf)	Relative density (%)	Stan-dard devia-tion	Coefficient of variance (%)
Third lift	5	110.2	107.8/111.2	34	1.4	1.3
Fifth lift test points within the trench tested on August 8, 2007	5	120.7	117.1/124.5	60	3.3	2.7
Fifth lift test points within the trench tested on August 10, 2007	5	122.3	120.1/125.7	65	1.8	1.5
Test points in the soil adjacent to the trench on August 10, 2007	3	123.4	122.3/125.4	N/A	1.7	1.4

Table 4.9. Moisture content results from the nuclear density tests on Trench A

Location	Number of test points	Average moisture content (%)	Min/Max moisture content (%)	Bulking moisture content (%)	Standard deviation	Coefficient of variance (%)
Third lift	5	6.5	5.9/8.3	4.0 to 8.0	1.0	15.4
Fifth lift test points within the trench tested on August 8, 2007	5	6.0	5.6/6.2	4.0 to 8.0	0.3	5.0
Fifth lift test points within the trench tested on August 10, 2007	5	6.1	5.0/9.7	N/A	2.0	32.8
Test points in the soil adjacent to the trench on August 10, 2007	3	5.0	5.0/9.7	N/A	2.5	32.1

Figure 4.31 shows the results from the relative density laboratory testing compared with measured dry unit weight and moisture content using the nuclear density gauge. The figure also shows the collapse potential index for the 3/8-inch minus limestone. The figure clearly shows the backfill was placed at a relative density ranging from 27% to 69% with moisture contents ranging from 5.0% to 8.3%, which was within the bulking moisture content range. This shows the backfill was placed at average moisture content with the highest collapse potential. This increased the trench's susceptibility to settlement. The circles around each lift show on which lift the various test points occurred and that the moisture content and dry unit weights were similar within each lift. There is one outlier point; this point corresponded to test point 1 on lift 3.

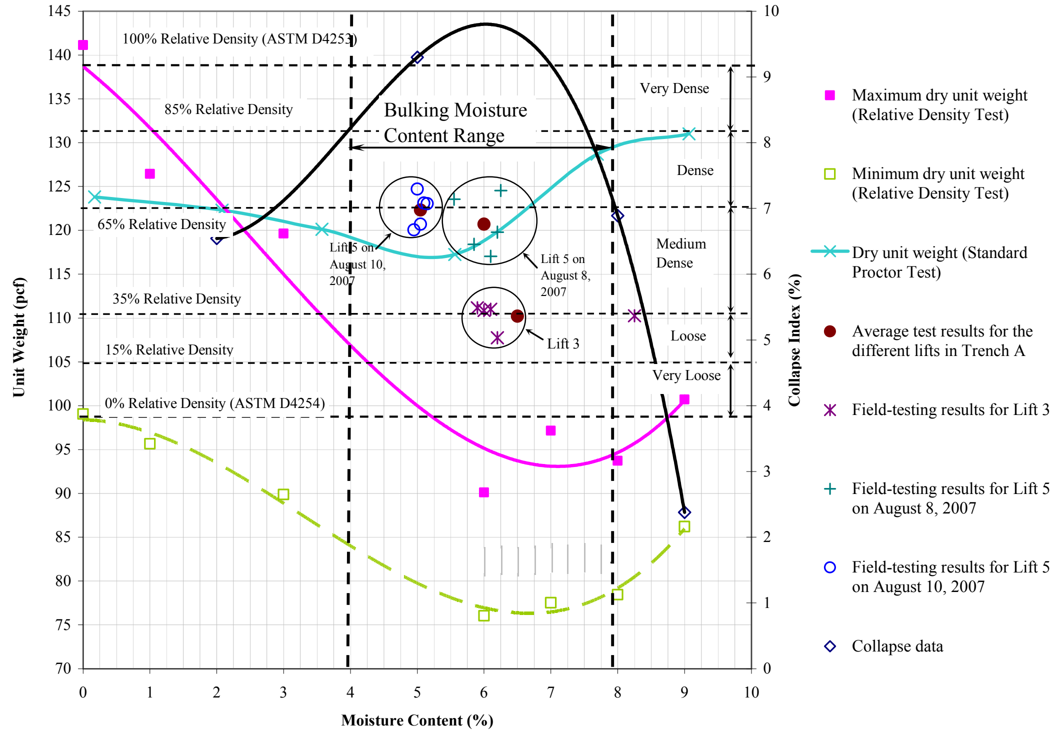


Figure 4.31. Relative density test results for 3/8-inch minus limestone with field-testing results for Trench A

DCP Test Results

The third and fifth lifts were constructed with 3/8-inch minus limestone. The limestone backfill was classified as SP-SM (poorly graded sand with silt). The typical CBR values were 10% to 40% for poorly graded sand from NAVFAC (1986) in Table 1.5.

Table 4.10 summarizes the average DCPI readings from the DCP tests for Trench A, and Table 4.11 summarizes the average CBR results from the DCP tests for Trench A. The DCPI was calculated for each depth of the profile, and then the CBR was estimated along the profile for each trench. The average CBR values were weighted values by distance (depth) from CBR profiles and were not calculated from the average DCPI. The same procedure was used for all DCPs performed on all trenches.

Table 4.11 shows the CBR values for the trench ranged from 8% for the third lift to 29% for the fifth (top) lift for the test points within the trench on August 10, 2007. Leaving the trench open for two days caused the CBR values from the DCP tests to increase from 17% to 29% for the test points within the trench, indicating an increase of 71%.

Table 4.10. DCPI results from the DCP tests for Trench A

Location	Number of test points	Depth of test (inches)	Average DCPI	Standard deviation	Coefficient of variance (%)
Third lift	5	26.5	61.1	58.6	96.1
Fifth lift test points within the trench tested on August 8, 2007	5	27.5	15.2	7.0	46.1
Fifth lift test points within the trench tested on August 10, 2007	5	26.6	8.7	3.0	33.8
Test points in the soil adjacent to the trench on August 10, 2007	3	26.1	47.7	26.3	55.1

Table 4.11. Average CBR results from the DCP tests for Trench A

Location	Number of test points	Depth of test (inches)	Average CBR (%)	Standard deviation	Coefficient of variance (%)
Third lift	5	26.5	8%	79.0	975.3
Fifth lift test points within the trench tested on August 8, 2007	5	27.5	17%	7.3	42.7
Fifth lift test points within the trench tested on August 10, 2007	5	26.6	29%	20.7	70.9
Test points in the soil adjacent to the trench on August 10, 2007	3	26.1	13%	20.4	156.9

For the third lift on August 8, 2007, the DCP test points within the trench had an average CBR value of 8%. Figure 4.32 shows the CBR values as a function of depth for lift 3. This figure shows a wide variation of the CBR values among the test points resulting from poor compaction. The CBR values range from 3% to 6.5% at the surface to 0.3% to 11.5% at the termination of the tests. The average CBR values range from 1% (point 5) to 14% (point 2). The tests ranged in depth from 22 inches to 28 inches below the elevation of lift 3.

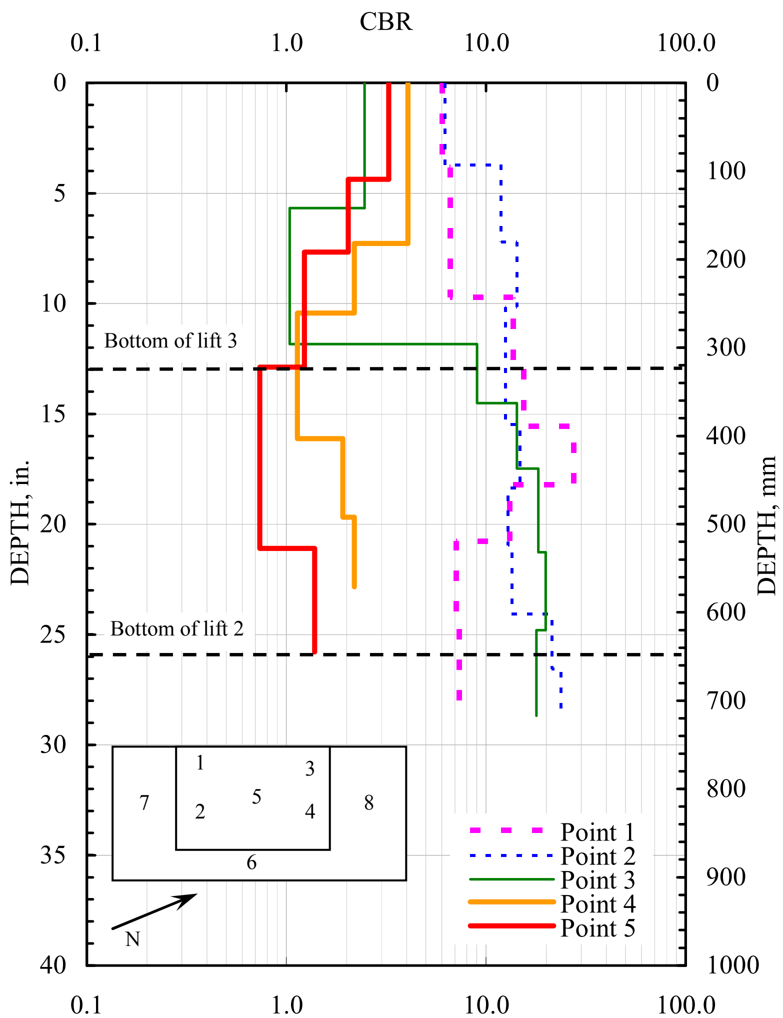


Figure 4.32. CBR results from the DCP test profiles for the third lift in Trench A (boundary locations are estimated based on the total depth of the trench and the number of lifts)

The fifth (top) lift was tested on August 8 and 10, 2007. The test points in the soil adjacent to the trench were tested on August 10, 2007. On August 8, 2007, the average CBR value within the trench was 17%. The CBR tests were conducted for a range of depths from 21 inches to 28 inches. At the surface of the lift the CBR values ranged from 5% to 12%. At the termination of the test the CBR values ranged from 20% to 47%. Figure 4.33 shows the CBR profiles for each test point.

When comparing the CBR profiles on August 8, 2007, to August 10, 2007, the CBR values at the surface had increased from an average of 7% to an average of 21%. The average values for the depth of the tests increased from 17% to 29%. The CBR values at the termination of the tests also increased but not to the same degree as at the surface. At all of the test points the CBR values increased with depth. The CBR profiles at each test point follow the same general profile through the depth of the profiles. This shows the compaction effect was evenly applied. When comparing Figures 4.32 and 4.33 for lifts 3 and 5, the compaction effort for lift 5 was more evenly applied and on the trench area was more effective through the depth of lift percentage, achieving better compaction (i.e., higher CBR) and less variability in CBR values.

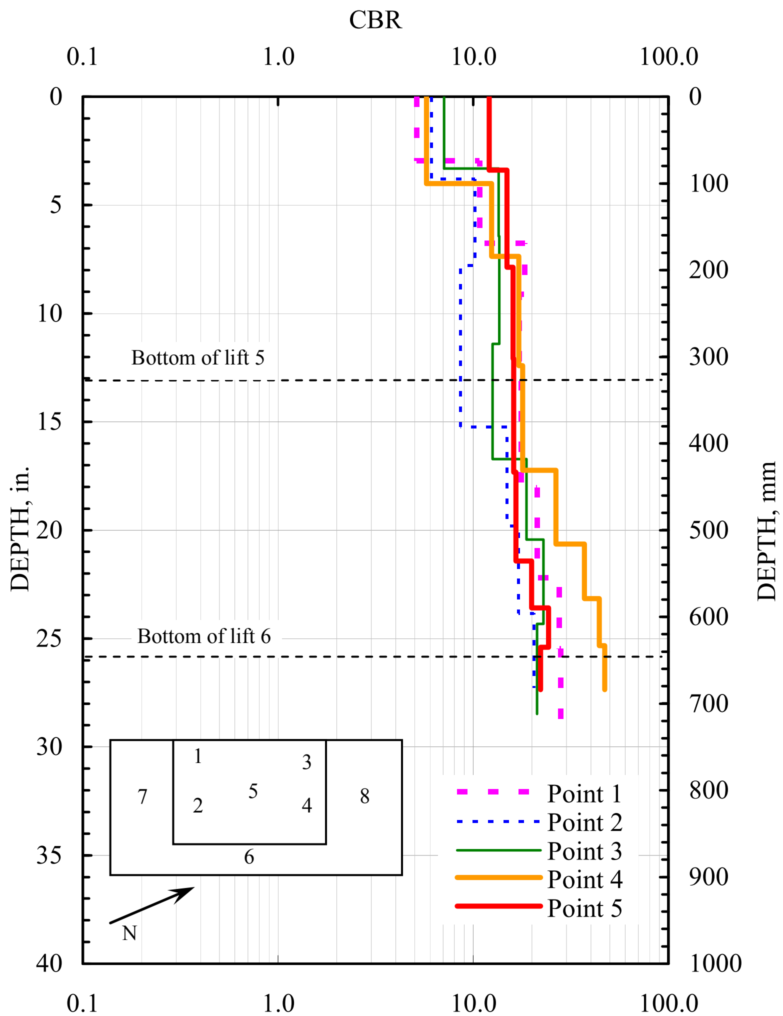


Figure 4.33. CBR results from the DCP test profiles for the fifth lift in Trench A on August 8, 2007

On August 10, 2007, the average calculated CBR value within the trench was 29% for the fifth lift. These tests ranged in depth from 25 inches to 27 inches (approximately the upper 2 feet of the trench, which affects the performance of the pavement the most). The figure shows at the surface the CBR values ranged from 16% to 24%. At the termination of the test, the CBR values ranged from 26% to 51%. Figure 4.34 shows the CBR profiles for all the test points.

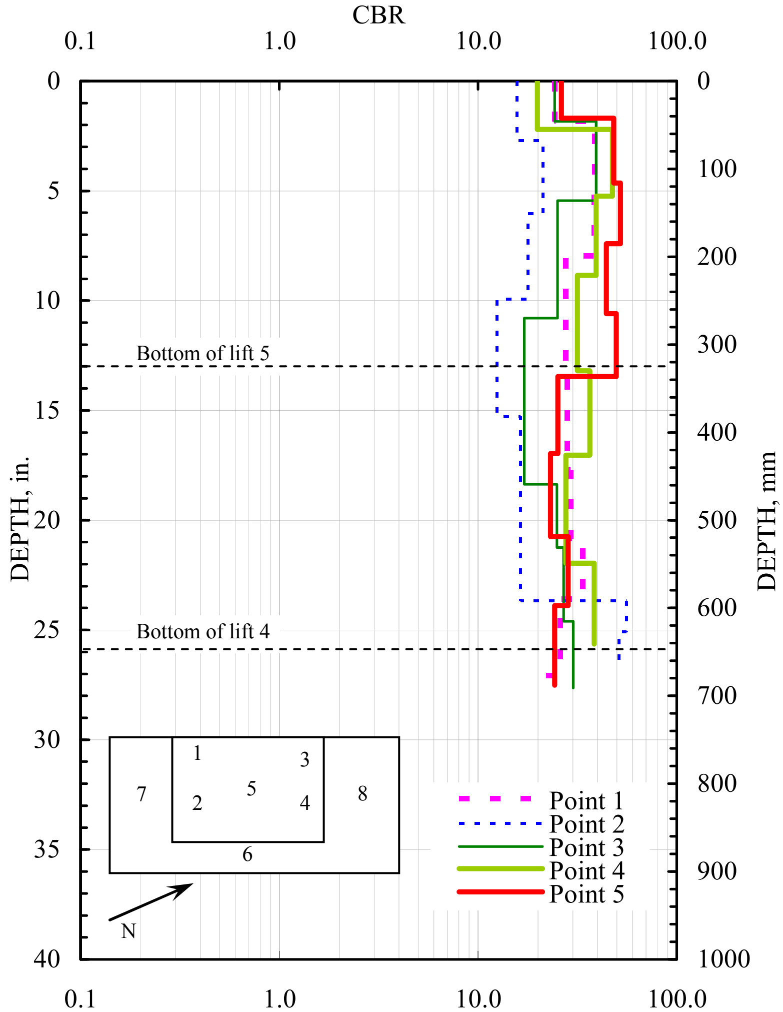


Figure 4.34. CBR results from the DCP test profiles for the fifth lift in Trench A on August 10, 2007

On August 10, 2007, the DCP test was conducted at three points in the soil adjacent to the trench. The depth of the CBR profiles ranged from 25 inches to 26 inches. For the top 2 feet, the CBR values ranged from 6% to 10%. At the surface, the CBR values ranged from 3% to 28%. At test points 7 and 8, the CBR values decreased with depth. This figure does not provide conclusive evidence that compacting around the trench improved the DCP values. Figure 4.35 plots the CBR values for each point as a function of depth for the test points outside the trench. When compared with compacted backfill, average CBR values in the trench within the top 2 feet show more than twice the CBR values of the natural soil around the trench. The average calculated CBR value within the trench was 13%.

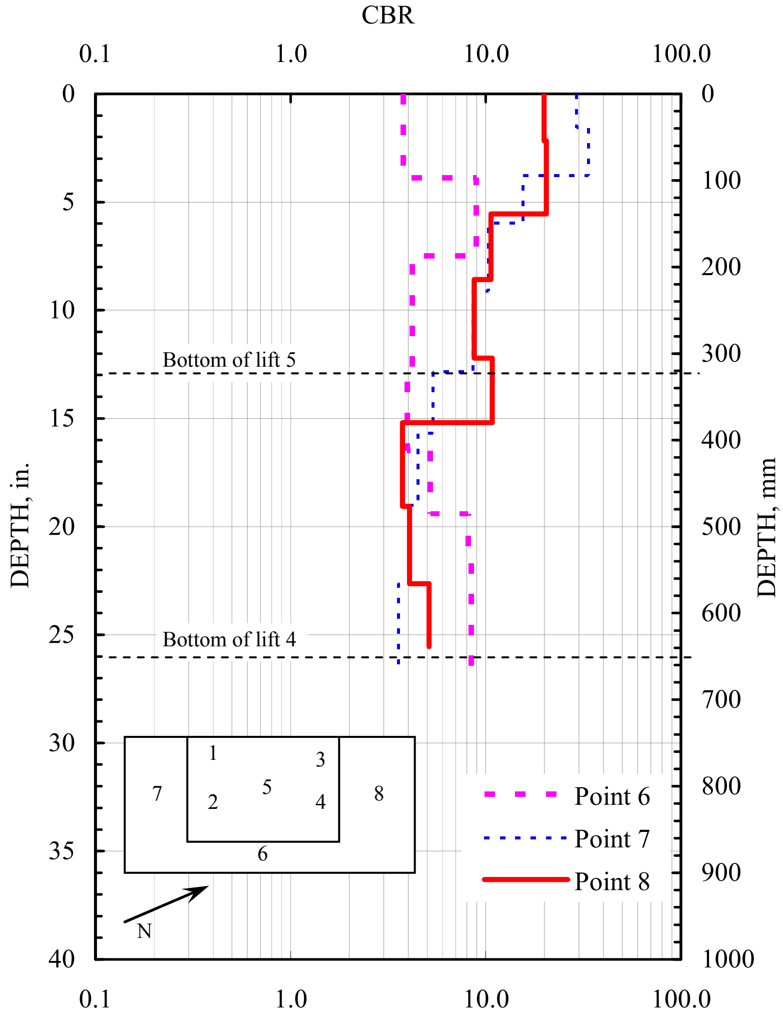


Figure 4.35. CBR results from the DCP test profiles for the soil adjacent to Trench A on August 10, 2007

FWD Test Results

To monitor the long-term performance of the constructed trenches, FWD testing was conducted. The FWD tests were performed to detect the zone of influence around the trench and associated weakening of the pavement and subgrade, as well as to monitor the change in trench response as a function of time. The surveys were conducted to measure the overall settlement of the trenches.

Falling weight deflectometer testing is often preferred over destructive testing methods because FWD testing is faster than destructive tests and does not entail the removal of materials. In addition, the testing apparatus is easily transportable. The FWD can either be mounted in a vehicle or on a trailer and is equipped with a weight and several velocity transducer sensors. To perform a test, the vehicle is stopped and the loading plate (weight) is positioned over the desired location (see Figure 2.9a). The sensors are then lowered to the pavement surface, and the weight is dropped at the desired load level. The advantage of an impact load response measuring device over a steady-state deflection measuring device is that it is quicker, the impact load can be easily varied, and it more accurately simulates the transient loading of moving traffic. Sensors located at specific radial distances (see Figure 2.9b) monitor the deflection history. The deflections measured at radial distances away from load form the deflection basin.

The Iowa DOT performed FWD testing for Trench A on November 5, 2007 (three months after construction). A second FWD test on June 25, 2008, is given in Figure 4.36. Figure 4.37 compares both dates and their deflection under the plate load for 15-kip drops. Note that 15-kip load level was chosen (instead of standard 9 kips) since the heavier drops give a better response of the backfill in the trench. It is important to note that the differences between the FWD deflections on the pavement surface not only result from the relative strength of the compacted subgrade due to seasonal changes (June vs. November) but also from the relative moduli of the hot mix asphalt at two different temperature levels. Figure 4.38 shows the FWD testing locations for Trench A. In Figure 4.39, the deflections for the 6-kip and 15-kip loads were labeled for key points. In these figures only the maximum deflections under the plate load center (D_0) are reported. The backfill had a stiffer response than the surrounding subgrade soil, which reflects the higher CBR values estimated from the DCP tests. The zone of influence was still present for Trench A. Figure 4.39 shows the zone of influence was detected on the northeast side of the trench with no distinct zone of influence on the southwest side. Based on Figures 4.3 and 4.4 (see above), construction equipment was located on both the northeast and southwest sides of the trench. This resulted in the trench having a softer response than the soil to the northeast of the trench, as seen by higher deflection of 65.3 mils on the southwest compared with 48.9 mils on the northeast side of the trench. The maximum deflection (D_0) from the 15-kip load at the center of the trench was 20.3 mils. The average deflection at the inside edges of the trench was 29.1 mils. The deflections of the subgrade within the trench averaged 26.1 mils. The surrounding soil that was compacted before the placement of the patch had an average deflection of 40.3 mils, while pavement away from the zone of influence in the soil around the trench that was not compacted had an average of 52.7 mils. This showed compacting the soil surrounding the trench improved its response. However, the compaction did not extend far enough to eliminate the zone of influence that formed when the lateral support was lost during excavation. The zone of influence extended about 4 feet beyond the trench (2 feet beyond the patch limits).

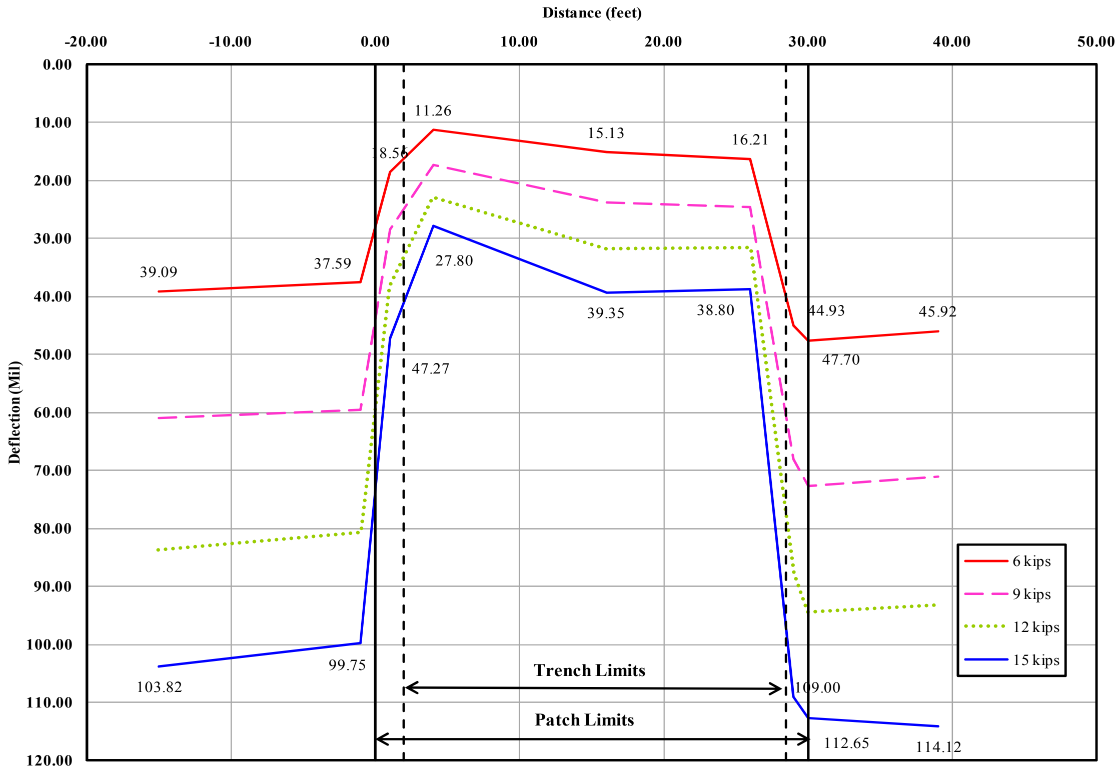


Figure 4.36. Falling weight deflectometer test results for Trench A on June 25, 2008, at a temperature of 99.5 ºF

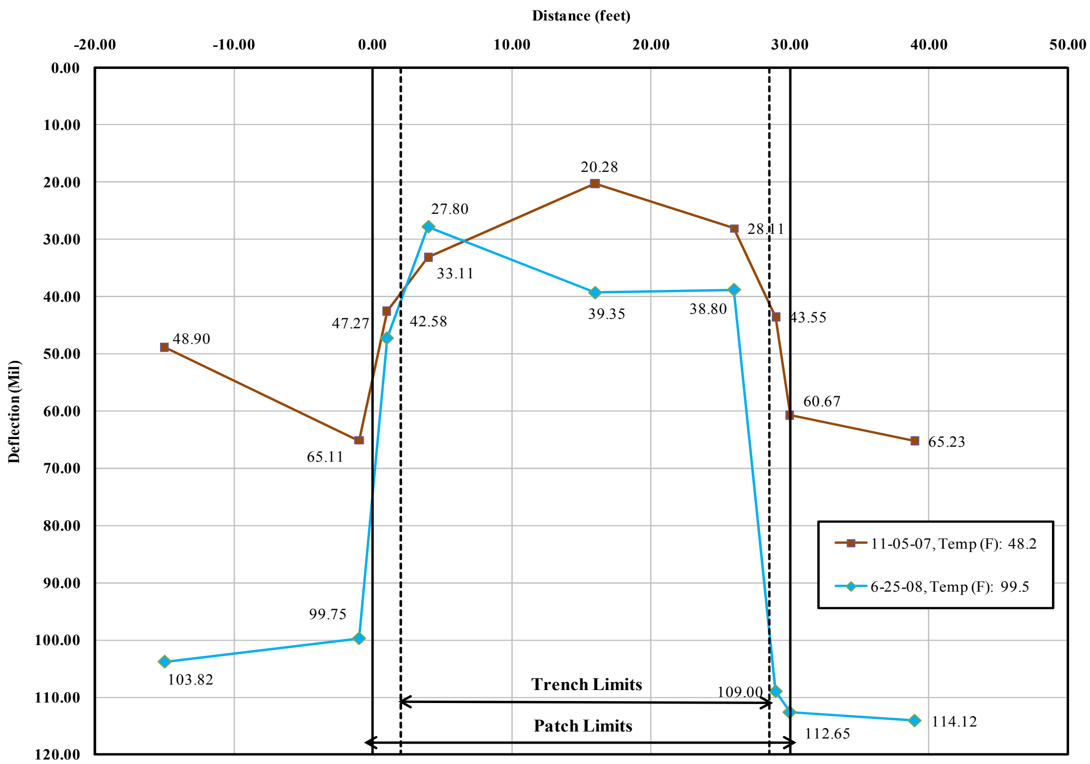


Figure 4.37. Falling weight deflectometer test results for Trench A (15 kip)

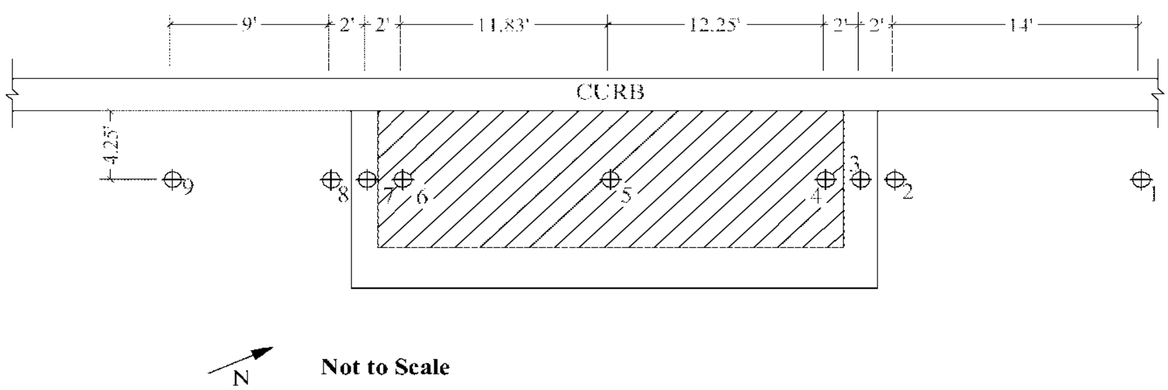


Figure 4.38. Falling weight deflectometer test locations for Trench A

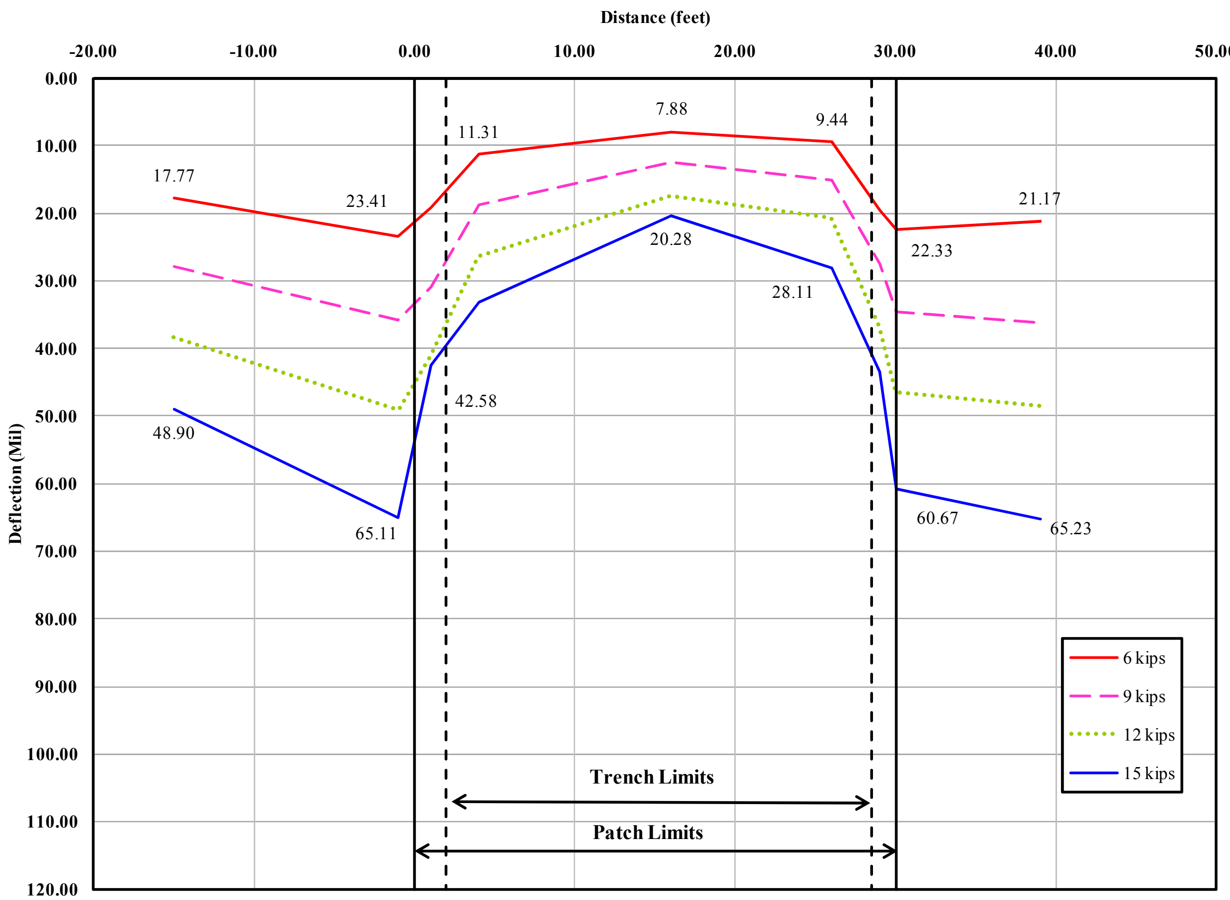


Figure 4.39. Falling weight deflectometer test results for Trench A on November 5, 2007, at a temperature of 48.2 ºF

Post-Construction Elevation Survey

The post-construction elevations were measured on September 25, 2007; March 19, 2008; June 10, 2008; August 20, 2008; November 20, 2008; and March 13, 2009.

A grid was placed across the trenches and then selected grid lines were extended outward from the trenches onto the original pavement. The extended lines were to monitor the settlement of the original pavement. The elevations were determined at each point.

At all sites, the dome bolt on fire hydrants was used for the benchmark (see Figure 4.40), which was given an elevation of 100 feet.

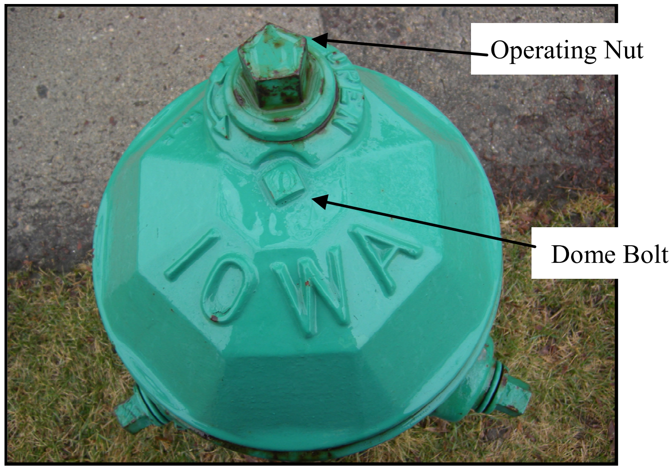


Figure 4.40. Dome bolt on fire hydrants

Trench A was surveyed with 43 grid points, as shown in Figure 4.41. The hydrant was located northeast of the restoration at the intersection of McKinley Drive and Van Buren Avenue. When the patch was originally placed, the difference between the highest and lowest elevation was 8.4 inches. During the March survey, the difference between the highest and lowest elevation increased to 8.54 inches. The patches had increased the difference between the highest and lowest elevations. The maximum uplift was 0.72 inches at survey points 2 and 13. The minimum uplift was 0.12 inches at survey points 26, 30, and 35.

Figure 4.42 shows the elevation profiles and the settlement of Trench A for the past five surveys. This shows the southwest edge of the trench experienced more uplift than other portions of the trench.

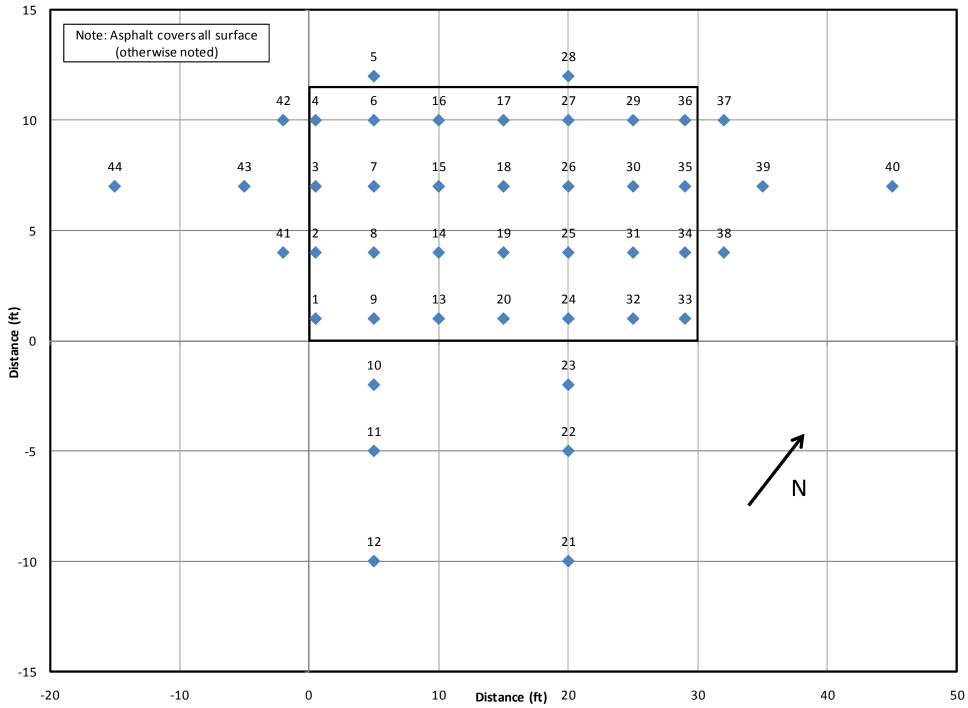


Figure 4.41. Survey locations for Trench A

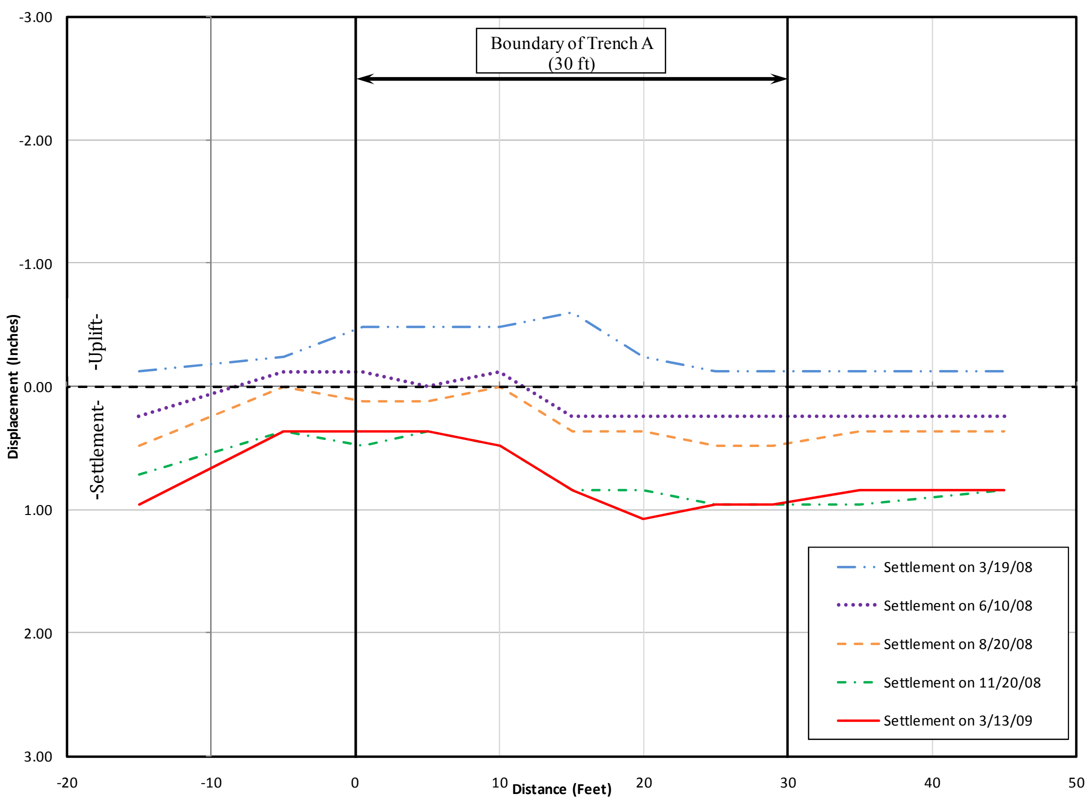


Figure 4.42. Settlement along the center line of Trench A (points 44, 43, 3, 7, 15, 18, 26, 30, 35, 39, 40)

Comparison of Field-Testing Results to Long-Term Monitoring

Figure 4.43 shows the FWD test results and the averaged field-testing data for test points near the FWD test locations. The CBR values calculated for the top 2 feet of the backfill and subgrade soil correspond with the response of the FWD response of patch for Trench A. At the center of the trench, the backfill and patch system provided the stiffest response (20.28 mils for the 15-kip load). This point corresponded to test point 5 for the field tests performed during construction. At test point 5, the average dry unit weight was 127.4 pcf and the CBR for the top 2 to 3 feet was 22%. At the southwest edge of the trench, test points 1 and 2 were in the vicinity of FWD test point number 6, the average dry unit weight was 120.4 pcf, and the CBR value was 25% for the top 2 feet. At the northeast edge of the patch, test points 3 and 4 were near the FWD test point 4. The average dry unit weight was 123.1 pcf, and the average CBR value was 12% for the top 2 feet.

For the test points located in the soil adjacent to the trench (test points 7 and 8 in Figure 4.43), the deflections were 41.17 mils at test point 7 and 38.89 mils at test point 8. The average dry unit weights at tests points 7 and 8 were 125.4 pcf at a moisture content of 9.7% and 122.3 pcf at a moisture content of 5.0%, respectively.

The same conclusion can be drawn when comparing the CBR values for the backfill and the subgrade soil surrounding the trench, where DCPs show an average value of 36% for backfill and the surrounding soil had an average value of 13%.

In Figure 4.44, the settlements measured between the summer survey after construction and the early survey were plotted with the FWD deflections. The figure shows that the southwest side had uplifts even though it had smaller deflections compared to the northeast side. This does not provide a correlation between settlement and FWD testing results.

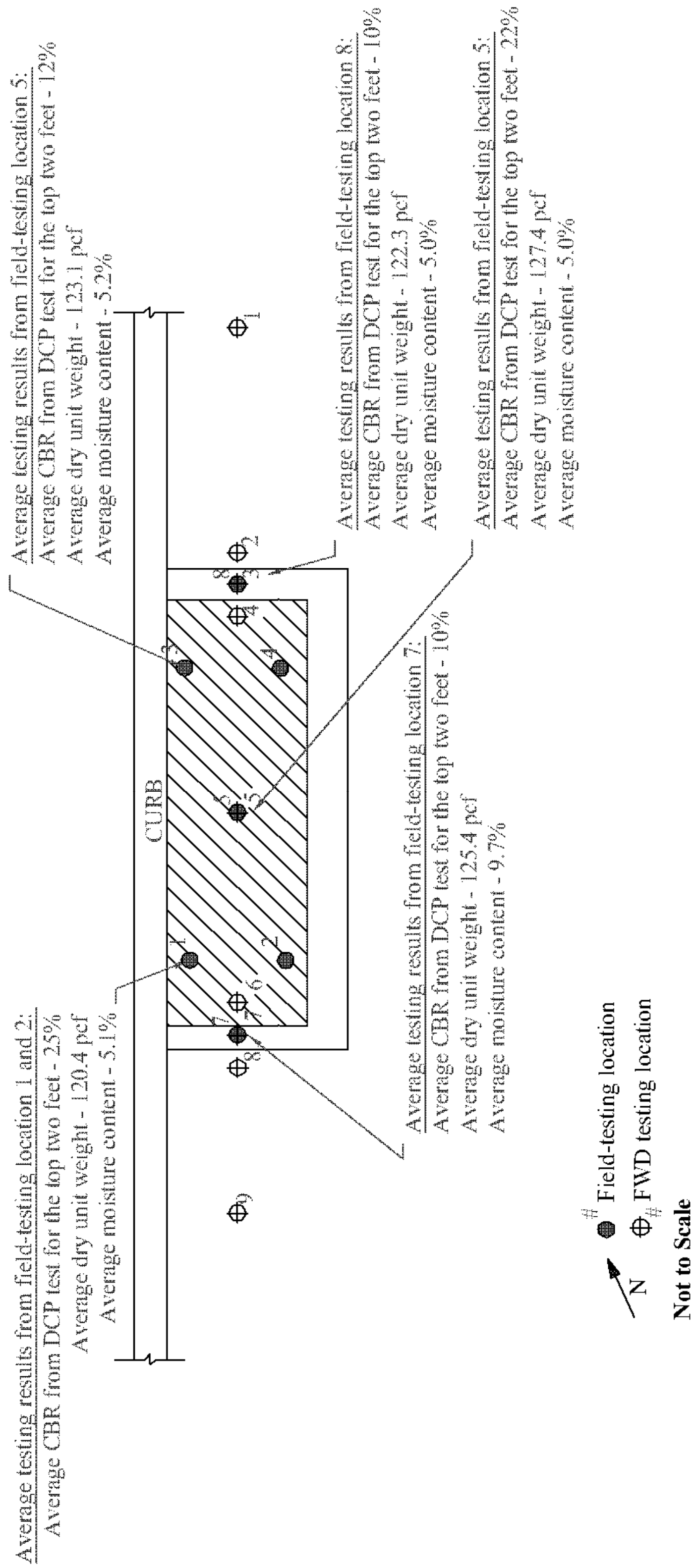


Figure 4.43. Comparison of CBR values, dry unit weights, and FWD testing results for Trench A

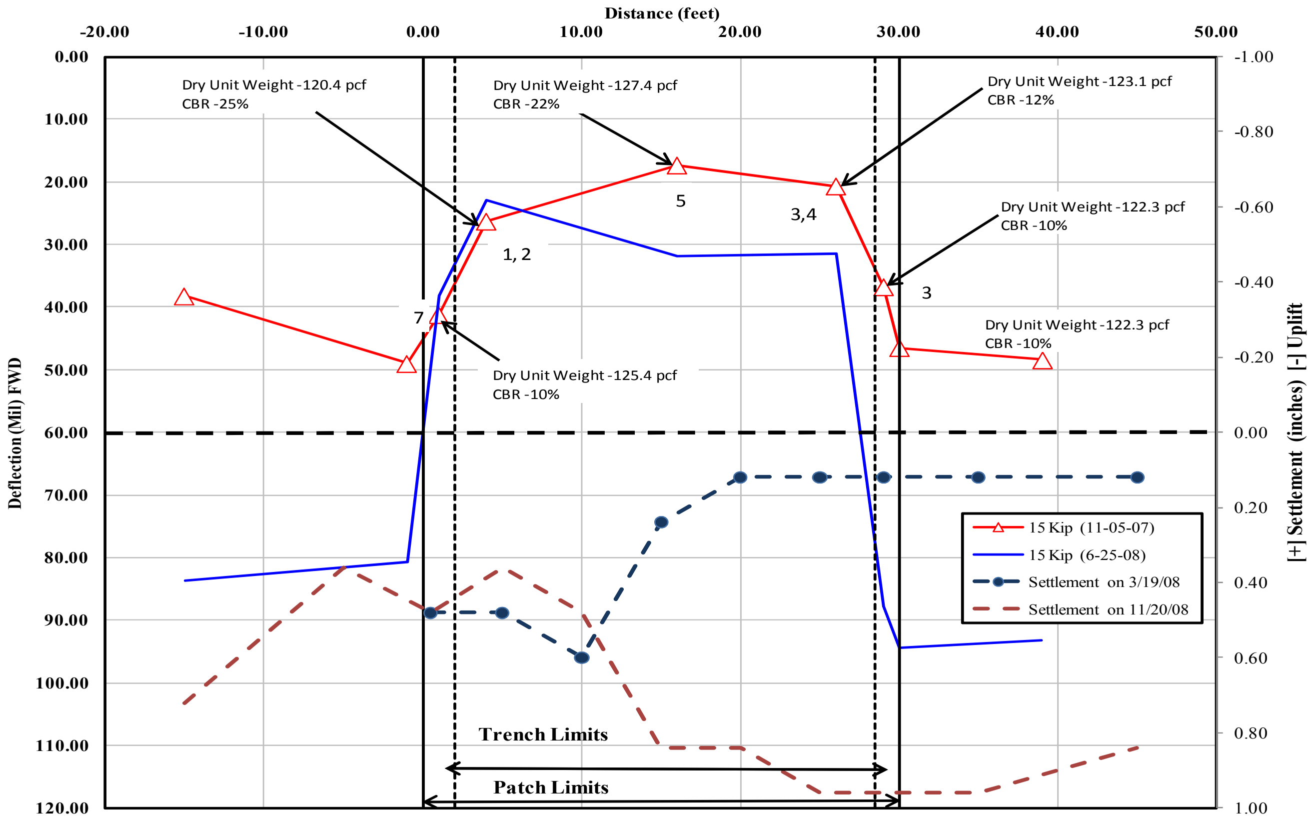


Figure 4.44. Falling weight deflectometer test (15 kip) and settlement for Trench A

Summary

- The granular backfill was placed at moisture contents ranging from 5.0% to 8.3%, which was mostly within the bulking moisture content range (i.e., 4.0% to 8.0%) for 3/8-inch minus backfill.
- The dry unit weights of the backfill within the trench ranged from 107.8 pcf to 124.7 pcf. For the soil surrounding the trench, the dry units ranged from 122.3 pcf to 122.5 pcf.
- The relative density of the 3/8-inch minus backfill within the trench ranged from 34% to 65% between the loose and dense compaction.
- The CBR values calculated from the DCP test for the top 2 feet ranged from 20% to 35%. The CBR values of the surrounding soil for the top 2 feet ranged from 13% to 21%.
- Leaving the trench open for two days resulted in a 1.7% increase in the density of the backfill.
- The FWD response and measured deflections across the trench and adjacent soil reflected the CBR values using the DCP test. Locations with higher CBR values show smaller deflections on the FWD test.
- The FWD testing indicated the zone of influence extended 4 feet beyond the trench (2

feet beyond the patch). Compacting the soil around the trench helped recover some of the stiffness that may have been lost when the trench was open during construction; however, the compaction needs to be extended further to 4 feet beyond the excavation.

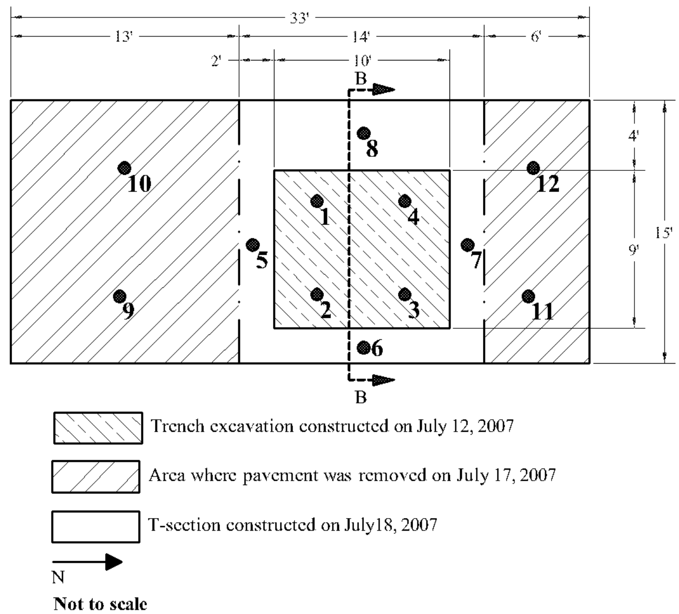
- Compared with the trenches constructed in Phase I, the backfill provided a stiffer response; however, the zone of influence was still present.

Recommended Trench B

At this site, nuclear density and DCP tests were performed during construction. On July 13, 2007, the tests were performed at four test points (points 1, 2, 3, and 4 in Figure 4.45) within the trench on the third and fifth (top) lifts. The trench was then left open for five days. On July 17, 2007, the pavement around the trench was removed. The pavement was removed beyond the extent of the trench because the surrounding pavement was damaged.

On July 18, 2007, the fifth lift was removed during the construction of the T-section. The T-section was completely backfilled with one lift. The replaced fifth (top) lift was tested at the same four points as the previous lifts as well as at four additional points (5, 6, 7, and 8 in Figure 4.45) in the T-section. A patch was not placed on the trench on July 18 because of time constraints. During the evening of July 18, 2007, the City of Ames received 1½ inches of rain. The trench was left open for an additional seven days after the T-section was constructed to allow the backfill to dry.

On July 25, 2007, the replaced fifth lift was retested at the same eight points used in previous testing as well as four additional points (9, 10, 11, and 12 in Figure 4.45) in the soil adjacent to the trench. This testing was performed to compare the changes in the backfill properties before it rained to after it rained and dried for seven days.



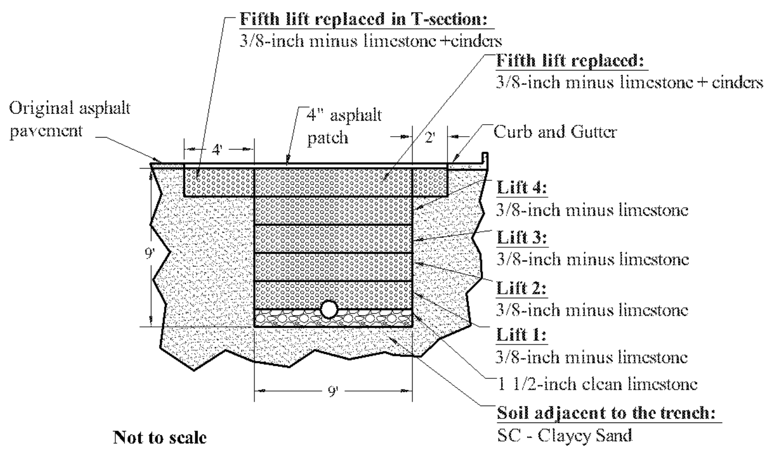


Figure 4.45. Location of test points in Trench B; cross-section of the trench

Nuclear Density Test Results

Tables 4.12 and 4.13 summarize the dry unit weights and moisture content measurements from the nuclear density testing, which was performed at a depth of 6 inches.

Figure 4.46 shows the relative density test results for 3/8-inch minus limestone with the average field-testing results for lifts 3 and 5.

The third and fifth lifts were constructed with 3/8-inch minus limestone. The limestone backfill was classified as SP-SM, poorly graded sand and silt. The replaced fifth (top) lift was constructed with two mixtures of 3/8-inch minus limestone, Backfill No. 1 and Backfill No. 2. The backfill was classified as SC – Clayey sand. The third lift for the four test points (1, 2, 3, and 4) had an average moisture content of 6.6% and a dry unit weight of 112.9 pcf. Lift 3 was placed at 41% relative density. According to laboratory testing, all backfill was placed at the upper boundary of the bulking moisture content.

The fifth (top) lift's four test points had an average dry unit weight of 118.2 pcf and a moisture content of 5.1%. Based on laboratory testing, the fifth lift was compacted to 54% relative density, at moisture content within the bulking moisture content found in laboratory testing. This lift was removed when the T-section was constructed.

The four test points within the trench (points 1, 2, 3, and 4) for the replaced fifth (top) lift after it rained (July 25, 2007) had an average moisture content of 7.0% and an average dry unit weight of 115.3 pcf with a reduction in dry unit weight of 2%. Based on laboratory testing results, the field placements were at 104% and 94% of the maximum density from the Standard Proctor test for Backfills No. 1 and 2, respectively. This was an increase of 96.8% and 87.5% of Standard Proctor from before it rained for Backfill No. 1 and Backfill No. 2. The total increase in the dry unit weights was 7.3%. This increase was larger than in other trenches, which were left open for several days. This large increase may be attributed to the change of moisture content increase due to the rain event.

The four test points in the T-section (points 5, 6, 7, and 8) of the replaced fifth (top) lift after it rained had an average moisture content of 7.9% and an average dry unit weight of 108.7 pcf. Based on laboratory test results, the backfill was placed at 98% of Standard Proctor for Backfill No. 1 and 88.5% of Standard Proctor for Backfill No. 2. These values were an increase of 92% and 83% of Standard Proctor for Backfill No. 1 and Backfill No. 2, respectively. The total increase in the dry unit weight was 7.2% for the backfill within the trench. The four test points in the soil adjacent to the trench (points 9, 10, 11, and 12) after it rained had an average moisture content of 8.6% and an average dry unit weight of 117.5 pcf. The soil was at 91% of Standard Proctor. Leaving the trench open resulted in a 6.1% increase in the dry unit weight of the soil in the T-section. The moisture content was below the optimum moisture content. However, the soil adjacent to the trench was consolidating over time (since the road was previously paved).

Table 4.12. Dry unit weight results from the nuclear density tests on Trench B, with backfill having γmax of 139 pcf and γmin of 98 pcf

Location	Number of test points	Average dry unit weight (pcf)	Min/Max dry unit weight from field testing (pcf)	Relative density (%)	Standard deviation	Coefficient of variance (%)
Third lift 3/8 inch	4	112.9	110.0/116.0	41.16	2.3	2.0
Fifth lift 3/8 inch	4	118.2	114.9/120.9	54.35	1.7	1.4
Replaced fifth lift *after rain event* for tests within the trench Backfill No. 1 and 2	4	115.3	111.0/117.0	N/A	2.4	2.0
Replaced fifth lift *after rain event* for tests in the T-section Backfill No. 1 and 2	4	108.7	106.7/110.8	N/A	2.1	1.9
Replaced fifth lift *after rain event* for tests in the soil adjacent to the trench Backfill No. 1 and 2	4	117.5	115.1/123.0	N/A	3.7	3.1

Table 4.13. Moisture content results from the nuclear density tests on Trench B

Location and material	Number of test points	Average moisture content (%)	Min/Max moisture content (%)	Bulking moisture content (%)	Standard deviation	Coefficient of variance (%)
Third lift	4	6.6	6.2/7.1	4.0 to 8.0	2.3	34.8
Fifth lift	4	5.1	4.8/5.5	4.0 to 8.0	0.2	2.9
Replaced fifth lift *after rain event* for tests within the trench	4	7.0	5.6/8.2	N/A	1.2	17.1
Replaced fifth lift *after rain event* for tests in the T-section	4	7.9	7.4/8.6	N/A	0.5	6.8
Replaced fifth lift *after rain event* for tests in the soil adjacent to the trench	4	8.6	7.0/11.0	N/A	1.7	19.8

Figure 4.46 shows the relative density testing with the field-testing results. The figure shows lifts constructed with 3/8-inch minus limestone were at a medium density with average moisture content of 5% to 7%, relative density of 35% to 65%, and collapse index of 5.5 to 7.

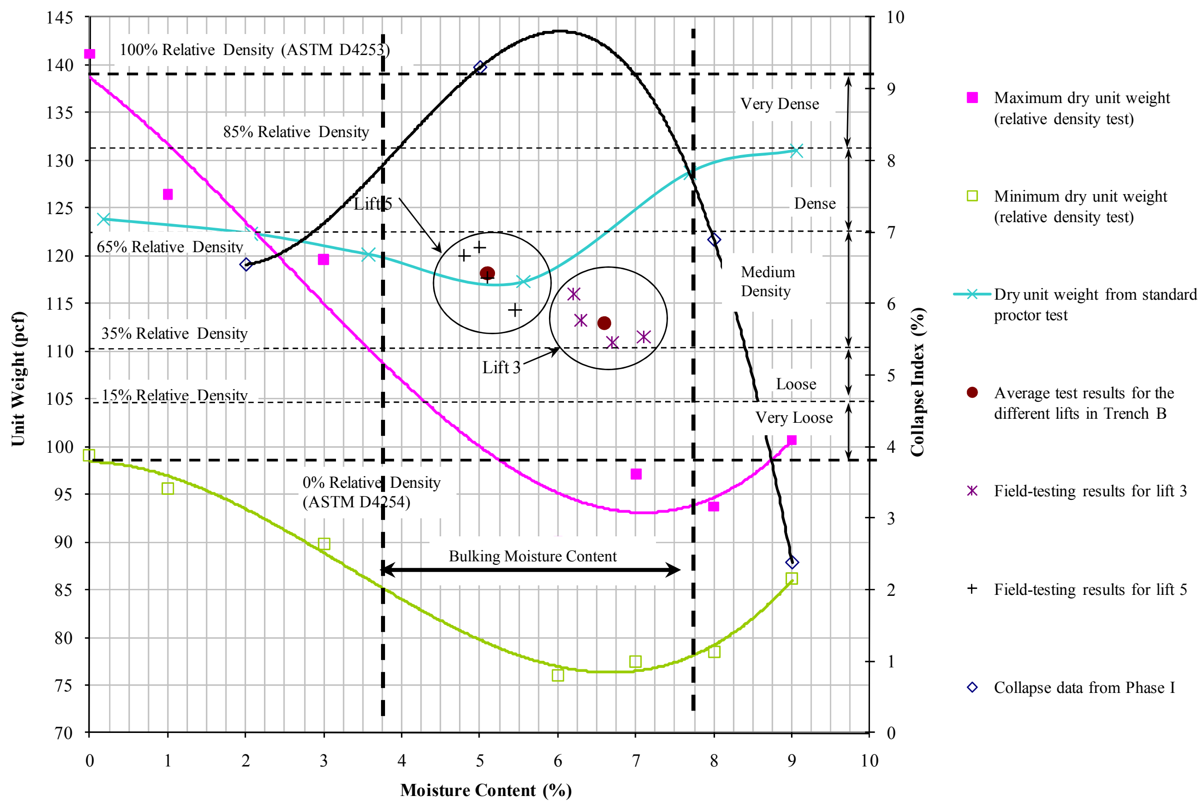


Figure 4.46. Relative density test results for 3/8-inch minus limestone with field-testing results for Trench B

DCP Test Results

Table 4.14 summarizes the DCPI readings from the DCP tests for Trench B, and Table 4.15 summarizes the average CBR results from the DCP tests for Trench B.

The third and fifth lifts were constructed with 3/8-inch minus limestone. The limestone backfill was classified as SP-SM, poorly graded sand and silt. The replaced fifth (top) lift was constructed with mixtures of 3/8-inch minus limestone, Backfill No. 1, and Backfill No. 2. The backfill was classified as SC, clayey sand.

Table 4.14. DCPI results from the DCP tests for Trench B

Location	Number of test points	Average depth of tests (inches)	Average DCPI	Standard deviation	Coefficient of variance (%)
Third lift	4	27.0	15.0	7.3	48.7
Fifth lift	4	26.7	16.6	9.0	54.2
Replaced fifth lift *after rain event* for tests within the trench	4	27.1	56.2	49.3	87.7
Replaced fifth lift *after rain event* for tests in the T-section	4	33.6	48.0	53.6	111.7
Replaced fifth lift *after rain event* for tests in the soil adjacent to the trench	4	32.3	63.2	34.7	54.9

Table 4.15. Average CBR results from the DCP tests for Trench B

Location	Number of test points	Average depth of tests (inches)	Average CBR (%)	Standard deviation	Coefficient of variance (%)
Third lift	4	27.0	16	7.4	46.3
Fifth lift	4	26.7	20	8.0	40.0
Replaced fifth lift *after rain event* for tests within the trench	4	27.1	8	48.7	608.7
Replaced fifth lift *after rain event* for tests in the T-section	4	33.6	1	2.6	260.0
Replaced fifth lift *after rain event* for tests in the soil adjacent to the trench	4	32.3	4	3.6	90.0

Figure 4.47 shows the CBR values for four test points as a function of depth for the third lift. The third lift had an average CBR value from the DCP of 16% for an average depth of 27.0 inches.

At the surface, the CBR values ranged from 4% to 8%. At the termination of the tests, the CBR values ranged from 15% to 26%. As the DCP test penetrated into the lift, the CBR values increased. The CBR profile for the third lift for the four test points increased slightly with depth. This indicates there was an increase in stiffness with depth. The four test points had lower standard deviations and were banded over a narrow range throughout the depth of the test.

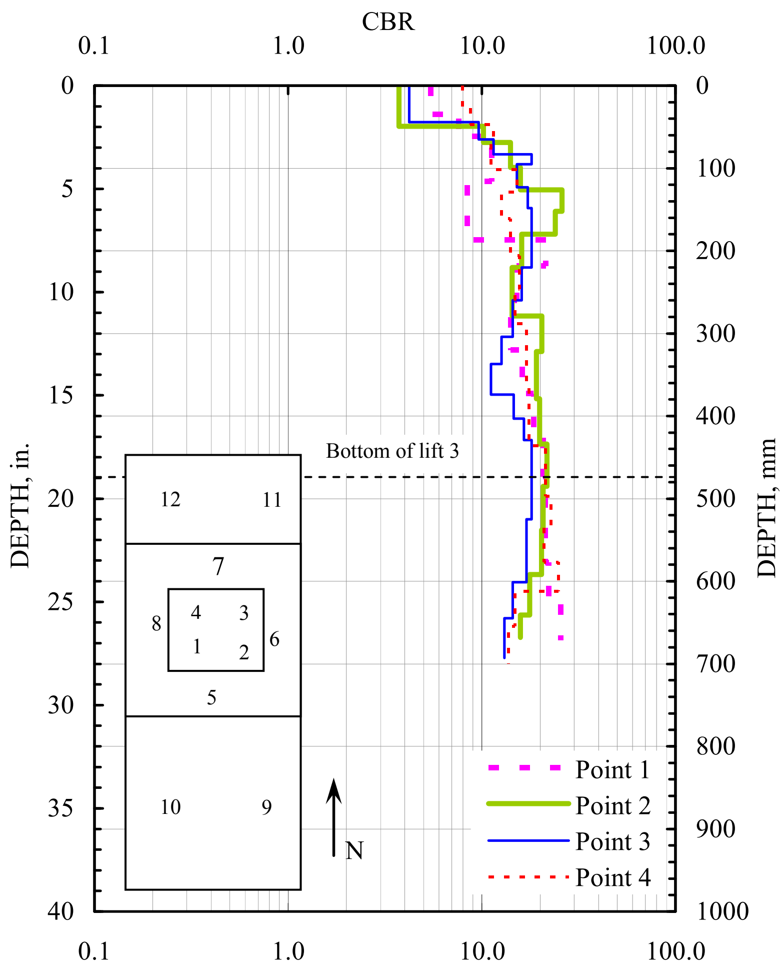


Figure 4.47. CBR results from the DCP test profiles for the third lift in Trench B

Figure 4.48 plots the calculated CBR values as a function of depth for the fifth lift. The DCP test had an average CBR value of 20%. At the surface, the CBR values ranged from 3% to 6%. At the termination of the tests, the CBR values ranged from 6% to 23%. The CBR values increased with depth, and each test point followed the same generalized pattern over the depth of the profile.

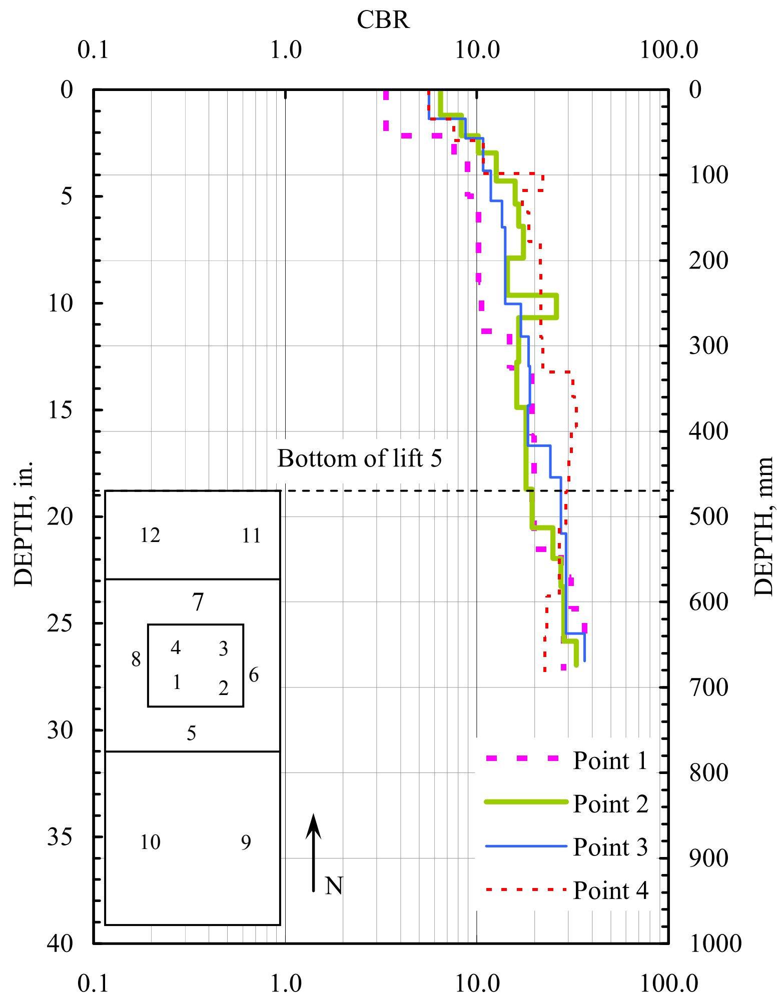


Figure 4.48. CBR results from the DCP test profiles for the fifth lift in Trench B

Figure 4.49 plots the CBR values for each point as a function of depth for the four test points within the trench for the replaced fifth lift after it rained. The four test points in the T-section region had an average CBR of 8% for 27.1 inches. At the surface of the lift, the CBR values ranged from 1% to 3%. At the termination of the tests, the CBR values ranged from 13% to 20%. The CBR profile for the test points within the trench had the same shape throughout the depth of the profile.

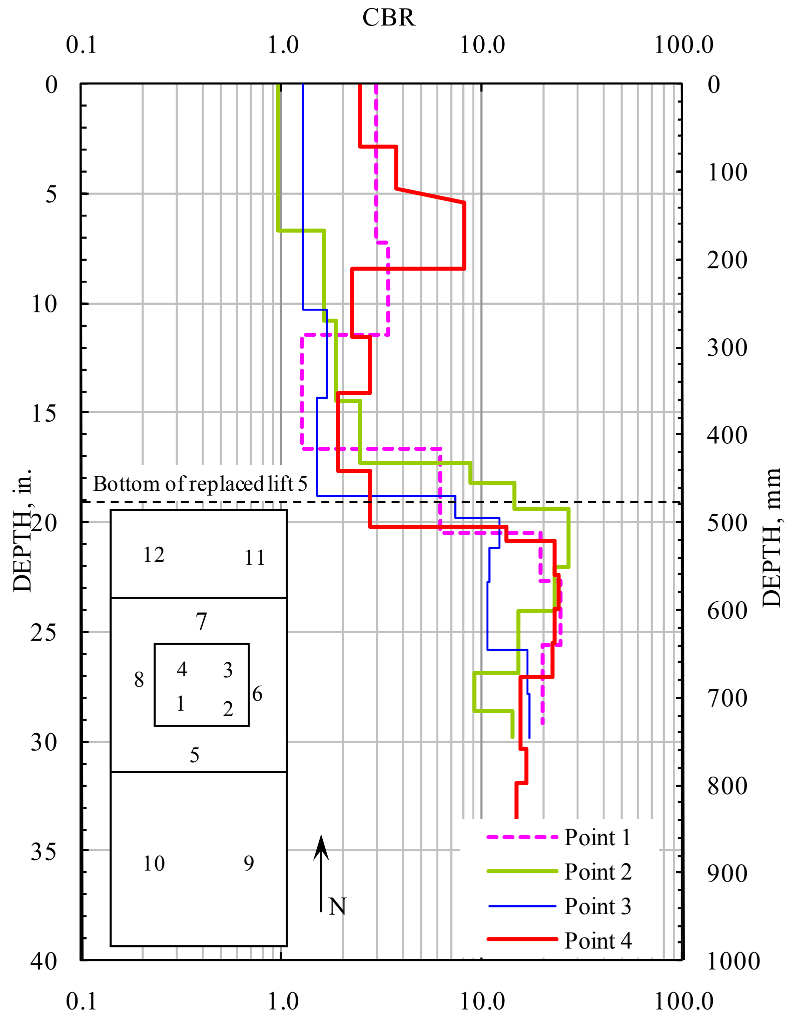


Figure 4.49. CBR results from the DCP test profiles for the replaced fifth lift of Trench B, after it rained

Figure 4.50 plots the CBR profiles for the four test points as a function of depth for the test points in the T-section for the final lift after it rained. For the replaced fifth lift after it rained, the four test points in the T-section had an average CBR value of 1% for 33.6 inches. At the surface of the lift, the CBR values ranged from 1% to 2%. At the termination of the test, the CBR values ranged from 1% to 5%. The CBR profiles for test points 5 and 6 increased until a depth of about 20 inches. After 20 inches, the CBR profiles decreased. This may indicate the effective depth of the construction equipment and technique was approximately 20 inches.

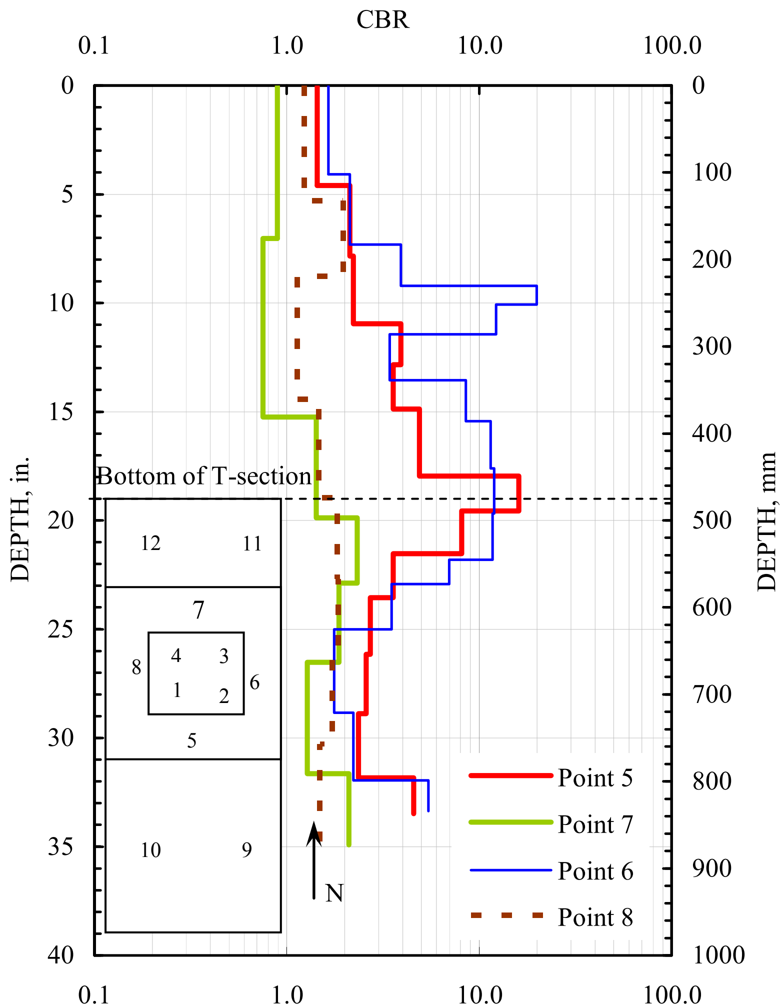


Figure 4.50. CBR results from the DCP test profiles for the replaced fifth lift of the T-section in Trench B, after it rained

Figure 4.51 plots the CBR values for each point as a function of depth for the test points in soil adjacent to the trench after it rained. After it rained, the four test points in the soil adjacent to the trench had an average calculated CBR value of 4% for 32.3 inches. At the surface, the CBR values ranged from 1% to 5%. At the termination of the tests, the CBR values ranged from 2% to 8%.

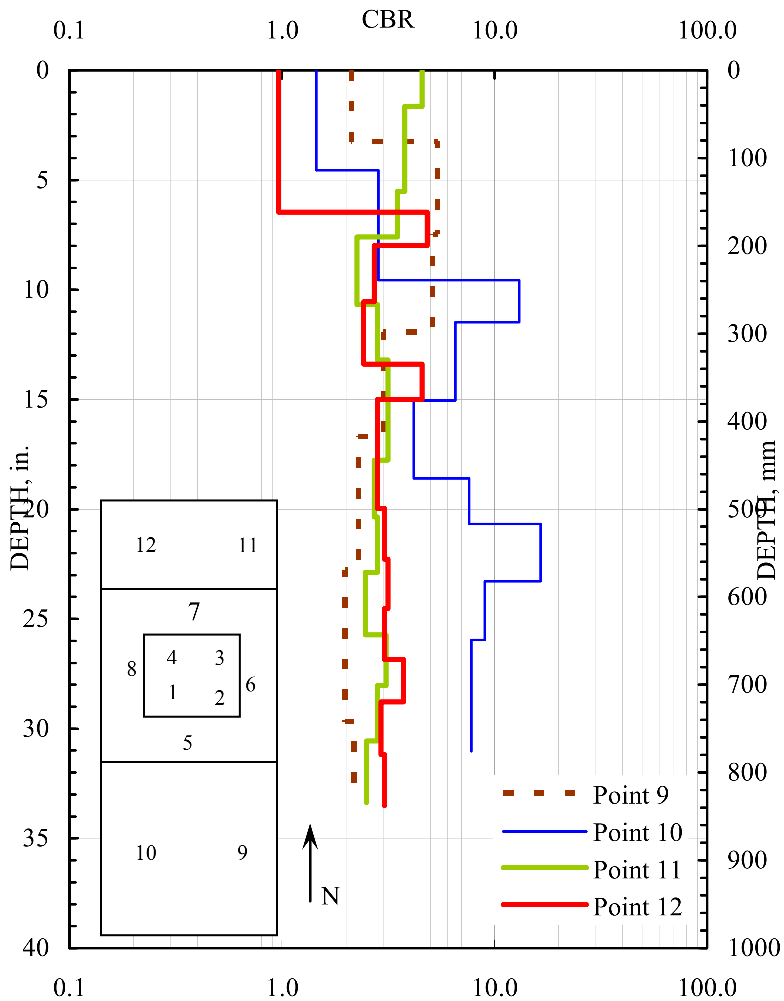


Figure 4.51. CBR results from the DCP test profiles for the soil adjacent to Trench B, after it rained

FWD Test Results

Trench B was not tested in 2007 because a car was parked on the trench for the two days the Iowa DOT FWD equipment was available for testing. On June 6, 2008, the first FWD testing was done on Trench B, see Figure 4.52.

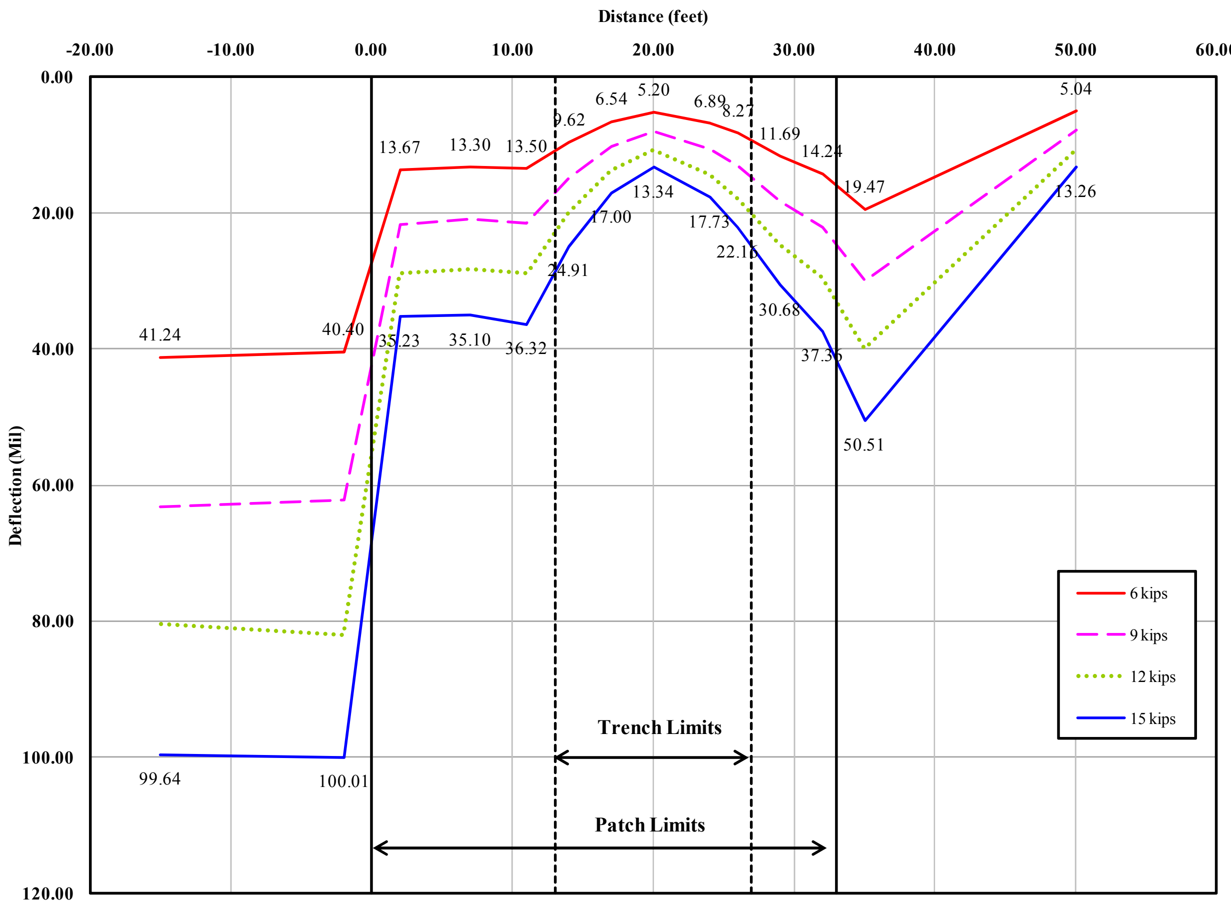


Figure 4.52. FWD test results for Trench B on June 26, 2008, at a temperature of 70.0°F

Post-Construction Elevation Survey

The post construction elevations were measured and 3-D surfaces were constructed using survey data collected on July 30, 2007; March 19, 2008; June 10, 2008; August 20, 2008; November 20, 2008; and March 13, 2009.

Trench B was surveyed with 51 grid points (see Figure 4.53). The benchmark was located at the dome bolt on the fire hydrant northwest of the trench at the intersection of 9th Street and Carroll.

The average uplift at the site was 0.10 inches and the maximum settlement was 0.30 inches at survey point 34. The maximum settlement occurred on the north edge of the patch in the soil adjacent to the trench. Figure 4.54 shows the elevation profiles of the trench as well as the settlement profile of the trench. This figure shows the settlement occurred at the north edge of the trench.

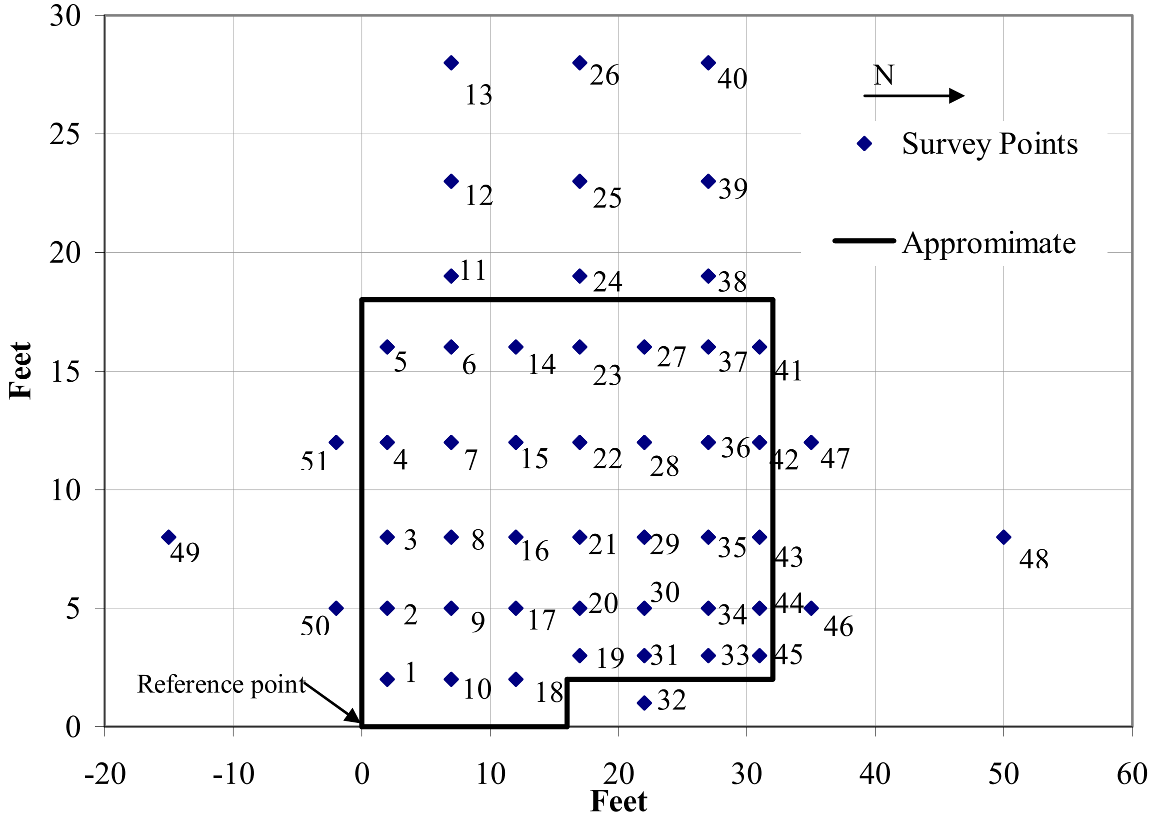


Figure 4.53. Survey locations for Trench B

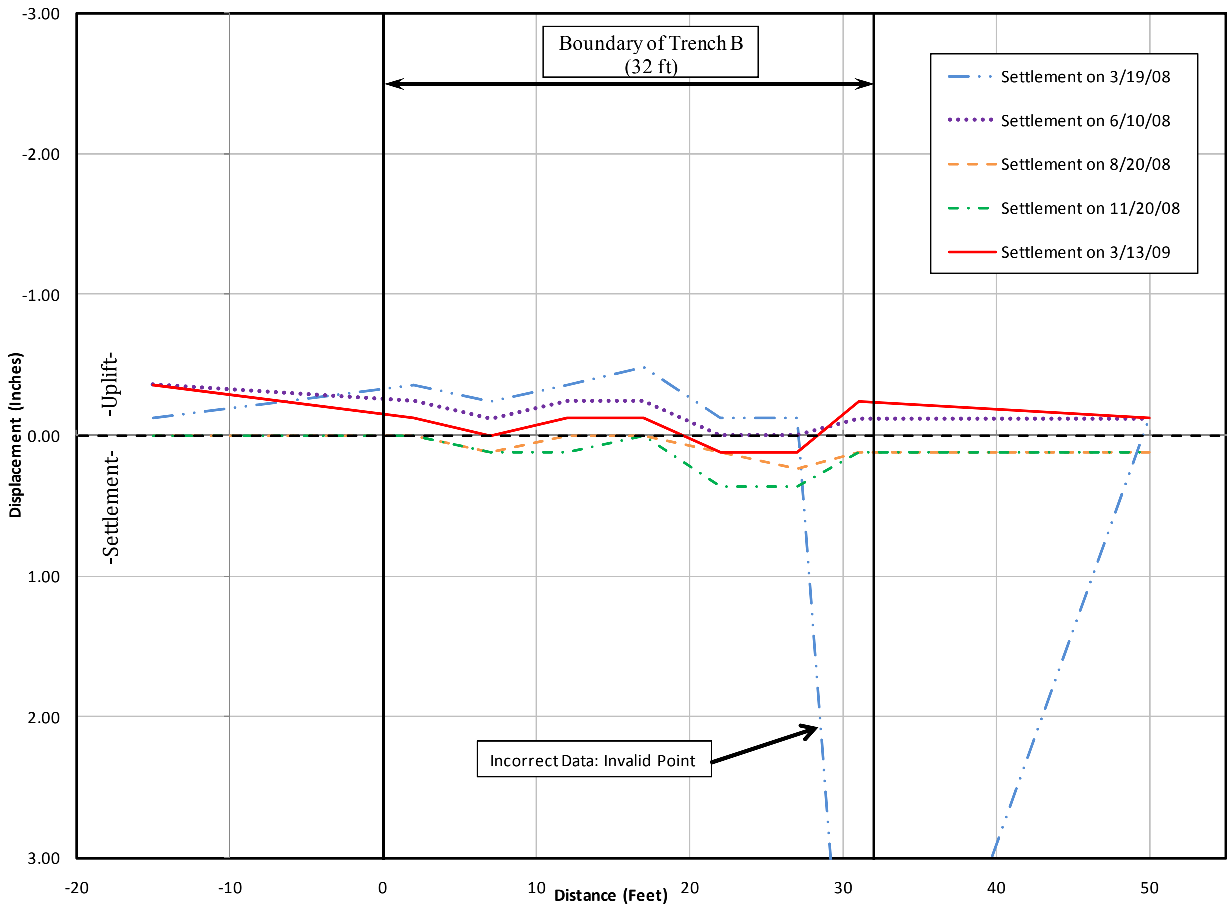


Figure 4.54. Settlement along center line of Trench B (points 49, 3, 8, 16, 21, 29, 35, 43, 48)

Comparison of Field-Testing Results to Long-Term Monitoring

The maximum settlement occurred on the north side of the trench (i.e., within the vicinity of test points 11 and 12 as shown in Figure 4.53). The dry unit weights at points 11 and 12 were 123.0 pcf and 116.3 pcf, respectively. The CBR values at the surface were 5% and 1% for points 11 and 12, respectively. The average CBR value for the depth of the test for both test points was 3%. These soils were extremely weak even though the densities for the soils were within the accepted range from NAVFAC. The CBR values from the DCP tests on the north edge of the trench at test point 11 showed a softening response, while depth and test point 12 showed the stiffness increasing with depth.

Summary

- The 3/8-inch minus limestone used in lift 3 was placed within the bulking moisture content of 5.0% to 9.0%.
- During initial compaction of the upper lifts, the moisture content of the backfill was below the optimum moisture content from the Standard Proctor test. The 1.5 inches of rain saturated the backfill and caused the moisture content to change from 5.1% to

7.0% and the density to change from 118.2 pcf to 115.3 pcf.

- When the patch was originally placed, there was a difference of 6.12 inches in elevation across the patch.
- The maximum settlement was 0.30 inches and occurred in the soil adjacent to the trench where there were low CBR values.
- The location of the maximum settlement corresponded to the location of lower dry unit weights and CBR values.

Recommended Trench C

Nuclear density and DCP tests were performed on Trench C located at McKinley Drive and Fillmore Avenue. Four different test points were used to test the third and final lifts on May 18, 2005 (i.e., at the end of Phase I). The locations of test points for the third and final lift are shown in Figure 4.55.

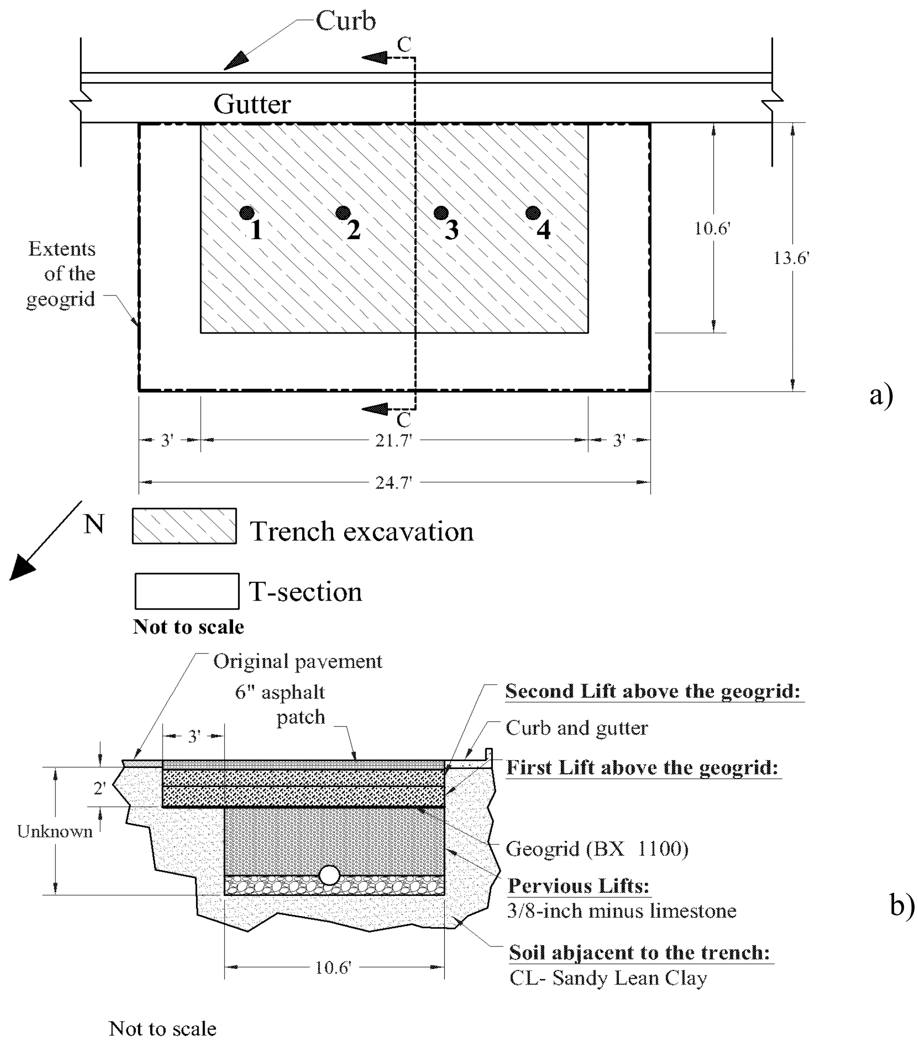


Figure 4.55. Location of test points in Trench C; cross-section of the trench

Nuclear Density Test Results

Tables 4.16 and 4.17 summarize the dry unit weight moisture content results from the nuclear density tests for Trench C, with the measurement made at a depth of 6 inches.

The nuclear density tests on the first lift above the geogrid had, for the four test points, an average moisture content of 11.3% and a dry unit weight of 111.0 pcf. The nuclear density tests on the final lift for the four test points had an average moisture content of 10.0% and a dry unit weight of 117.8 pcf.

Table 4.16. Dry unit weight results from the nuclear density tests on Trench C

Location	Number of test points	Average dry unit weight (pcf)	Min/Max dry unit weight (pcf)	Stan-dard devia-tion	Coefficient of variance (%)
First lift above the geogrid	4	111.0	101.2/116.8	6.9	6.2
Second/final lift above the geogrid	4	117.8	113.1/121.2	3.5	3.0

Table 4.17. Moisture content results from the nuclear density tests on Trench C

Location	Number of test points	Average moisture content (%)	Min/Max moisture content (%)	Stan-dard devia-tion	Coefficient of variance (%)
First lift above the geogrid	4	11.3	9.7/12.5	1.4	12.4
Second/final lift above the geogrid	4	10.0	8.9/11.0	1.0	3.0

DCP Test Results

Table 4.18 summarizes the DCPI readings from the DCP tests for Trench C, and Table 4.19 summarizes the average CBR results from the DCP tests for Trench C.

Table 4.18. DCPI results from the DCP tests for Trench C

Location	Number of test points	Average depth of tests (inches)	Average DCPI	Standard deviation	Coefficient of variance (%)
First lift above the geogrid	4	30.6	6.2	8.7	140.3
Second/final lift above the geogrid	4	30.4	46.1	56.7	127.3

Table 4.19. Average CBR results from the DCP tests for Trench C

Location	Number of test points	Average depth of tests (inches)	Average CBR (%)	Standard deviation	Coefficient of variance (%)
First lift above the geogrid	4	30.6	55	66.2	120.4
Second lift above the geogrid	4	30.4	9	10.9	121.1

Figure 4.56 shows the CBR values as a function of depth for the top 45 inches at the four test points within the trench. The first life above the geogrid had an average CBR value of 55%. For the second/final lift (see Figure 4.57), the average calculated CBR value within the trench was 9%. At the surface, the CBR values ranged from 3% to 6%. At the termination of the test, the CBR values ranged from 3% to 21%. At a depth of about 19 inches, test points 2, 3, and 4 show a large decrease in CBR values. The decrease in CBR values identified a boundary between either lifts or material types.

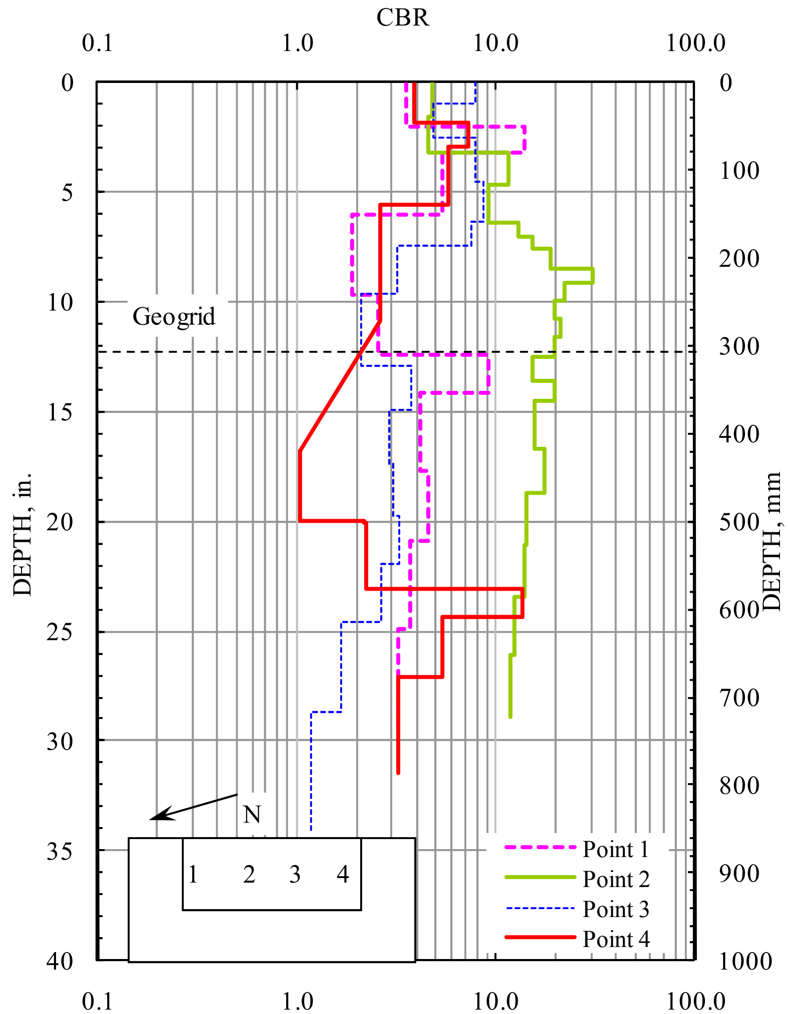


Figure 4.56. CBR results from the DCP test profiles for the first lift above the geogrid in Trench C

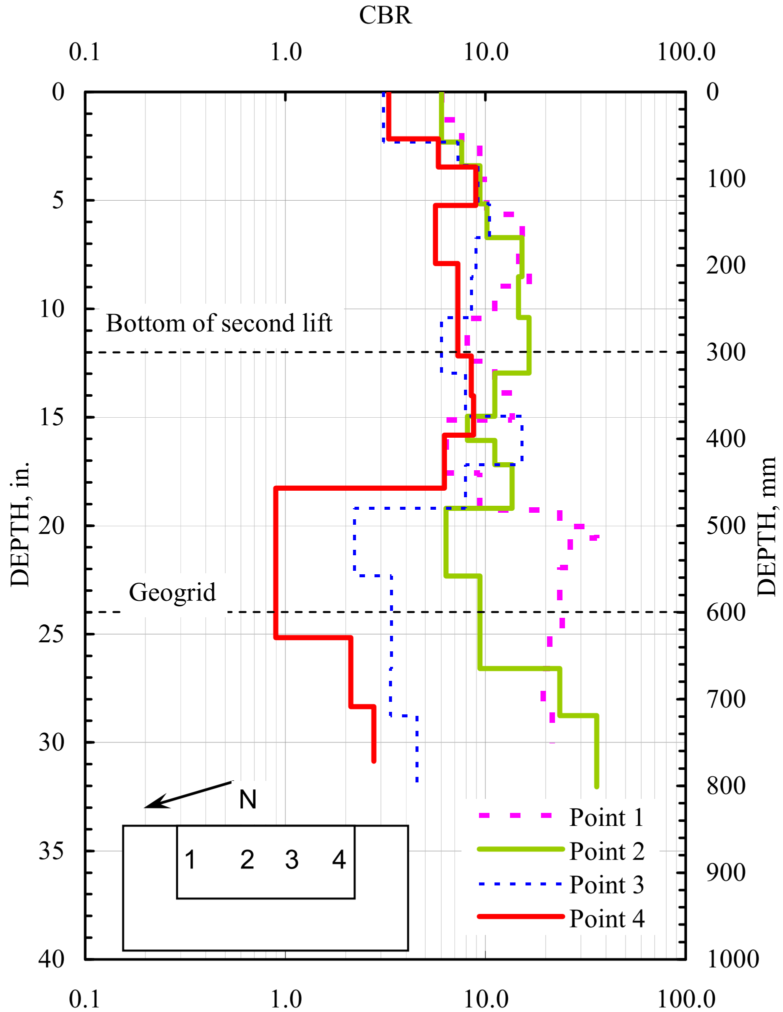


Figure 4.57. CBR results from the DCP test profiles for the second/final lift above the geogrid in Trench C

FWD Test Results

The FWD testing locations are shown in Figure 4.58. Falling weight deflectometer testing was performed in June 2007, June 2008, and November 2008. The June 2007 testing results are shown in Figure 4.59. The June 2008 testing results are shown in Figure 4.60. The November 2008 testing results are shown in Figure 4.61. Figure 4.62 shows the comparison between June 2008 and November 2008.

During the June 2007 FWD testing, the backfill within the trench was stiffer than the surrounding soil. The deflection in the trench for the 15-kip load was 37.7% less than the deflection outside the trench. The deflections for the 15-kip load show that the deflections for the testing points furthest from both sides of the trench were less than the deflections for the 12-kip load.

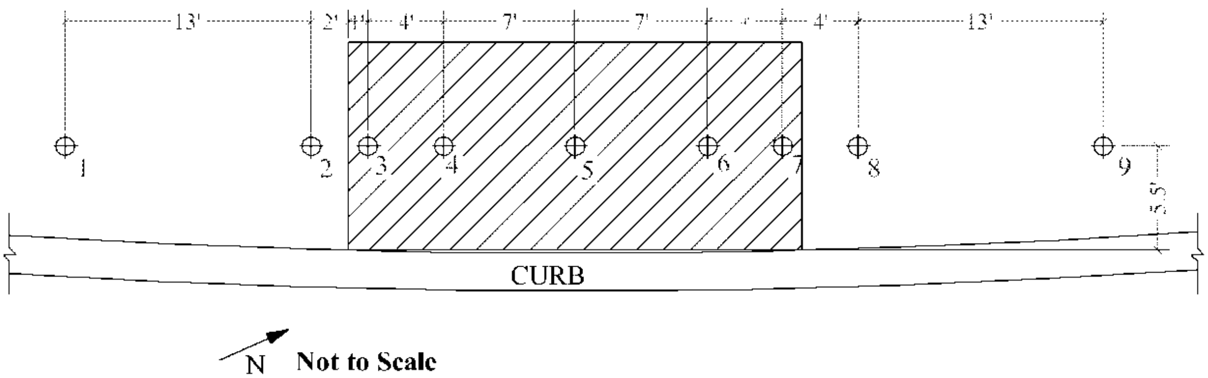


Figure 4.58. Falling weight deflectometer test locations for Trench C

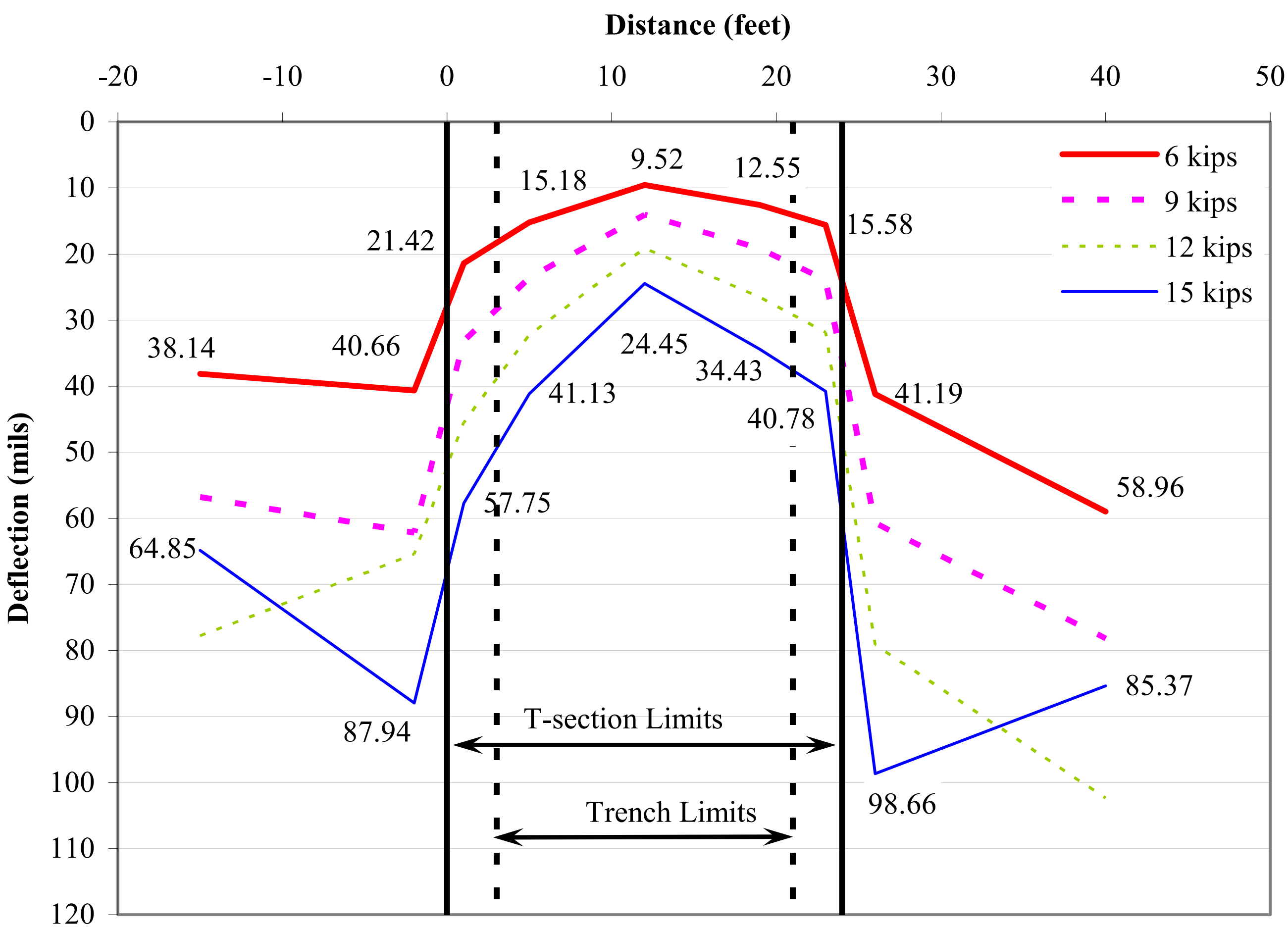


Figure 4.59. Falling weight deflectometer test results for Trench C in June 2007

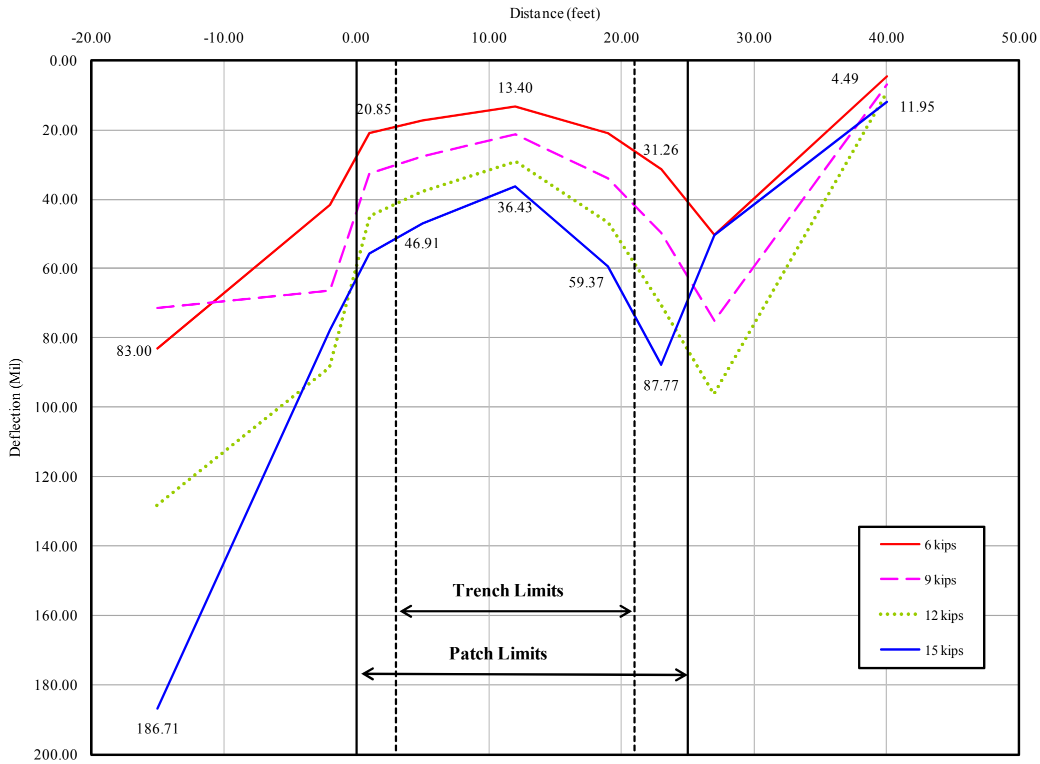


Figure 4.60. Falling weight deflectometer test results for Trench C in June 2008 at a temperature of 88.8 °F

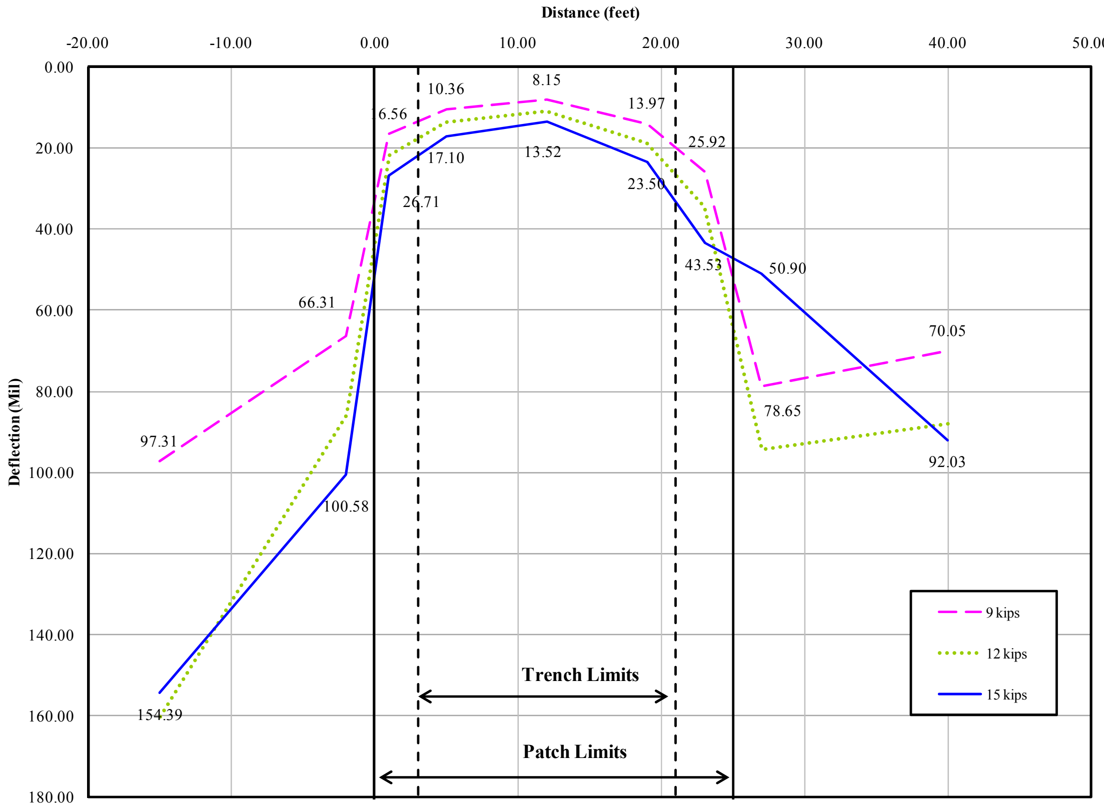


Figure 4.61. Falling weight deflectometer test results for Trench C in November 2008 at a temperature of 50.9ºF

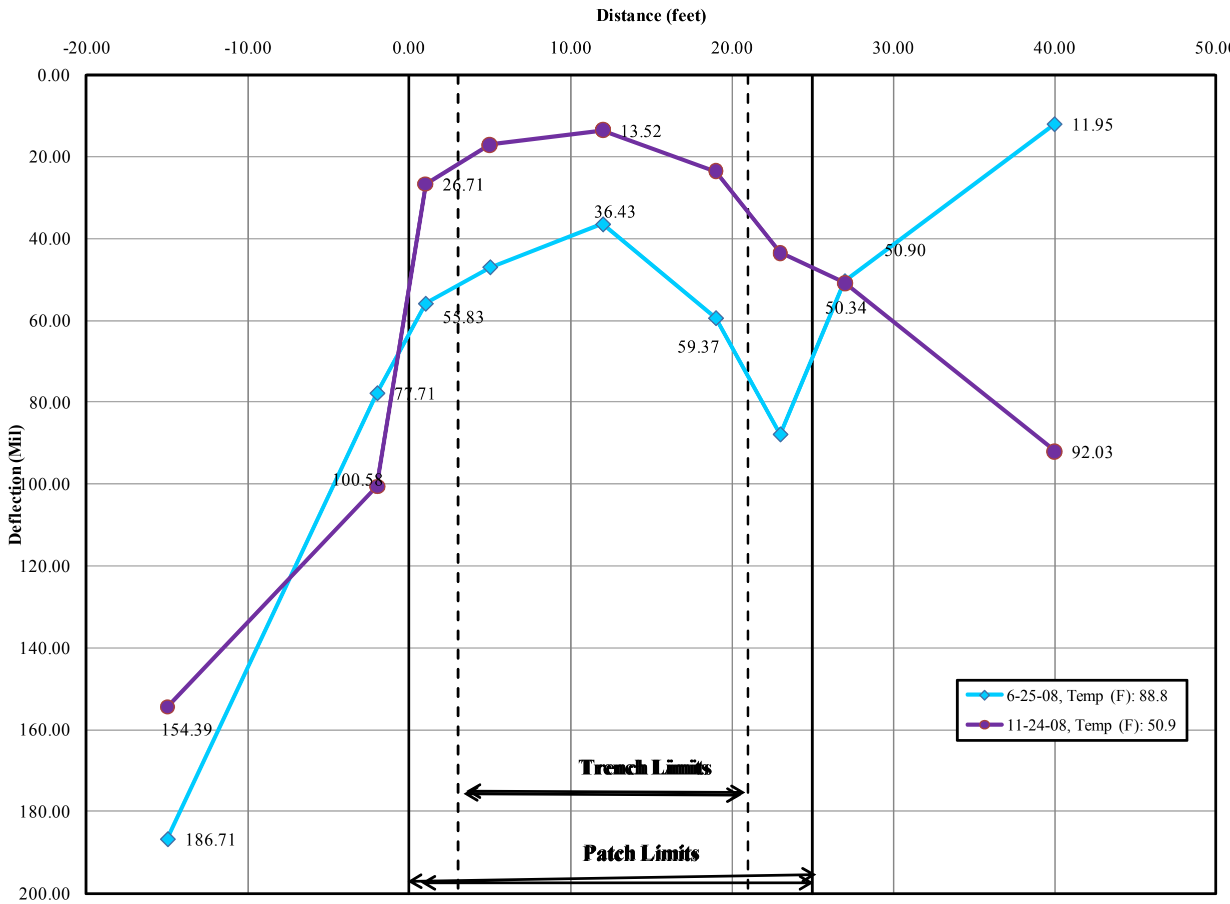


Figure 4.62. Comparison of the 15-kip load deflections for Trench C; June 25, 2008, and November 26, 2008

Post-Construction Elevation Survey

The post-construction elevation surfaces were constructed using survey data collected on May 11, 2007; March 19, 2008; June 10, 2008; August 20, 2008; December 06, 2008; and March 13, 2009. No survey date earlier than March 11, 2008, was available.

Trench C was surveyed with 51 grid points (see Figure 4.63). The benchmark was the dome bolt (see Figure 4.40) on the hydrant northeast of the trench located at the intersection of McKinley Drive and Van Buren Avenue.

From the elevation survey on March 19, 2008, the average uplift of the trench was 2.05 inches and the maximum settlement was 1.7 inches (survey point 17). The maximum settlement occurred in the west corner of the patch. Figure 4.64 presents the elevation profiles and settlement profiles for Trench C. This figure shows that the middle of the trench did not experience the same magnitude of uplift during the winter as the area surrounding it. Because of this, the patch had a bump in it.

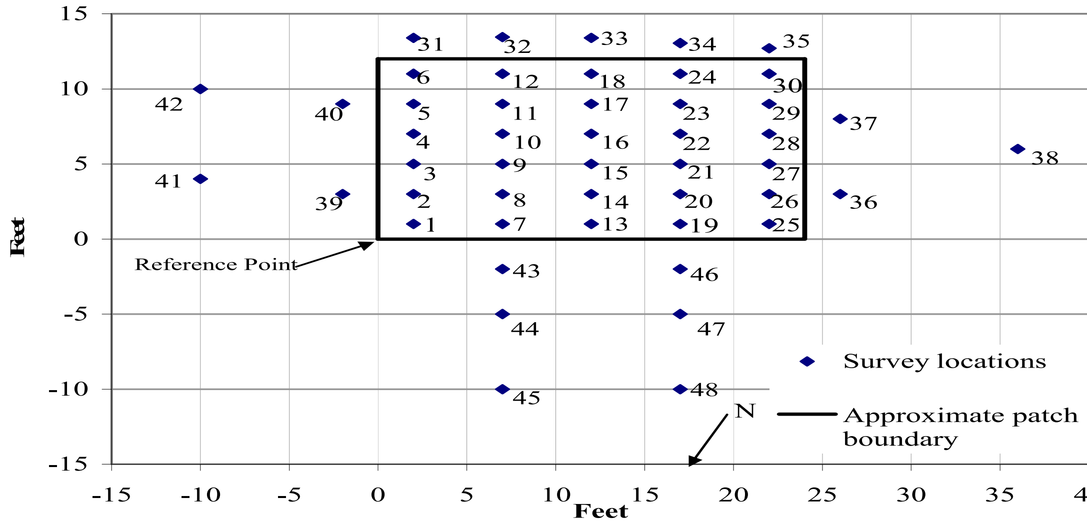


Figure 4.63. Survey locations for Trench C

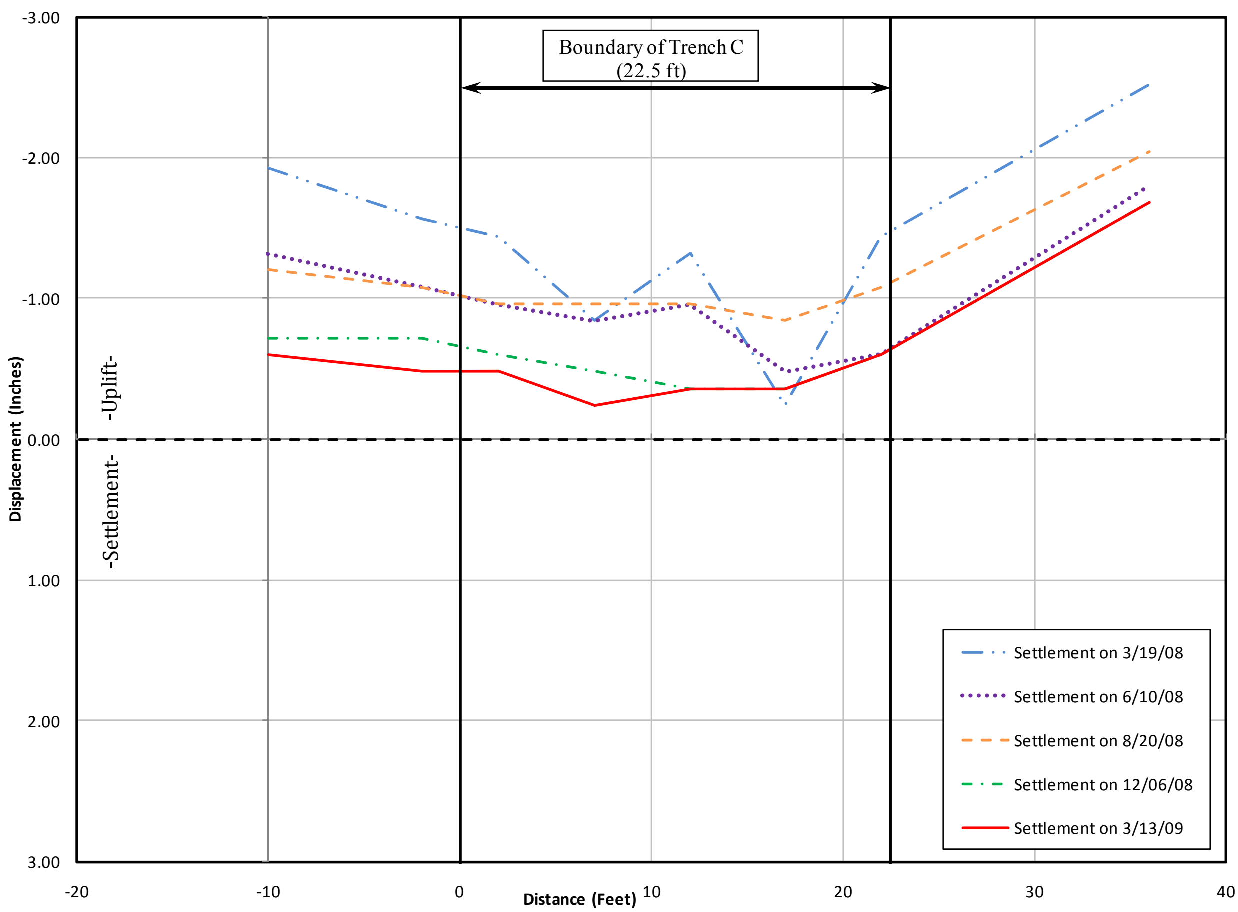


Figure 4.64. Settlement along center line of Trench C (points 41, 39, 3, 9, 15, 21, 27, 38)

Comparison of Field-Testing Results to Long-Term Monitoring

Figure 4.65 shows the FWD testing locations with the field-testing locations superimposed with the averaged field-testing results for various points. The deflection in the middle of the trench was 24.25 mils. At the edge of the trench, the deflections ranged from 40.78 mils to 41.13 mils. Test point 1 (see Figure 4.65) was near the FWD test point 7 that yielded the 34.43 mils. At this test point, the dry unit weight of the backfill after compaction was 117.2 pcf. The average CBR value for the depth of the DCP test was 7%. The CBR value at the surface was 6%. Test point 4 was located near the FWD test point 5 with the deflection of 41.13 mils. At this test point, the dry unit weight was 113.1 pcf. The CBR average value for the depth of the DCP test was 32%, and the CBR value at the surface of the trench was 3%. In the center of the trench, test points 2 and 3 (see Figure 4.65) were averaged together. The average dry unit weight was 120.4 pcf. The average CBR value at these points over the depth of the DCP tests was 14%. At the surface, the average CBR value was 5%. Since no bulk sample of the backfill was collected and its classification is not known, no other comparisons can be drawn.

Figure 4.66 shows a plot with FWD deflections and settlement. A load of 15 kip was used to accurately show the deflection of the subgrade. This shows the settlements are independent of the FWD deflections. On the chart, it can be seen that the higher dry unit weights measured within the trench occurred where there were lower deflections. It also shows the CBR values did not predict the FWD testing results.

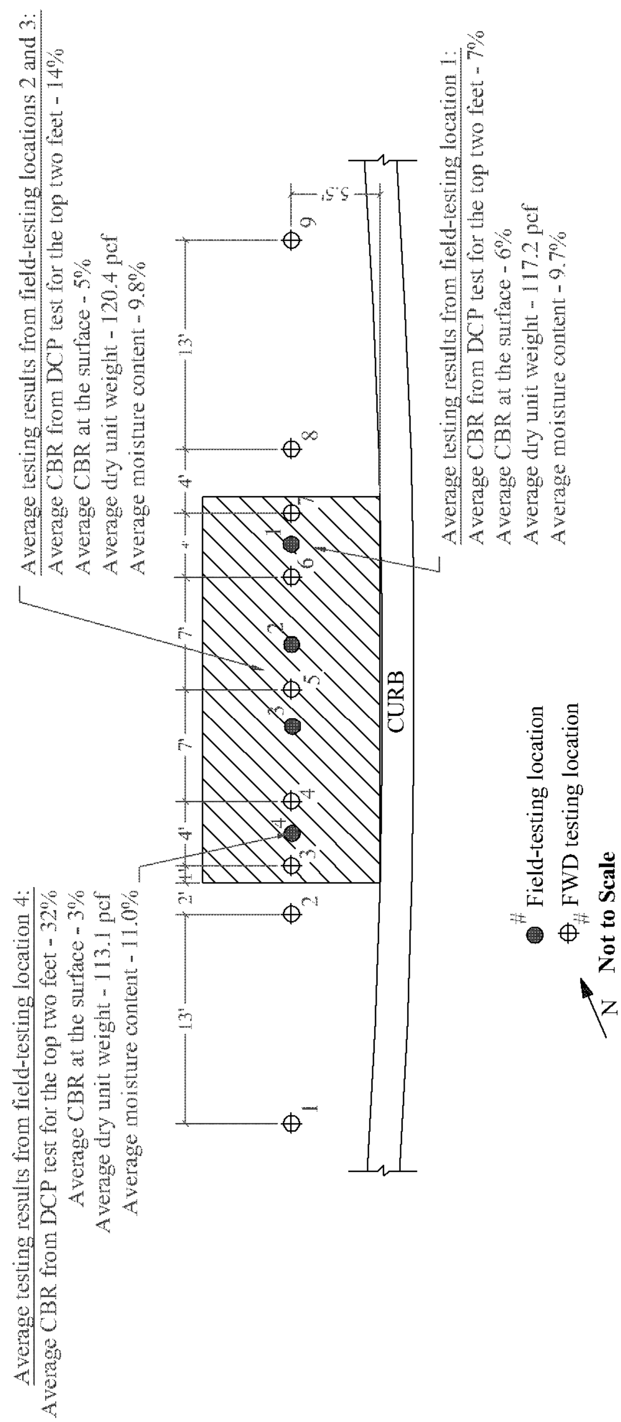


Figure 4.65. Comparison of CBR values, dry unit weights, and FWD testing results for Trench C

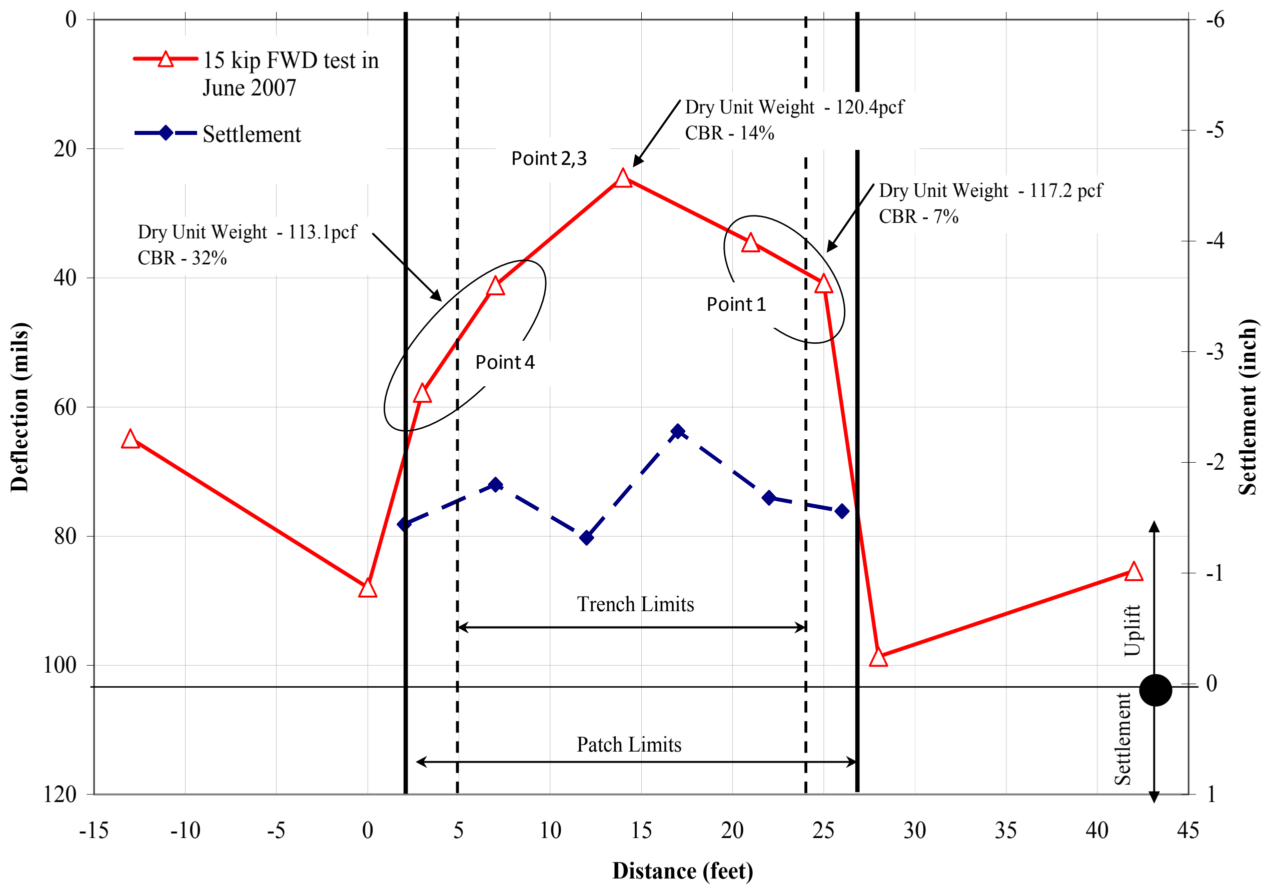


Figure 4.66. Falling weight deflectometer test (15 kip) and settlement for Trench C

Summary

- The trench settled 1.70 inches between the spring 2007 survey and the March 2008 survey.
- The FWD test results showed the backfill within the trench stiffened over time.
- The zone of influence was present on both sides of the trench.
- Falling weight deflectomter deflections did not correlate with CBR results or settlements. The FWD results showed lower deflections occurred where there were higher dry unit weights.

Recommended Trench D

Nuclear density and DCP tests were performed on Trench D. On July 23, 2007, the backfill was tested at four test points on the third lift. On July 25, 2007, the final lift was tested at the same four points and four additional test points in the T-section for a total of eight test points (see Figure 4.67).

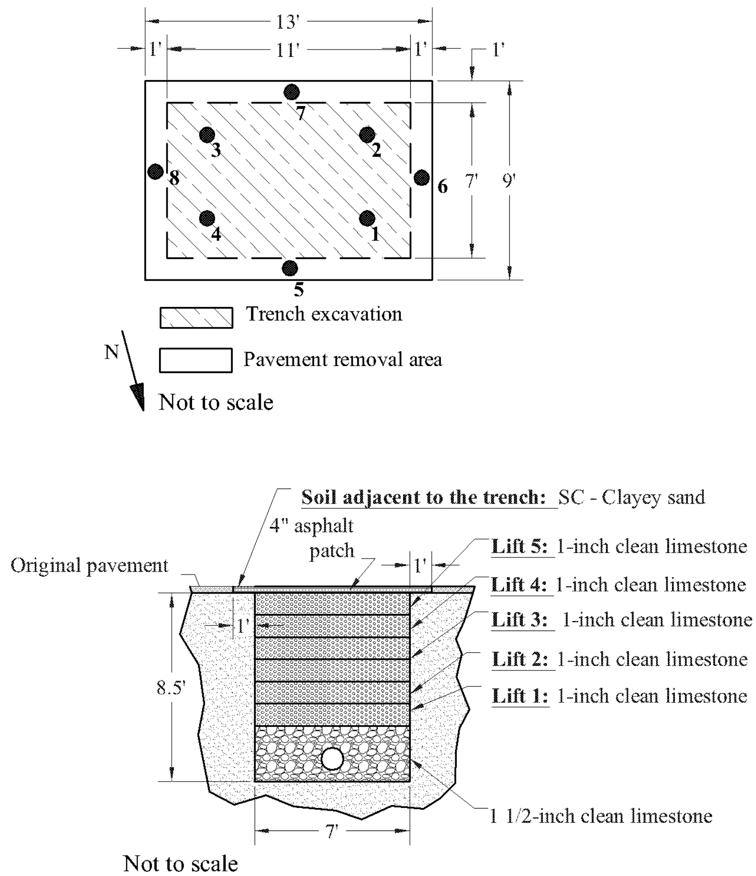


Figure 4.67. Location of test points in Trench D; cross-section of the trench

Nuclear Density Test Results

Tables 4.20 and 4.21 summarize the dry unit weights and moisture contents from the nuclear density testing results for Trench D. The probe depth was 6 inches.

The backfill within the trench was classified as GP—poorly graded gravel. The typical range of dry unit weights for compacted poorly graded gravel is 115 to 125 pcf, and the typical moisture content range is from 11% to 14% according to NAVFAC (1986) in Table 1.5. Using the vibratory table test, the maximum dry unit weight at zero moisture content was 132.2 pcf. Given the large size of aggregate particles and the free drainage, there was no bulking moisture content range. The smallest value of maximum dry unit weight was 108.6 pcf at approximately 4.3%.

The maximum dry unit weight within the trench was 114.5 pcf, and the minimum dry unit weight within the trench was 104.3 pcf. The nuclear density tests for the third lift at the four test points had an average dry unit weight of 106.9 pcf and average moisture content of 3.0%. Based on the laboratory testing, the backfill was compacted to 57% relative density, which corresponded to a medium dense compaction.

The average dry unit weight of the fifth lift from the nuclear density tests for the four test points within the trench was 110.0 pcf and the moisture content was 2.0%, which corresponds to 63% relative density (i.e., medium dense compaction).

The four test points in the adjacent soil where the pavement was removed had an average moisture content of 5.1% and an average dry unit weight of 118.5 pcf. The average dry unit weight of the soil was 98% of the maximum from the Standard Proctor test.

Table 4.20. Dry unit weight results from the nuclear density tests on Trench D, with backfill having γmax of 133 pcf and γmin of 86 pcf

Location	Number of test points	Average dry unit weight (pcf)	Min/Max dry unit weight from field testing (pcf)	Relative density (%)	Stan-dard devia-tion	Coefficient of variance (%)
Third lift	4	106.9	104.3/108.6	57	19	1.8
Fifth lift test points within the trench	4	110.0	107.1/114.0	63	2.9	2.6
Soil adjacent to the trench	4	118.5	107.1/122.5	N/A	7.6	6.4

Table 4.21. Moisture content results from the nuclear density tests on Trench D

Location	Number of test points	Average moisture content (%)	Min/Max moisture content (%)	Stan-dard devia-tion	Coefficient of variance (%)
Third lift	4	3.0	2.5/3.0	0.3	10
Fifth lift test points within the trench	4	2.0	1.8/2.8	0.2	10.2
Soil adjacent to the trench	4	5.1	2.5/7.3	2.0	39.2

Figure 4.68 shows the relative density testing results and the average testing results from Trench D. In the figure, the boundary between the different relative densities is shown as dashed lines. The collapse index of the material is also shown. Although the backfill material has a minimum collapse potential, the moisture content that the backfill was placed at was within the range of maximum collapse potential. This figure shows that the average dry unit weight for each lift corresponded to medium dense compaction.

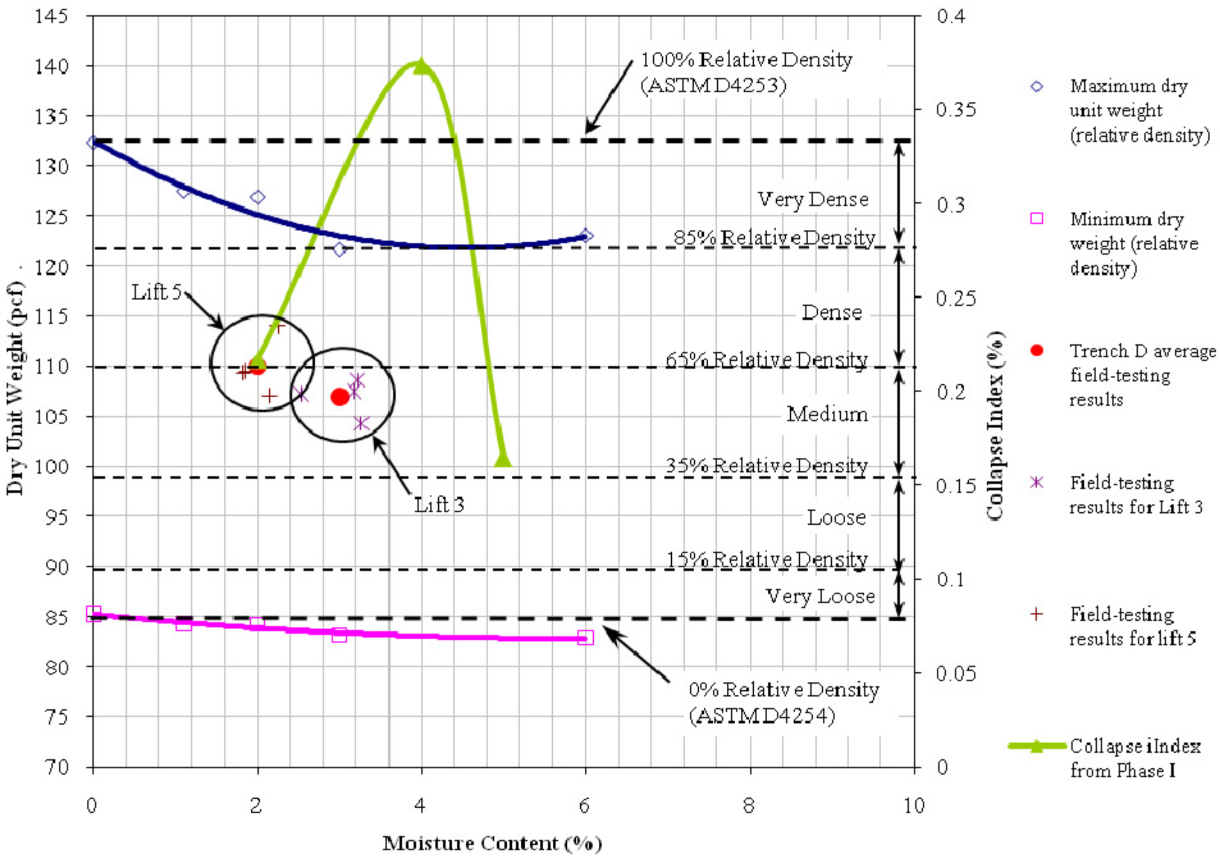


Figure 4.68. Relative density test results from Phase I with field-testing results for Trench D

The maximum dry unit weight and optimum moisture content of the soil around the trench was 122.5 and 5.1% from the Standard Proctor test.

DCP Test Results

Tables 4.22 and 4.23 summarize the DCPI readings and calculated CBR values from the DCP tests for Trench D. Trench D was constructed with 1-inch clean limestone. This backfill was classified as GP—poorly graded gravel. The reported CBR values for poorly graded gravel were from 30% to 60%. The average values for the trench were below the typical values from NAVFAC.

Table 4.22. DCPI results from the DCP tests for Trench D

Location	Number of test points	Average depth of tests (inches)	Average DCPI	Standard deviation	Coefficient of variance (%)
Third lift	4	27.3	33.2	34.6	103.0
Fifth lift test points within the trench	4	29.4	15.9	16.7	105.0
Soil adjacent to the trench	4	29.8	40.5	30.8	76.0

Table 4.23. Average CBR results from the DCP tests for Trench D

Location	Number of test points	Average depth of tests (inches)	Average CBR (%)	Standard deviation	Coefficient of variance (%)
Third lift	4	27.3	26	26.8	64.6
Fifth lift test points within the trench	4	29.4	23	12.7	55.2
Soil adjacent to the trench	4	29.8	11	9.7	88.2

The profiles of the CBR values as a function of depth for the third and fifth lifts are shown in Figures 4.69 and 4.70. The third lift for the four test points within the trench had an average CBR value of 26% extending 27.3 inches. At the surface of the lift, the CBR values ranged from 2% to 5%. These CBR values were below the typical values from NAVFAC. At the termination of the DCP tests, the CBR values ranged from 24% to 34%. The CBR profiles increased slightly with depth. This indicates there was an increase in strength with depth. The four test points were tightly banded through the profile. This indicates the lift was placed consistently over the trench.

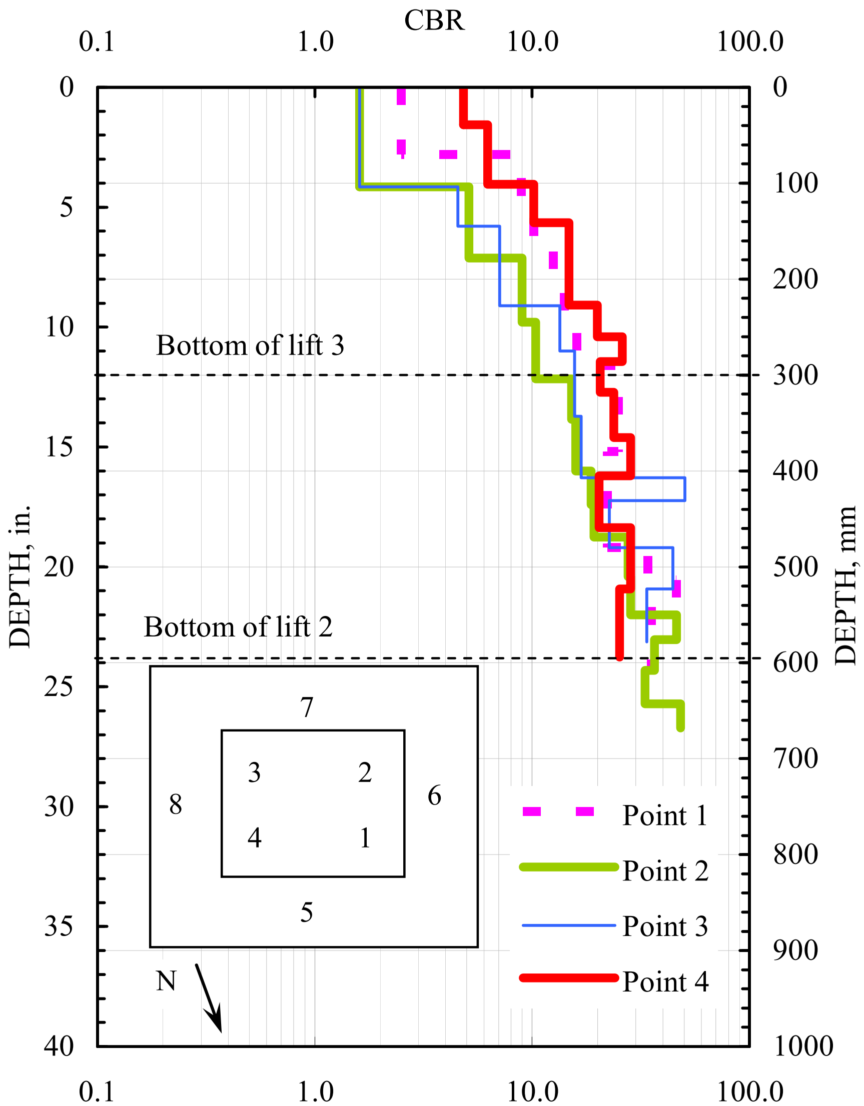


Figure 4.69. CBR results from the DCP test profiles for the third lift in Trench D

The four test points within the trench had an average CBR of 23% extending 29.4 inches. At the surface, the CBR values ranged from 2% to 4%. The CBR profile increased with depth. The profiles were tightly banded for the entire profile, indicating consistent compaction during construction.

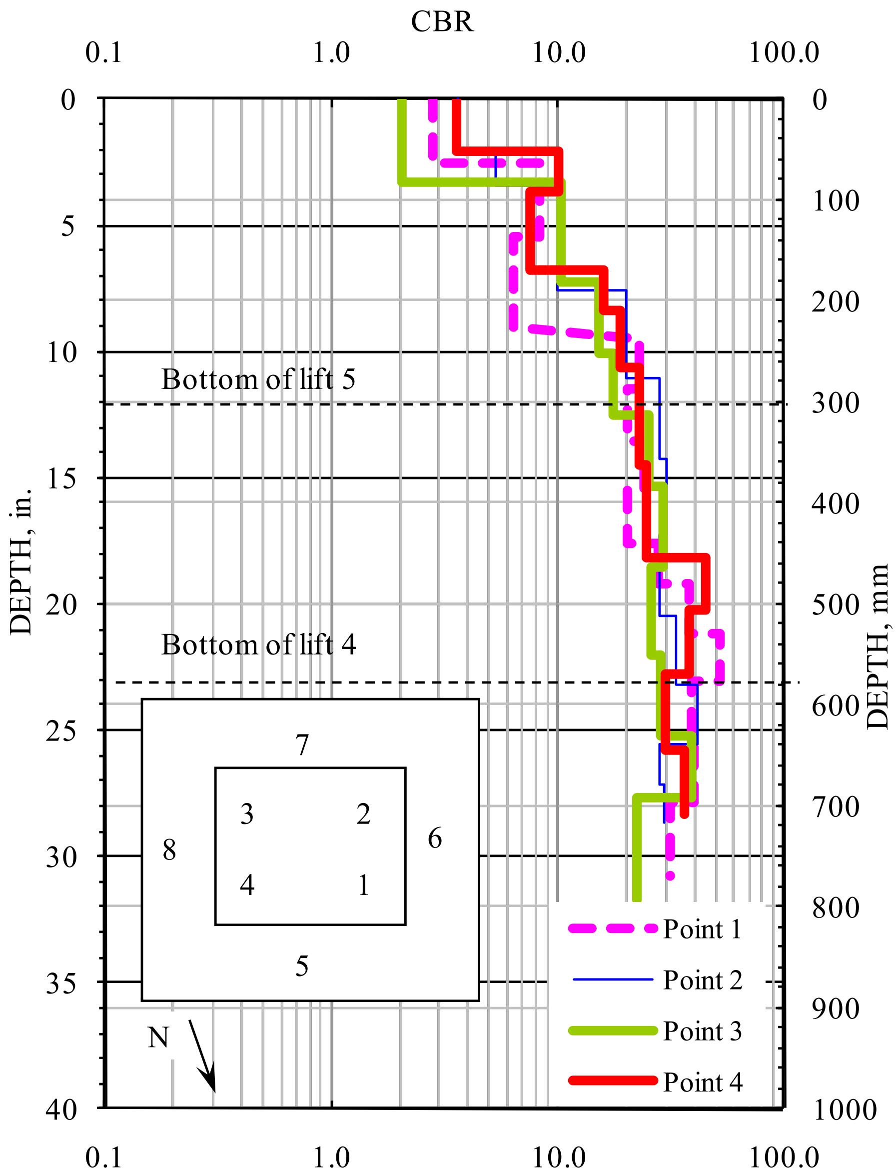


Figure 4.70. CBR results from the DCP test profiles for the fifth lift in Trench D

Figure 4.71 shows the profiles of the CBR values as a function of depth in the soil adjacent to Trench D. The four test points had an average CBR of 11%. At the surface, the CBR results ranged from 1% to 9%. At the termination of the DCP tests, the CBR values ranged from 2% to 20%. The CBR profiles for the four test points in the soil adjacent to the trench were widely varied, which could lead to differential settlement. The profiles also show that at test points 7 and 8 the soil softened with depth and at test points 5 and 6 the soil strengthened with depth.

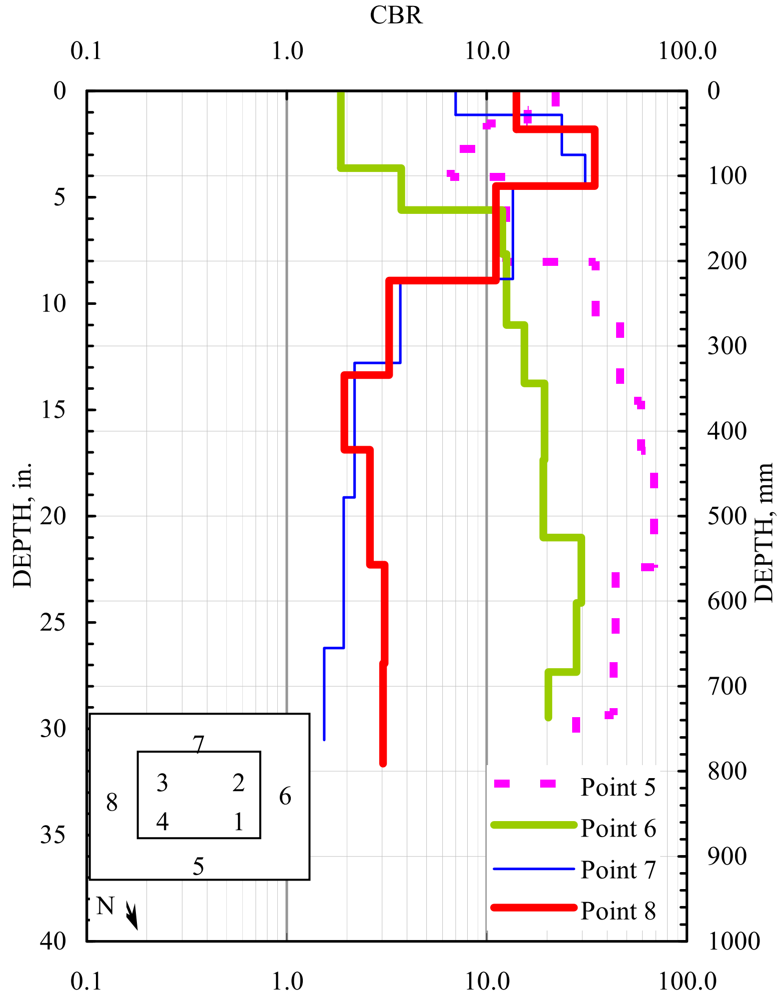


Figure 4.71. CBR results from the DCP test profiles for the soil adjacent to Trench D

FWD Test Results

Figure 4.72 shows the testing locations for Trench D. Figure 4.73 shows the results from the November 5, 2007, FWD testing. The FWD testing shows the zone of influence on the west side of the trench. On the east edge of the trench, the zone of influence was not present. The deflection at the center of the trench was 15.36 mils, and the average deflection at the inside edge of the trench was 20.68 mils for the 15-kip loads. This figure shows that the trench was placed at a density higher than the surrounding soils. Figure 4.74 shows the results from the June 28, 2008, FWD testing; Figure 4.75 shows results from the November 20, 2008, FWD testing; and Figure 4.76 shows results from the March 3, 2009, testing. All are proportional to the testing of November 2007. Figure 4.77 compares June 2008, November 2008, and March 2009 at a 15-kip load.

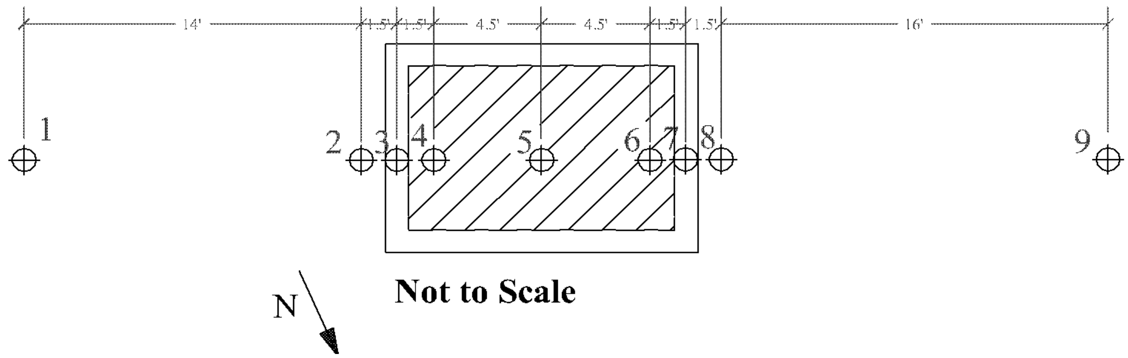


Figure 4.72. Falling weight deflectometer test locations for Trench D

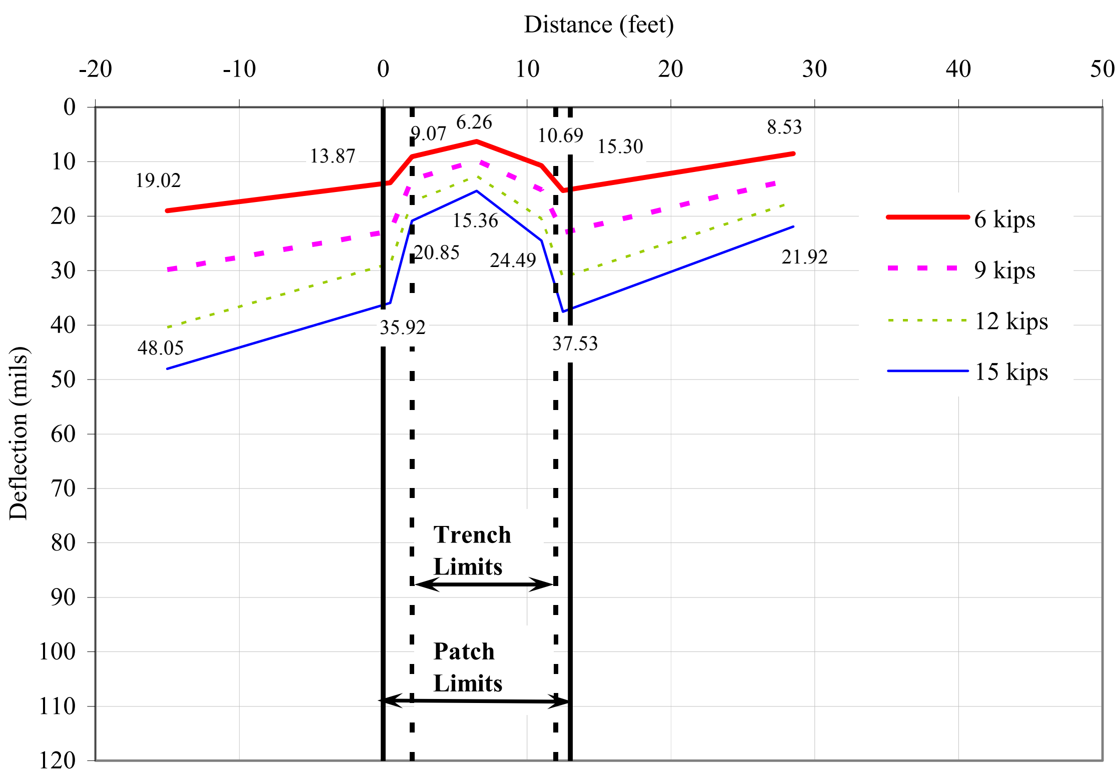


Figure 4.73. Falling weight deflectometer test results for Trench D on November 5, 2007

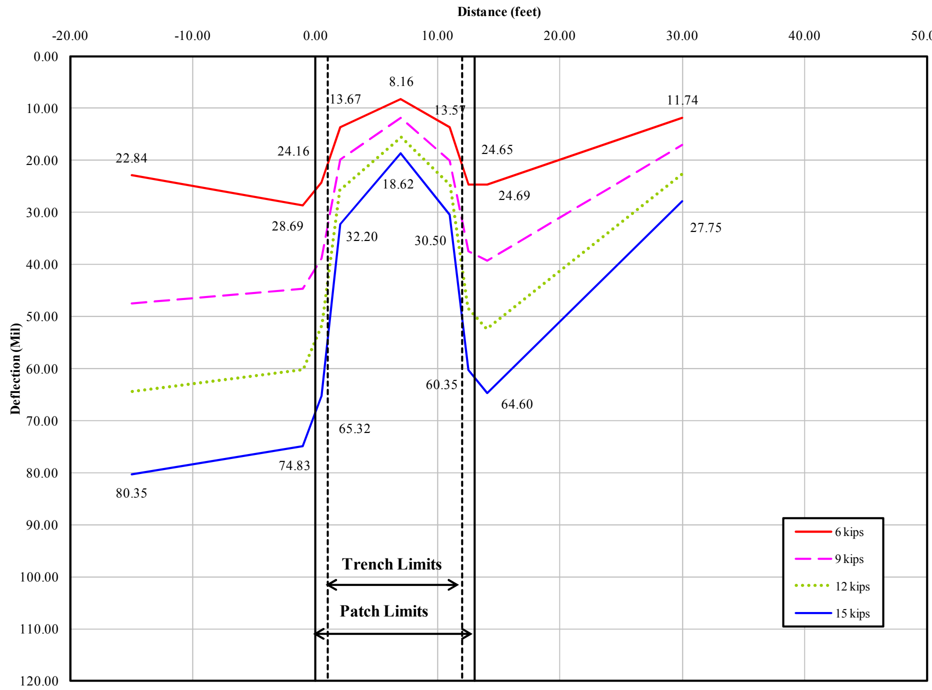


Figure 4.74. Falling weight deflectometer test results for Trench D on June 28, 2008, at a temperature of 67ºF

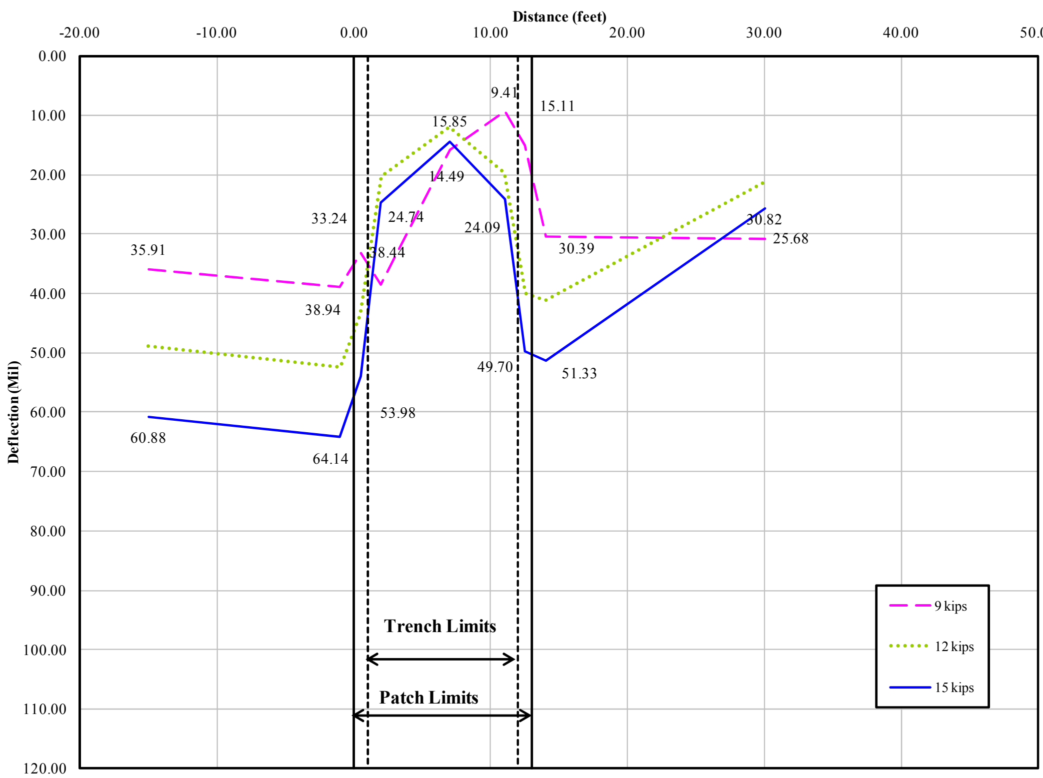


Figure 4.75. Falling weight deflectometer test results for Trench D on November 20, 2008, at a temperature of 45.8ºF

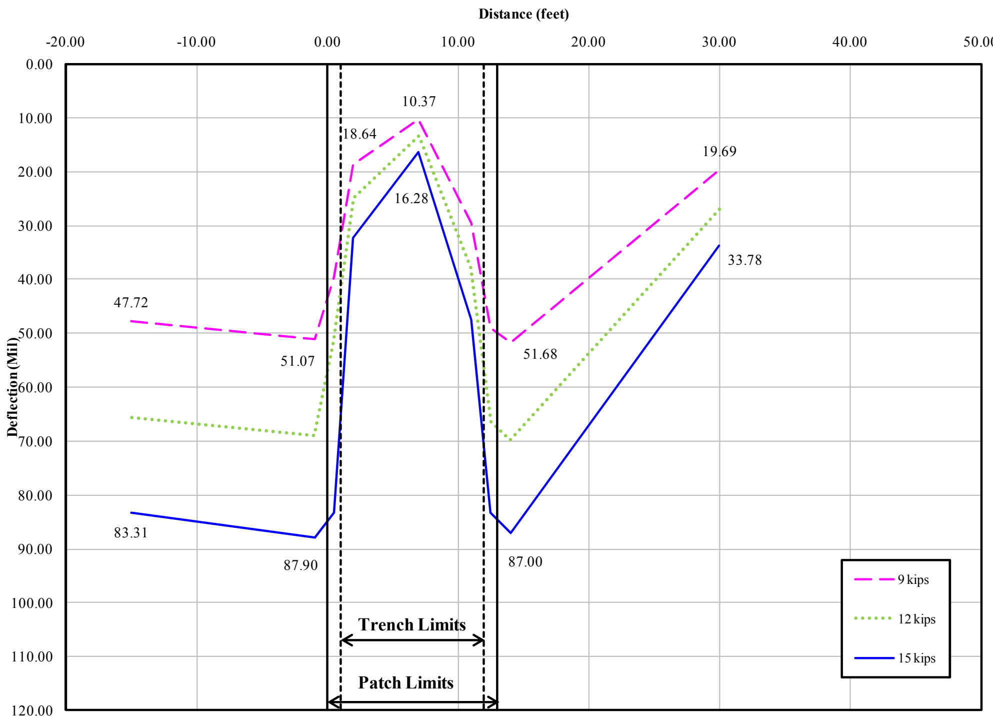


Figure 4.76. Falling weight deflectometer test results for Trench D on March 3, 2009, at a temperature of 37.7ºF

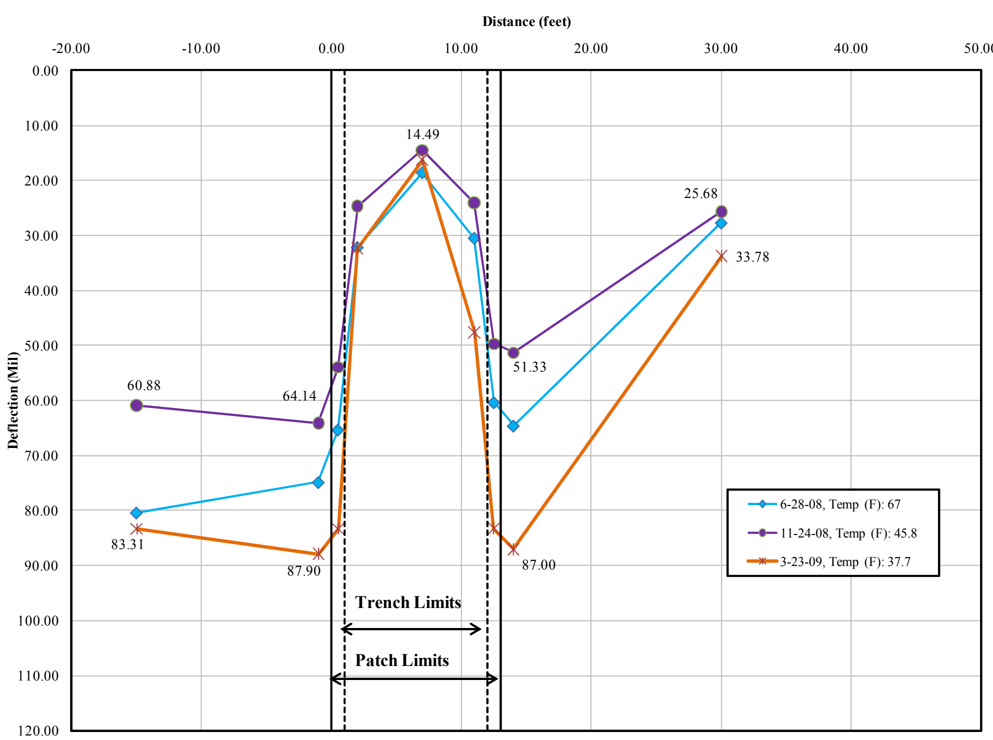


Figure 4.77. Comparison of FWD test (15 kip) results from June 2008, November 2008, and March 2009 for Trench D

Post-Construction Elevation Survey

The post-construction surfaces were constructed using survey data collected on July 30, 2007; March 19, 2008; June 10, 2008; August 20, 2008; November 18, 2008; and March 13, 2009. Figure 4.78 shows the location of the grid points.

Trench D was surveyed with 27 grid points. The benchmark was the dome bolt (see Figure 4.40) on a hydrant northeast of the trench. The difference between the maximum (point 1) and lowest (point 16) elevation was 1.44 inches on July 30, 2007. This was a reduction in the total difference between the highest and lowest level. The uplift was 0.96 inches and the maximum settlement was 0.24 inches at survey point 15 along the south edge of the trench in the adjacent soil.

Figure 4.79 shows the movement profiles for Trench D. This shows that the larger uplifts during the winter occurred outside the trench. This would result in the patch appearing like a dip in the road.

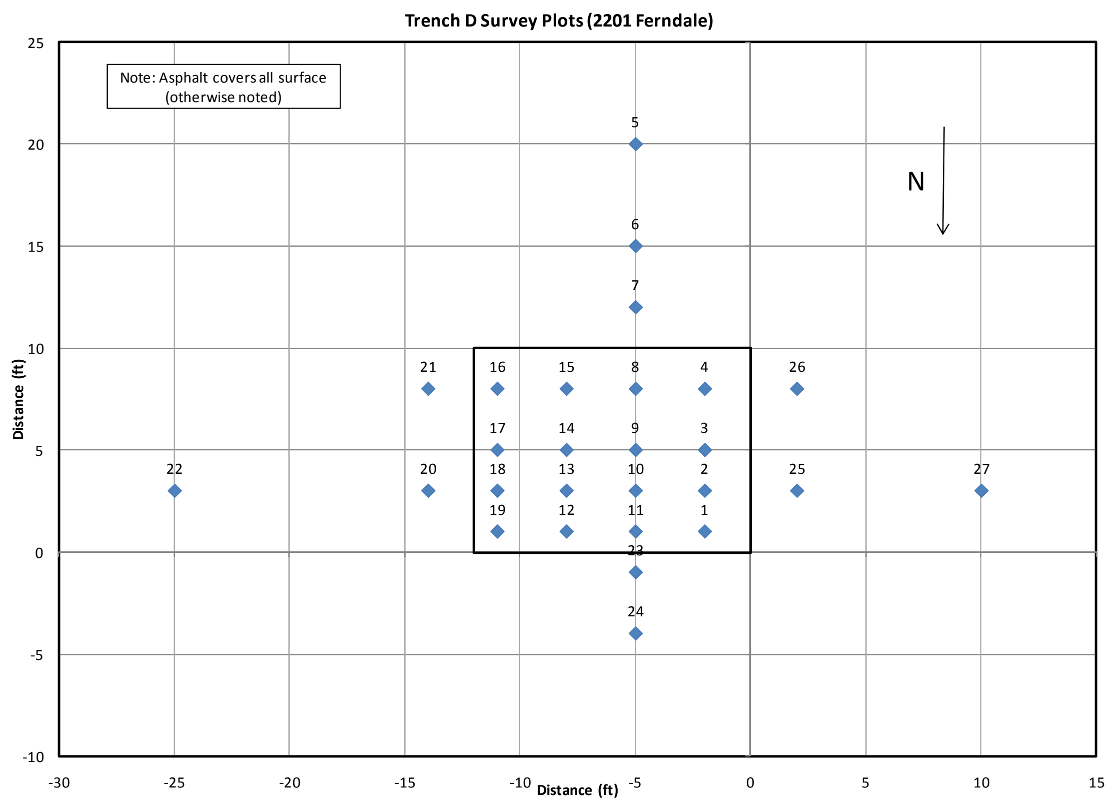


Figure 4.78. Survey locations for Trench D

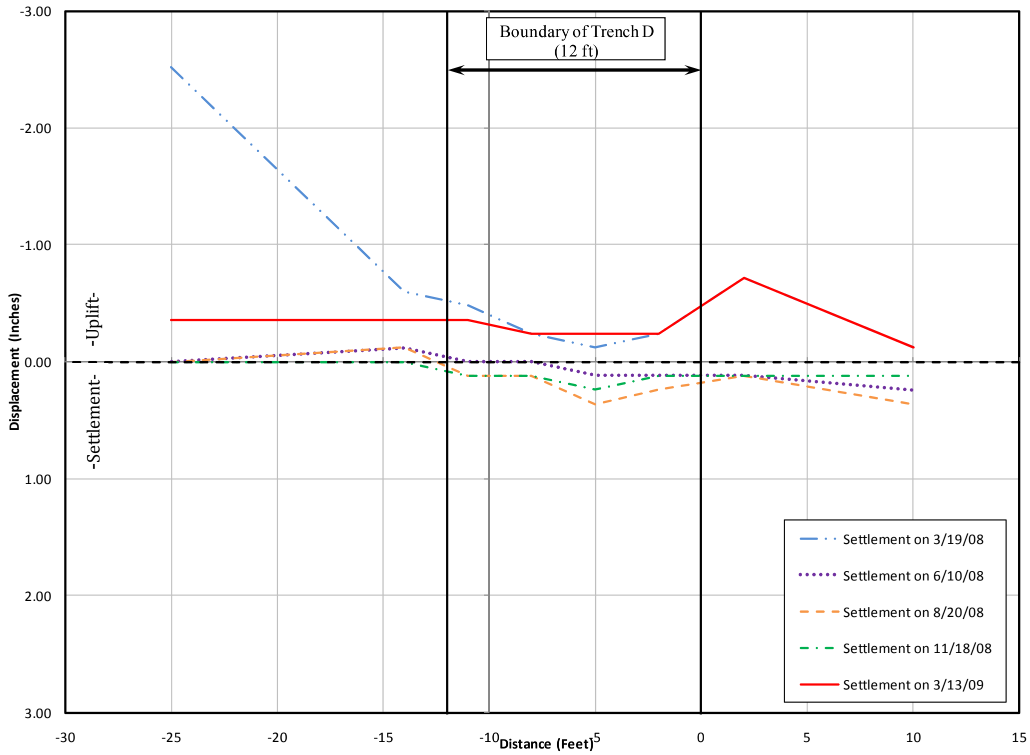


Figure 4.79. Settlement along center line of Trench D (points 22, 20, 18, 13, 10, 2, 25, 27)

Comparison of Field-Testing Results to Long-Term Monitoring

Falling weight deflectometer test point 7 (see Figure 4.80) was near the location of maximum settlement. The CBR value at the surface and over the depth of the trench at this point was 7%. Comparing this value to other places in the trench, the CBR value was below the CBR values at other test points in the soil adjacent to the trench.

The field-testing results during construction from the four test points within the trench were compared with the FWD testing. Figure 4.80 shows the FWD and field-testing locations superimposed. Also in the figure are the average field-testing results for field-testing points that correspond to FWD testing locations. The average dry unit weight was 110 pcf. The CBR value at the surface was 3%, and the average CBR value for the depth of the test was 23%. At the west edges of the trench, test points 3 and 4 were within the vicinity of FWD testing point 4 and had an average dry unit weight of 108.3 pcf. The CBR value at the surface was 3% and the average CBR value for the depth of the test was 22%. On the east side of the trench, test points 1 and 2 were in the vicinity of FWD test point 6. The dry unit weight was 111.7 pcf. The CBR value at the surface was 3% and the average CBR result was 24%.

The dry unit weight of the soil adjacent to the trench on the west side (FWD point 7) at test point 6 (see Figure 4.80) was 107.1 pcf at 2.5% moisture content. The average CBR value for the length of the test was 17%, and at the surface the CBR was 2%.

The dry unit weight of the soil adjacent to the trench on the east side (FWD point 3) at test point 8 (see Figure 4.80) was 122.2 pcf at 5.9% moisture content. The average CBR value for the length of the test was 7%, and at the surface the CBR was 2%.

Figure 4.81 shows an overlay of the pavement settlement between the summer and early spring surveys with the FWD testing results. A load of 15 kip was used to accurately show the deflection of the subgrade. Stiffer responses to FWD testing correlate to reduced movement during the freeze/thaw cycle. The higher deflections from the FWD testing were located where the backfill had lower CBR values.

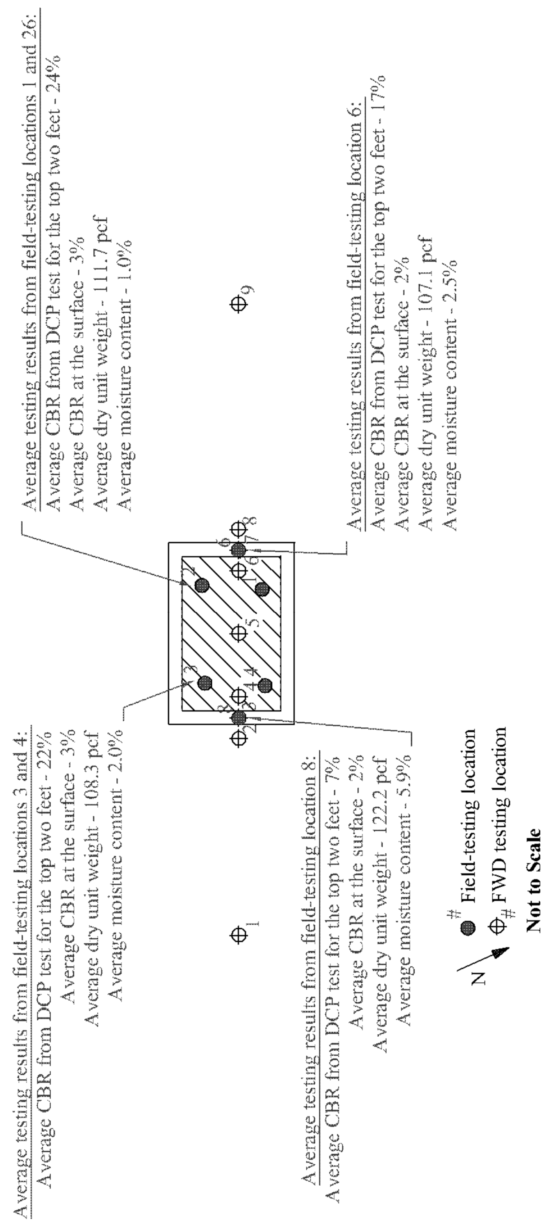


Figure 4.80. Comparison of CBR values, dry unit weights, and FWD testing results for Trench D

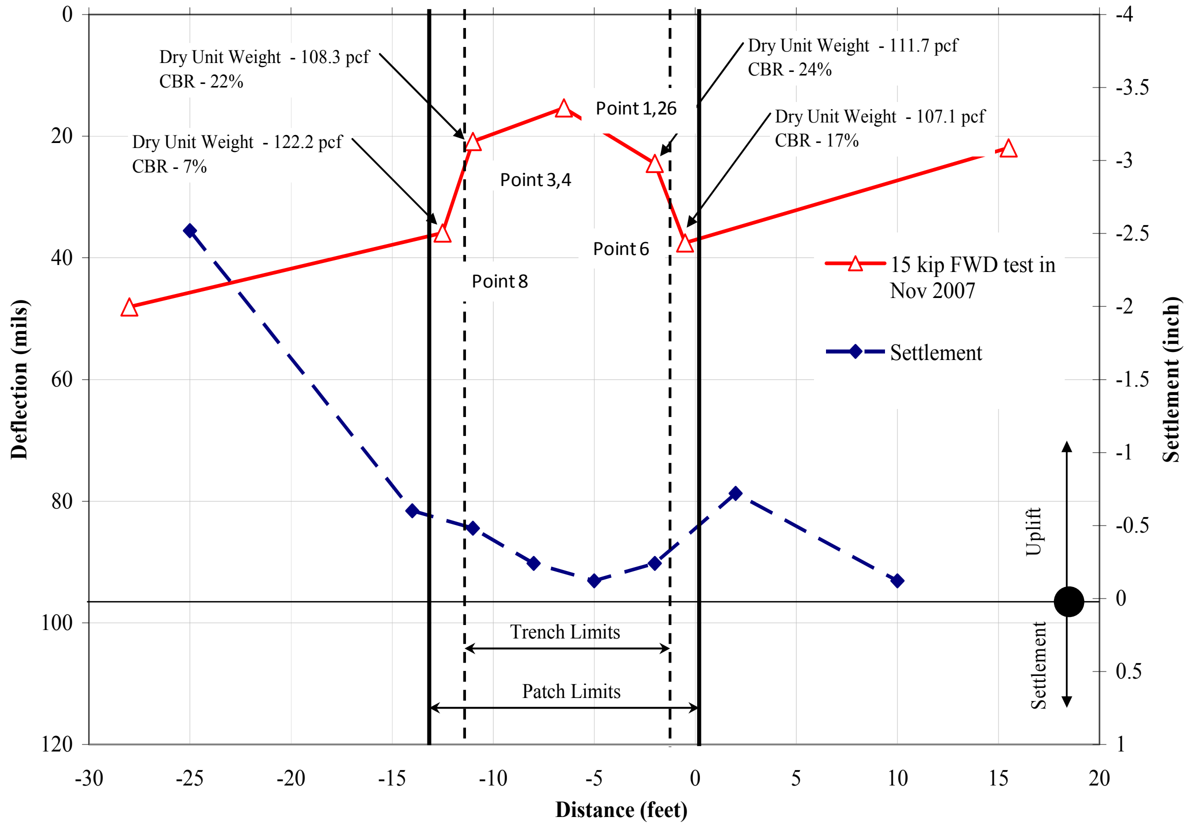


Figure 4.81. Falling weight deflectometer test (15 kip) and settlement for Trench D

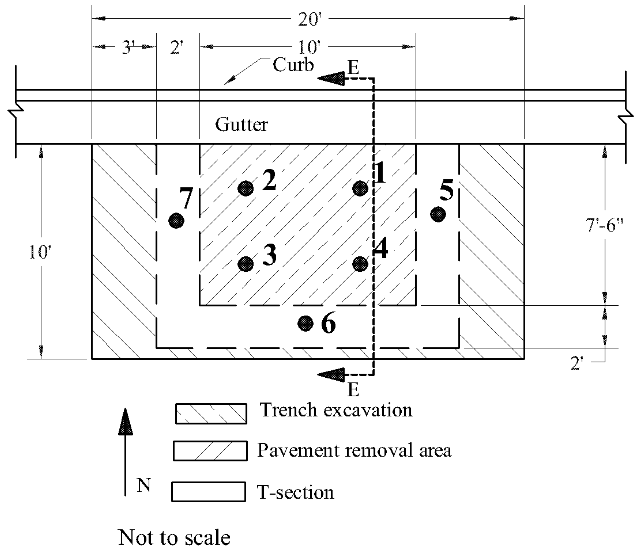
Summary

- The backfill was placed at medium-to-dense relative densities ranging from 57% to 63%.
- The maximum settlement was 0.24 inches in the soil adjacent to the trench on the south side of the trench. At this location, there were low CBR values.
- The patch moved independent of the surrounding pavement during the winter. The patch experienced less uplift than the surrounding pavement. Even though the patch was performing better during winter conditions, it would still be perceived as a dip in the road during the winter.
- The FWD test results showed high deflections occurred where the dry unit weights for the trench were the largest. The low FWD deflections occurred where the CBR values were higher in the trench.

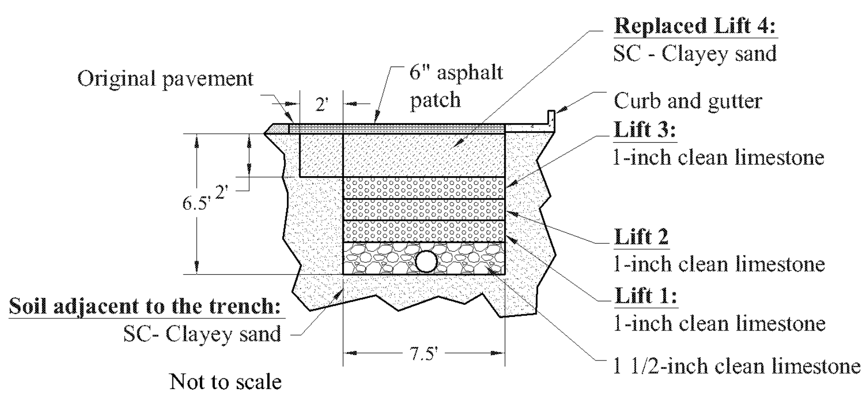
Recommended Trench E

Nuclear density gauge and DCP tests were performed in Trench E located on 7th Street and Carroll Avenue. On July 12, 2007, the third and fifth lifts were tested at four points (see Figure

4.82). The trench was left open for six days. On July 18, 2007, the T-section was excavated 2 feet, which removed the fifth lift. Testing was performed on the same four points (1, 2, 3, and 4) used to test the third and fifth lifts and then three additional test points (5, 6, and 7) in the T-section.



(a)



(b)

Figure 4.82. Location of test points in Trench E; cross-section of the trench

Nuclear Density Test Results

Tables 4.24 and 4.25 summarize the dry unit weights and moisture contents from the nuclear density testing results for Trench E. The probe depth was 6 inches.

The third and fifth lifts were constructed with 1-inch clean limestone. The classification was GP, poorly graded gravel. The maximum dry unit weight was 132.2 pcf.

The replaced fifth lift was constructed with mixtures of 1-inch clean limestone and soil from the City of Ames soil supply piles. These backfills were classified as SC, clayey sand. The Standard Proctor test results of the SC backfill show a maximum dry unit weight of 106.2 pcf at a moisture content of 17.0%. The range of optimum moisture content was 11% to 19%.

On July 12, 2007, the third lift, for the four test points, had an average dry unit weight of 105.6 pcf (54% relative density) and an average moisture content of 5.6%. The fifth lift, for four test points, had an average dry unit weight of 102.2 pcf (47% relative density) and an average moisture content of 3.4%. Both lifts were at a medium density after compaction. The fifth lift was removed during the construction of the T-section.

On July 18, 2007, the replaced fifth lift, for the four test points within the trench, had an average dry unit weight of 99.8 pcf and an average moisture content of 10.6%. The average dry unit weight was placed at 94% of the maximum from the Standard Proctor test. The moisture content of the backfill for all test points was below the optimum moisture content.

The replaced fifth lift, for the three points located in the T-section (points 5, 6, and 7), had an average dry unit weight of 98.7 pcf and an average moisture content of 12.2%. Based on laboratory testing, the backfill in the T-section was placed at 93% of the Standard Proctor. The moisture content of the backfill during placement was below the optimum moisture content, which increases the collapse potential of the backfill.

Table 4.24. Dry unit weight results from the nuclear density tests on Trench E, with backfill having γmax of 133 pcf and γmin of 86 pcf

Location	Number of test points	Average dry unit weight (pcf)	Relative density (%)	Min/Max dry unit weight (pcf)	Standard deviation	Coefficient of variance (%)
Third lift	4	105.6	54%	103.3/111.3	3.8	3.6
Fifth lift tested on July 12, 2007	4	102.2	47%	99.5/104.6	2.3	2.3
Replaced fifth lift at test points within the trench tested on July 18, 2007	4	99.8	N/A	94.5/102.7	5.0	5.0
Replaced fifth lift at test points in the T-section tested on July 18, 2007	3	98.7	N/A	95.5/104.5	5.0	5.1

Table 4.25. Moisture content results from the Nuclear Density tests on Trench E

Location	Number of test points	Average moisture content (%)	Min/Max moisture content (%)	Stan-dard devia-tion	Coefficient of variance (%)
Third lift	4	3.0	4.9/6.0	0.5	16.6
Fifth lift tested on July 12, 2007	4	3.4	3.2/3.5	0.2	5.9
Replaced fifth lift at test points within the trench tested on July 18, 2007	4	10.6	6.8/12.9	2.7	25.5
Replaced fifth lift at test points in the T-section tested on July 18, 2007	3	12.2	8.8/14.7	3.1	25.4

Figure 4.83 shows the average field-testing results for the third and fifth lifts. These lifts were placed at medium dense compaction. The field-testing results are circled and show the test points were uniform within the trench.

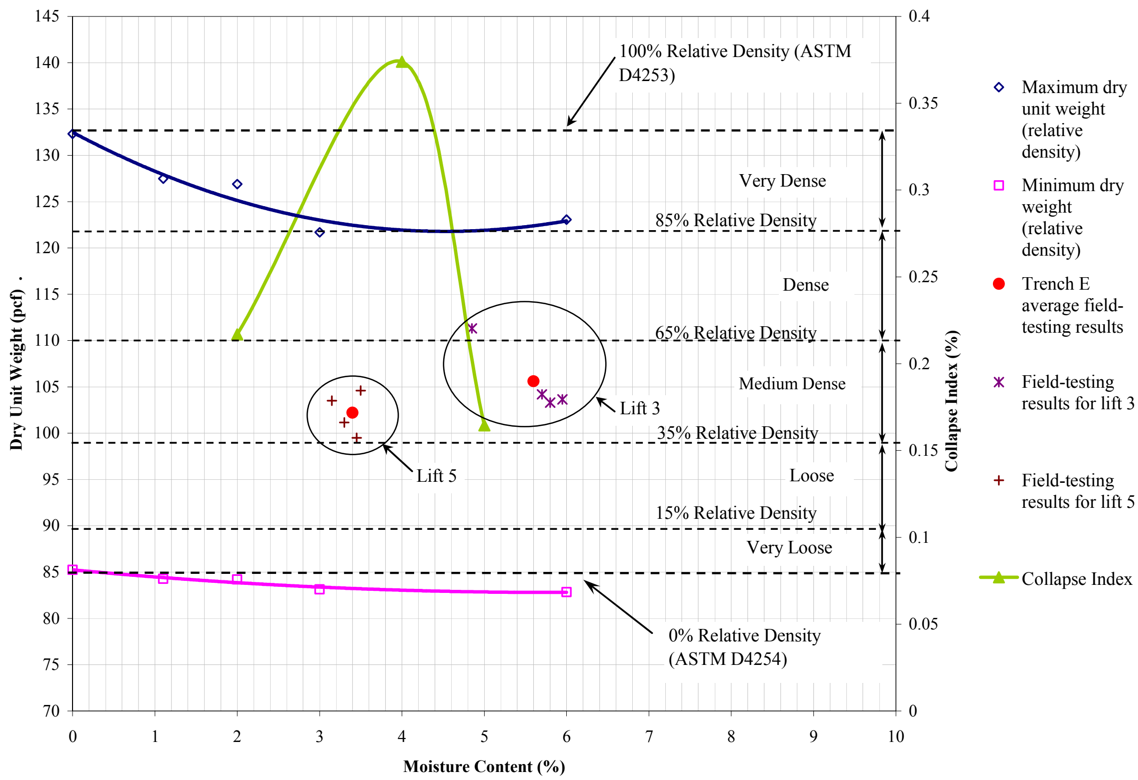


Figure 4.83. Relative density test results from Phase I with field-testing results from the third and fifth lifts of Trench E

The additional backfill was placed at a dry unit weight ranging from 98.7 to 105.6 pcf and moisture content of 3% and 12.2%.

DCP Test Results

Table 4.26 summarizes the DCPI readings and Table 4.27 summarizes the average CBR results from the DCP test for Trench E. The third and fifth lifts were constructed with 1-inch clean limestone.

The replaced fifth lift was constructed with two mixtures of 1-inch clean limestone and soil from the City of Ames soil supply piles. These backfills were classified as SC (clayey sand).

Table 4.26. DCPI results from the DCP tests for Trench E

Location	Number of test points	Average depth of tests (inches)	Average DCPI	Standard deviation	Coefficient of variance (%)
Third lift	4	22.7	14.3	9.2	
Fifth lift tested on July 12, 2007	4	24.5	19.8	9.3	47.0
Replaced fifth lift at test points within the trench tested on July 18, 2007	4	27.1	44.5	29.7	66.7
Replaced fifth lift at test points in the T-section tested on July 18, 2007	3	28.5	74.2	16.1	21.7

Table 4.27. Average CBR results from the DCP tests for Trench E

Location	Number of test points	Average depth of tests (inches)	Average CBR (%)	Standard deviation	Coefficient of variance (%)
Third lift	4	22.7	28	12	
Fifth lift tested on July 12, 2007	4	24.5	16	12.7	79.4
Replaced fifth lift at test points within the trench tested on July 18, 2007	4	27.1	13	17.6	135.4
Replaced fifth lift at test points in the T-section tested on July 18, 2007	3	28.5	3	0.8	26.7

For the four test points in the third lift, the CBR values had an average of 28% for an average depth of 22.7 inches. It should be noted the DCP test at point 1 was terminated at 12.6 inches. At the surface of the lift, the CBR values ranged from 4% to 10%. At the termination of the DCP tests, the CBR values ranged from 12% to 56%. The average CBR value at the termination of the tests was 42%. The profile shows backfill follows the same generalized pattern through the depth of the profiles. This indicated the backfill was placed and compacted in a uniform manner. The profiles increased with depth, indicating the backfill increased in resistance with depth. Figure 4.84 plots profiles of the CBR values as a function of depth.

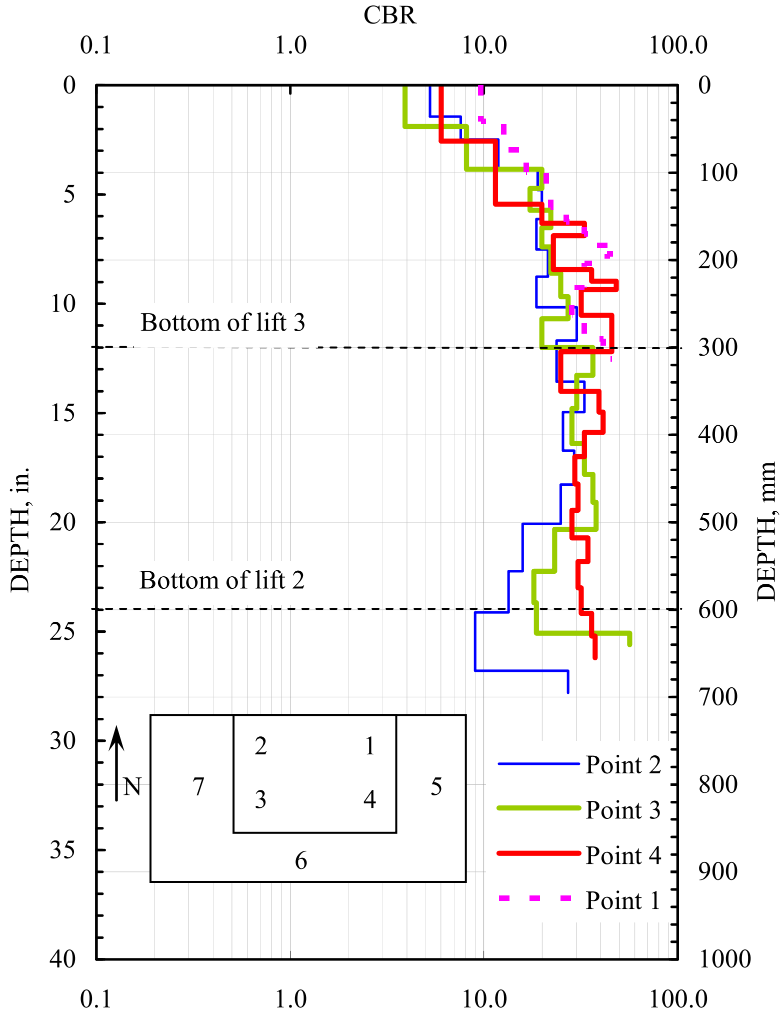


Figure 4.84. CBR results from the DCP test profiles for the third lift in Trench E

The fifth lift, tested on July 12, 2007, for the four test points within the trench, had an average CBR of 16% extending 24.5 inches (see Figure 4.85). At the surface of the lift, the CBR values ranged from 3% to 5%. The CBR values at the termination of the DCP tests ranged from 22% to 53%. The profiles all followed the same pattern with increasing CBR as a function of depth, suggesting the lifts were compacted in an even manner and indicating the increases in soil resistance and strength.

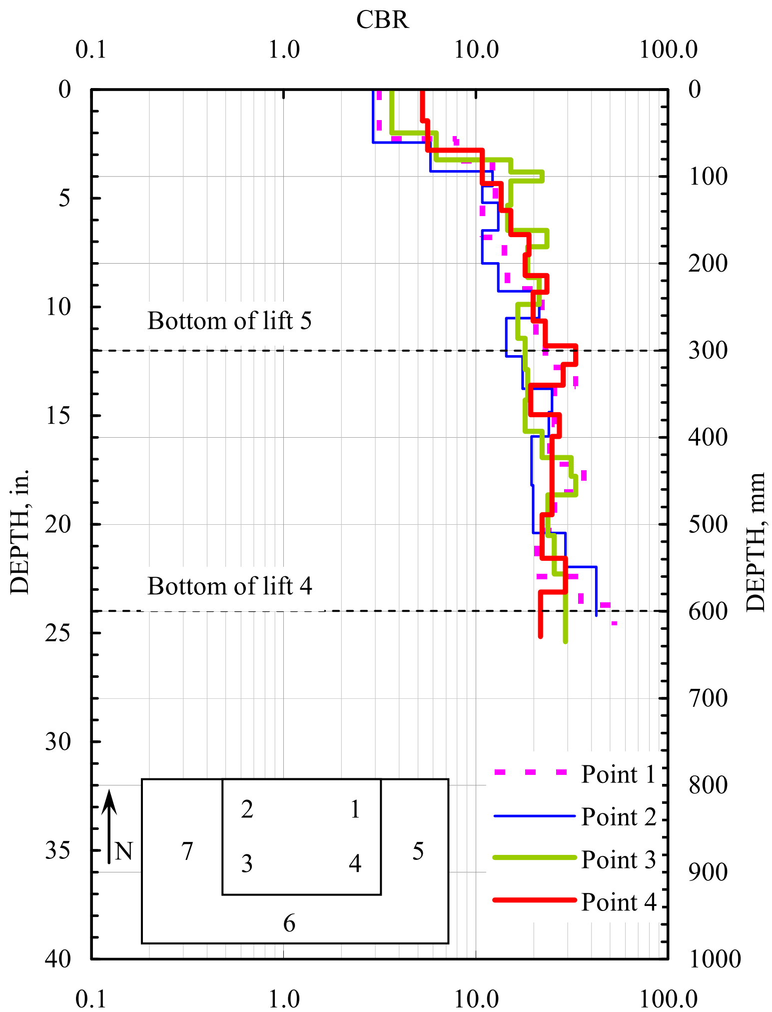


Figure 4.85. CBR profiles for test points within the fifth lift in Trench E on July 12, 2007

The replaced fifth lift, tested on July 18, 2007, for the four test points within the trench, had an average CBR of 13% extending 27.1 inches (see Figure 4.86). At the surface of the lift, the CBR values ranged from 2% to 3%. At the termination of the test, the CBR values ranged from 4% to 65%. At a depth between 17 and 20 inches for test points 2, 3, and 4, the CBR values increased from between 12% and 13% to a range of 32% to 54%. This indicated the DCP probe penetrated to the 1-inch clean limestone below.

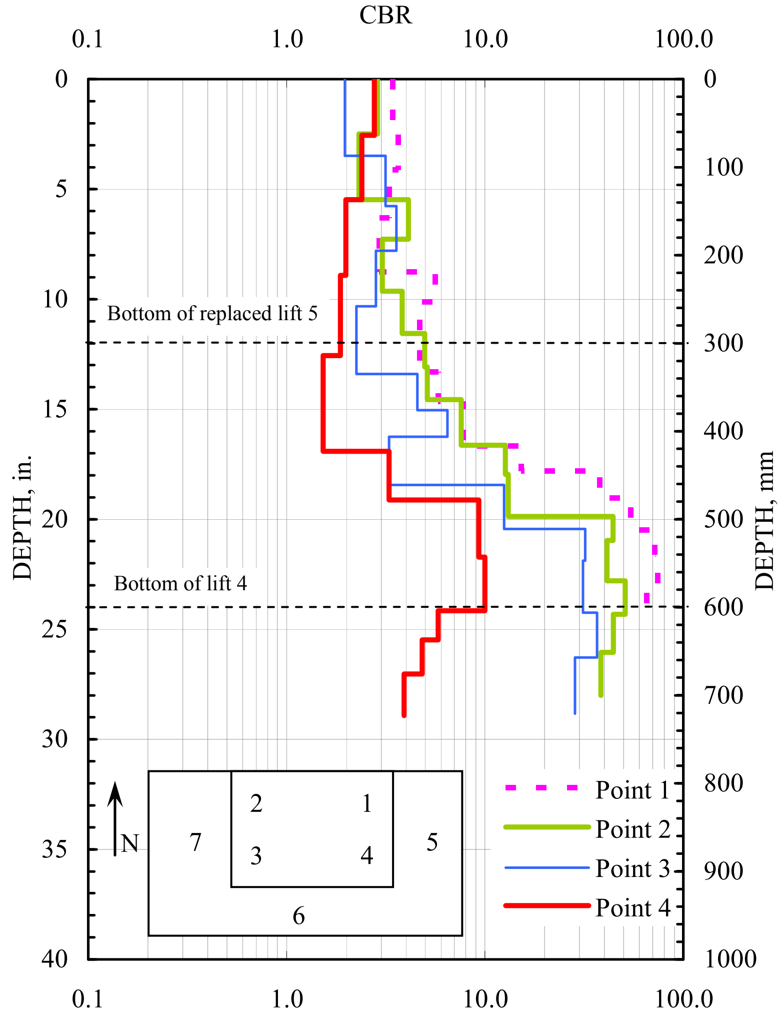


Figure 4.86. CBR profiles for test points within the replaced fifth lift in Trench E on July 18, 2007

Figure 4.87 shows the CBR profiles for the three points in the T-section tested on July 18, 2007. The replaced fifth lift had an average CBR of 13% extending 28.5 inches. The average CBR values for the top 2 feet ranged from 2% to 3%. At the surface and termination of the lift, the CBR values ranged from 2% to 3%. This indicates the backfill within the T-section was not compacted sufficiently.

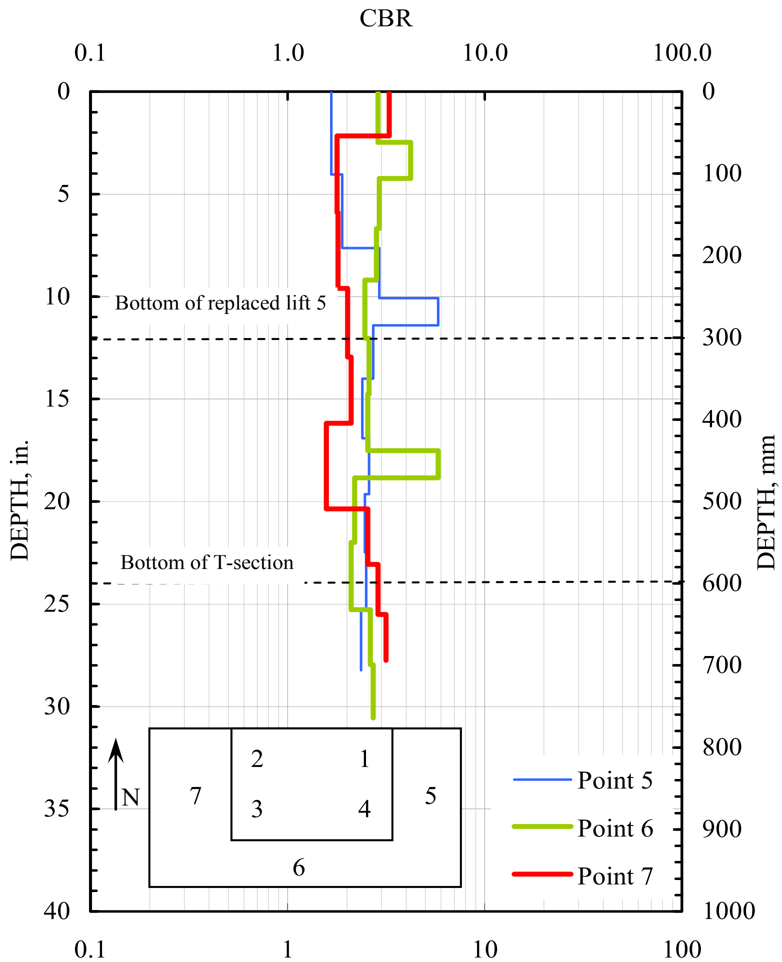


Figure 4.87. CBR profiles for test points within the T-section of the fifth lift on July 18, 2007

FWD Test Results

Falling weight deflectometer testing was performed on November 5, 2007. Figure 4.88 shows the testing locations for Trench E. Figure 4.89 shows the FWD testing results for Trench E on November 5, 2007. Figure 4.90 shows the results from the June 26, 2008, FWD testing, and Figure 4.91 shows results from the March 25, 2009, testing. Figure 4.92 shows overall comparison of these two data sets with 12-kip load (which are almost identical). Note a 15-kip drop was chosen since it gives a good estimation of the trench's subgrade. The FWD testing for Trench E showed the zone of influence was present on both the east and west sides of the trench. The FWD tests within the trench showed the backfill was placed relatively evenly across the trench without the peak stiffness seen in the other trenches.

For the 15-kip load, the deflection in the middle of the trench was 14.7 mils. The center of the trench did not exhibit the decrease in deflection as seen in the other trenches. Comparatively, the backfill in this trench gives a rather uniform response. At the edge of the trench at FWD testing locations 5 and 7, the deflections were 14.7 mils and 18.7 mils, respectively. At FWD testing location 1, the deflection was 15.8 mils. The T-section on the west side had a higher deflection than on the east side of the trench.

The FWD diagram shows, on the east side of the trench, the zone of influence is present. The zone of influence was beyond the outer limits of the T-section. On the west side of the trench, the deflections were high in comparison to other locations. Figure 4.93 shows there was no visible cracking in the pavement. Based on the photographs taken during construction of the trench, equipment was only located on the east side of the trench (see Figure 4.19). This does not account for the difference in the response to the FWD testing.

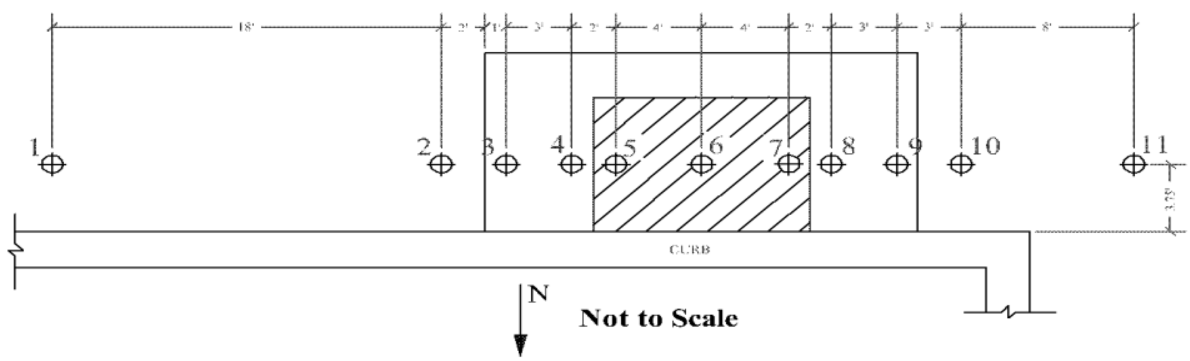


Figure 4.88. Falling weight deflectometer test locations for Trench E

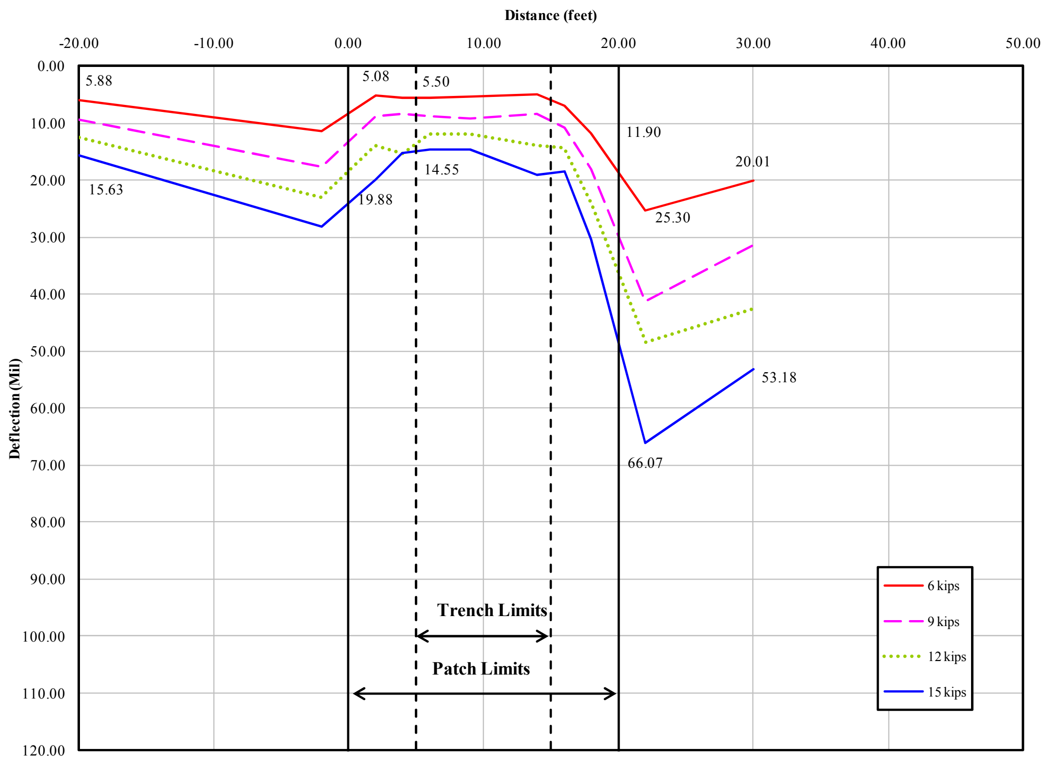


Figure 4.89. Falling weight deflectometer test results for Trench E on November 5, 2008, at a temperature of 41.3ºF

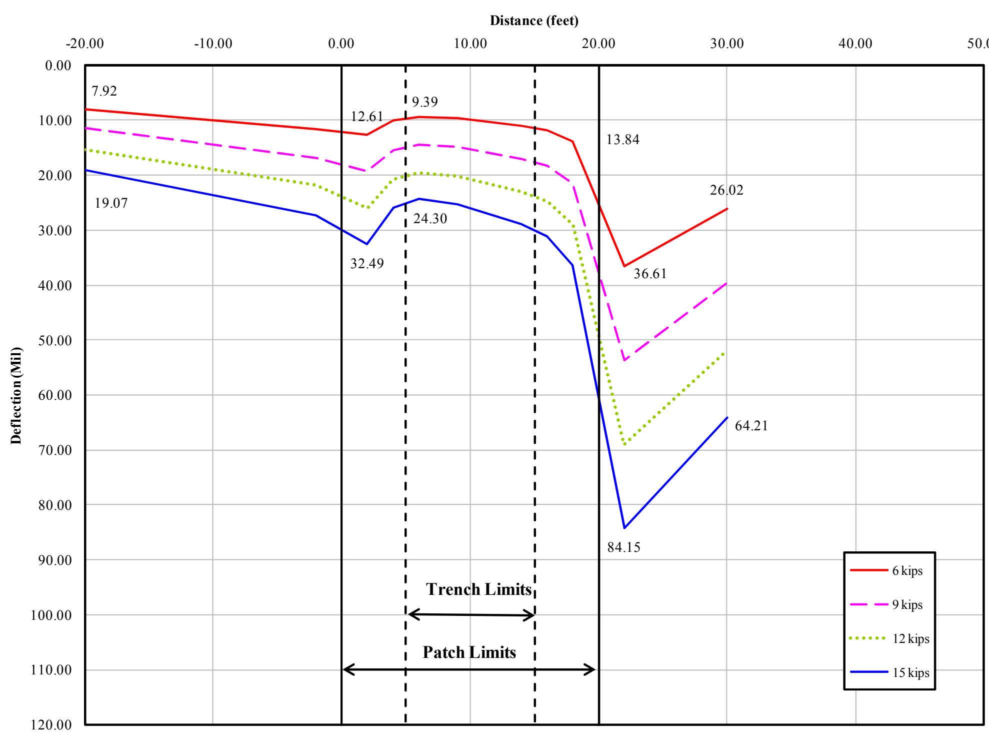


Figure 4.90. Falling weight deflectometer test results for Trench E on June 26, 2008, at a temperature of 70.3ºF

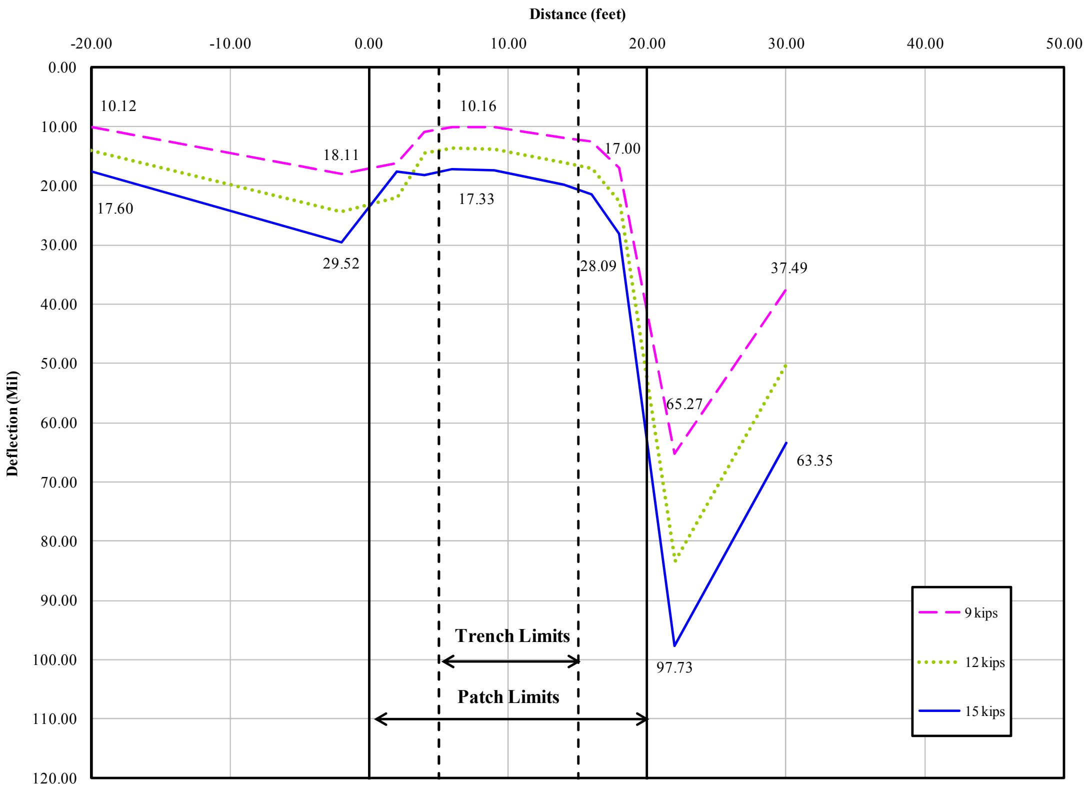


Figure 4.91. Falling weight deflectometer test results for Trench E on March 25, 2009, at a temperature of 37.7ºF

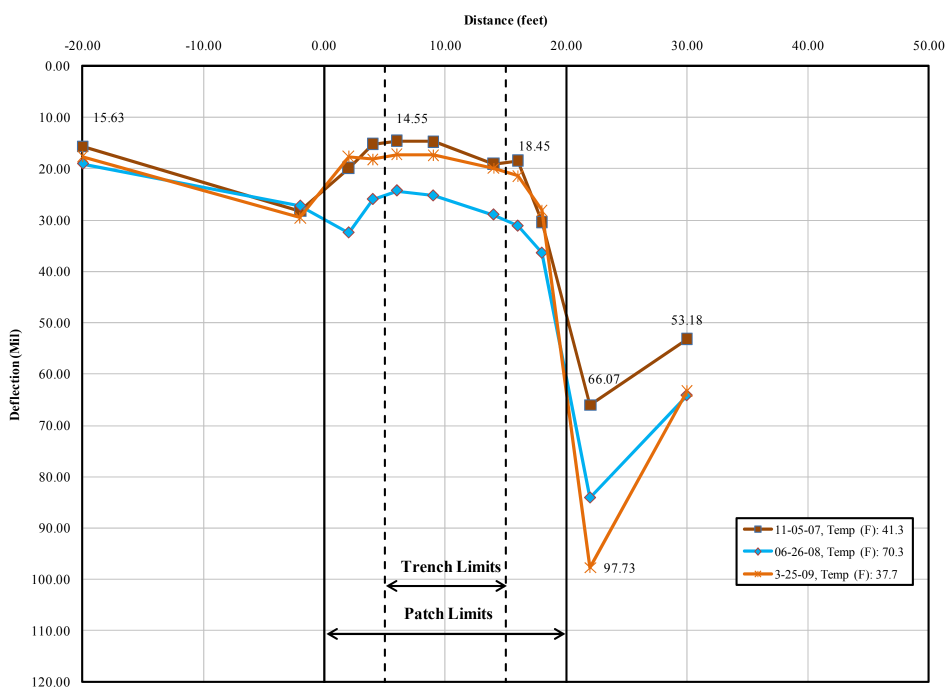


Figure 4.92. Falling weight deflectometer test results for Trench E compared (15-kip load)

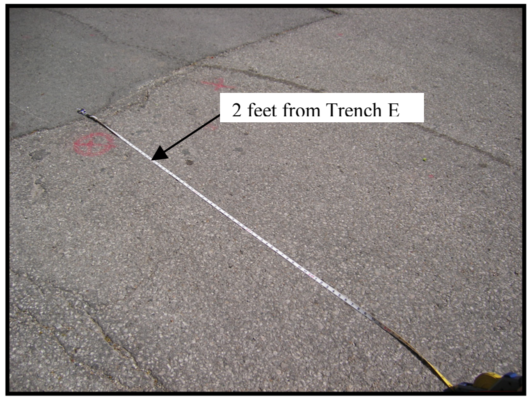


Figure 4.93. Condition of the pavement 2 feet west of Trench E at FWD testing location 10

Post-Construction Elevation Survey

The post-construction surfaces were constructed using survey data collected on July 20, 2007; March 19, 2008; June 10, 2008; August 19, 2008; November 18, 2008; and March 13, 2009. Figure 4.78 shows the location of the grid points.

Trench E was surveyed at 46 grid points. The benchmark was the hydrant northwest of the trench at the intersection of 6th Street and Carroll Avenue. The average settlement was 0.36 inches, and the maximum settlement was 0.72 inches along the south edge of the trench at survey points 22 and 26. Figure 4.94 shows the location of the grid points.

Figure 4.95 shows the settlement and elevation profiles for Trench E. This shows the trench settled since construction. Five feet west of the trench, the pavement was in a state of uplift. This caused the settlement of the patch to appear greater.

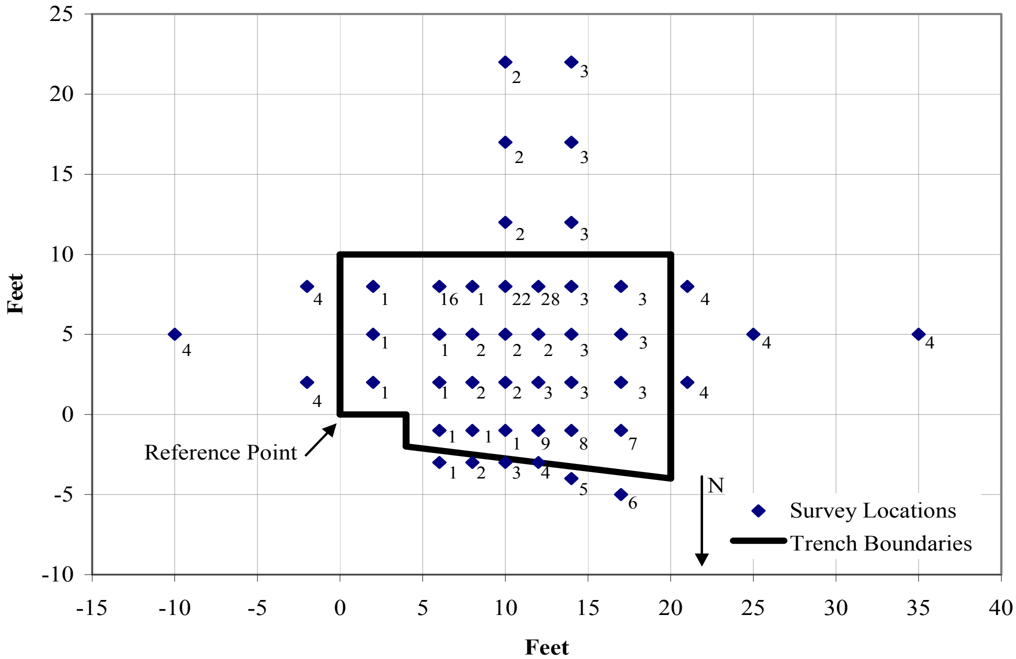


Figure 4.94. Pavement surface elevations of Trench E

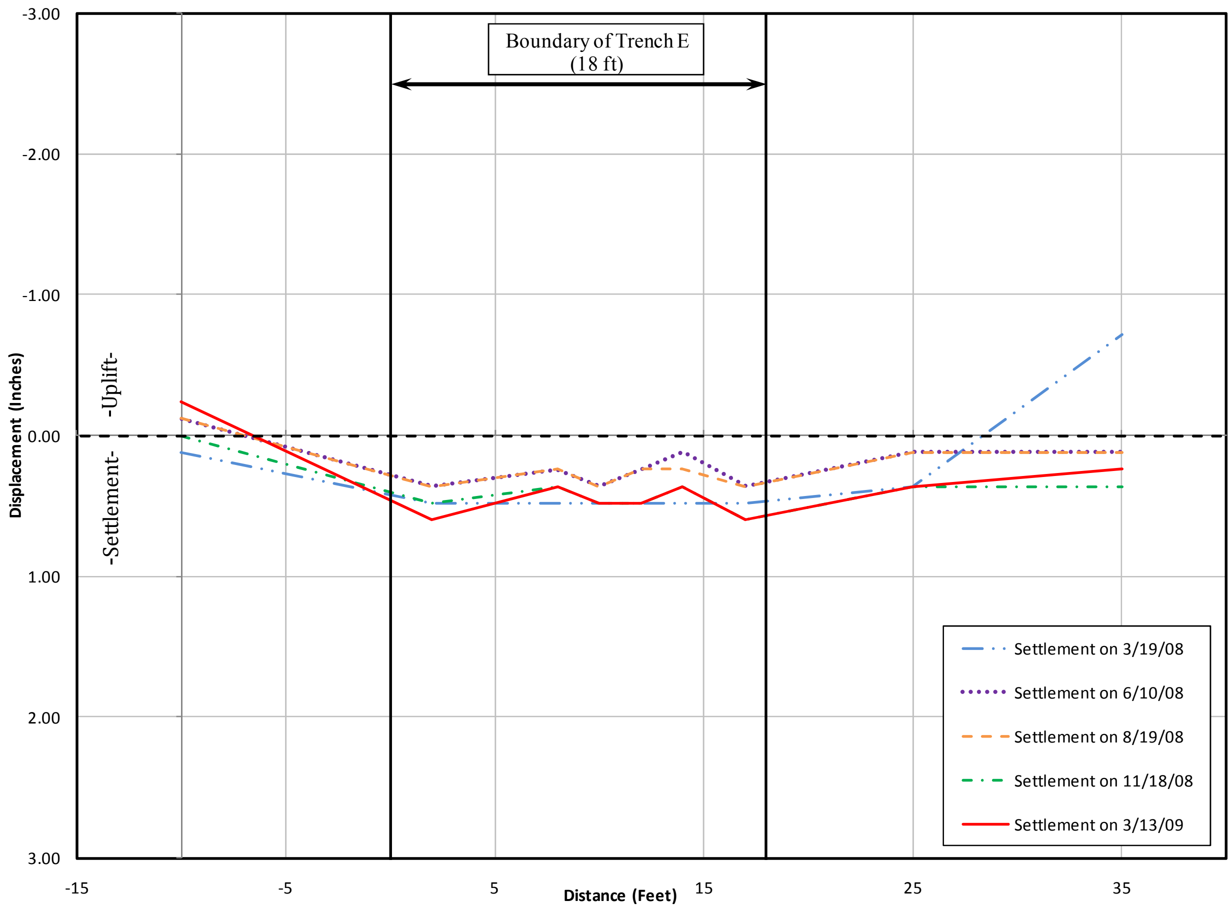


Figure 4.95. Settlement along the center line of Trench E (points 46, 14, 17, 20, 23, 19, 32, 38, 42, 43)

Comparison of Field-Testing Results to Long-Term Monitoring

Figure 4.96 shows the FWD locations and the field-testing locations superimposed with field-testing results averaged. The dry unit weights at test points 5 and 7 were 104.5 pcf and 95.5 pcf, respectively. At the top of the fifth lift, the CBR value was 2% and 3% for test points 5 and 7, respectively. The higher deflection on the west side of the trench was the result of the backfill not being compacted to a dense state. Figure 4.97 shows the settlement of the trench with the FWD deflections and field-testing results. A load of 15 kip was used to accurately show the deflection of the subgrade. The maximum settlement of Trench E was 0.72 inches along the south edge of the trench. The dry unit weight at test point 6 on the south side of the trench was 96.1 pcf at a moisture content of 14.7%. The CBR values from the DCP tests were constant with depth. This figure shows that where the FWD deflections were lowest, the settlement was the highest. Outside the trench, the highest deflection on the FWD results occurred where the trench had uplifted during the winter.

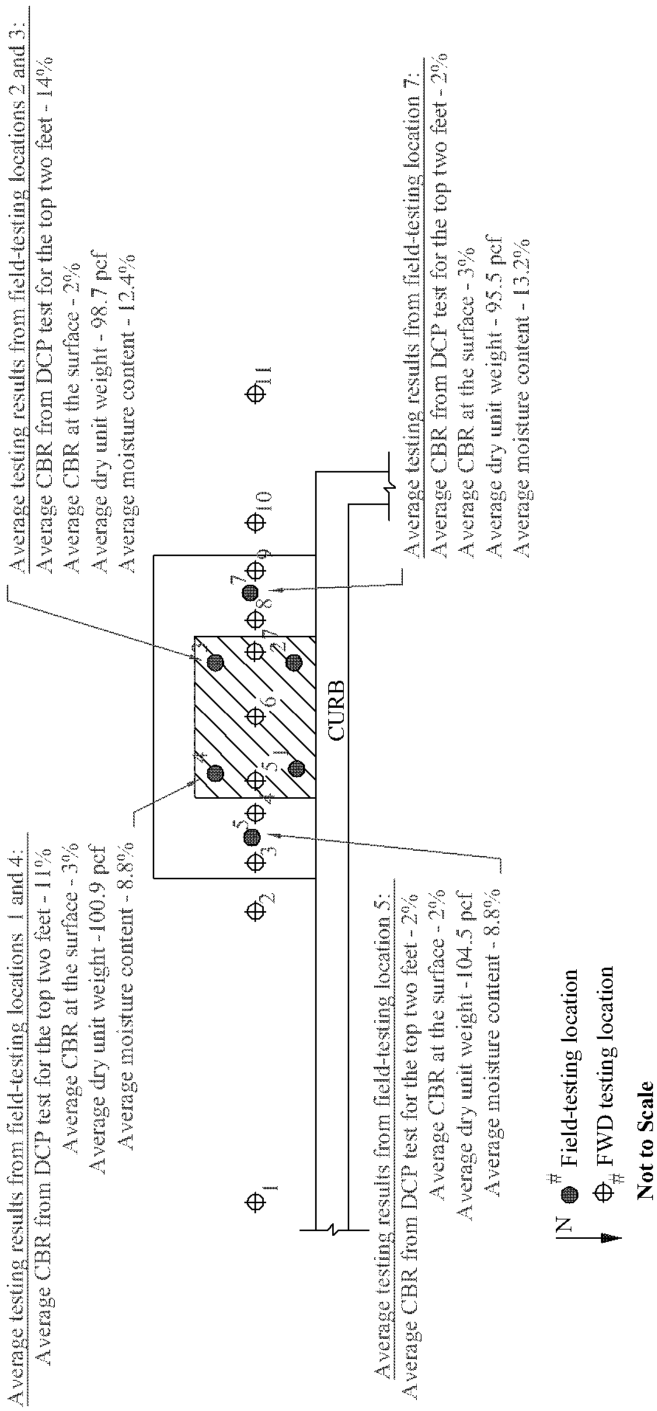


Figure 4.96. Comparison of CBR values, dry unit weights, and FWD testing results for Trench E

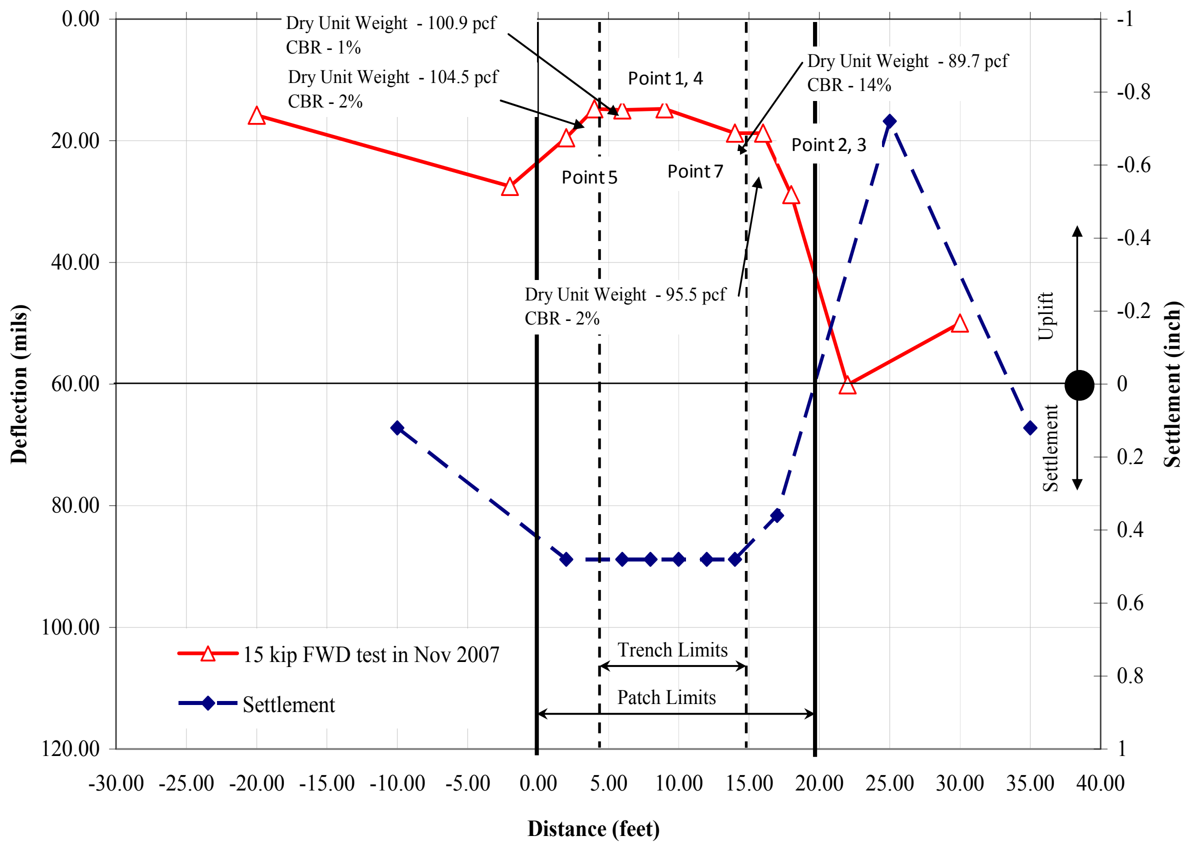


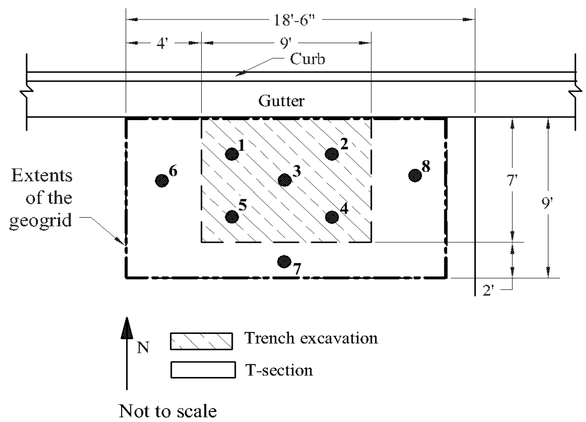
Figure 4.97. Falling weight deflectometer test (15 kip) and settlement for Trench E

Summary

- The third lift was placed at a medium dense compaction.
- The backfill used in the T-section was placed at 93% to 94% of the Standard Proctor. The backfill was dry of optimum, which will increase its collapse potential.
- The T-section was not compacted to the same dry unit weights as the backfill within the trench. The CBR values in this T-section were also below the CBR values in the center of the trench.
- The highest deflection in the trench from the FWD testing was the result of the low density of the backfill during construction.
- The zone of influence was present on both sides of the trench outside the T-section.
- The highest settlement was located on the south edge of the trench where the backfill was placed at a low density.
- West of the trench the pavement was uplifted during the winter, causing the settlement of the patch to be perceived as greater than what it actually was.

Recommended Trench F

Nuclear density and DCP tests were performed on Trench F located on 6th Street and Carroll Avenue. On July 12, 2007, the second and fourth lifts were tested at five points (points 1, 2, 3, 4, and 5). On July 18, 2007, after the excavation of the fourth lift and additional soil from the T-section, the trench was tested at the same five test points within the trench as well as three additional points in the T-section for a total of eight test points (see Figure 4.98).



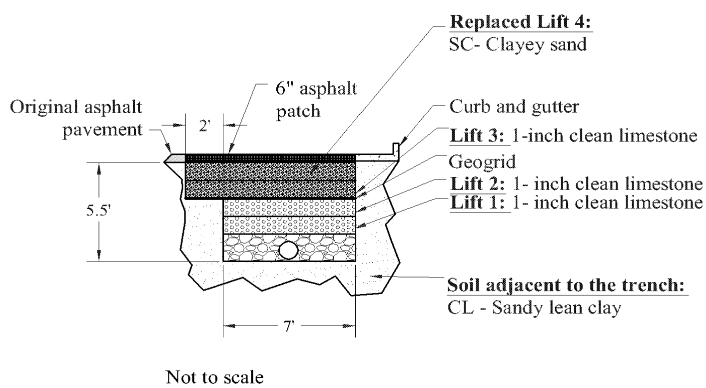


Figure 4.98. Location of test points in Trench F; cross-section of the trench

Nuclear Density Test Results

Table 4.28 summarizes the dry unit weights, and Table 4.29 summarizes the moisture content results from the nuclear density tests for Trench F. The probe depth was 6 inches.

The second and fourth lifts were constructed with 1-inch clean limestone. The classification was GP (poorly graded gravel). The replaced fourth lift was constructed with mixtures of 1-inch clean limestone and soil from the City of Ames soil supply piles. To evaluate the placement of the backfill, the Standard Proctor tests were performed on the additional backfill material. Laboratory testing found the maximum dry unit weight was 106.2 pcf at a moisture content of 17.0%.

On July 12, 2007, the second lift at the five test points had an average dry unit weight of 95.2 pcf and an average moisture content of 5.2%. Based on laboratory testing, the backfill was placed at 29% relative density. This corresponded to loose compaction according to Table 1.6. The fourth lift, tested at five test points, had an average dry unit weight of 99.0 pcf and an average moisture content of 3.6%. Based on laboratory testing, the backfill was placed at 39% relative density. This corresponded to medium dense compaction from Table 1.6. This lift was removed for the construction of the T-section.

The replaced fourth lift, for the five test points within the trench, had an average dry unit weight of 108.8 pcf and an average moisture content of 12.3%. Based on laboratory testing, the backfill was placed at 92% of the Standard Proctor at 106.2 pcf. The moisture content was below the optimum moisture content.

The replaced fourth lift, for the three test points located in the T-section, had an average dry unit weight of 110.6 pcf and an average moisture content of 12.6%. Based on laboratory testing, the backfill was placed at 93% of the Standard Proctor test.

Table 4.28. Dry unit weight results from the nuclear density tests on Trench F, with backfill having γmax of 133 pcf and γmin of 86 pcf

Location	Number of test points	Average dry unit weight (pcf)	Relative density (%)	Min/Max dry unit weight (pcf)	Standard deviation	Coefficient of variance (%)
Second lift	5	95.2	29	90.7/99.1	4.2	4.4
Fourth lift tested on July 12, 2007	5	99.0	39	85.7/104.1	3.5	12.1
Replaced fourth lift for test points within the trench tested on July 18, 2007	5	108.8	N/A	99.7/114.3	5.7	5.2
Replaced fourth lift for test points in the T-section tested on July 18, 2007	3	110.6	N/A	108.1/112.3	5.1	2.1

Table 4.29. Moisture content results from the nuclear density tests on Trench F

Location	Number of test points	Average moisture content (%)	Min/Max moisture content (%)	Standar d deviatio n	Coefficient of variance (%)
Second ift	5	5.6	3.4/7/3	1.0	18.8
Fourth lift tested on July 12, 2007	5	3.6	3.5/4.0	0.2	5.4
Replaced fourth lift for test points within the trench tested on July 18, 2007	5	12.3	9.3/14.5	1.8	14.5
Replaced fourth lift for test points in the T-section tested on July 18, 2007	3	12.6	9.3/12.3	3.0	23.8

Figure 4.99 shows the field placement results for the second and fourth lifts for Trench F. The backfill was placed at loose density with nonuniform compaction.

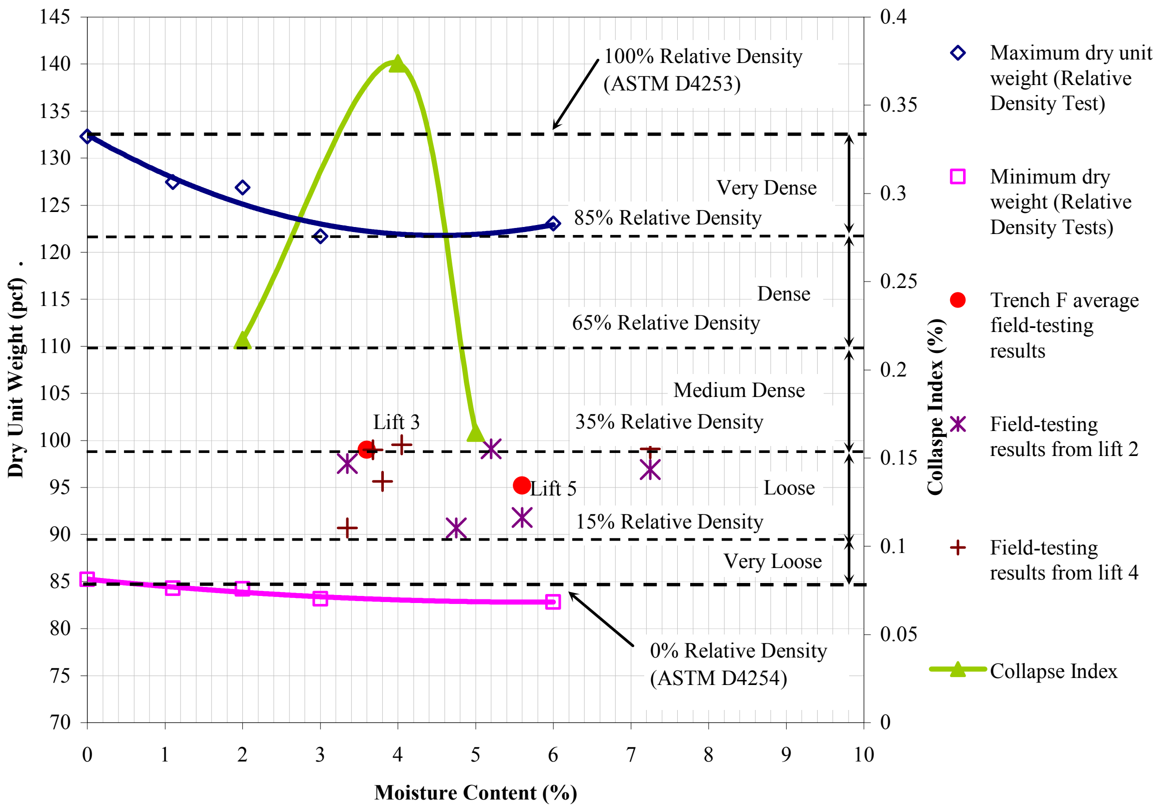


Figure 4.99. Relative density test results with field-testing results for Trench F

The Standard Proctor test results for the backfill used in the top 2 feet of the trench show a maximum dry unit weight of 119.1 pcf at an optimum moisture content of 13.2%. The figure shows the field placement of the backfill was between 90% and 95%. The backfill within the T-section was placed at a higher dry unit weight than within the trench.

DCP Test Results

Tables 4.30 and 4.31 summarize the DCPI readings and the average CBR results from the DCP tests for Trench F. The second and fourth lifts were constructed with 1-inch clean limestone. The classification was GP (poorly graded gravel). The replaced fifth lift was constructed with two mixtures of 1-inch clean limestone and soil from the City of Ames soil supply piles. Standard Proctor laboratory testing found the maximum dry unit weight was 106.2 pcf and the optimum moisture content was 17.0%.

Table 4.30. DCPI results from the DCP tests for Trench F

Location	Number of test points	Depth of test (inches)	Average DCPI	Standard deviation	Coefficient of variance (%)
Second lift	5	24.6	30.6	22.7	74.9
Fourth lift tested on July 12, 2007	5	24.5	22.1	9.6	43.2
Replaced fourth lift for test points within the trench tested on July 18, 2007	5	30.5	2.4	11.6	493.6
Replaced fourth lift for test points tested in the T-section on July 18, 2007	3	30.2	58.4	21.7	37.2

Table 4.31. Average CBR results from the DCP tests for Trench F

Location	Number of test points	Depth of test (inches)	Average CBR (%)	Standard deviation	Coefficient of variance (%)
Second lift	5	24.6	11	7.0	63.6
Fourth lift tested on July 12, 2007	5	24.5	15	7.1	47.3
Replaced fourth lift for test points within the trench tested on July 18, 2007	5	30.5	13	0.4	3.1
Replaced fourth lift for test points in the T-section tested on July 18, 2007	3	30.2	3	1.7	45.8

Figure 4.100 shows the CBR profiles for the five test points within the trench for the second lift. The second lift, tested on July 12, 2007, had an average CBR value of 11% extending 24.6 inches. At the surface of the lift, the CBR values ranged from 2% to 3%, and at the termination of the tests the CBR values ranged from 2% to 19%. At the start of the test, the CBR values were similar; however, at the termination of the test the CBR values were scattered. At a depth of 10 inches, the profiles diverge. This is the interface between the 1-inch and 1½-inch clean limestone, which was used as bedding for the pipe.

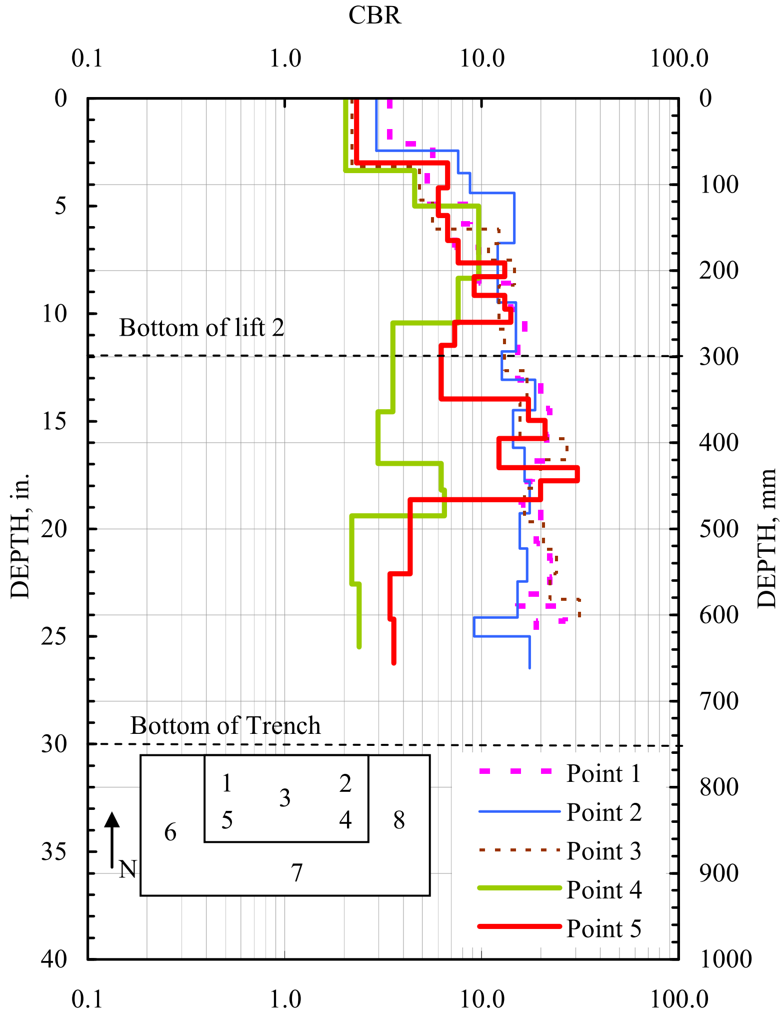


Figure 4.100. CBR results from the DCP test profiles for the second lift in Trench F

Figure 4.101 shows the CBR profiles for five test points tested on July 12, 2007, within the trench for the fourth lift. The fourth lift, tested on July 12, 2007, had an average CBR value of 15% extending 24.5 inches. At the surface of the lift the CBR values ranged from 2% to 3%, while at the termination of the test the CBR values ranged from 16% to 25%. The CBR profiles increased with depth. The profiles also followed the same pattern throughout the profile. This indicated the lift was evenly compacted.

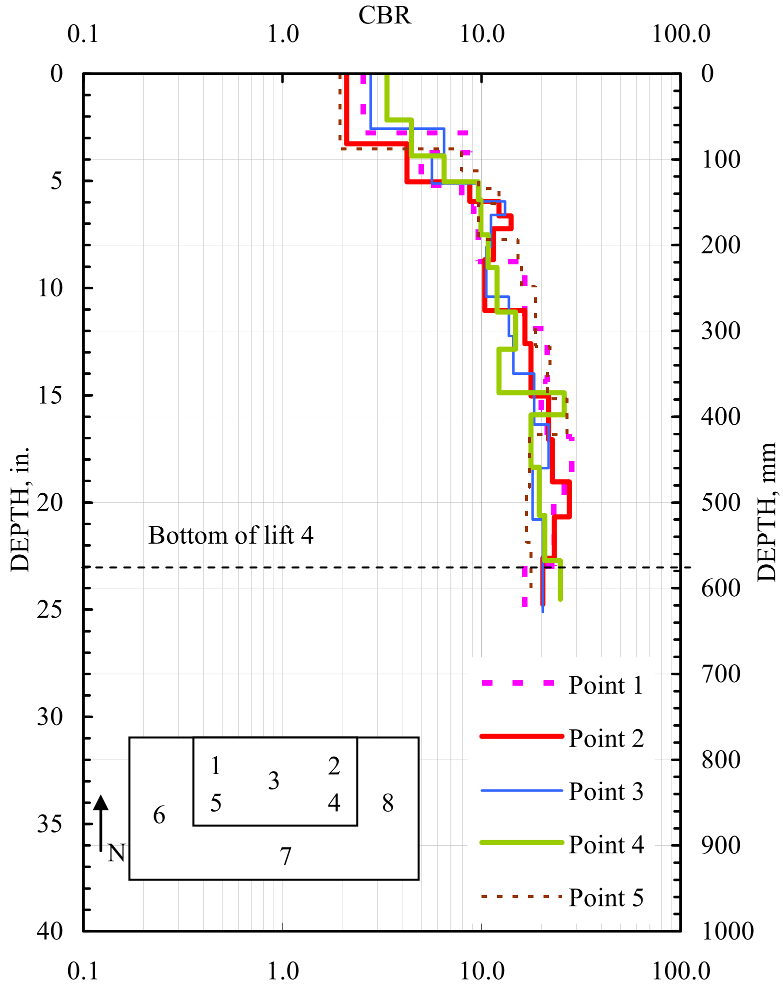


Figure 4.101. CBR profiles for test points within the fourth lift in Trench F on July 12, 2007

The replaced fourth lift, tested on July 18, 2007, at the five test points within the trench, had an average CBR value of 13% extending 30.5 inches (see Figure 4.102). At the surface of the lift, the CBR values ranged from 2% to 5%. At the termination of the DCP tests, CBR values ranged from 22% to 34%. However, as the depth increased, the CBR values increased. The CBR profiles had wide variation at the bottom of the lift, indicating compaction was not applied evenly across the trench.

Figure 4.103 shows the CBR profile for the three points tested on July 18, 2007, in the T-section, which had an average CBR value of 3% extending 32.0 inches. At the start of the test, the CBR values ranged from 2% to 4%. At the termination of the test, the CBR values ranged from 1% to 4%. These values are low and indicate the T-section did not receive significant compaction.

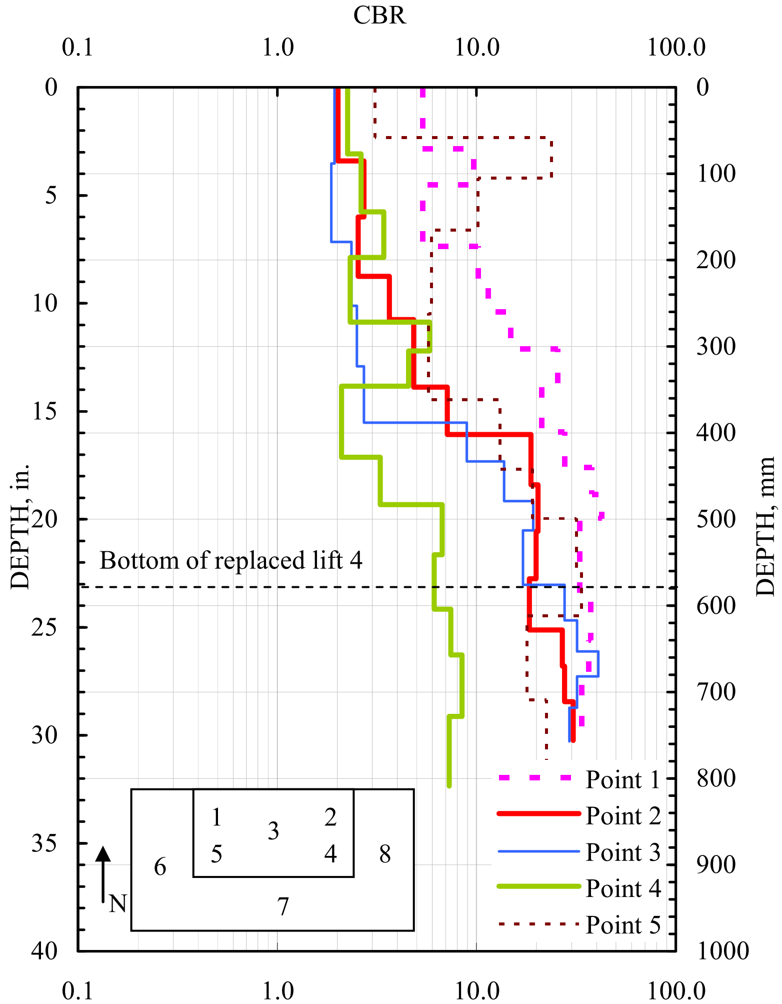


Figure 4.102. CBR profiles for test points within the replaced fourth lift in Trench F, July 18, 2007

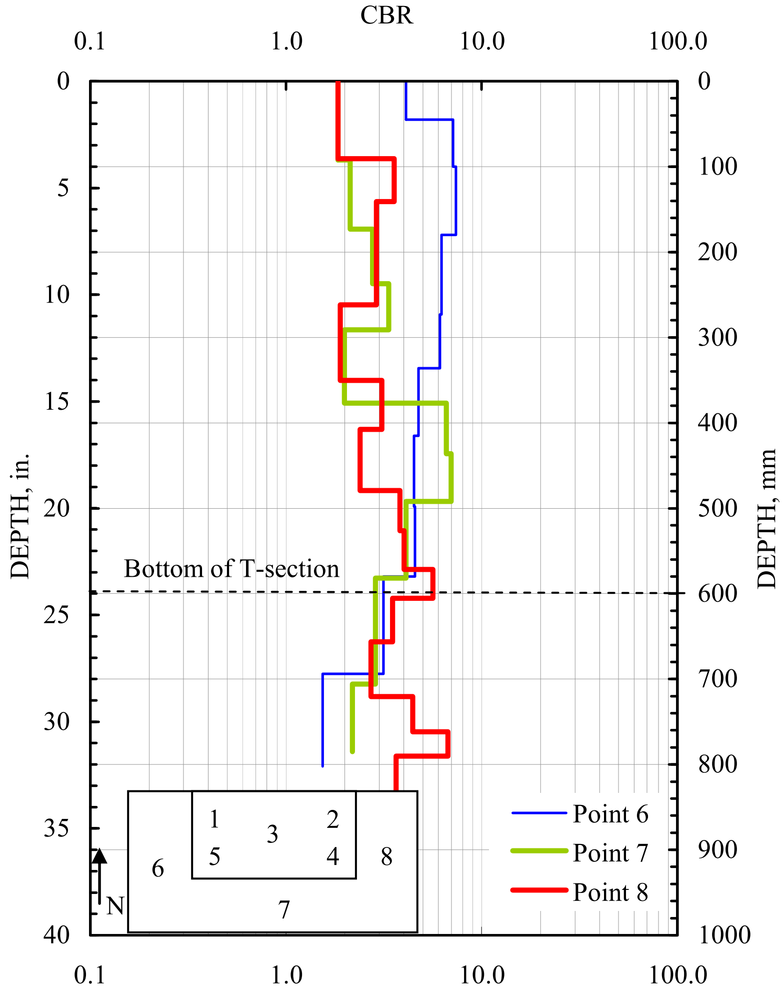


Figure 4.103. CBR profiles for test points within the replaced fourth lift of the T-section in Trench F, July 18, 2007

FWD Test Results

Figure 4.104 shows the FWD testing locations and Figure 4.105 shows the FWD testing results on November 5, 2007. The deflection from the 15-kip load in the center of the trench was 11.4 mils. The deflections at the edges of the trench were 15.9 mils and 21.3 mils. The zone of influence is reflected on both sides of the trench. To the west of the trench, there were no visible cracks in the pavement. Figure 4.106 shows the results from the June 28, 2008, FWD testing; Figure 4.107 shows the results from November 20, 2008; and Figure 4.108 shows results from the March 25, 2009, testing. Figure 4.109 shows overall comparison of the data sets with 15-kip load. Note a 15-kip drop was chosen since it gives a good estimation of the trench's subgrade. Figure 4.110 shows there was no visible cracking of the pavement where the high FWD deflections occurred. The construction equipment does not account for the difference in the FWD responses between the east and west sides of the trench. The pavement away from the trench deflected 8.5 mils. The backfill in the trench was not placed as stiff as the surrounding soil.

The FWD testing on Trench F shows the zone of influence was in the T-section on the east side of the trench; however, the zone of influence was present in the soil adjacent to the trench on the west side of the trench.

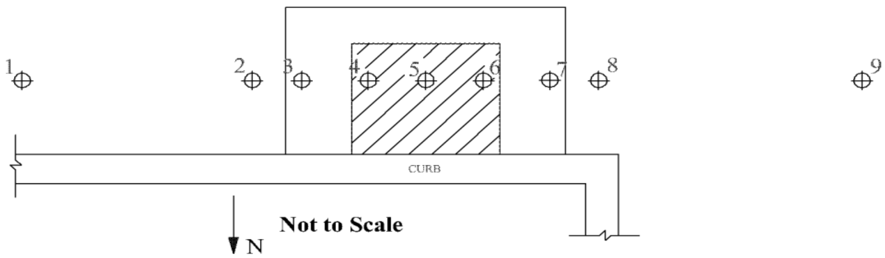


Figure 4.104. Falling weight deflectometer test locations for Trench F

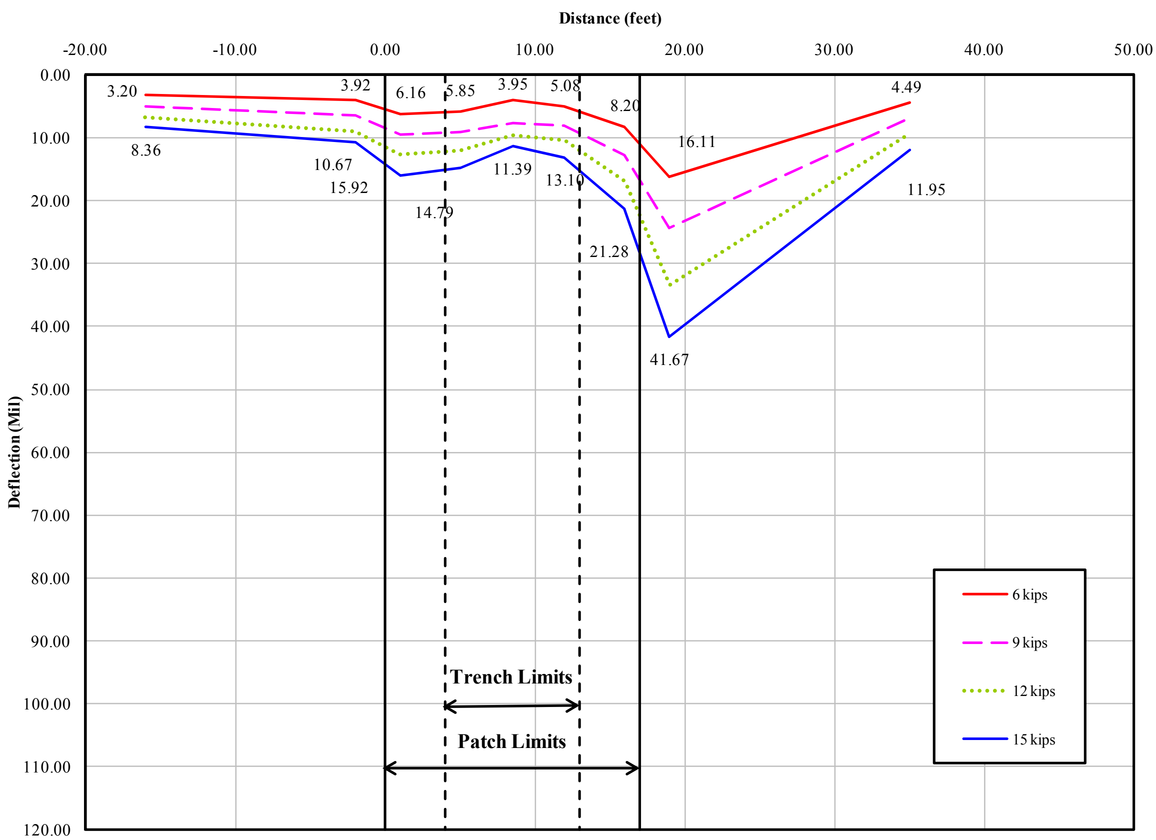


Figure 4.105. Falling weight deflectometer test results for Trench F on November 5, 2007, at a temperature of 44.4°F

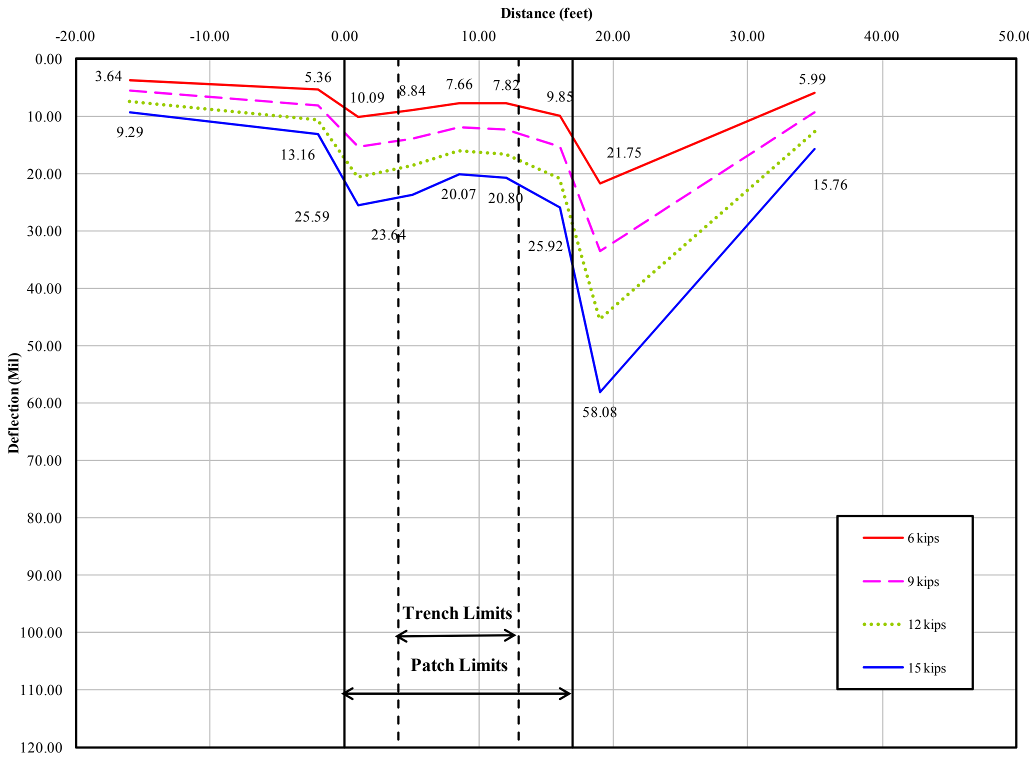


Figure 4.106. Falling weight deflectometer test results for Trench F on June 28, 2008, at a temperature of 70.0ºF

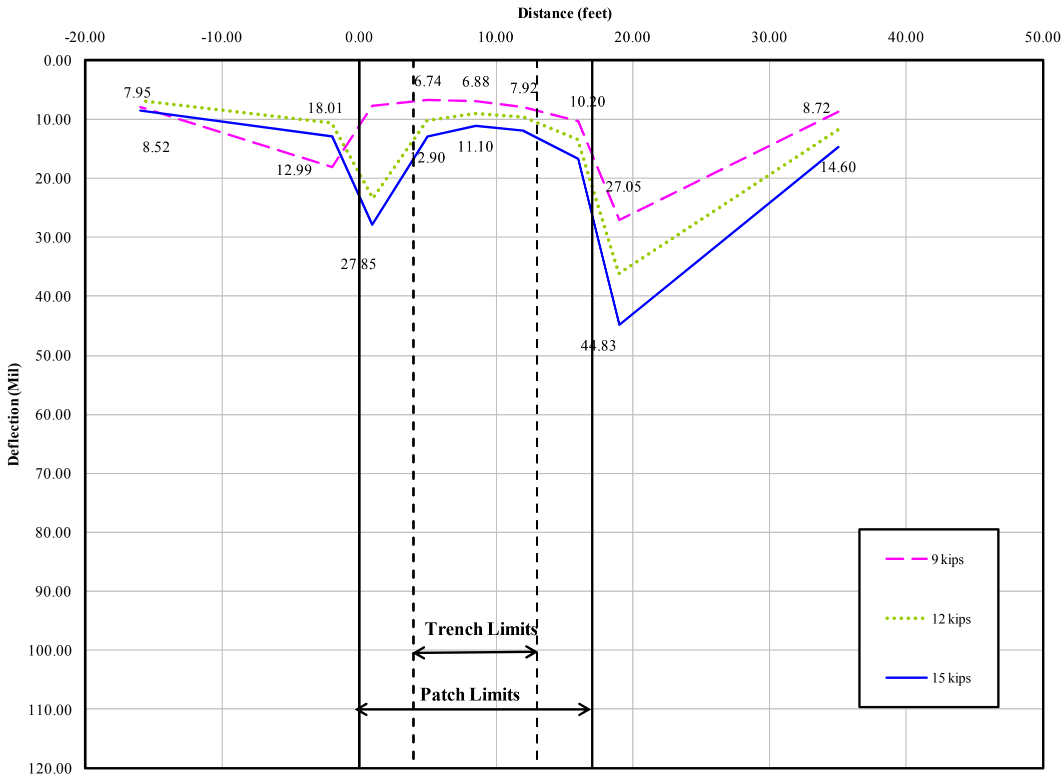


Figure 4.107. Falling weight deflectometer test results for Trench F on November 20, 2008, at a temperature of 43.4ºF

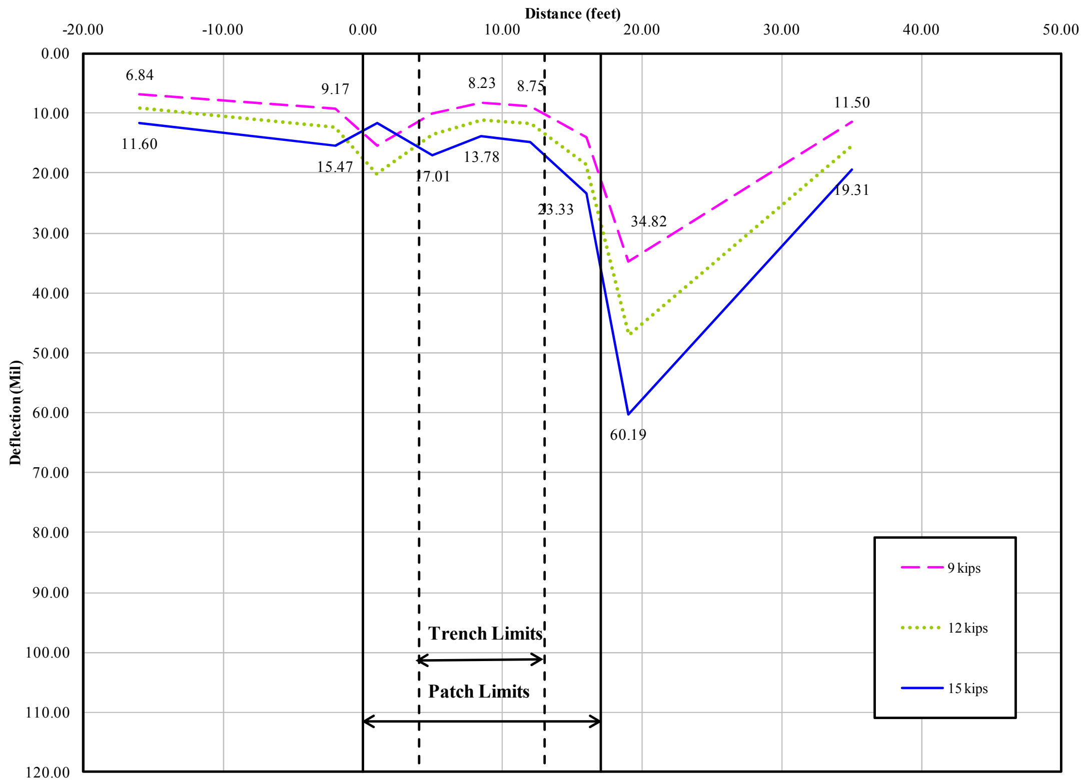


Figure 4.108. Falling weight deflectometer test results for Trench F on March 25, 2009, at 37.7ºF

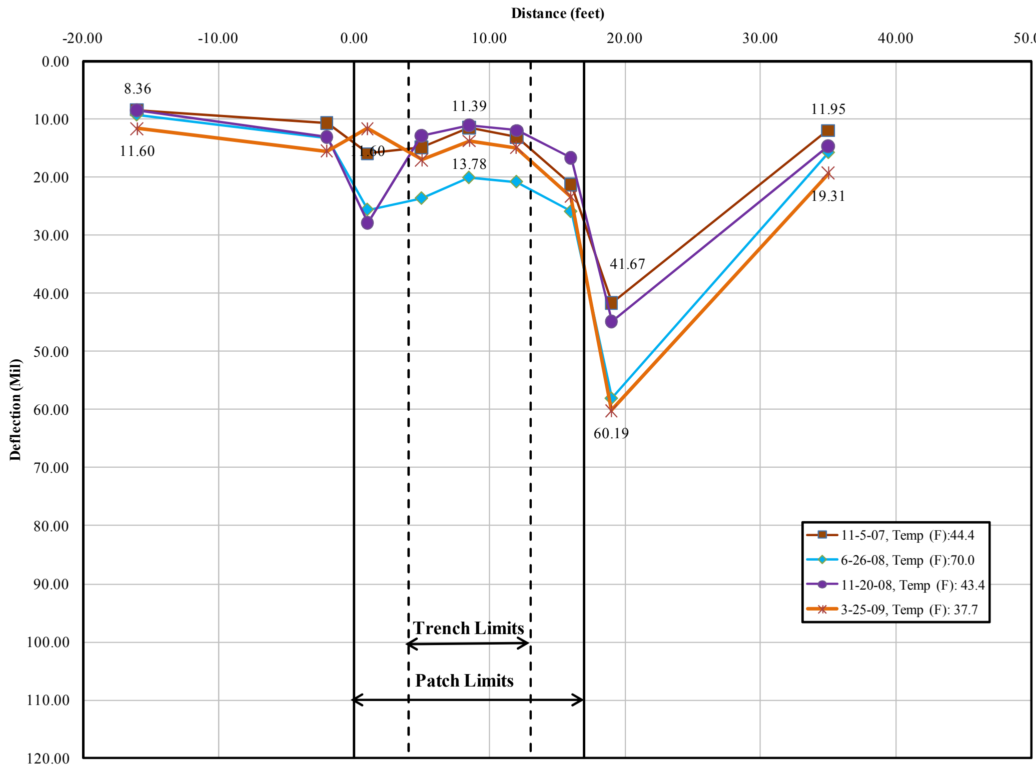


Figure 4.109. Falling weight deflectometer test results for Trench F (15-kip load)

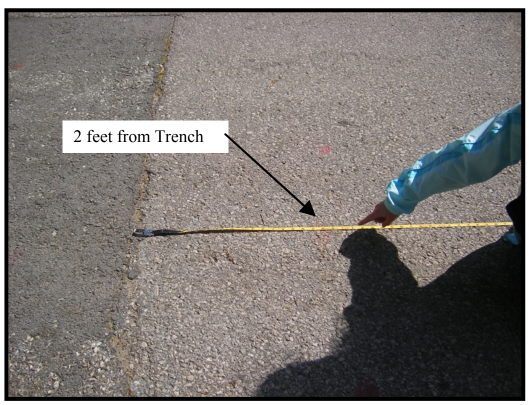


Figure 4.110. Condition of the pavement 2 feet west of Trench F at FWD testing location 8

Post-Construction Elevation Survey

The post-construction surfaces were constructed using survey data collected on July 20, 2007; March 19, 2008; June 10, 2008; August 19, 2008; November 18, 2008; and March 13, 2009.

Trench F was surveyed at 36 points. Figure 4.111 shows the location of the grid points. The benchmark was the dome bolt on the hydrant northwest of the trench at the intersection of 6th Street and Carroll Avenue. The difference between the highest and the lowest elevation was 5.04 inches. This shows the patch was placed in uneven fashion. From the elevation survey on March

19, 2008, the average settlement was 0.24 inches and the maximum settlement was 2.51 inches, which occurred along the southwest corner of the patch. The difference between the highest elevation and the lowest elevation was 5.04 inches. Figure 4.112 shows the elevation profiles and settlement for Trench F. This shows the trench settled since its construction.

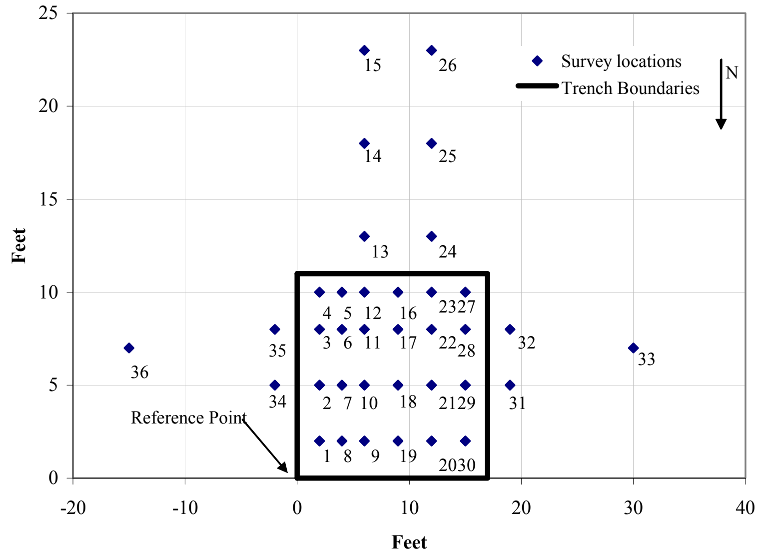


Figure 4.111. Survey locations for Trench F

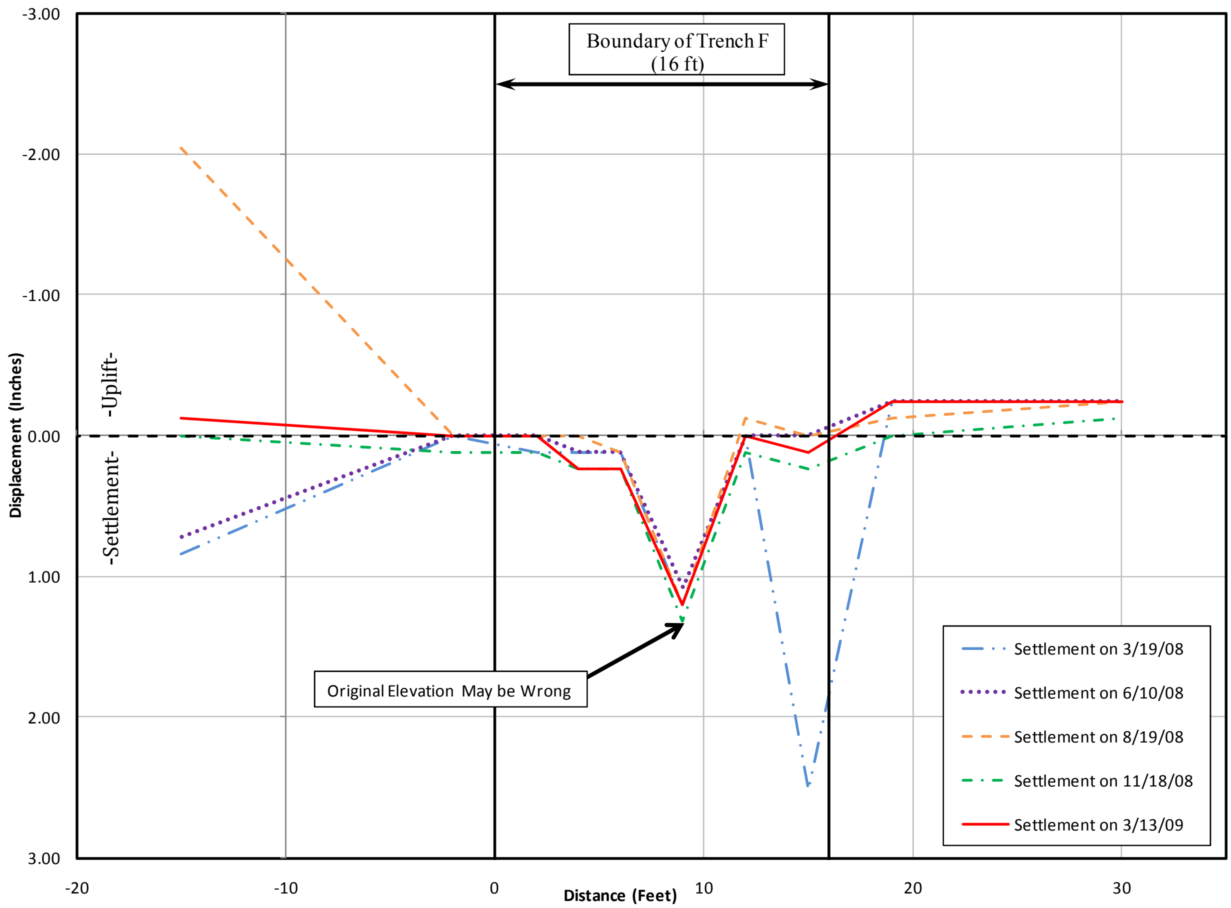


Figure 4.112. Settlement along center line of Trench F (points 36, 34, 2, 7, 10, 18, 21, 29, 31, 33)

Comparison of Field-Testing Results to Long-Term Monitoring

Figure 4.113 shows the FWD locations and the field-testing locations with the average field-testing results corresponding to the test points. On the east side of the trench, test point 8 (see Figure 4.113) corresponds to FWD location 3. According to the nuclear density test, at test point 8 the dry unit weight was 112.3 pcf at a moisture content of 13.4%. The DCP test at point 8 shows the soil stiffness increased with depth. The CBR value at the surface was 4%, and the average CBR value was 3%. In the center of the trench, test point 3 corresponded to FWD location 5. The nuclear density test measured a dry unit weight of 107.1 pcf. The CBR value at the surface was 2%, and the average CBR value was 12%. On the west side of the trench, test point 6 corresponded to FWD location 7. The dry unit weight was 111.6 pcf. The CBR value at the surface was 4%, and the average CBR value was 5%. This shows the higher measured FWD deflections occurred where there were lower CBR values. The dry unit weights did not correspond to the FWD deflections.

Figure 4.114 shows the settlement profile of the trench with the FWD test results. A load of 15 kip was used to accurately show the deflection of the subgrade. The maximum settlement of the trench was 2.51 inches in the southwest corner of the trench (not shown on Figure 4.114). The nuclear density test on the backfill of the T-section at test point 6 on the west side of the trench was 111.6 pcf at a moisture content of 9.3%. The DCP test at point 6 shows the soil softened with depth. According to the nuclear density test on the backfill on the east side of the trench at test point 8, the dry unit weight was 112.3 pcf at a moisture content of 13.4%. The nuclear density test on the backfill at test point 7 on the south side of the trench had a dry unit weigh of 108.1 pcf at a moisture content of 15.3%. The DCP test for this point shows the soil's CBR values were constant over the depth of the trench. The maximum settlement occurred where the FWD testing showed the softest response and the lowest dry unit weights.

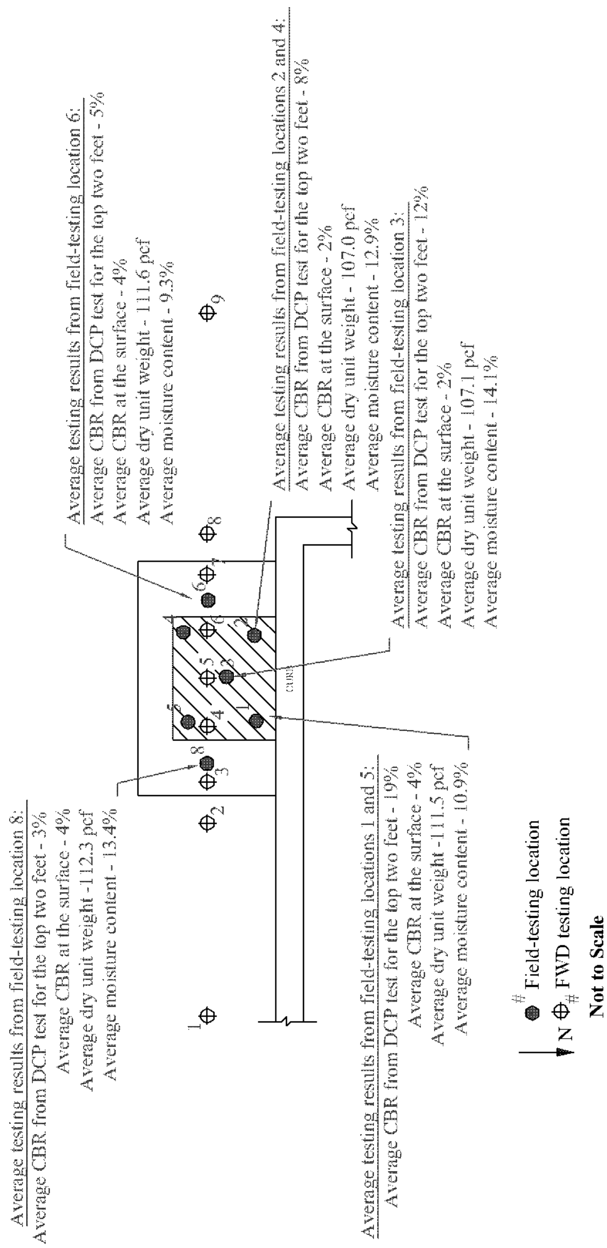


Figure 4.113. Comparison of CBR values, dry unit weights, and FWD testing results for Trench F

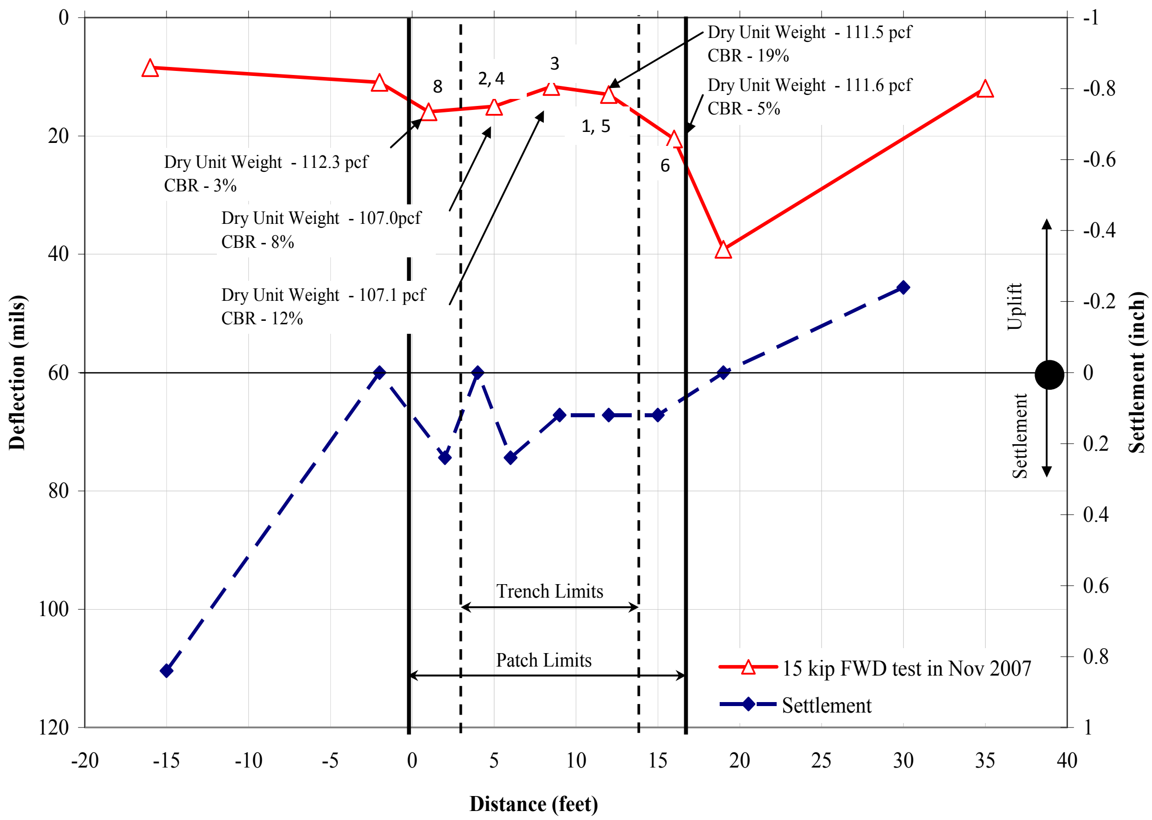


Figure 4.114. Falling weight deflectometer test (15 kip) and settlement for Trench F

Summary

- The 1-inch clean backfill was placed at low dry unit weights.
- The upper lifts were placed above 90% of the Standard Proctor.
- The maximum settlement was 2.51 inches.
- The maximum settlement occurred where lowest dry unit weights are measured.
- The zone of influence was present in the T-section on the east side of the trench, and on the west side of the trench the zone of influence was present outside the trench.
- The maximum settlement occurred where the FWD testing showed the softest response and the lowest dry unit weights.

Comparison of the Trenches

Three different trench designs were tested with two different primary backfills: 3/8-inch minus and 1-inch clean limestone. Table 4.32 summarizes the backfill used in the top 2 feet of each trench. The top 2 feet of Trenches A and D were constructed with granular limestone backfill. The top 2 feet of Trenches B, C, E, and F were constructed with granular backfill mixed with other soils. The backfills in Trenches B, E, and F were classified as SC (clayey sand). In Trench C, no bulk sample of the backfill used in the top 2 feet was collected for laboratory testing;

therefore, there are no typical values from NAVFAC or laboratory data to compare the field-testing results.

Table 4.33 summarizes the field-testing results for the top 2 feet of the trenches. In trenches with T-sections, the test points within the trench and in the T-section were averaged together. The dry unit weights of the backfills were all placed within the typical range of dry unit weights from NAVFAC, except for Trench E. The moisture contents of the backfills were all below the typical values, except for Trenches E and F. Trench A had the highest average dry unit weights and the highest average CBR values.

Shown in Table 4.34 are the settlement results from spring/summer 2007 to March 2009. The date of March 13, 2009, was used to compare the trench settlement fairly over the same climate conditions. Trench C was first surveyed in May 2007, and no survey data from before 2007 is available for comparison. The settlement data presented for Trench C spanned from May 2007 to March 2009. In Table 4.35, FWD testing results from the tests performed after construction are presented. The month of June 2008 was chosen to easily compare all the trenches in the same climate and temperature. The load of 15 kip was chosen to give a good estimation of the trenches' subgrade.

Table 4.32. Backfills used in the top 2 feet of each trench with typical NAVFAC values

Trench	Backfill	Typical dry unit weights from NAVFAC	Typical moisture content from NAVFAC	Typical CBR values from NAVFAC
Trench A	SP-SM	100 to 120	12 to 21	10 to 40
Trench B	SC	105 to 125	11 to 19	5 to 20
Trench C*	N/A	N/A	N/A	N/A
Trench D	GP	115 to 125	11 to 14	30 to 60
Trench E	SC	105 to 125	11 to 19	5 to 20
Trench F	SC	105 to 125	11 to 19	5 to 20

*No soil classification was available for the top 2 feet of backfill

Table 4.33. Field-testing results compared to laboratory-testing results for the top 2 feet of each trench

Trench	Average dry unit weight (pcf)	Relative density (%)	% of Standard Proctor	Average moisture content (%)	Bulking moisture content (%)	Optimum moisture content (%)	Average CBR (%) from DCP
Trench A	122.3	64	---	5.1		---	29
Trench B	112.0	---	96	7.4	---	13.1	5
Trench C*	117.8	N/A	N/A	10.1	N/A	N/A	9
Trench D	118.5	63	----	5.1	---	---	23
Trench E	99.3	---	94	11.3	---	17.0	8
Trench F	109.5	---	92	12.5	---	13.2	10

*No laboratory data is available for the backfill used in the top 2 feet of Trench 2

Table 4.34. Summary of the settlement for each trench

Trench	Date of construction	Dates of testing	Average displacement within the trench (inches)	Maximum/ Minimum displacement within the trench (inches)	Average displacement adjacent to the trench (inches)	Maximum/ Minimum displace-ment adjacent to the trench (inches)
Trench A	08/10 2007	3/13/09	0.60	1.08/0.24	0.35	-2.16/1.2
Trench B	07/25/2007	3/13/09	-0.13	-0.48/0.24	-0.44	-0.96/0.00
Trench C	05/16/2005	3/13/09	-0.50	-2.68/1.44	-0.64	-2.64/1.80
Trench D	07/18/2007	3/13/09	-0.29	-0.60/0.12	-0.45	-0.96/-0.12
Trench E	07/25/2007	3/13/09	0.37	0.72/0.12	-0.55	-3.66/-0.24
Trench F	07/18/2007	3/13/09	0.14	1.20/-0.12	-0.02	0.84/-0.48

*Negative values indicate uplift

Table 4.35. Summary of the deflections from the FWD test (15 kip load) after construction

Trench	Date of construction	Dates of testing	Deflection at the center of the trench (mils)	Average deflection at the edges of the trench (mils)	Deflection of the surrounding soils (mils)
Trench A	08/10/2007	6/25/2008	39.35	78.14	109.0
Trench B	07/25/2007	6/26/2008	13.34	36.29	56.45
Trench C	05/16/2005	6/25/2008	36.43	71.8	99.33
Trench D	07/18/2007	6/28/2008	18.62	62.84	54.05
Trench E	07/25/2007	6/26/2008	25.29	34.48	41.64
Trench F	07/18/2007	6/26/2008	20.07	24.78	12.53

Trench A and Trench D were constructed using the vertical wall cross-section, with two different backfills. Trench A was constructed using 3/8-inch minus limestone, and Trench D was constructed using 1-inch clean limestone. Trench A was uplifted during the winter survey by 0.12 inches, and Trench D experienced a maximum settlement of 0.24 inches. The difference in the settlements measured was the result of 3/8-inch minus backfill being frost susceptible and the 1-inch clean limestone being non-frost susceptible. The deflections from the 15-kip load in the FWD tests were higher in Trench A than in Trench D, and both trenches were stiffer than the surrounding soil. Smaller settlement in these two trenches was associated with higher CBR values. Trenches B and E were constructed using the T-section cross-section. The maximum settlement in Trench B was larger than the settlement in Trench E. However, the average settlement in Trench B was less than the average settlement in Trench E. The backfill in the top 2 feet of the trenches was placed at similar relative densities. The larger differential settlements in Trench B could be the result of using cinders from the Ames Power Plant in the top 2 feet.

Trench C and Trench F were constructed using the T-section with geogrid cross-section. The maximum settlement since March 2009 in Trench C was larger than in Trench F. However, the FWD testing results show Trench F provided a stiffer response.

Trenches A, B, and C were constructed using various cross-sections and the same 3/8-inch minus backfill. Trench A was the best performing trench of the three trenches constructed with 3/8-inch minus limestone. It experienced the smallest settlements and had the stiffest response from the FWD tests. The backfill in Trench A had higher average dry unit weight and CBR values than in Trenches B and C.

Trenches B and C were T-sections, except Trench C had geogrid placed across the bottom of the T-section. During field testing, these trenches had similar CBR values. Trench B had a maximum settlement of 5.64 inches in March 2008, and Trench C had a maximum settlement of 2.40 inches also in March 2008. The average settlement in Trench B was less than the average settlement in Trench C. However, the settlements measured in Trench C were from June 2007 to March 2008 and started two years after the construction of the trench was complete. There was no survey data for Trench C dating before May 2007.

Trenches D, E, and F were constructed using the same cross-section and 1-inch minus limestone backfill. Trench D is the best performing trench of these three trenches. Trench D experienced the smallest settlements. The FWD deflections were higher than the other three trenches; however, the surrounding soils for Trench D were also softer than the soil surrounding the other two trenches.

Trenches B, C, E, and F were constructed with limestone backfill mixed with other cohesive materials in the top 2 feet of the trench. Based on the settlement criteria for performance, these four trenches performed poorly with settlements ranging from 0.72 inches to 5.64 inches. All backfill used in the top 2 feet was classified as a cohesive material, and, therefore, the Standard Proctor test results apply. When comparing the field-testing results for the top 2 feet of these trenches, the backfill was placed either below optimum moisture content or at low dry unit weights. Placing backfill below the optimum moisture content caused the backfill to have collapse behavior and high settlements. The reason the backfill was at low moisture content was that the granular material, which was at low moisture content, was mixed with cohesive material that was allowed to air dry during construction. These two factors resulted in moisture contents lower than desired.

Figure 4.115 plots average settlement as a function of time for the six trenches.

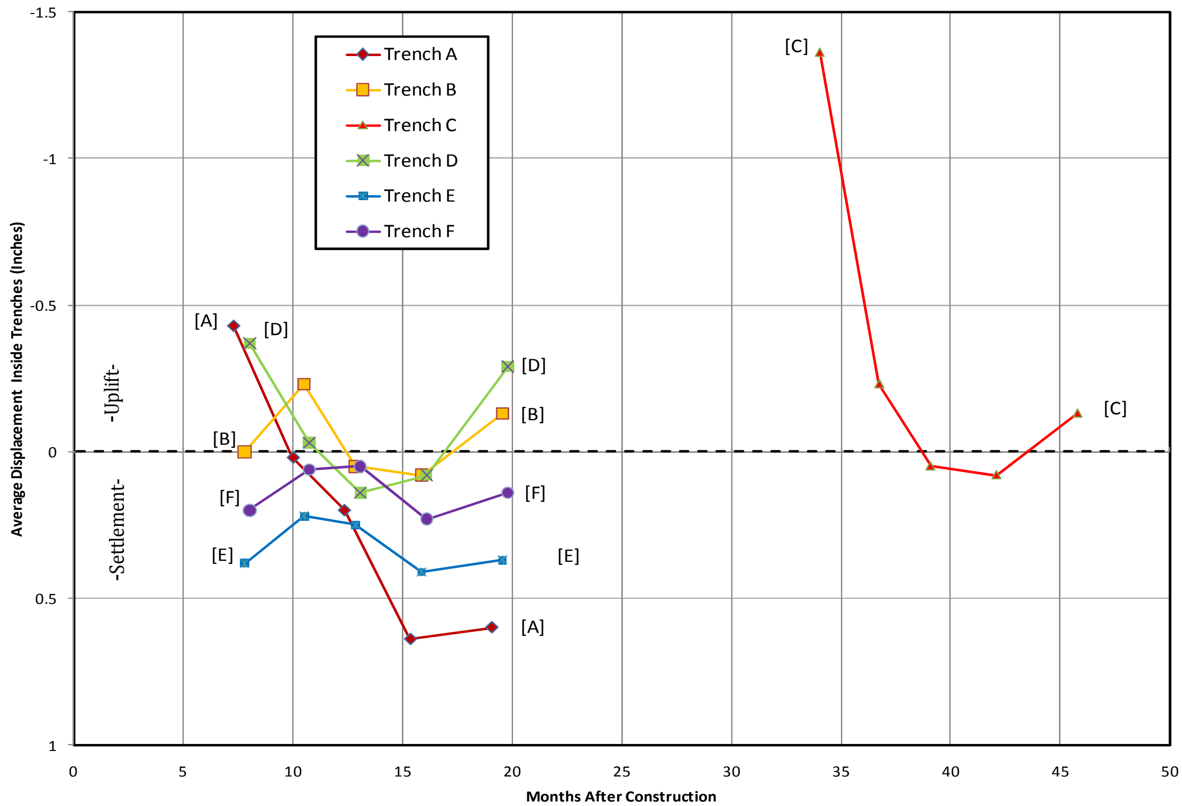


Figure 4.115. Settlement as a function of time for the six recommended trenches

Conclusion

Based on the monitoring results, the following conclusions can be drawn:

- The use of 1-inch clean limestone improves the performance of the trenches. It stiffens the response of the trench in FWD testing, and the settlement within the trenches is less than in trenches constructed with 3/8-inch minus limestone.
- Mixing the soil excavated from the trench with the backfill was not proved to be a successful construction practice, which could be attributed to the low compaction levels achieved during construction and minimal moisture control of the fine-grained materials.
- The use of geogrid in the trenches did not improve the performance of the trenches compared to the trenches constructed without the geogrid for the trenches using 3/8-inch minus limestone. The geogrid appears to have stiffened the response of the trench for the FWD testing; however, these trenches had larger settlements than trenches A and D.
- The T-section did not abate the zone of influence on the trenches. Rather, the zone of influence moved outside of the T-section for all trenches except Trench E, where it was in the T-section on one side.
- The T-section did not reduce the settlement in the trenches. The trenches without the T-section (Trenches A and D) performed better. During the construction of the T-section, a larger area of the road was disturbed and a large volume of backfill had to

be evenly compacted. Because there was no quality management of the placement of the backfill to ensure it was compacted to appropriate relative densities across the trenches, uneven settlements occurred. Another reason for the poor performance of the T-section trenches could be the result of mixing the limestone backfills with other soils.

- The increased effort and resources used to construct the T-section trenches did not yield a better trench performance.

Recommendations

Based on the performance of the six trenches, the following recommendations were made:

- The standard vertical-walled cross-section with 1-inch clean limestone is recommended as a construction practice.
- Soils excavated from the trenches or other soils should not be mixed with the granular backfills unless previous laboratory testing yielding a range of recommended moisture content and densities to be achieved in the field are conducted.
- Pavement should be removed from four feet around the parameter of the trench, and the area should be recompacted if a T-section is not constructed.
- The T-section should be modified to use walls that are beveled outward to facilitate compaction of backfill. Beveled edges will reduce the amount of disturbance to the surrounding soil and also eliminate the vertical excavation, which makes compacting the backfill more difficult.
- Quality-control measures should be implemented in the field to ensure compaction requirements are met. This includes achieving at least medium relative density with moisture contents above the bulking moisture content for cohesionless soils and above 95% of Standard Proctor and +/-2% of optimum moisture content for cohesive soils.
- Cities should implement moisture control practices. The construction industry has implemented the practice of wetting dry soil before compaction to insure it meets specifications for performance.

Further Research

After the completion of this report, the following research is recommended:

- Reconstruct the T-section trenches with beveled edges without mixing the soil excavated from the trench with granular backfill. Quality-control management should be used on each lift (i.e., ensure each lift is placed with moisture control and at the appropriate relative density for the backfill being used). This will provide more conclusive results that the T-section trench works to reduce the zone of influence or the zone of influence moves further outside the trench.
- Continue FWD testing and settlement monitoring on the existing six recommended trenches.

CHAPTER 5. INSTRUMENTED TRENCHES

Three instrumented trenches were constructed near 2709 Kellogg Avenue between Luther Drive and 28th Street in north Ames to monitor the effects of different utility cut construction techniques. One trench (Trench AI) was constructed using the current City of Ames standard construction practices and the other two trenches (Trenches BI and CI) were constructed using the recommended construction practices. Figure 5.1 shows the site before trench construction. This location was selected as one of the locations where the City of Ames planned to conduct routine resurfacing with an estimated annual traffic of less than 1,000 vehicles.

Site Conditions of Kellogg Avenue

The asphalt pavement was sawed around the trenches. The existing asphalt pavement ranged from 1 to 3 inches in thickness across the site. A layer of approximately 6 inches of fly-ash-treated soil was found beneath the asphalt.

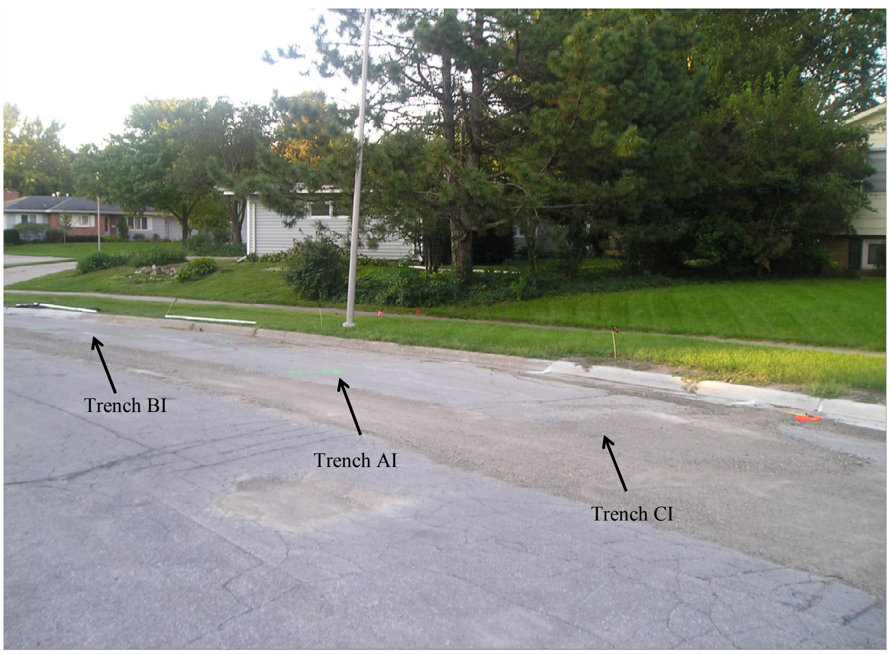


Figure 5.1. Kellogg Avenue before construction of the instrumented trenches

Three Shelby tube samples were collected from a boring made north of Trench BI on Kellogg Avenue. Six unconfined compression tests were conducted on these samples using ASTM D2166 (2006). The samples were then classified according to the Unified Soil Classification System (USCS).

Figure 5.2 shows the boring log of the subgrade soils at the instrumented trenches. The moisture content of the samples ranged from 15.5% to 21.5%. The dry unit weights ranged from 105.9 pcf to 120.6 pcf. The undrained shear strengths ranged from 1,180 psf to 7,910 psf.

Table 5.1 and Figure 5.3 show the gradations of the soil on Kellogg Avenue. More than 30% of the soil particles were smaller than 0.02 mm (see Figure 5.3). This high percentage increased the soil's susceptibility to frost heave, which, along with the soil's classification of SC, allowed for a frost heave classification of F3, as determined by the Army Corps of Engineers (see Table 2.9). The Army Corps of Engineers also determined the frost heave suitability of the soil to be medium to high. This means the soil was expected to experience 2.0 to 8.0 mm per day heave when temperatures were below freezing. These soils were classified using the USCS and AASHTO standards (see Tables 2.2, 2.3, and 2.4).

Log of Boring 1

Site: Instrumented Trenches on Kellogg Avenue

Project:

Phase II:

Depth	Description (Approximate Surface Elevation: 97.6 ft)	Elevation	USCS Symbol	AASHTO Symbol	Sample: Number	Sample: Type	Sample: in	Tests: Content	Tests: Dry Unit Weight, pcf	Tests: Compressiv e Strength, psf	Tests: Other
0.3	Pavement	95.7				PA					
	Sandy Lean Clay		CL	A-6							
	Brown										
1.3											
		94.7				ST	30.0				LL=37
					1			19.5	111.0	7911.2	PL=14
											PI=23
2.6		93.4									
					2			19.1	106.7	3331.0	
						PA					
3.6											
	Clayey Sand	92.4	SC	A-6		ST	20.0				LL=30
	Brown				3			19.1	111.6	1526.7	PL=13
											PI=17
					4			19.5	107.0	1526.7	
5.0											
5.3	**Sandy Lean Clay**	91.0	CL	A-6		PA					
	Brown					ST	22.0				LL=29
					5			21.5	105.9	1665.5	PL=13
6.0											PI=16
6.5		89.5									
					6			15.5	120.6	1179.3	
7.0		89.0									
10.0		86.0									
	Bottom of Boring										

Figure 5.2. Boring log for trenches on Kellogg Avenue

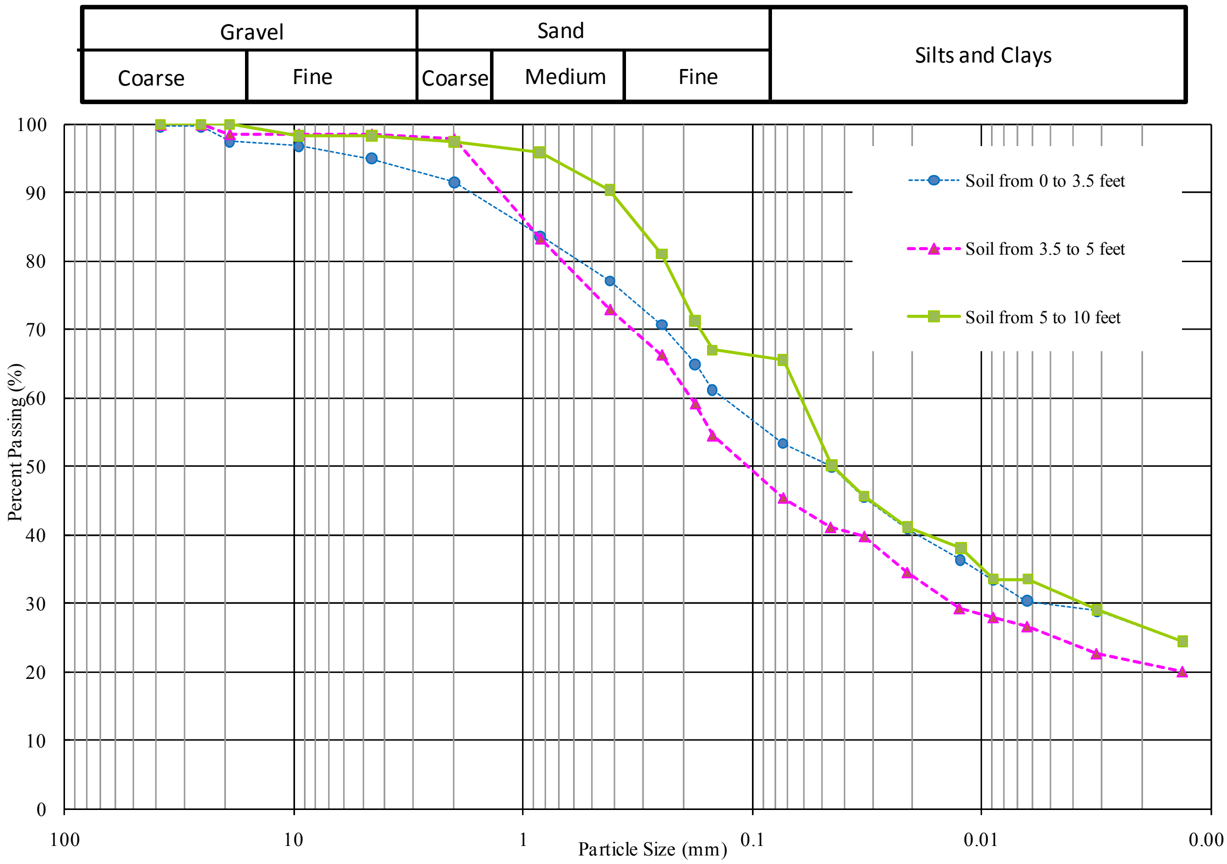


Figure 5.3. Gradation of soils excavated from the Kellogg Avenue trenches

Table 5.1. Classification of the soils removed from the trenches on Kellogg Avenue and the 3/8-inch minus backfill

Sample	**Soil classification**			
	USCS		**AASHTO**	
Kellogg bulk sample from 0 to 3½ feet	CL	Sandy lean clay	A-6	Clayey soil
Kellogg bulk sample from 3½ to 5 feet	SC	Clayey sand	A-6	Clayey soil
Kellogg bulk sample from 5 to 10 feet	CL	Sandy lean clay	A-6	Clayey soil

Design of the Instrumented Trenches

Trench AI used the City of Ames standard trench restoration practices and was used as a control. Trench BI was constructed using the Trench A technique shown in Figure 4.1. Trench CI was constructed using the Trench B technique (also shown in Figure 4.1), except that it included instrumentation and all backfill was 3/8-inch minus limestone (no soil excavated from the trench was placed back in the trench). General descriptions of the Instrumented Trenches AI, BI, and CI are summarized below:

Trench AI characteristics (using City of Ames practices):

- Vertical side walls with no T-section for the trench excavation
- Placing 3/8-inch minus limestone in lifts of 2 feet or greater
- No moisture control
- Compaction using a vibratory plate compactor attached to a backhoe

Trench BI characteristics:

- Vertical side walls with no T-section for the trench excavation
- Placing 3/8-inch minus limestone in lifts of 1 foot or less
- Moisture control
- Compaction using an impact rammer

Trench CI characteristics:

- Vertical side walls with the T-section for the trench excavation
- Placing 3/8-inch minus limestone in lifts of 1 foot or less
- Moisture control
- Compaction using an impact rammer (the first two lifts were compacted using a vibratory plate compactor attached to a backhoe)

As shown in Figure 5.4, the trenches were instrumented to measure settlement, vertical earth pressure, and moisture content within the trenches, and also to measure the temperature within the trenches and adjacent soil. The trenches had instrumentation installed vertically every 2 feet in the loose backfill placement. The data logger synchronized instrumentation readouts.

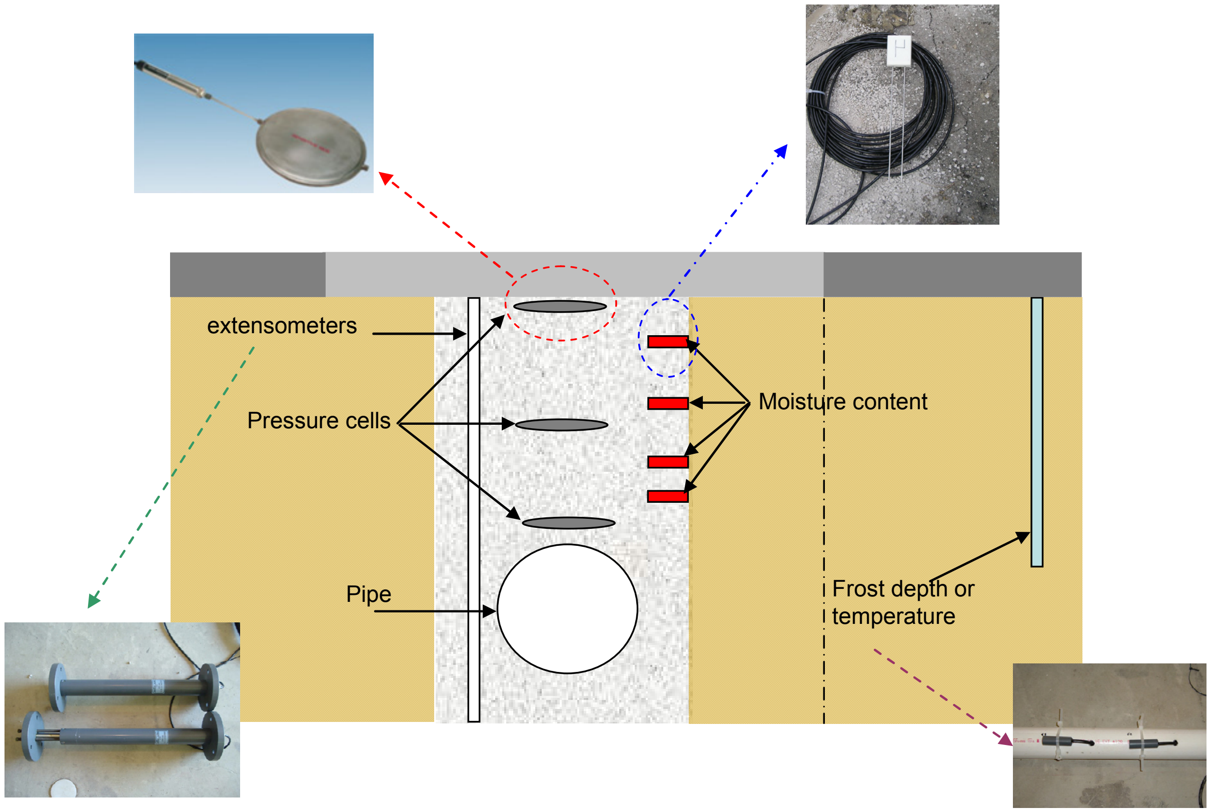


Figure 5.4. General configuration of instrumentation in trenches

Instrumentation Installation Procedure

The extensometers were partially extended during installation to allow for both extension and retraction. Bolts secured the position of the partially extended extensometers and prevented them from fully retracting (see Figure 5.5) before installation. A concrete base extended at least 1 foot below the base of the trench, which acted as an anchoring point for the extensometers (see Figure 5.6). This concrete anchoring point was assumed to be fixed and at offset zero. The extensometers were installed in a series by bolting the top of one extensometer to the bottom of the extensometer above it. After the extensometers were connected, the spacing bolts were removed.

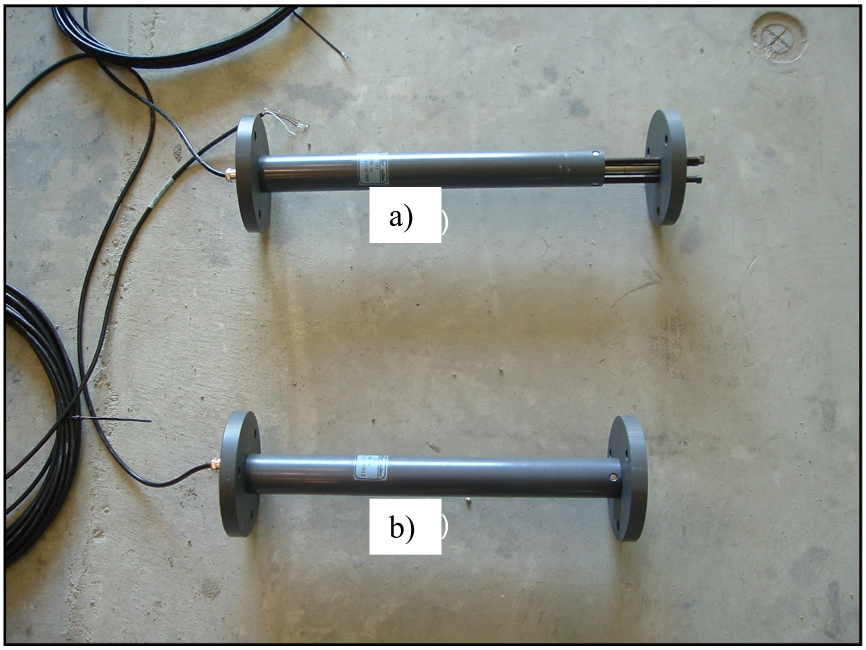


Figure 5.5. Extensometers used in instrumented trenches (a) with temporary bolts maintaining the extended head, and (b) fully retracted head

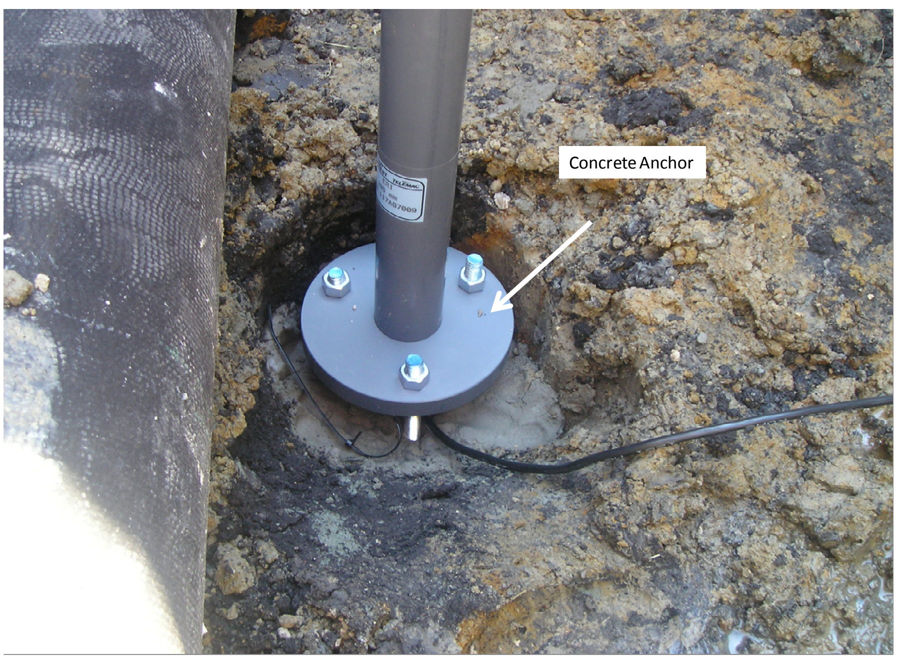
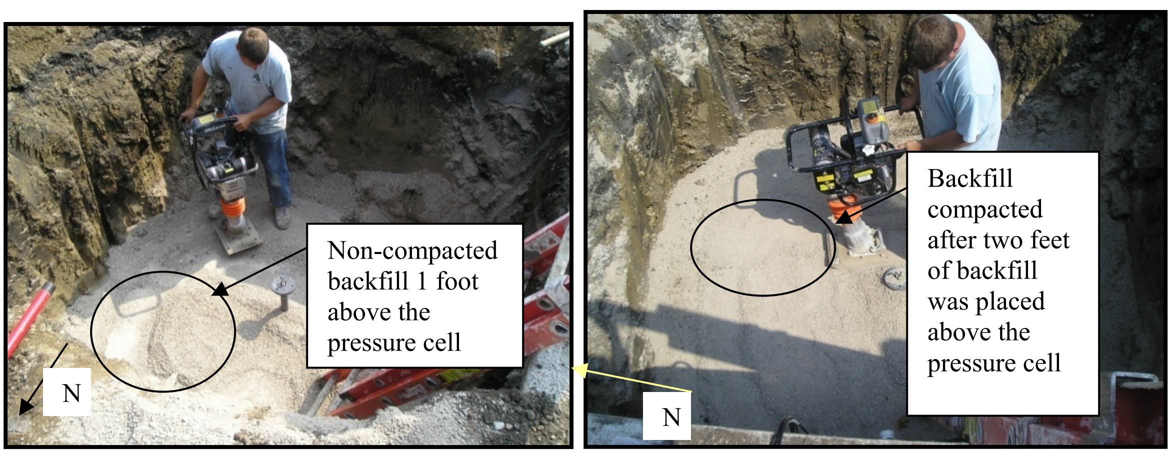


Figure 5.6. Concrete based at the bottom of the trench

The pressure cells were installed in the trenches and protected using sand. The maximum sand particle size was less than 1/8 of the total thickness of the pressure cell (less than 4.7 mm, No. 4 sieve). The small sand particles protected the pressure cells from being punctured or dented by the 3/8-inch minus limestone backfill (see Figure 5.7). During construction, the pressure cells were further protected by not compacting the 3/8-inch minus limestone backfill above the cells until the lift thickness was approximately 2 feet (see Figure 5.8).

a) b)

Figure 5.7. Pressure cell being placed within a lens of sand: (a) sand lens below the pressure cell, and (b) sand lens covering the pressure cells



a) b)

Figure 5.8. Procedure for compacting backfill over the pressure cells: (a) 1 foot above the pressure cells the backfill was not compacted, and (b) 2 feet above the pressure cell the backfill was compacted

At each layer of instrumentation, moisture sensors were installed lying flat on the surface to avoid bending the sensors' prongs (see Figure 5.9).

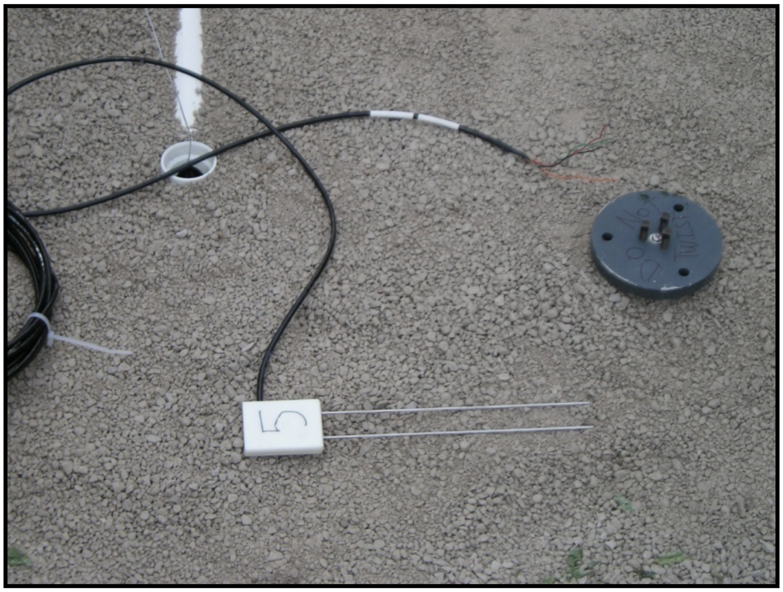


Figure 5.9. TDR moisture sensor used in instrumenting all trenches

Eight temperature sensors were mounted to a PVC pipe in a configuration consisting of increasing interval lengths with increasing depth. The upper four sensors were mounted at 6-inch intervals, followed by two sensors placed at 1-foot intervals. The deepest two sensors were mounted at 2-foot intervals (see Figure 5.10). The PVC pipe was installed vertically in a soil boring north of Instrumented Trench CI so that the first temperature sensor was located 11.5 inches below the newly paved surface.

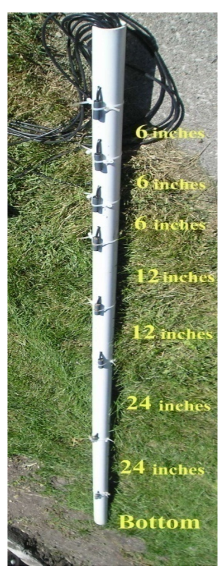


Figure 5.10. Temperature sensors mounted on the PVC pipe

Four multiplexers connected each type of instrument to the data logger, which collected the data (Figure 5.11). The system was powered by a solar panel and a battery pack. Data from the data logger was downloaded approximately every 10 days.

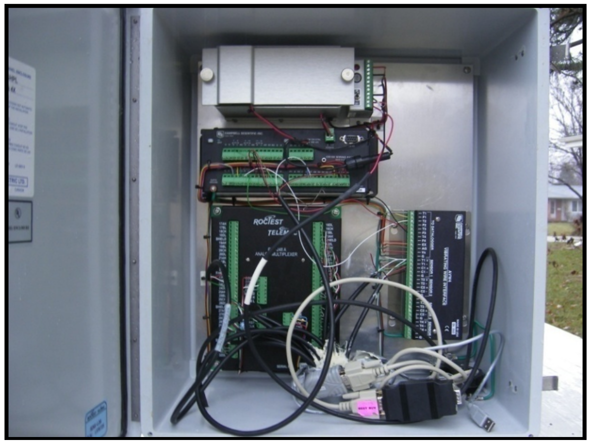


Figure 5.11. Multiplexer, vibrating wire interface, and data logger used to read the instruments

Initially, the data logger was programmed to begin scanning at one-minute intervals; however, a complete scan of all the multiplexers required one minute and ten seconds. This caused the scans to repeatedly abort before data was recorded in temporary storage. Once the program was modified to scan every two minutes, the data logger was able to record the maximum, minimum, and average readings in two-hour intervals.

Construction of Instrumented Trenches

The City of Ames, in conjunction with the Iowa State research team, completed the excavation and construction of the instrumented trenches. The Iowa State research team monitored and documented each step in the trench construction. The benchmark for the site was the dome bolt (see Figure 5.12) on a fire hydrant located 100 feet southeast of the site. The dome bolt, which held the cap of the hydrant in place, was assigned an elevation of 100.0 feet. This benchmark was used throughout the elevation monitoring of the site. The elevation of the pre-excavation asphalt road ranged from 97.2 feet to 98.5 feet.

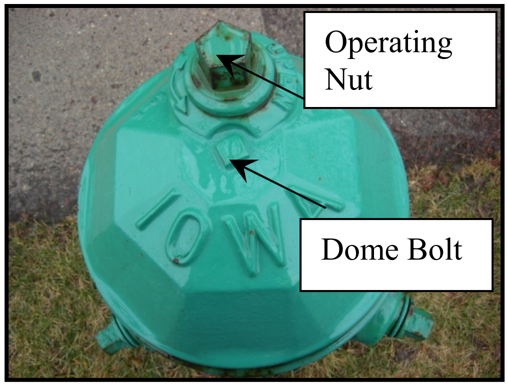


Figure 5.12. Top of the Iowa valve fire hydrant in Ames, Iowa with dome bolt

Figure 5.13 shows the location of the boring for the temperature sensors and the trenches along Kellogg Avenue. Also illustrated are site features including existing sewers, power pole, and fire hydrant.

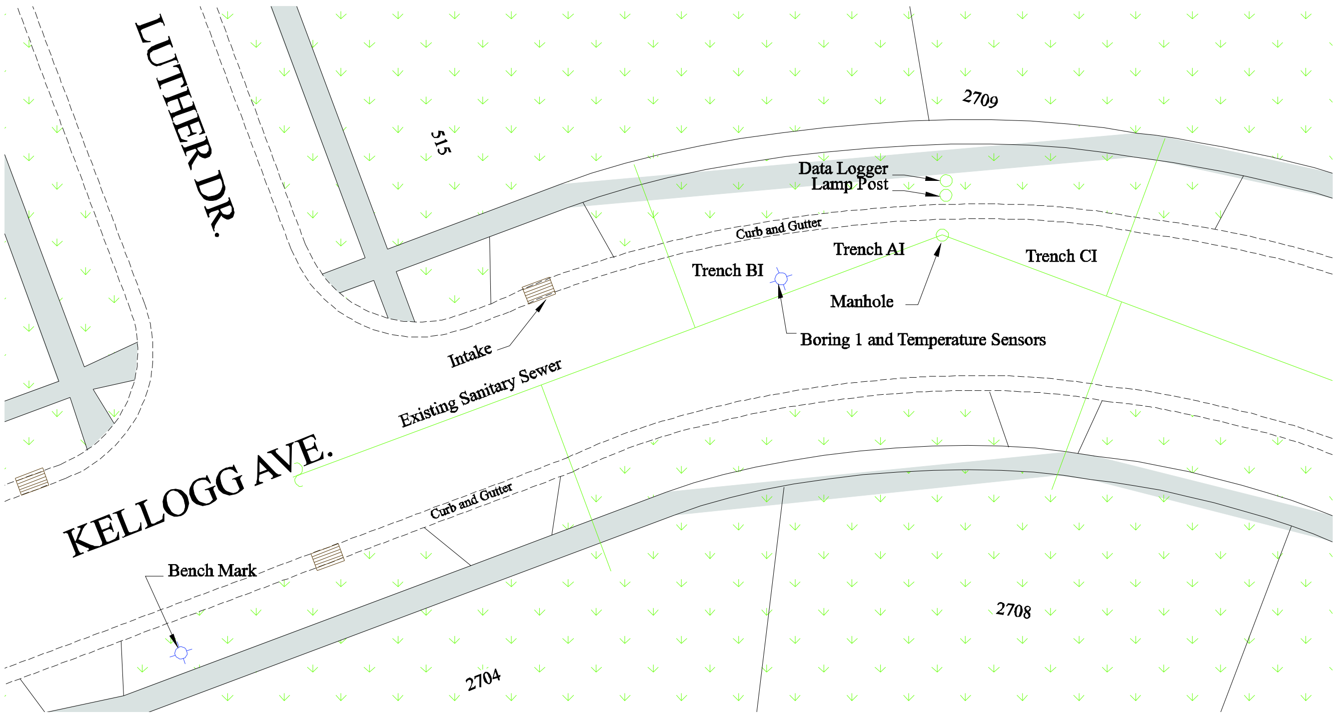


Figure 5.13. Plan view of the instrumented trench site showing the location of the trenches and the temperature sensors

Construction Summary: Trench AI

The installation of Trench AI took place on September 20, 2007, by the City of Ames. The Iowa State research team documented the construction process. Observations include the following:

- The trench was excavated using a backhoe to the approximate elevation of 90.3 feet (7.8 feet below existing pavement). The trench dimensions were about 10 feet long paralleling the curb and 8 feet wide extending from the edge of the concrete curb gutter (see Figure 5.14).
- A clay drainpipe lined with a plastic tube was broken while excavating the north end of the trench. According to the City of Ames, this drain line was not marked as an active utility. Thus, the hole in the clay pipe was filled with concrete.
- A City of Ames worker probed to find the exact location of the sewer line in the northwest corner of the trench. The sewer line was below the trench floor.
- A drainpipe located between lifts 2 and 3 on the west side of the trench was also broken. This pipe was also filled with concrete.
- A section of ductile iron pipe was placed horizontally at the base of the trench to simulate an underground utility. The ductile iron pipe was filled with 3/8-inch minus limestone, and the ends of the pipe were capped with clay.

- The first extensometer was installed next to the ductile iron pipe.
- Each lift was approximately 2 feet of loose 3/8-inch minus limestone backfill.
- A plate compactor attached to a backhoe was used to compact each lift (see Figure 5.15).
- The City of Ames personnel were responsible for placing the backfill material in Trench AI, with no requirements for moisture control or density. The Iowa State research team performed four nuclear density tests in each lift after compaction.
- The Iowa State research team performed DCP tests on lifts 1, 2, 3, and 4 to estimate the CBR of the compacted backfill. Lift 5 (the top lift) was not tested using the DCP tests.
- Extensometers were installed to span the center of lifts 1, 2, 3, and 4. An extensometer spanning lift 1 was also installed on the side of the trench. This additional extensometer did not work.
- On lifts 1, 2, 3, and 4, pressure cells and moisture sensors were installed. An additional moisture sensor was installed on the side of the trench on lift 1. This moisture sensor did not work.
- The City of Ames placed a temporary asphalt patch over the trench after completion.
- On October 1, 2007, the City of Ames resurfaced the street with 6 inches of asphalt pavement.

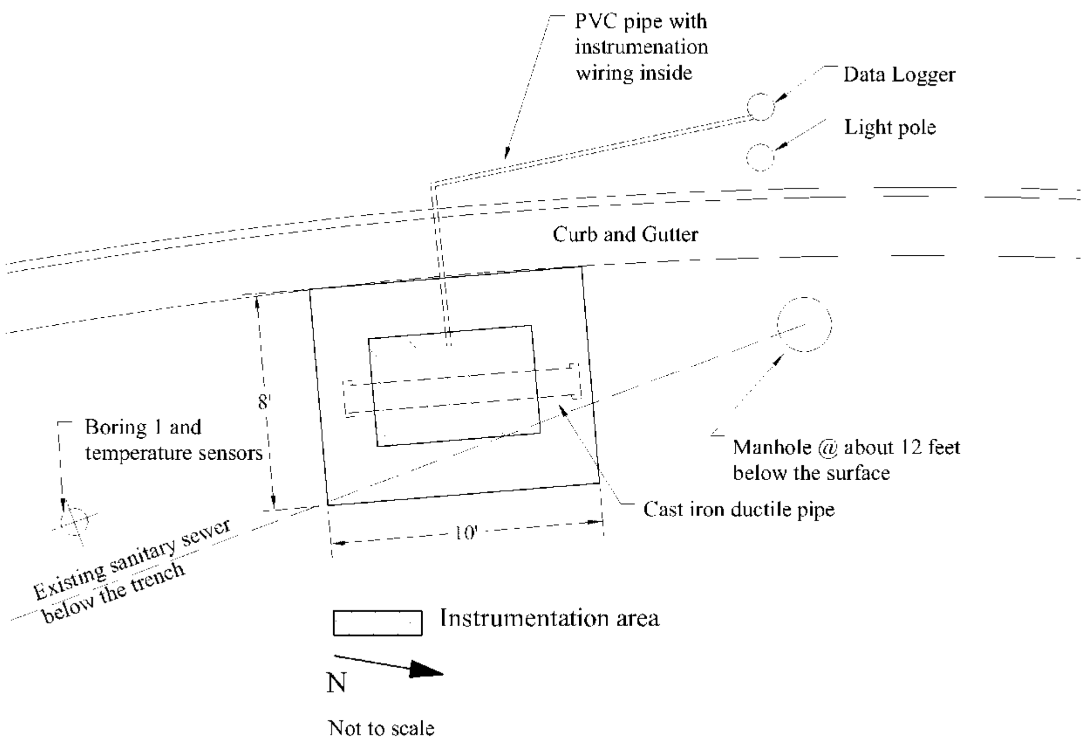


Figure 5.14. Plan view showing details of Trench AI

Figure 5.15. Vibratory plate compactor attached to the backhoe used to compact the backfill in Trench AI

Construction Summary: Trench BI

Trench BI was constructed on September 27, 2007. The following is a summary of the construction process:

- The trench was excavated using a backhoe to the elevation of about 89.8 feet (about 7.25 feet below existing pavement). The trench dimensions were roughly 10 feet long paralleling the curb and 8 feet wide extending from the edge of the concrete curb gutter pan (see Figure 5.16).
- After the trench was excavated, a section of ductile iron pipe was placed horizontally at the base of the trench to simulate an underground utility. The ductile iron pipe was filled with 3/8-inch minus limestone, and the ends of the pipe were capped with clay.
- The first extensometer was installed next to the iron pipe.
- The first lift consisted of 2 feet of loose backfill to cover the pipe and was compacted with an impact rammer compactor.
- The remaining eight lifts consisted of roughly 1 foot of loose 3/8-inch minus limestone backfill.
- An impact rammer compactor was used to compact each lift (see Figure 5.17).
- Each lift was moisture conditioned by sprinkling water on it from a hose attached to a watering truck.
- After completing lift compaction, four nuclear density tests were performed to verify that each lift met moisture content and dry unit weight requirements (moisture content greater than 8% and unit weight to achieve 65% relative density). Figure 5.18 shows a nuclear density test about to be performed in Trench BI.
- Dynamic cone penetration tests were also performed on lifts 2, 4, 6, 7, and 8 to estimate the CBR of the compacted backfill (see Figure 5.19).
- The instruments were installed on lifts 2, 4, 6, and 8. On the uppermost instrumented lift (lift 8), an additional moisture sensor was inserted into the wall of the trench to

measure the moisture content of the soil adjacent to the trench.

- Upon the completion of the trench, a temporary asphalt patch was placed over the trench (see Figure 5.20).
- On September 30, 2007, the asphalt was stripped from the road for repaving. The trenches were left with only a partial patch. During that afternoon, the City of Ames experienced a large amount of rain (0.82 inches). The runoff flowed into the trenches (see Figure 5.21).
- On October 1, 2007, the street was resurfaced with 6 inches of asphalt.

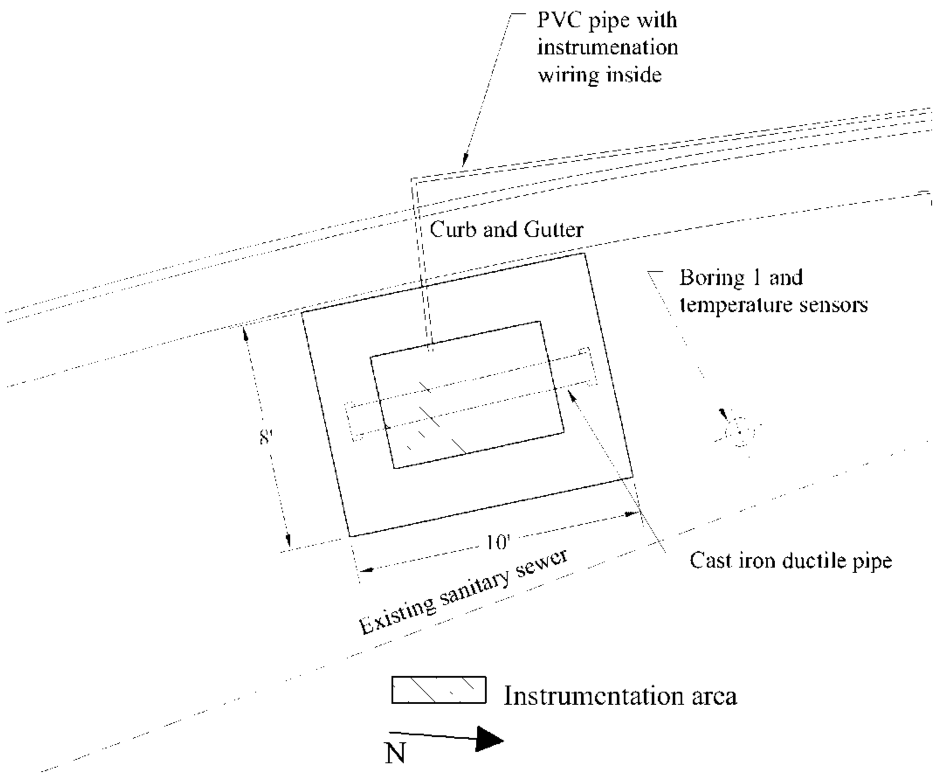


Figure 5.16. Plan view showing details of Trench BI

Figure 5.17. Impact rammer used to compact the backfill in Trench BI

Figure 5.18. Nuclear density gauge used in Trench BI

Figure 5.19. Dynamic cone penetration test being performed in Trench BI

Figure 5.20. The placement of the temporary patch on Trench BI

Figure 5.21. Runoff flowing into the trench around the temporary patch

Construction Summary: Trench CI

Trench CI was constructed on September 21, 2007. The following was documented by the Iowa State research team:

- The trench was excavated using a backhoe to the elevation of 91.3 feet (7.25 feet below existing pavement). The trench dimensions were roughly 10 feet long paralleling the curb (including the cutbacks) and 8 feet wide extending from the edge of the concrete curb gutter (see Figure 5.22).
- An unused drain tile was broken during excavation, and it was patched with concrete.
- After the excavation was completed, water seepage was present in the northwest corner of the trench. The water prevented the concrete anchor of the extensometer from curing. This resulted in about an hour delay before the trench could be backfilled.
- Lifts were performed by placing about 1 foot of loose 3/8-inch minus limestone fill.
- A vibratory plate compactor was initially used to compact the first lift until the workers noticed that the vibrations caused the extensometer to tip off vertically. An impact rammer compactor was then used to adjust the extensometer back to vertical alignment and to complete this and all subsequent lift compactions. No further problems occurred with the extensometers during this process.
- Each lift was moisture conditioned with a water hose (see Figure 5.23).
- The Iowa State research team performed nuclear density tests in each lift after compaction to verify that they met moisture content and dry unit weight requirements (moisture content greater than 8% and a relative density greater than 65%).
- The Iowa State research team performed DCP tests on lifts 3, 4, 5, 7, and 9 to estimate the CBR of the compacted backfill.
- At the completion of lift 6, the T-section was constructed by removing 2 feet of soil on each side of the trench (see Figure 5.24). Additional nuclear density and DCP tests

were performed on the T-section.

- The completed trench contained nine lifts with a column of extensometers extending from a depth of 8.30 feet to 2.42 feet, and pressure cells and moisture sensors were installed on top of lifts 2, 4, 6, and 7.
- On the T-section, a pressure cell and moisture sensor was installed.
- The City of Ames placed a temporary asphalt pavement patch over the completed trench.
- On October 1, 2007, the City of Ames resurfaced the street with 6 inches of asphalt pavement.

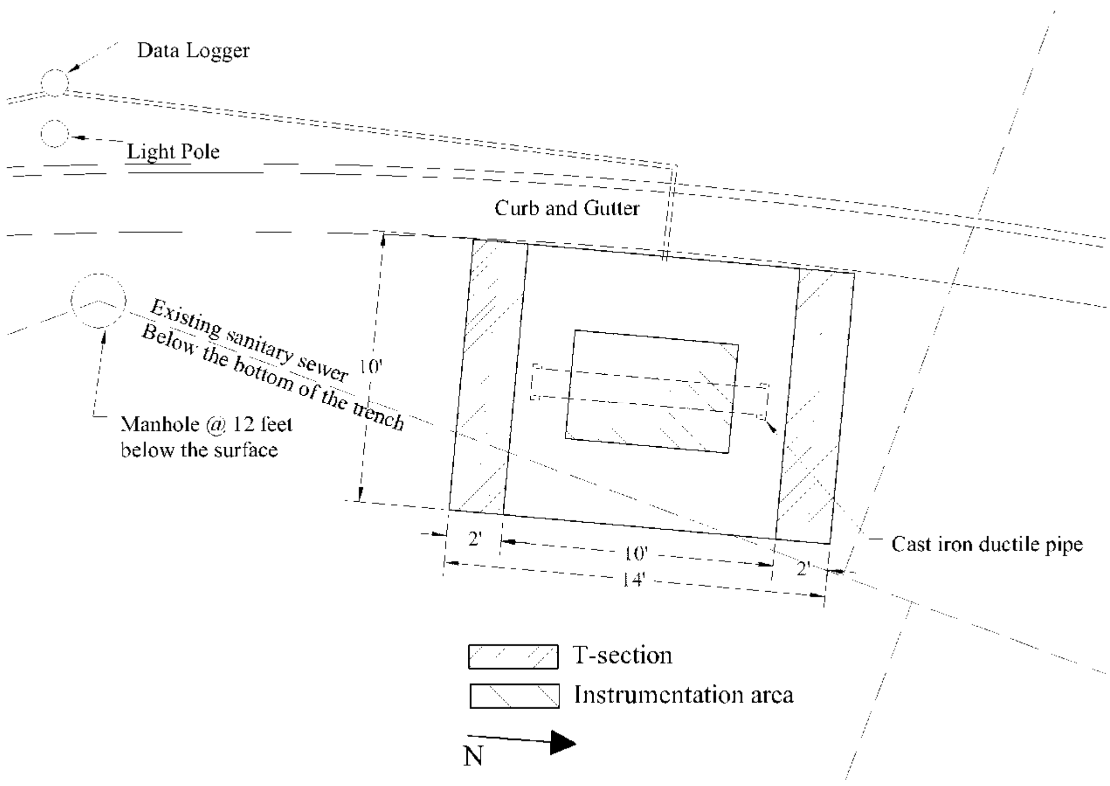


Figure 5.22. Plan view showing details of Trench CI

Figure 5.23. Adding water to the 3/8-inch minus backfill from above in Trench CI

Figure 5.24. The excavation of the T-section being constructed for Trench CI

Laboratory-Testing Results

Laboratory tests were conducted on the backfill from the instrumented trenches on Kellogg Avenue. These tests included particle size distribution with sieve and hydrometer analyses, Atterberg limits, water content, Standard Proctor, and minimum and maximum relative density tests. All tests were performed according to the corresponding ASTM standards.

Soil Classification

The 3/8-inch minus backfill had 3% of the particle by mass passing 0.02 mm. The percent of fines along with the classification of SP-SM resulted in a frost heave classification of F2 by the Army Corps of Engineers. The frost susceptibility of the soils was negligible to medium according to Figure 2.14 by the Army Corps of Engineers. The frost susceptibility rating accounts for the rate of frost heave per day. This means that these soils were expected to experience 0.5 mm/day to 4.0 mm/day of temperatures below freezing. The gradation chart in Figure 5.25 presents the gradations of the backfill used in the instrumented trenches and the soils excavated from the trenches.

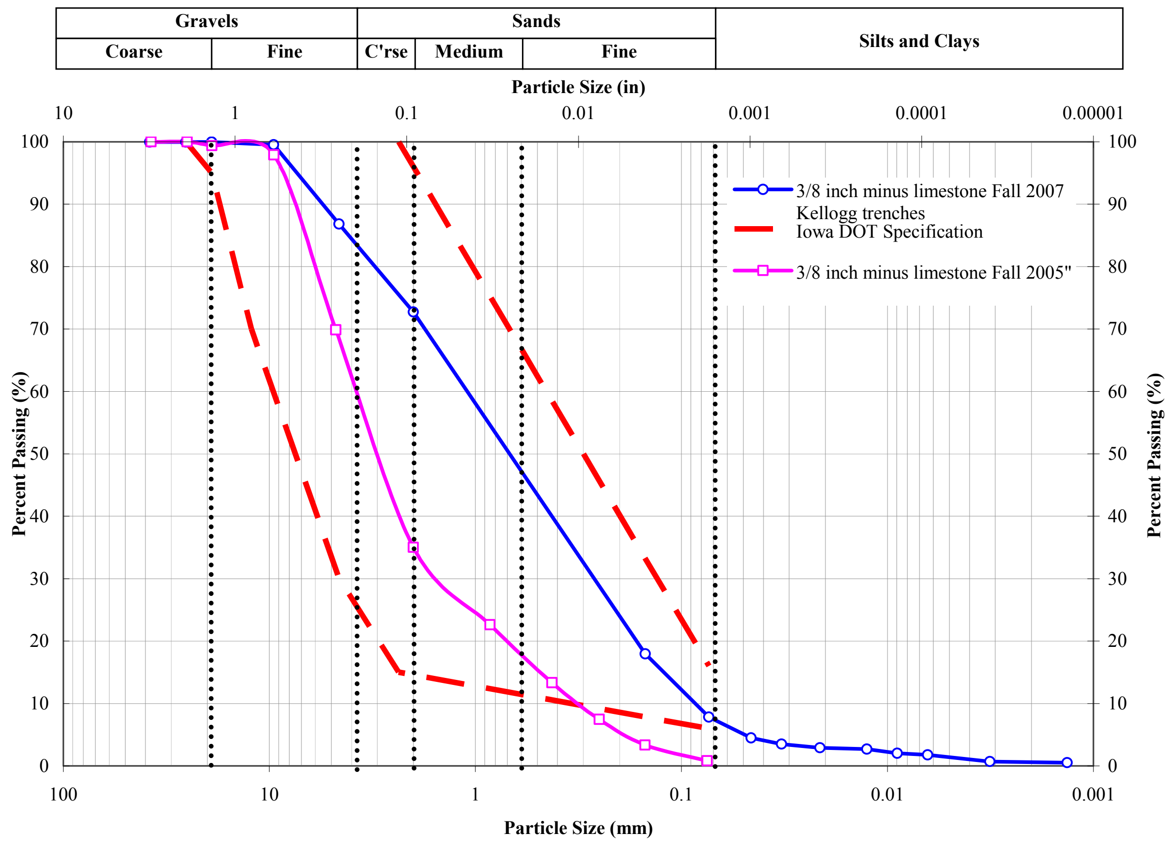


Figure 5.25. Gradation of backfill and soils excavated from the trenches

All soils were classified using the USCS and AASHTO standards. The soil classification results are summarized in Table 5.2. This table shows that the backfill was classified as SP-SM (poorly graded sand with silt) with the USCS and the AASHTO classification was A-1-B (stone fragments, gravel, and sand).

Table 5.2. Classification of the soil removed from the trenches on Kellogg Avenue and the 3/8-inch minus backfill

Sample	Soil classification			
	USCS		AASHTO	
3/8-inch minus backfill for from 2007	SP-SM	Poorly graded sand with silt	A-1-B	Stone fragment, gravel, and sand
3/8-inch minus backfill for from 2005	SM	Sand/silt	A-1-A	Stone fragment, gravel, and sand

Soil Compaction

Figure 5.26 compares the relative density test results with field measurements of the backfill. The bulking moisture content lowest dry unit weight occurred at 7% moisture content, and the bulking moisture content ranged from 4% to 8%.

Based on the classification of the 3/8-inch minus limestone as SP-SM, the recommended maximum dry unit weight from the Standard Proctor tests was 110 pcf to 125 pcf. The optimum moisture content ranged from 11% to 16%.

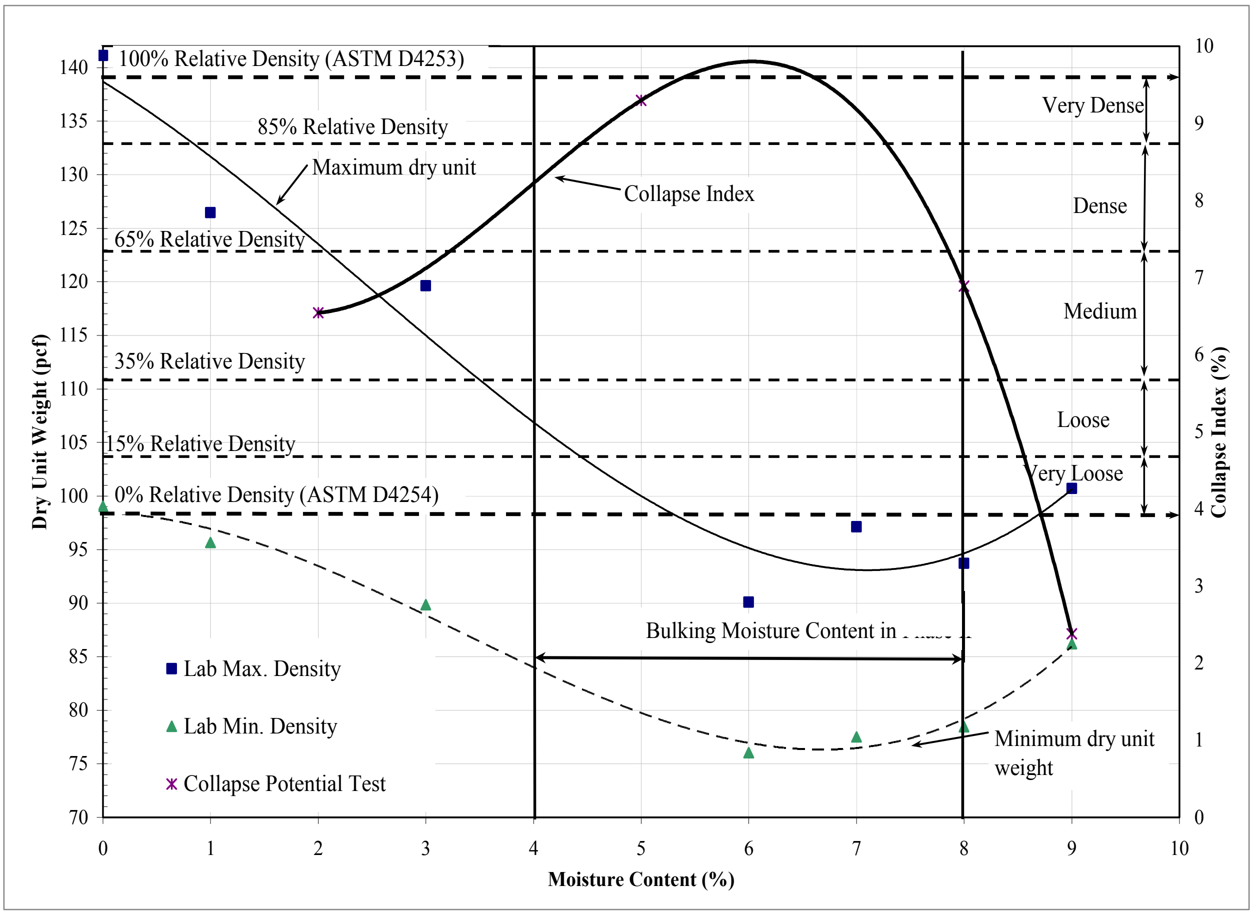


Figure 5.26. Relative density test results with the bulking moisture content and collapse index

Field Testing during Site Construction

Nuclear density and DCP tests were conducted on the three instrumented trenches. The trenches were constructed with 3/8-inch minus limestone that was classified as SP-SM (poorly graded sand and silt). The field-test results of the backfill were compared to the following NAVFAC recommendations for silty soils: (1) dry unit weights ranged from 110 pcf to 125 pcf; (2) optimum moisture contents ranged from 11% to 16%; and (3) CBR values ranged from 5% to

30%. The field-test results were also compared to values obtained during laboratory testing of the backfill used in Phase II: (1) maximum dry unit weight of 140 pcf; (2) minimum dry unit weight of 99.0 pcf; and (3) bulking moisture content from 4% to 8%.

Instrumented Trench AI

The locations of the test points in Trench AI are shown in Figure 5.27. The first lift was tested at points 2, 3, and 5 because the bottom of the trench was uneven. In lift 3 the nuclear density tests were performed at all five test points (1 through 5) and the DCP tests were performed at test points 1, 2, 3, and 4. Lifts 2 and 4 were tested at test points 1, 2, 3, and 4. Lift 5 was only tested using the nuclear density tests at field-testing points 2, 3, and 4. The fifth lift was not tested using DCP testing. Figure 5.28 shows the trench cross-section with the average testing results.

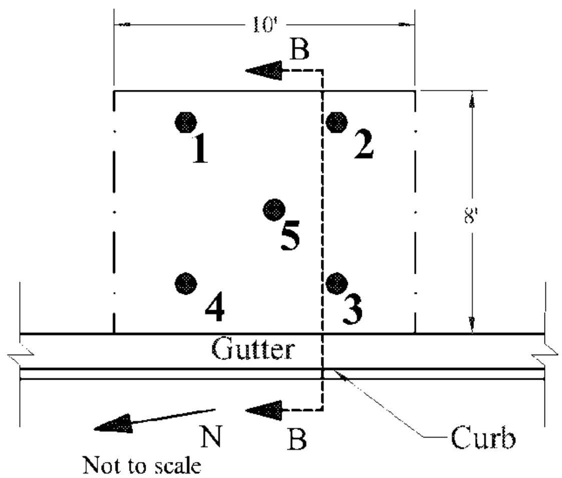


Figure 5.27. Locations of test points in Trench AI

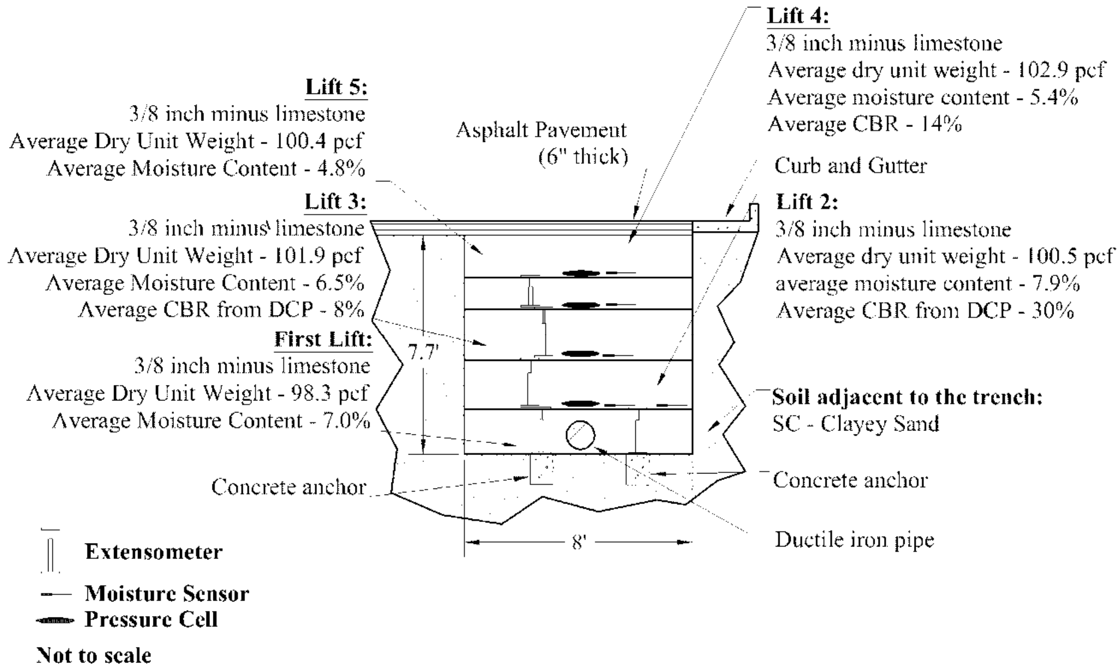


Figure 5.28. East-west cross-section of completed Trench AI with testing

Nuclear Density Test Results

Tables 5.3 and 5.4 summarize the average dry unit weight results and the moisture content results from the nuclear density tests for the various lifts in Trench AI. The probe depth was 4 inches.

All measured dry unit weights corresponded to very loose compaction densities. The average moisture contents measured in the trench for all lifts were within the bulking moisture content range. The average dry unit weights and moisture contents for each lift are plotted in Figure 5.29. This figure illustrates that all the lifts were placed at very loose relative densities and within the bulking moisture content range of 4% to 8%.

Table 5.3. Dry unit weights measured using nuclear density tests for Trench AI

Lift	Average dry unit weight (pcf)	Relative density (%)	Relative compaction	Minimum and maximum dry unit weights from field testing (pcf)	Standard deviation	Coefficient of variance (%)
Lift 1	98.3	0	Very Loose	92.3/104.1	5.9	6.0
Lift 2	100.5	5	Very Loose	97.4/105.1	3.5	3.4
Lift 3	101.9	9	Very Loose	96.7/106.9	4.1	4.0
Lift 4	102.9	13	Very Loose	97.6/107.3	4.1	4.0
Lift 5	100.4	4	Very Loose	97.1/102.6	2.9	2.9
Average for all lifts	101.0	7	Very Loose	---	4.9	---

Table 5.4. Average moisture contents measured using nuclear density tests for Trench A1

Lift	Average moisture content (%)	Degree of saturation (%)	Minimum and maximum moisture contents (%)	Standard deviation	Coefficient of variance (%)
Lift 1	7.0	12.9	5.2/8.3	1.6	22.9
Lift 2	7.9	14.8	7.0/10.1	1.5	7.0
Lift 3	6.5	12.2	5.1/9.8	1.9	29.8
Lift 4	5.4	10.1	4.9/5.7	0.4	27.9
Lift 5	4.8	8.7	3.9/6.3	1.3	3.9
Average for all lifts	6.3	11.4	---	1.7	---

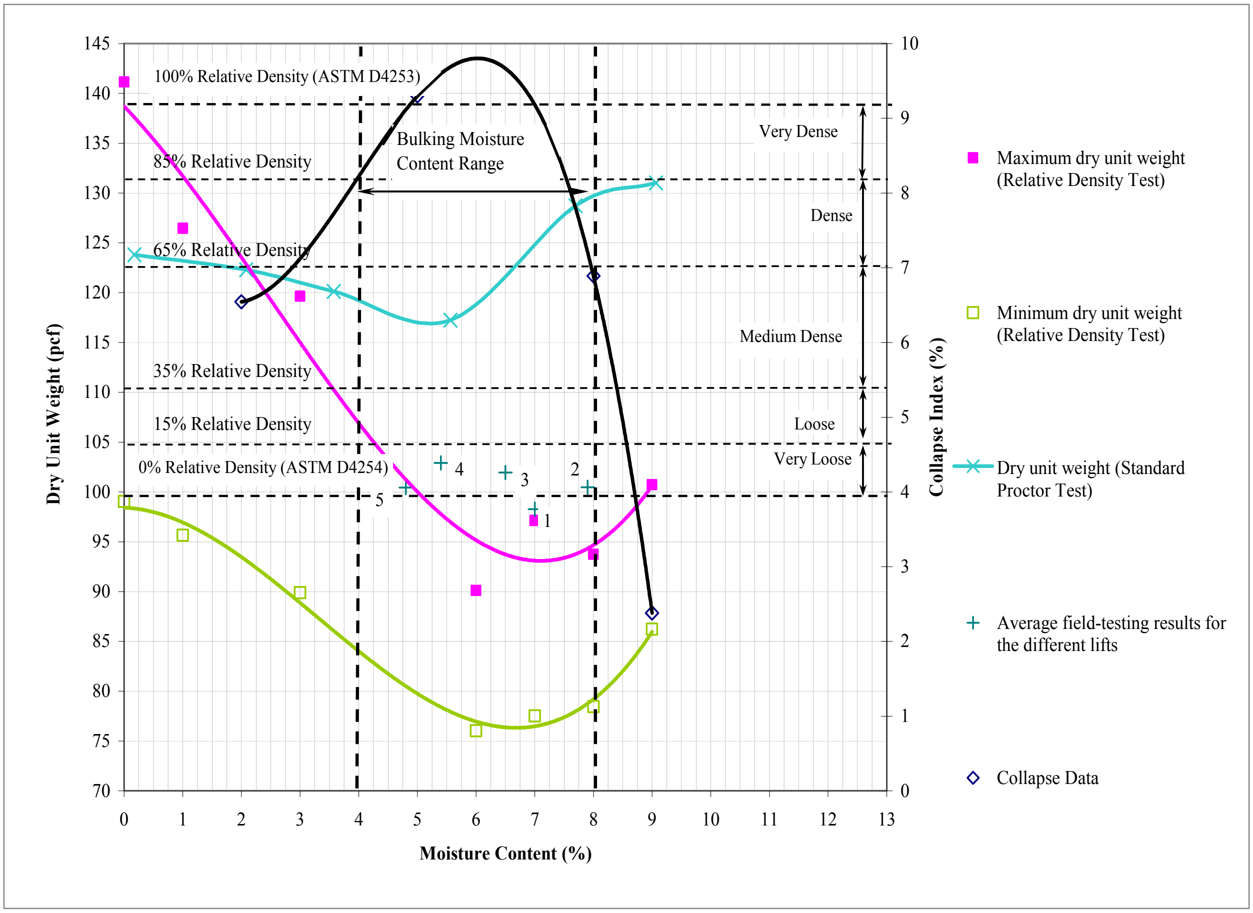


Figure 5.29. Dry unit weights and moisture contents measured during the construction of Trench AI compared with the relative density and Standard Proctor test results with the collapse index

DCP Test Results

Tables 5.5 and 5.6 summarize the DCPI and the CBR results from the DCP tests for the various lifts. The typical CBR values range from 10% to 40% according to NAVFAC in Table 2.5. The CBR values for lifts 1 and 4 were within the recommended range from NAVFAC. However, lifts 2 and 3 had CBR values below the recommended values from NAVFAC. The locations of low and high average CBR values did not correspond to the maximum and minimum dry unit weights measured during construction.

Table 5.5. Average DCPI calculated from DCP test results for Trench AI

Lift	Average DCPI	Average depth of DCP tests (inch)	Standard deviation	Coefficient of variance (%)
Lift 1	43.2	13.9	37.8	87.3
Lift 2	35.1	19.6	24.9	70.9
Lift 3	31.8	21.2	14.0	43.9
Lift 4	19.7	26.1	13.4	67.9
Lift 5	N/A	N/A	N/A	N/A

Table 5.6. Average CBR calculated using DCP test results for Trench AI

Lift	Average CBR (%)	Average depth of DCP tests (inch)	Standard deviation	Coefficient of variance (%)
Lift 1	30	13.9	27.8	92.6
Lift 2	9	19.6	6.0	68.2
Lift 3	8	21.2	4.1	49.3
Lift 4	14	26.1	7.2	49.8
Lift 5	N/A	N/A	N/A	N/A

Instrumented Trench BI

The nuclear density gauge and DCP tests were performed at four different test points for all lifts. The location of the test points is shown in Figure 5.30. Test points 1 through 4 were used to test all lifts. Figure 5.31 shows the trench cross-section with the average testing results.

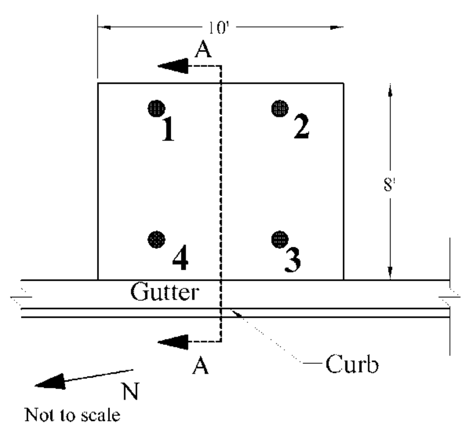


Figure 5.30. Location of test points in Trench BI

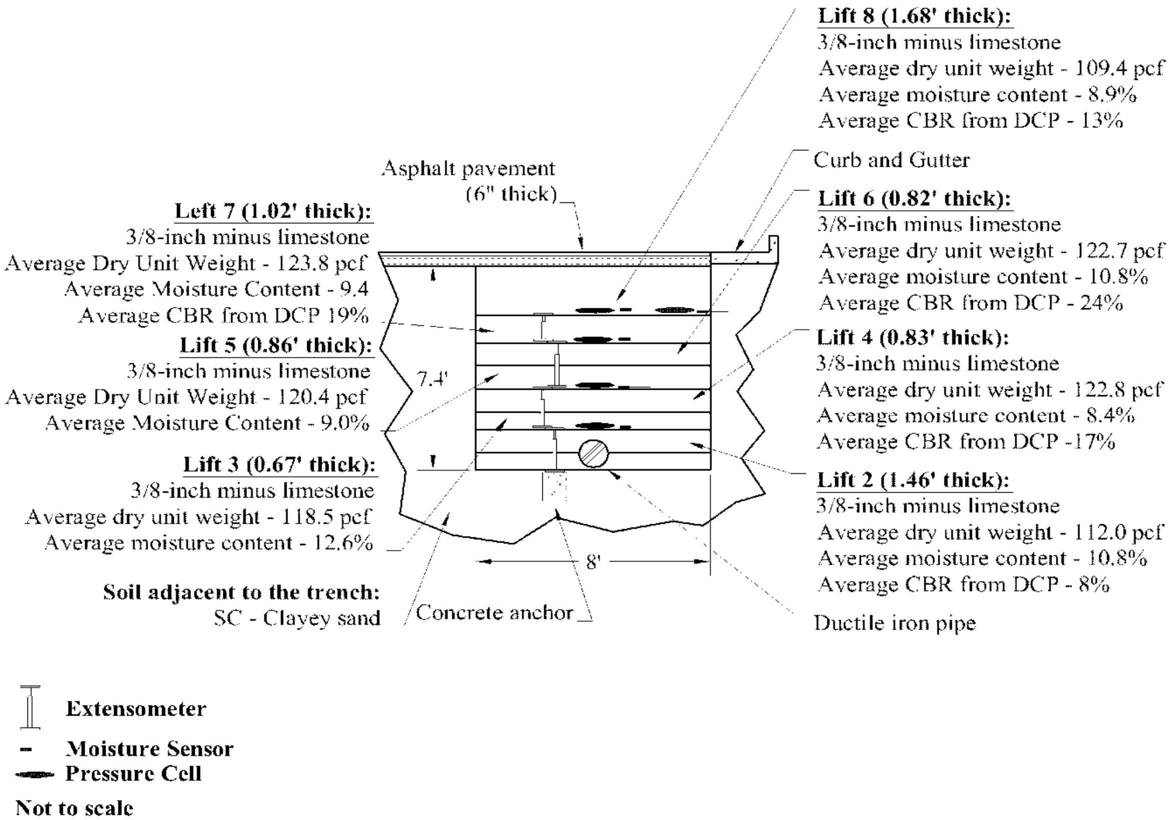


Figure 5.31. East-west cross-section of completed Trench BI with testing results

Nuclear Density Test Results

Tables 5.7 and 5.8 summarize the average dry unit weight and the moisture content results from the nuclear density tests for the various lifts in Trench B1. The probe depth during testing was 4 inches. As reported in the laboratory testing section, the maximum dry unit weight was 140 pcf and the bulking moisture content ranged from 4.0% to 8.0%.

The average dry unit weights for lifts 2 through 7 ranged from 112.0 pcf to 123.8 pcf with average relative densities ranging from 39% to 67%, which corresponds to medium dense to dense compaction. The average dry unit weight for lift 8 was 109.4 pcf. The relative density of lift 8 was 32%, which corresponded to loose compaction density.

The average moisture contents for all lifts ranged from 8.4% to 12.6%. All the moisture contents were above the bulking moisture content.

The lower lifts were compacted to medium dense to dense compactions. The upper lift was loose at 32% relative density. Figure 5.32 shows the average field-testing results for each lift compared to lab-test results. The average dry unit weight for all the lifts was 118.5 pcf. This is a 55% relative density or medium dense compaction. The average moisture content for all lifts was 9.7%. The moisture content was above the bulking moisture content.

Table 5.7. Dry unit weights measured using nuclear density testing for Trench BI

Lift	Average dry unit weight (pcf)	Relative density (%)	Relative compaction	Minimum and maximum dry unit weights from field testing (pcf)	Standard deviation	Coefficient of variance (%)
Lift 2	120.0	59	Medium	115.1/123.4	3.6	3.2
Lift 3	118.5	55	Medium	113.2/121.4	3.8	3.2
Lift 4	122.8	65	Dense	120.0/123.5	1.9	1.5
Lift 5	120.4	59	Medium	111.4/125.2	6.2	5.1
Lift 6	122.7	65	Dense	121.2/124.8	1.5	1.2
Lift 7	123.8	67	Dense	121.4/125.8	1.1	1.9
Lift 8	109.4	32	Loose	106.4/111.0	2.2	2.0
Average for entire trench	119.6	58	Medium	---	5.5	

Table 5.8. Average moisture contents measured using nuclear density testing for Trench BI

Lift	Average moisture content (%)	Degree of saturation (%)	Minimum and maximum moisture contents (%)	Standard deviation	Coefficient of variance (%)
Lift 2	10.8	74.2	9.4/12.6	1.3	12.0
Lift 3	12.6	49.6	11.7/14.4	1.3	10.3
Lift 4	8.4	28.6	7.7/9.1	0.8	9.5
Lift 5	9.0	26.3	8.6/9.0	0.3	3.3
Lift 6	10.8	30.1	9.2/11.7	1.1	11.7
Lift 7	9.4	25.2	8.2/10.8	1.5	16.0
Lift 8	8.6	18.9	8.4/9.6	0.6	7.0
Average for entire trench	10.0	23.9	---	1.7	

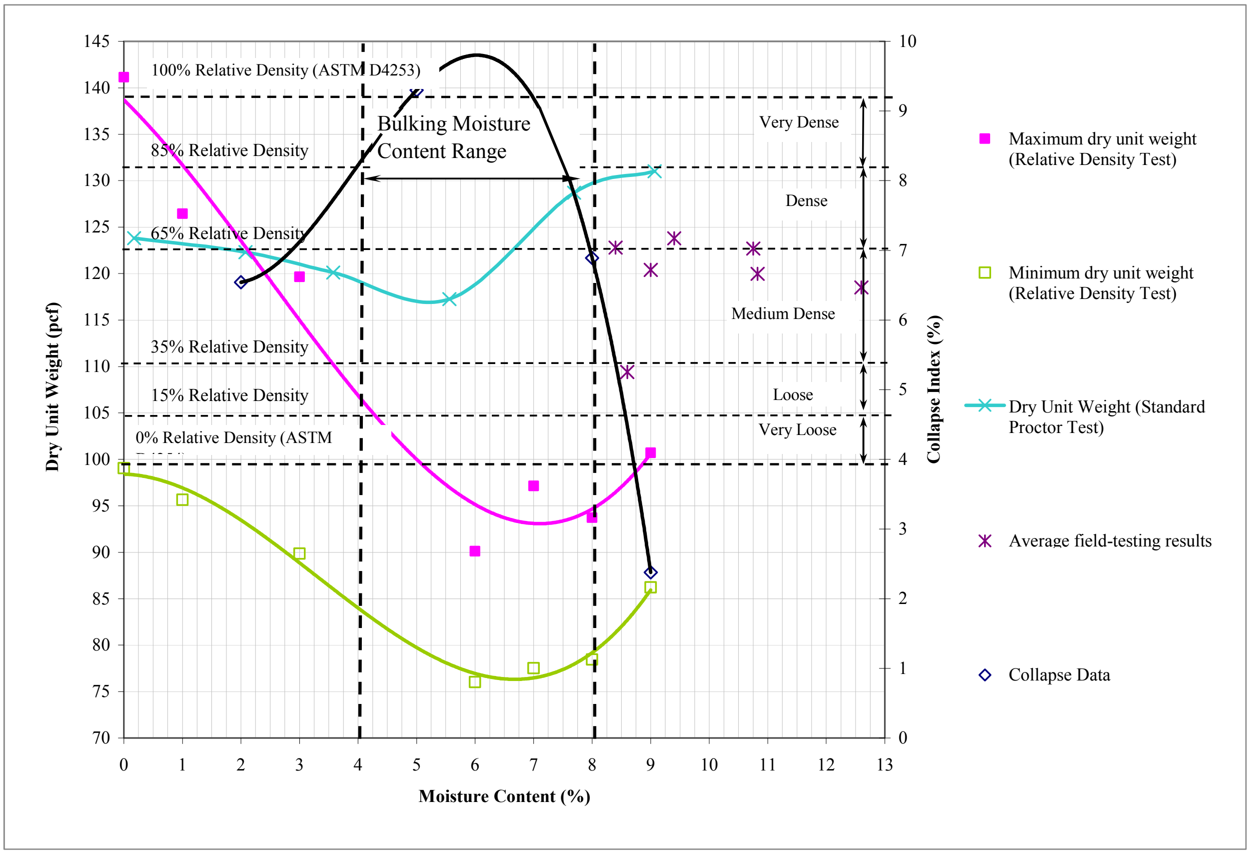


Figure 5.32. Summary of average dry unit weights and moisture contents measured for different lifts during the construction of Trench BI compared with the relative density and Standard Proctor test results with the collapse index

DCP Test Results

Tables 5.9 and 5.10 summarize the DCPI and CBR results from the DCP tests for the various lifts. The fifth lift was not tested using the DCP. The average CBR value for lift 2 was below the typical values from NAVFAC. The remaining lifts had CBR values that were within the recommended CBR values from NAVFAC.

Table 5.9. Average DCPI calculated from DCP testing for Trench BI

Lift	Average DCPI	Average depth of test (inch)	Standard deviation	Coefficient of variance (%)
Lift 2	42.5	25.5	46.5	108.3
Lift 3	N/A	N/A	N/A	N/A
Lift 4	16.2	19.5	8.3	51.2
Lift 6	18.7	19.5	8.7	46.5
Lift 7	13.4	19.1	26.6	19.8
Lift 8	23.8	20.7	17.0	73.1

Table 5.10. Average CBR results calculated using DCP test results for Trench BI

Lift	Average CBR	Average depth of test (inch)	Standard deviation	Coefficient of variance (%)
Lift 2	8	25.5	4.9	62.6
Lift 3	N/A	N/A	N/A	N/A
Lift 4	17	19.5	9.1	54.2
Lift 6	24	19.5	12.4	51.7
Lift 7	19	19.1	8.8	78.2
Lift 8	13	20.7	8.2	61.6

Instrumented Trench CI

The locations of the test points in the trench are shown in Figure 5.33. Figure 5.34 shows the trench cross-section with the average testing results.

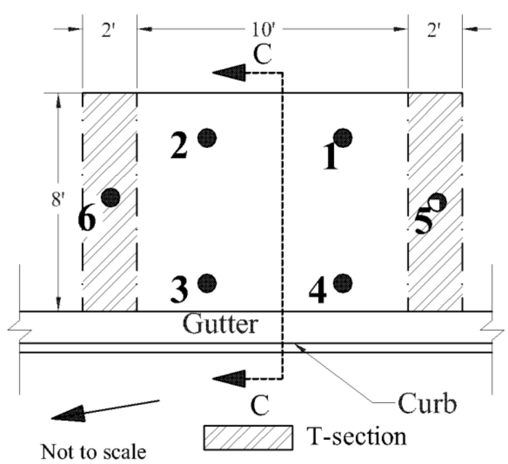


Figure 5.33. Location of test points in Trench CI

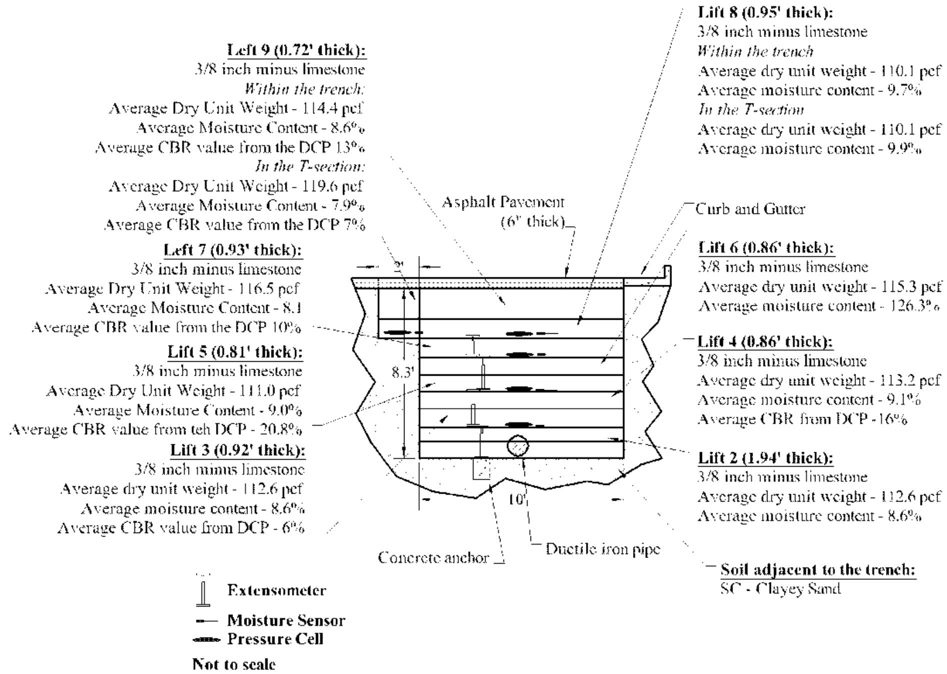


Figure 5.34. Cross-section of completed Trench CI

Nuclear Density Test Results

Table 5.11 summarizes the average dry unit weight results from the nuclear density tests for each layer. Table 5.12 summarizes the moisture content results from the nuclear density tests for the various lifts in Trench CI. The typical dry unit weights from NAVFAC ranged from 110 pcf to 125 pcf, and the typical optimum moisture content ranged from 11% to 16%. From laboratory testing, the maximum dry unit weight was 140 pcf and the minimum dry unit weight was 99.0 pcf. The range of the bulking moisture contents was from 4% to 8%. The probe depth of the nuclear density gauge was 4 inches.

Figure 5.35 illustrates the average dry unit weights and moisture contents for each tested lift. The average dry unit weights for all tested lifts ranged from 109.4 pcf to 123.8 pcf. The average dry unit weights for all the lifts were within the recommended values by NAVFAC, except for lift 9 in the T-section. The relative densities ranged from 32% to 67%, which corresponded to medium dense and dense compaction.

The average moisture contents ranged from 8.4% to 12.6% for all tested lifts. The average moisture contents for all tested lifts were below recommended values from NAVFAC. All lifts had average moisture contents above the range of bulking moisture contents.

Table 5.11. Dry unit weights measured using nuclear density tests for Trench CI

Lift	Average dry unit weight (pcf)	Relative density (%)	Relative compaction	Minimum and maximum dry unit weights	Standard deviation	Coefficient of variance (%)
Lift 2	115.1	47	Medium	104.4/127.6	9.5	8.2
Lift 3	112.6	40	Medium	107.4/119.7	5.2	4.6
Lift 4	113.2	42	Medium	96.7/106.9	5.1	4.5
Lift 5	111.0	36	Medium	107.4/119.4	4.1	3.7
Lift 6	115.3	47	Medium	105.9/114.9	3.7	3.2
Lift 7	116.5	50	Medium	114.3/119.0	2.1	1.8
Lift 8 within the trench	110.1	34	Loose	107.8/112.8	2.3	2.1
Lift 8 in the T-section	110.1	45	Medium	107.5/112.7	3.7	3.3
Lift 9 within the trench	114.4	34	Loose	113.3/117.1	1.8	1.6
Lift 9 in the T-section	119.6	57	Medium	119.2/119.9	0.5	0.4
Average for all lifts	113.7	43	Medium	---	4.8	

Table 5.12. Average moisture contents measured using nuclear density tests for Trench CI

Lift	Average moisture content (%)	Degree of saturation (%)	Minimum and maximum moisture contents (%)	Standard deviation	Coefficient of variance (%)
Lift 2	7.8	16.2	7. 3/9.4	1.4	17.5
Lift 3	8.6	17.3	7.0/10.1	0.9	10.9
Lift 4	9.1	18.3	7.3/9.5	0.8	9.1
Lift 5	9.0	17.6	8.4/10.1	0.8	9.1
Lift 6	9.6	19.5	8.0/10.0	0.2	2.3
Lift 7	8.1	16.5	9.4/9.9	0.4	5.4
Lift 8 within the trench	9.7	18.6	9.1/11.0	0.9	9.4
Lift 8 in the T-section	9.9	18.9	9.9/10.1	0.1	1.4
Lift 9 within the trench	8.6	17.0	8.3/9.0	0.3	4.0
Lift 9 in the T-section	7.2	14.9	7.9 / 8.5	1.9	11.7
Average for all lifts	8.8	17.2	---	1.0	

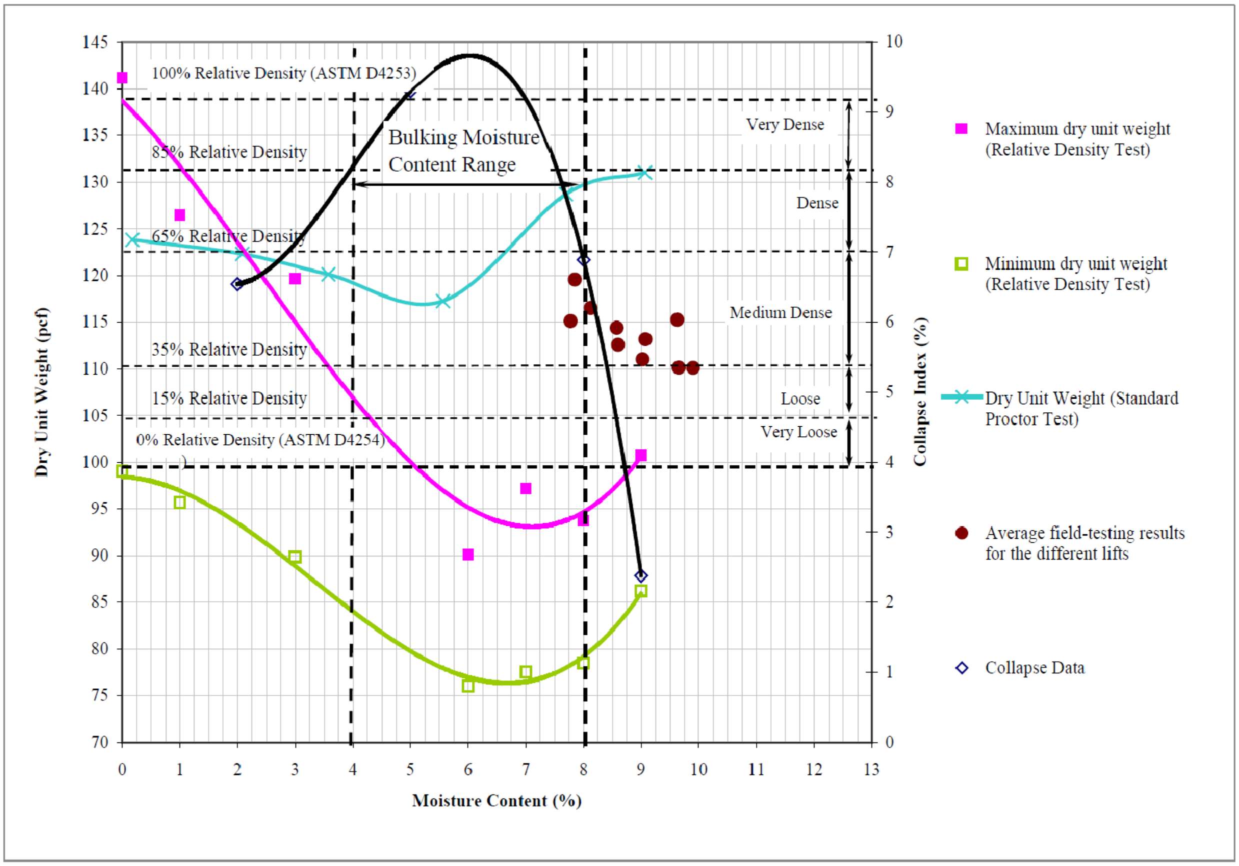


Figure 5.35. Dry unit weights and moisture contents measured during the construction of Trench CI compared with the relative density and Standard Proctor test results with the collapse index

DCP Test Results

Table 5.13 summarizes the DCPI, and Table 5.14 summarizes the CBR results from the DCP tests for the various lifts. The typical CBR values for compacted soil from NAVFAC range from 10% to 40%.

The average CBR values for lifts 2, 3, and 9 in the T-section were below the recommended values from NAVFAC. The average CBR values from lifts 4, 5, 7, and 9 within the trench were within the recommended range of values from NAVFAC.

Table 5.13. Average DCPI results from DCP testing for Trench CI

Lift	Average DCPI	Average depth of DCP test	Standard deviation	Coefficient of variance (%)
Lift 2	78.0	22.6	72.3	
Lift 3	87.8	22.3	107.2	112.2
Lift 4	18.2	23.5	10.9	60.1
Lift 5	20.8	22.2	15.5	74.5
Lift 7	23.1	21.6	13.1	57.1
Lift 9 within the trench	20.6	25.5	10.5	51.0
Lift 9 in T-section	32.9	18.6	14.9	45.4

Table 5.14. Average CBR results from DCP testing for Trench CI

Lift	Average CBR (%)	Average depth of DCP test	Standard deviation	Coefficient of variance (%)
Lift 2	3	22.6	2.4	
Lift 3	6	22.3	5.8	91.8
Lift 4	16	23.5	9.7	59.1
Lift 5	16	22.2	10.5	64.6
Lift 7	10	21.6	6.5	63.3
Lift 9 within the trench	13	25.5	6.1	48.2
Lift 9 in T-section	7	18.6	3.9	53.7

FWD Monitoring and Elevation Surveys of Trenches

Instrumented Trench AI

To monitor the long-term performance of Trench AI, FWD testing and elevations surveys were conducted on the site. The first FWD test on Trench AI was on November 5, 2007. This was 45 days after the construction of the trench.

FWD Test Results

Falling weight deflectometer tests were performed on the trenches on November 05, 2007; June 28, 2008; and March 13, 2009. Forces of 6 kips, 9 kips, 12 kips, and 15 kips were applied to each test point along the trenches. The deflections of the pavement were measured at eight points across each trench. The locations of the test points for Trench AI are shown on Figure 5.36.

Figure 5.37 shows the deflection of 15 kip at the test points for Trench AI for all test dates. Values correspond to the test on November 11, 2007. The backfill in the trench had smaller deflections than the surrounding soil. This shows that the backfill was placed stiffer than the surrounding soil. Testing north of the trench was limited due to a sewer crossing the site. At FWD testing point 5, in the center of the trench the deflection was 18.1 mils from the 15-kip load. The average dry unit weight for the fifth lift was 100.4 pcf at an average moisture content of 3.9%. No DCP testing was performed on the top lift (lift 5). The average CBR value calculated for lift 4 from the DCP test was 14%.

At FWD testing point 12, the deflection was 21.0 mils under a 15-kip load. Test points 2 and 3 had an average dry unit weight for lift 5 of 99.8 pcf at a moisture content of 4.0%. The CBR at the surface of lift 4 (i.e., 1.5 feet below the surface) was 3%, and the average CBR for lift 4 value was 12%.

At FWD testing point 14 on the north edge of the trench, the deflection was 22.4 mils. Near this deflection was test point 4 on the fifth lift; test point 1 was not tested on the fifth lift. The dry unit weight at test point 4 was 101.6 pcf at a moisture content of 6.3%. The CBR value at the surface of lift 4 (1.5 feet below the surface) was 4% and the average CBR for lift 4 value was 17%. The lower deflections from the FWD testing occurred at lower dry unit weights and CBR values.

Figure 5.37 also shows two survey dates with the settlement and the FWD deflections for the 15-kip load. This shows that the trench had a uniform response to the FWD test. The uplift was uniform. This also shows that where the FWD shows softer soils the uplift during the winter was uneven.

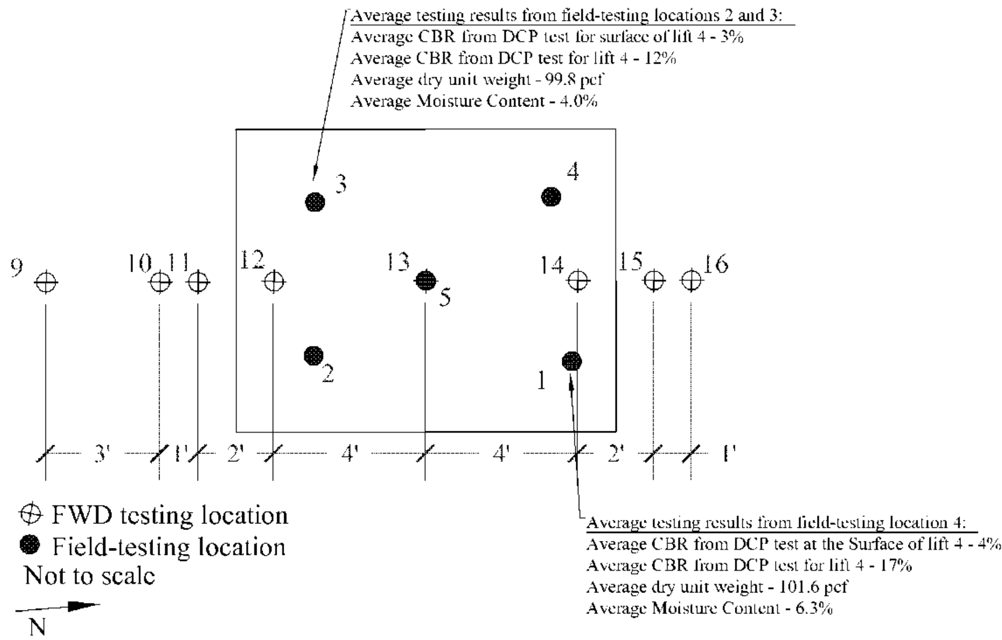


Figure 5.36. Location of FWD testing for Trench AI with the field-testing locations and average field-testing results

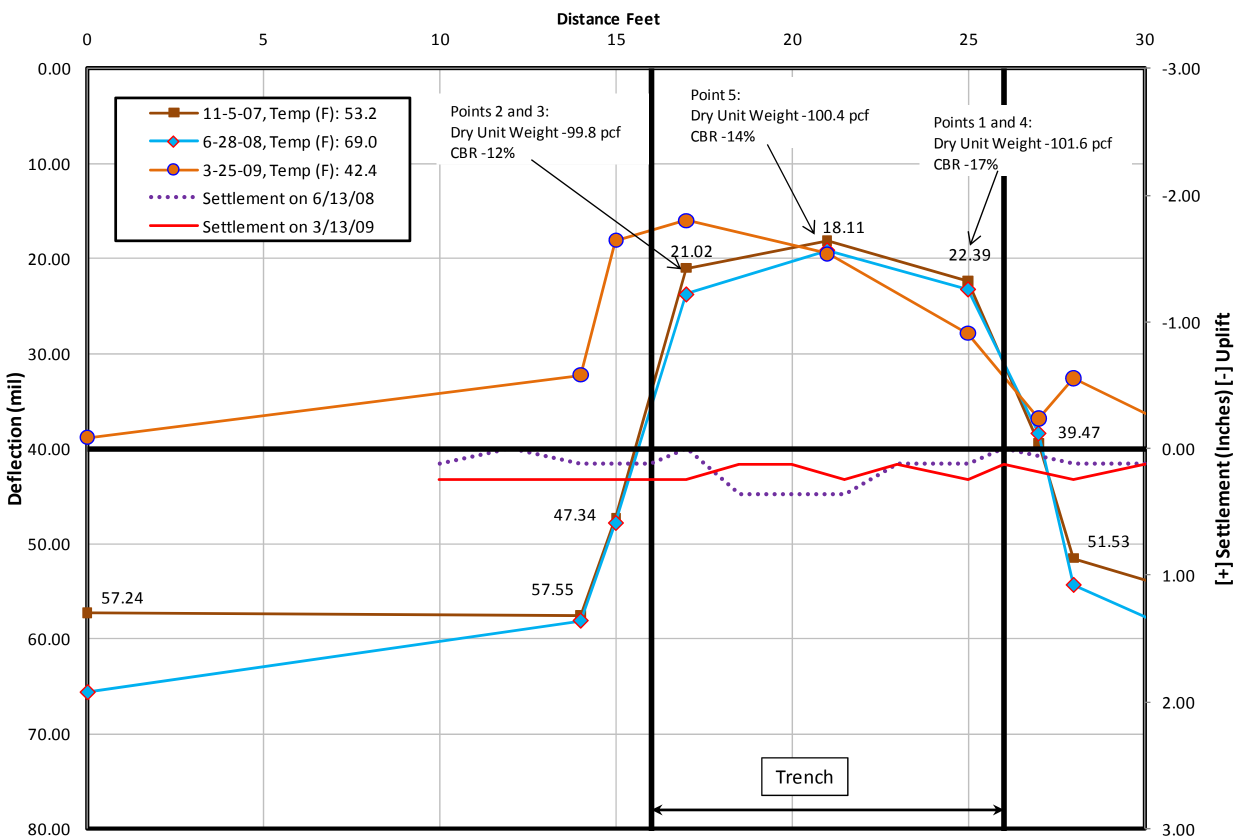


Figure 5.37. Displacement of Trench AI with the 15-kip load superimposed

Post-Construction Elevation Survey

Trench AI was surveyed March 19, 2008; June 13, 2008; August 19, 2008; November 18, 2008; and March 13, 2009. The site survey contained 41 grid points. Figure 5.38 shows the location of the grid points across the trench.

The average uplift between the two surveys was 0.31 inches, and the maximum uplift was 0.48 inches at survey point 2. The minimum uplift between the two surveys was 0.12 inches at survey points 27 and 33 (see Figure 5.39).

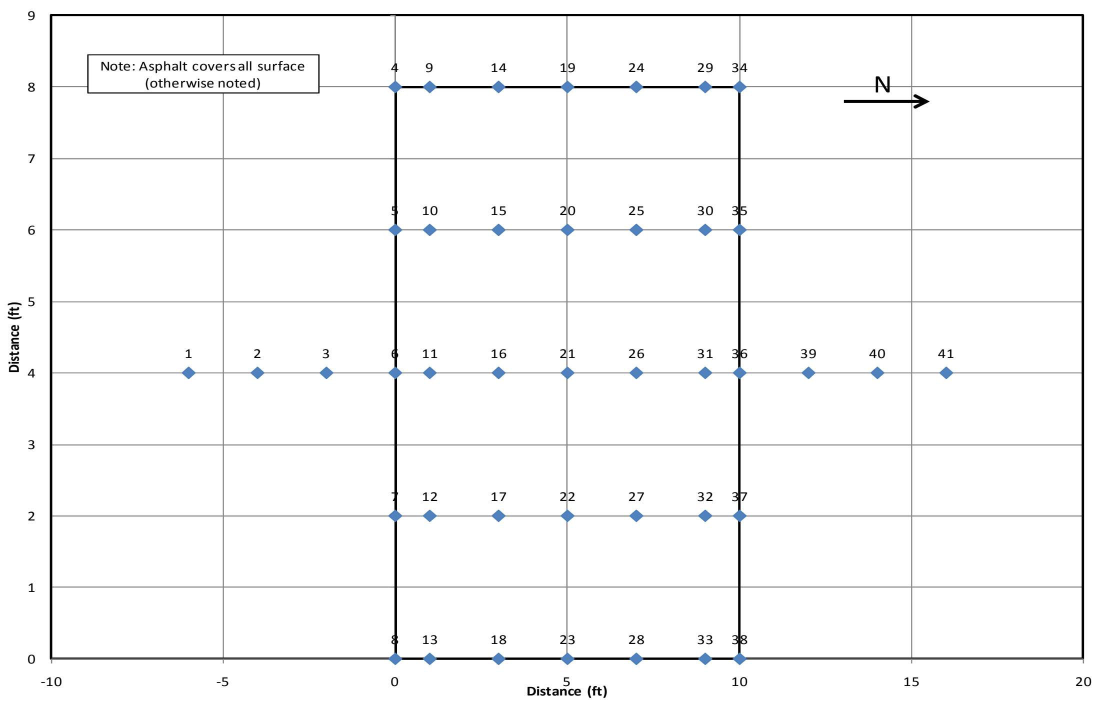


Figure 5.38. Location of points used to survey the elevation of the surface of the pavement on top of the trench and surrounding area

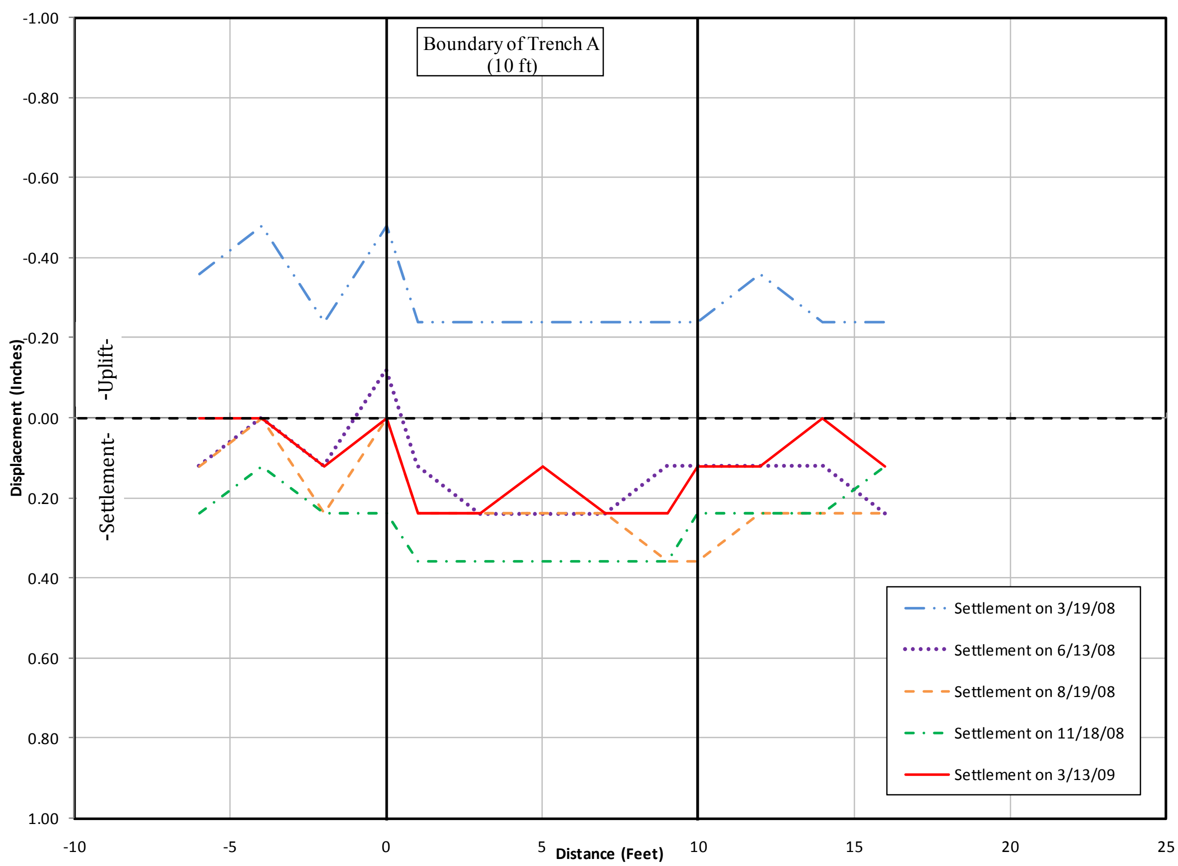


Figure 5.39. Settlement along Trench AI (points 1, 2, 3, 6, 11, 16, 21, 26, 31, 36, 39, 40, 41)

Instrumented Trench BI

To monitor the long-term performance of Trench BI, FWD testing and elevation surveys were conducted on the site. The first FWD test on Trench BI was on November 5, 2007. This was 38 days after the construction of the trench.

Falling weight deflectometer Test Results

Falling weight deflectometer tests were performed on the trenches on November 5, 2007; June 28, 2008; and March 13, 2009. Forces of 6 kips, 9 kips, 12 kips, and 15 kips were applied to each test point along the trenches. The deflections of the pavement were measured at eight points across each trench. The locations of the test points are shown in Figure 5.40 for Trench BI.

Figure 5.41 shows the deflections from the 15-kip load at the test points for Trench BI for all test dates. Values correspond to tests on November 11, 2007. This figure shows that the trench had a stiffer response than the surrounding soil for all test dates. At the south edge of the trench, the deflection was 21.0 mils at FWD testing point 4. The average dry unit weight of test points 2 and 3 on the south edge of the trench for the top lift was 110.1 pcf at a moisture content of 9%. The CBR value at the surface was 3%, and the average CBR value for the top 2 feet was 15%. At the north edge of the trench, the deflection from the 15-kip load was 22.4 at FWD testing point 6. Within the vicinity of this deflection were test points 1 and 4. The average dry unit weight was 108.7 pcf at 8.8%. The CBR value at the surface was 3%, and the average for the top 2 feet was 12%. The higher deflections corresponded to lower dry unit weights and CBR values. As the load increased, the difference between the backfill within the trench and the surrounding soil increased. This was the result of the difference in the soil densities and stiffness.

Figure 5.41 also shows the settlement between the summer and spring of 2008. Superimposed on the graph is the deflection measured from the 15-kip load in the fall. This shows that the locations of the highest deflections from the FWD testing corresponded to the smallest uplifts. This may be attributed to frost heave effects. The soils with lower stiffness were classified as clayey sand, which was less susceptible to frost heave than the 3/8-inch minus limestone.

At the inside edges of the trench, the FWD deflection increased. At 2 feet outside the trench (both south and north sides), the FWD deflections were larger than those recorded within the trench. According to elevation surveys, the soil at these locations experienced smaller uplifts during freezing conditions. This was a function of the soil type and the density of the surrounding soil.

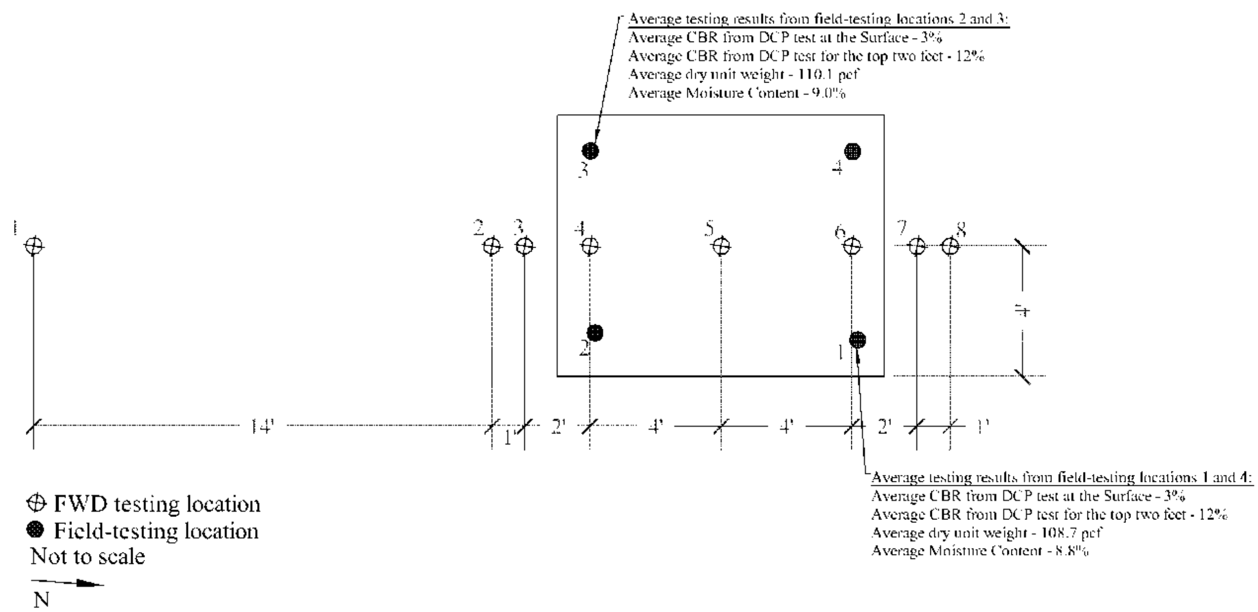


Figure 5.40. Falling weight deflectometer testing locations for Trench BI and average field-testing results for the uppermost lift for the north and south edges of the trench

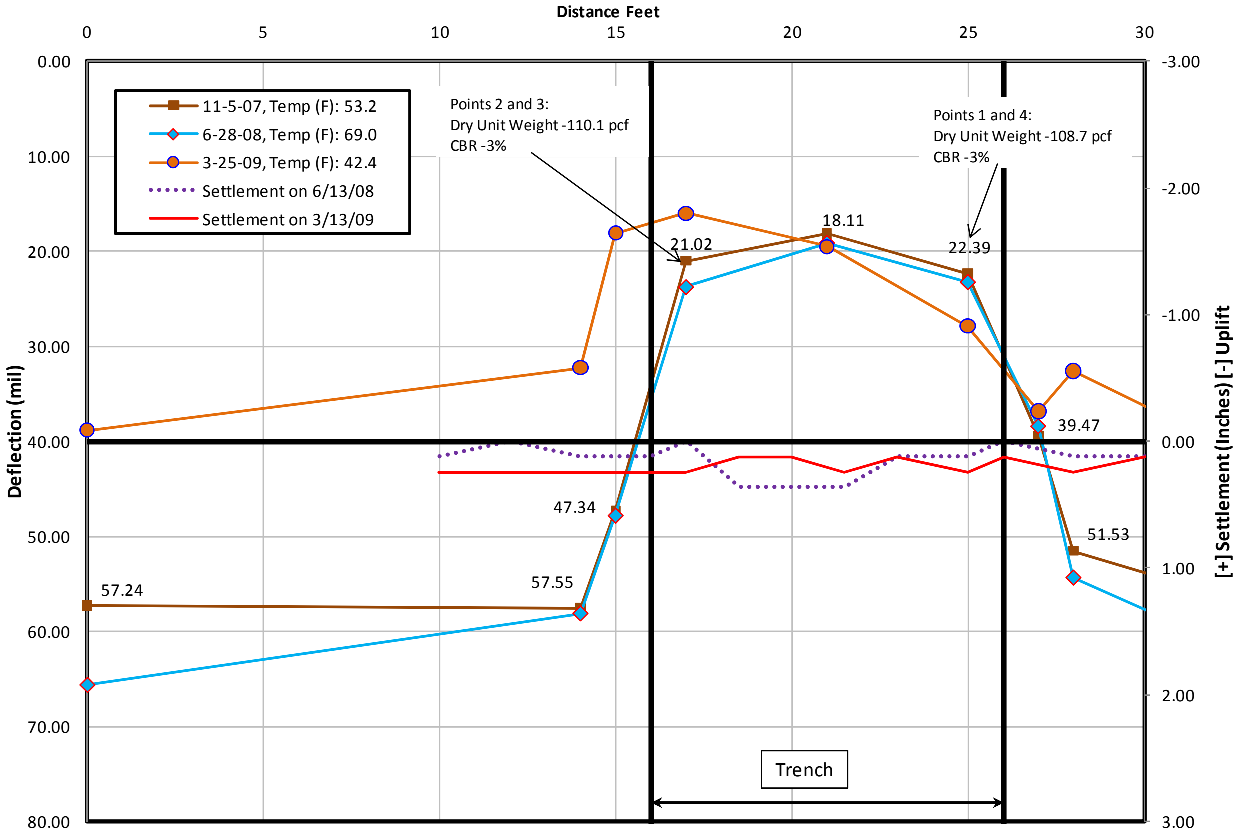


Figure 5.41. Displacement of Trench BI with the 15-kip load superimposed

Post Construction Elevation Survey

Trench BI was surveyed March 19, 2008; June 13, 2008; August 19, 2008; November 18, 2008; and March 13, 2009. The site survey contained 41 grid points. Figure 5.42 shows the location of the grid points across the trench. On March 19, 2008, the difference between the maximum and minimum elevation was 0.2 inches.

On June 13, 2008, the difference between the maximum and minimum elevation was 0.3 inches. The difference between the maximum and minimum elevation did not change between the two surveys. This shows the trench behaved in a uniform manner and differential movement did not occur. The average uplift between the two surveys was 0.32 inches, the maximum uplift was 0.48 inches (at survey points 15, 25, and 39), and the minimum uplift was 0.12 inches (at survey points 8, 19, and 29). The pavement across the trench sloped from north to south and from the center of the road to the curb (see Figure 5.43).

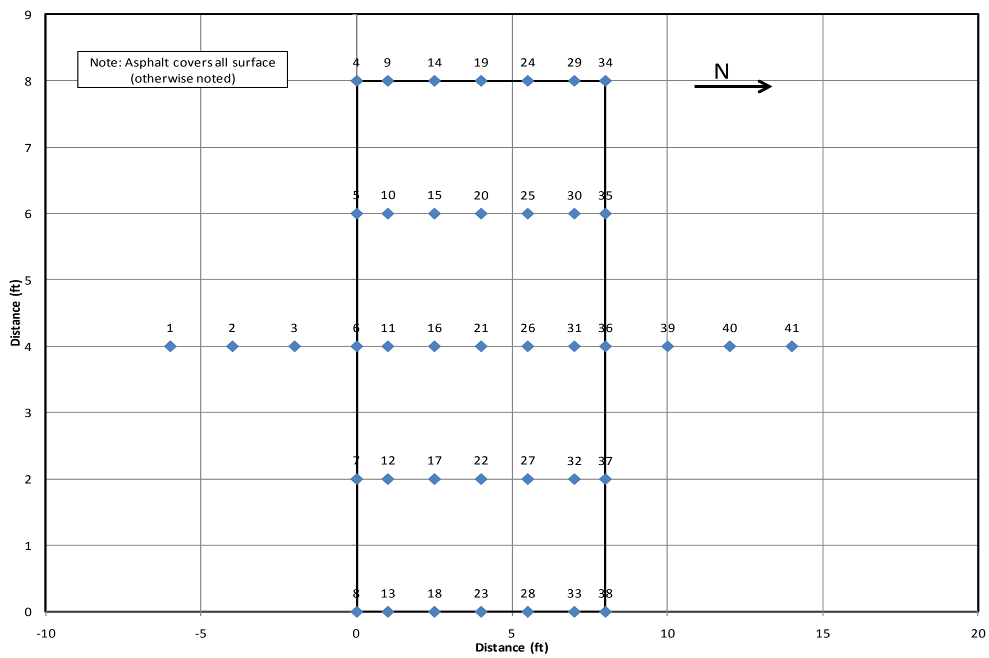


Figure 5.42. Location of points used to survey the elevation of the surface of the pavement on top of the trench and surrounding area

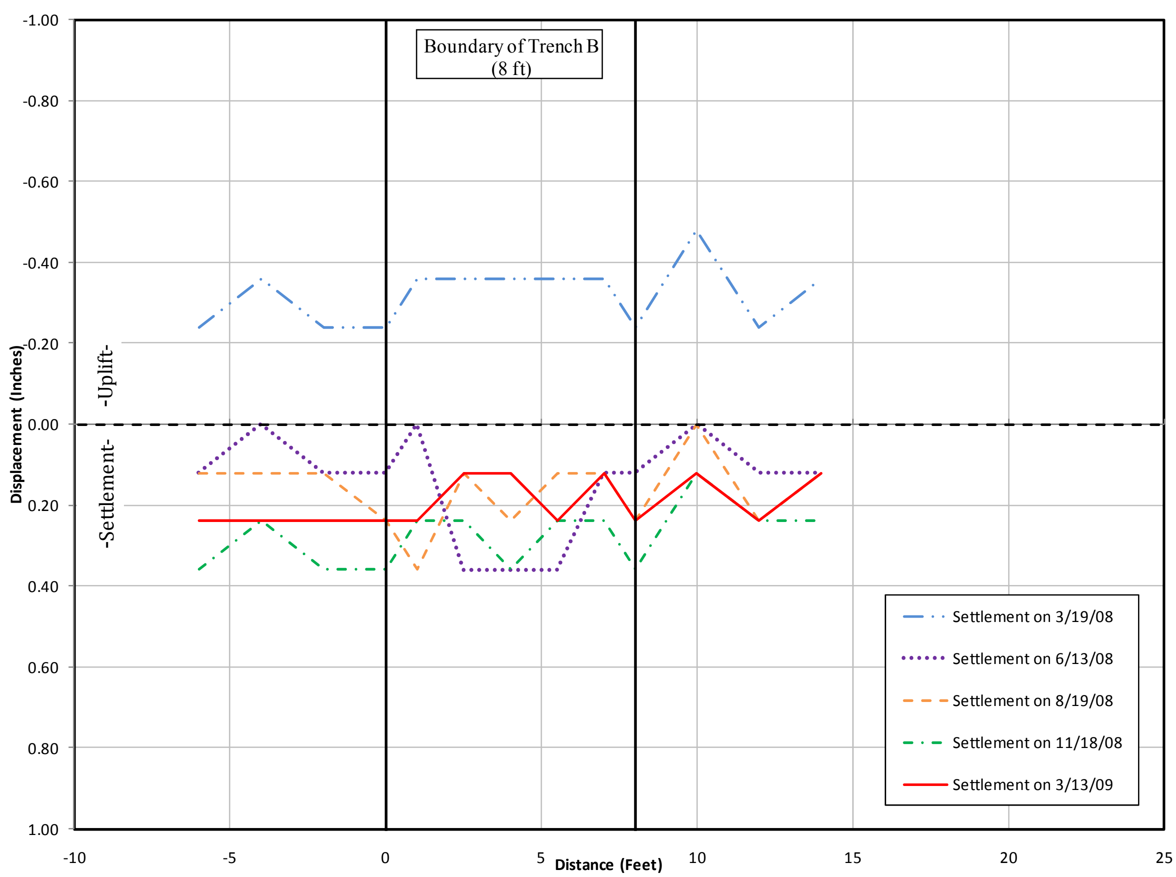


Figure 5.43. Settlement along Trench BI (points 1, 2, 3, 6, 11, 16, 21, 26, 31, 36, 39, 40, 41)

Instrumented Trench CI

To monitor the long-term performance of Trench CI, FWD testing and elevation surveys were conducted on the site. The first FWD test on Trench CI was on November 5, 2007. This was 44 days after the construction of the trench.

FWD Test Results

Falling weight deflectometer tests were performed on the trenches on November 5, 2007; June 28, 2008; and March 13, 2009. Forces of 6 kips, 9 kips, 12 kips, and 15 kips were applied to each test point along the trenches. The deflections of the pavement were measured at eight points across each trench. The locations of the test points are shown in Figure 5.44 for Trench CI.

Figure 5.45 shows the deflection at 15 kip for the test points for Trench CI for all test dates. In the trenches, the deflections were smaller than the deflections in the surrounding soil. This indicates that backfill was stiffer than the surrounding soils. The sewer prevented the test from being extended on the south side of the trench.

At the edge of the trench were FWD testing points 20 and 22. At FWD testing point 20, the deflection was 17.0 mils. Near this point were field-testing points 2 and 3. The average dry unit weight was 115.2 pcf at a moisture content of 8.9%. The CBR value at the surface was 4%, and the average CBR value for the top two feet was 12%. At FWD test point 22, the deflection from the 15-kip load was 18.8 mils. Near this point were FWD testing points 2 and 3 (see Figure 5.44). The average dry unit weight was 113.6 pcf at a moisture content of 8.3%. The CBR value at the surface was 4%, and the average was 14%.

In the T-section of the trench were FWD testing points 19 and 23. At FWD test point 19, the deflection from the 15-kip load was 19.1 mils. Field test point 5 was near this location. The dry unit weight was 119.9 pcf at a moisture content of 7.2%. The CBR value at the surface was 5%, and the average CBR value was 7%. At FWD test point 23, the deflection was 26.1 mils. Near this location was test point 6, where the dry unit weight was 119.2 pcf at a moisture content of 8.5%. The CBR value at the surface was 1%, and the average CBR value was 7%.

The lower CBR values and the dry unit weights measured during construction corresponded to higher deflections from the FWD testing. The zone of influence was present on both sides of the trench. The zone of influence extended 1 foot beyond the trench.

Figure 5.45 shows the settlement between the two surveys, with the FWD 15-kip test results. The south edge of the trench settled evenly, while the north edge of the trench did not settle evenly. These settlement values do not correspond with in-field measurements of dry unit weights and moisture contents. This shows on the south edge of the trench in the zone of influence, where there were larger settlements.

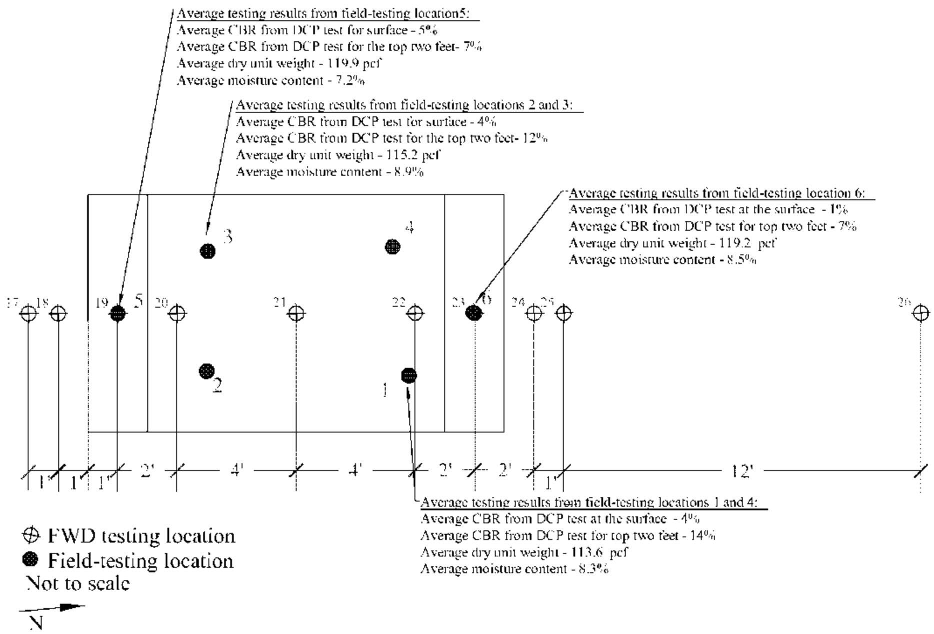


Figure 5.44. Location of FWD testing locations for Trench CI

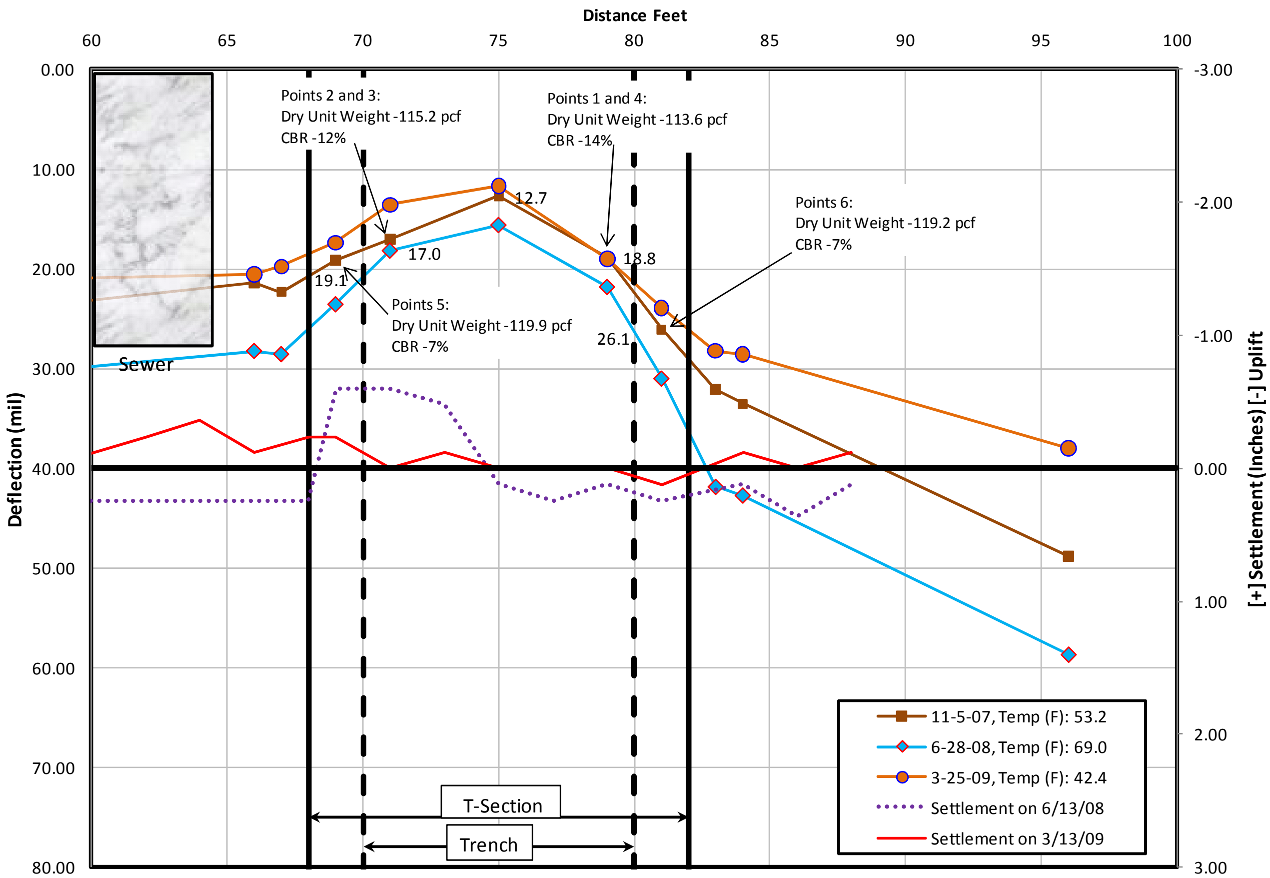


Figure 5.45. Displacement of Trench CI with the 15-kip load superimposed

Post-Construction Elevation Survey

Trench CI was surveyed March 19, 2008; June 13, 2008; August 19, 2008; November 18, 2008; and March 13, 2009. The site survey contained 41 grid points. Figure 5.46 shows the location of the grid points across the trench. On March 19, 2008, the difference between the maximum and minimum elevation was 0.22 inches.

On June 13, 2008, the difference between the maximum and minimum elevation was 0.8 inches. The difference between the maximum and minimum elevation changed between the two surveys. This showed that the trench did not behave in a uniform manner and differential movement did occur. The average uplift between the two surveys was 0.34 inches, and the maximum uplift was 0.60 inches (survey point 9). The pavement across the trench sloped from north to south and from the center of the road to the curb (see Figure 5.47).

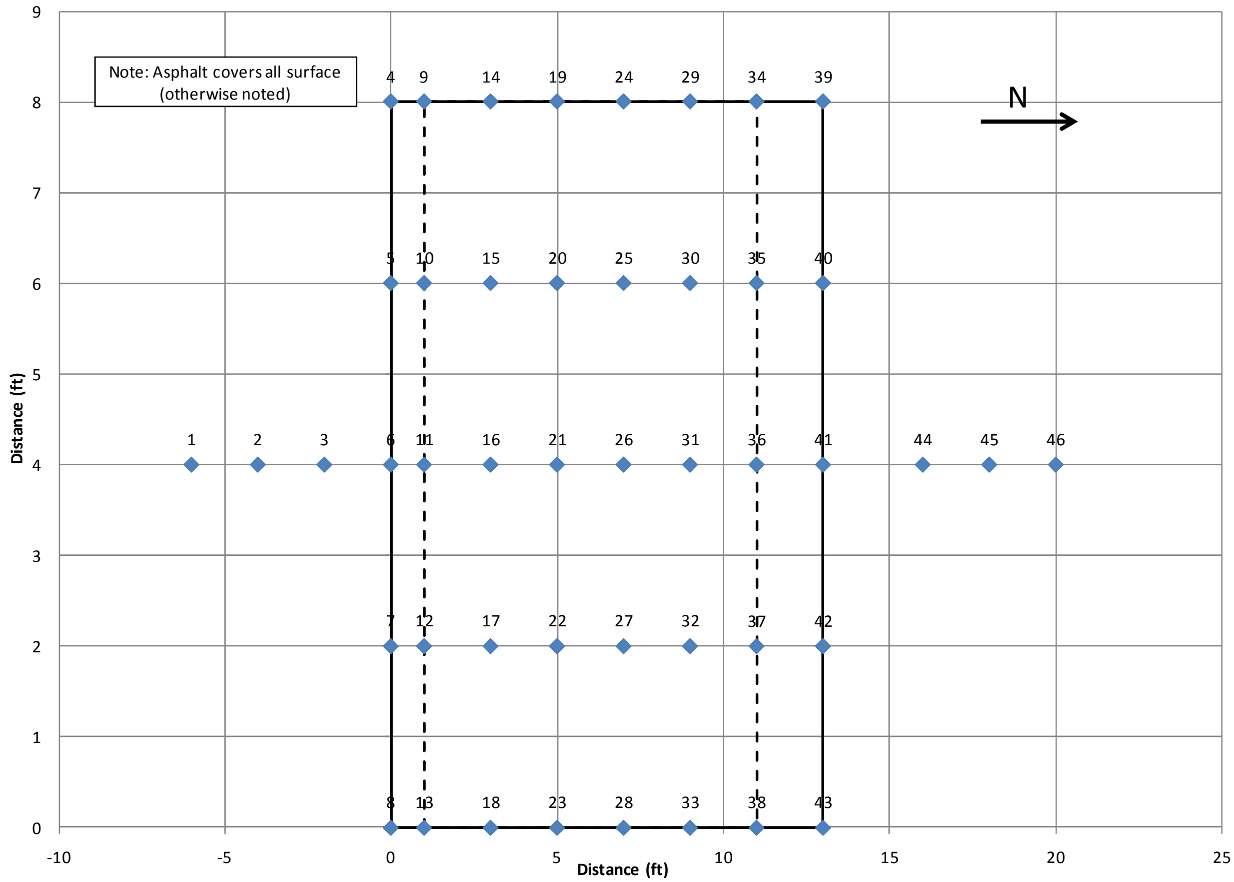


Figure 5.46. Survey locations for Trench CI

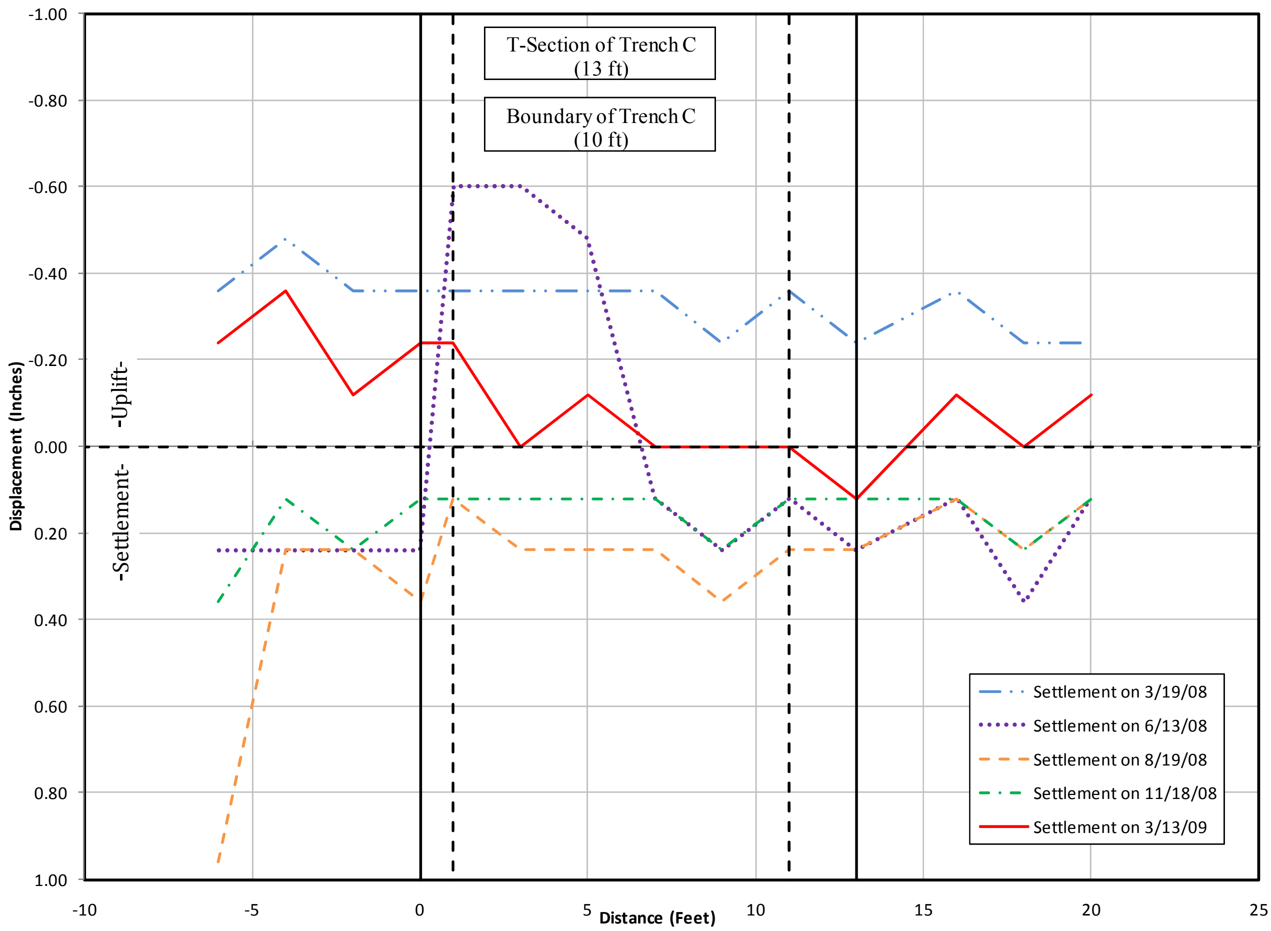


Figure 5.47. Settlement along Trench CI (points 1, 2, 3, 6, 11, 16, 21, 26, 31, 36, 41, 44, 45, 46)

Comparison of the Trenches

The backfill used in the trenches was classified as SP-SM (poorly graded sand with silt). According to Table 2.5, the typical CBR values for this backfill ranged from 10% to 40%. The typical dry unit weights for compacted backfill were 100 pcf to 120 pcf. The optimum moisture content for compacted backfill was 12% to 20% according to Table 2.5.

The top lift of Trench AI was placed at 4% relative density and was in the bulking moisture content. The average CBR value was 14%. The top lift of Trench BI was placed at 39% relative density and in the upper range of the bulking moisture content. The average CBR value was 13%. The top lift of Trench CI was placed at 49% relative density and was at the upper end of the bulking moisture content. The average CBR value was 11%.

The backfill in Trench AI was at or below the bulking density of the backfill. The backfill for Trench BI and Trench CI for all lifts was above the bulking moisture content obtained in the relative density testing. The results from the field testing are plotted in Figure 5.48. This figure shows that the trench constructed using the City of Ames standard method without moisture control yielded lifts with lower densities and moisture contents.

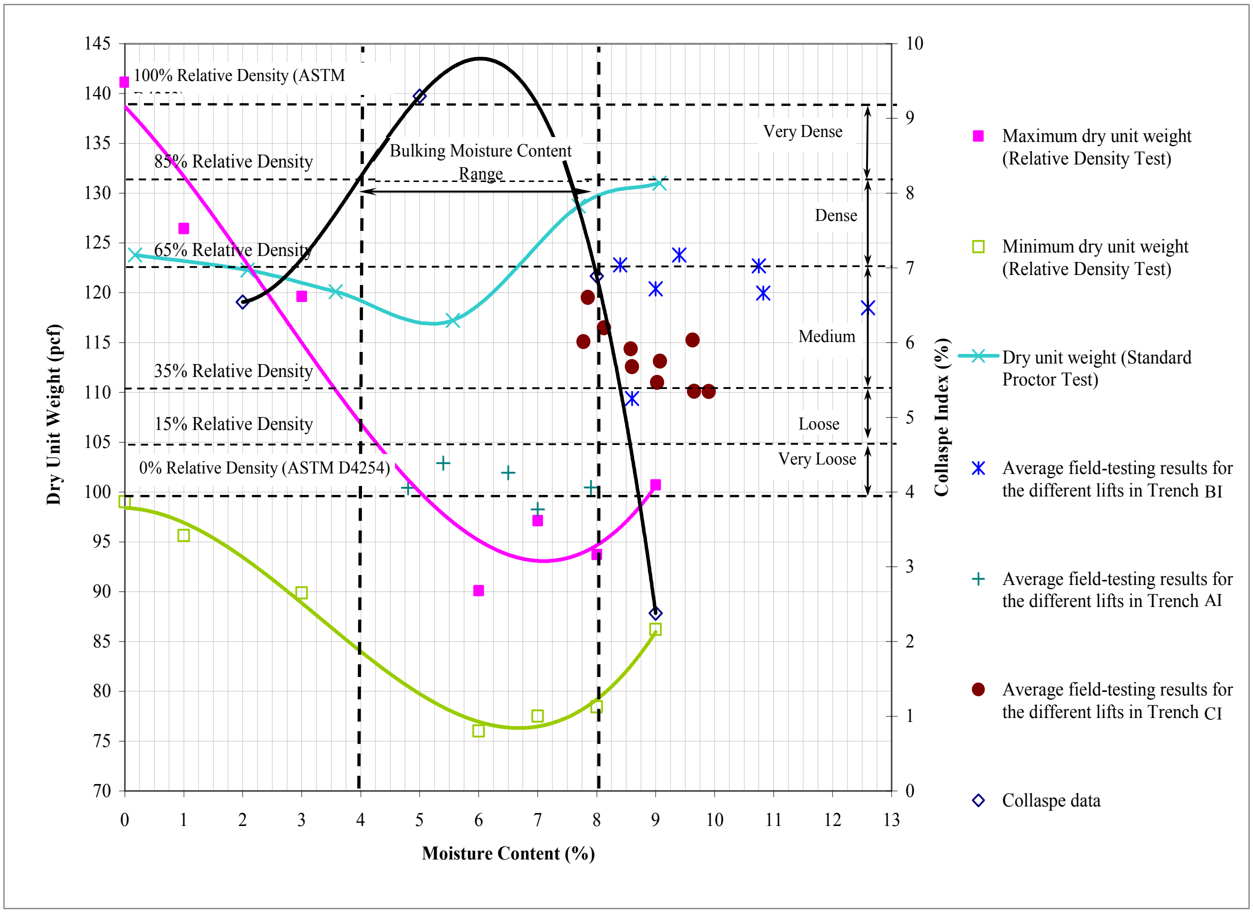


Figure 5.48. Results from relative tests with the average nuclear density testing results for the three instrumented trenches

Figures 5.49, 5.50, and 5.51 show FWD test results from three test days across all three trenches. Figure 5.52 shows a comparison of the trenches with a 15-kip load over the three test dates. Displacement of two surveys is also superimposed. The figure shows that Trench CI provided the stiffest (smallest deflections) response to the FWD test. Trench CI was placed at the highest relative density during compaction; however, the CBR values from Trench CI were lower than in the other trenches. Outside of Trenches AI and BI, the deflections are larger than the deflections outside of Trench CI. However, at the test point further from the trenches, the subgrade had similar high deflections from the FWD testing. This shows the effect of the zone of influence was minimal using the T-section construction. A manhole located between Trenches AI and CI did not allow the research team to conduct FWD testing to measure the deflection of the subgrade away from the trench.

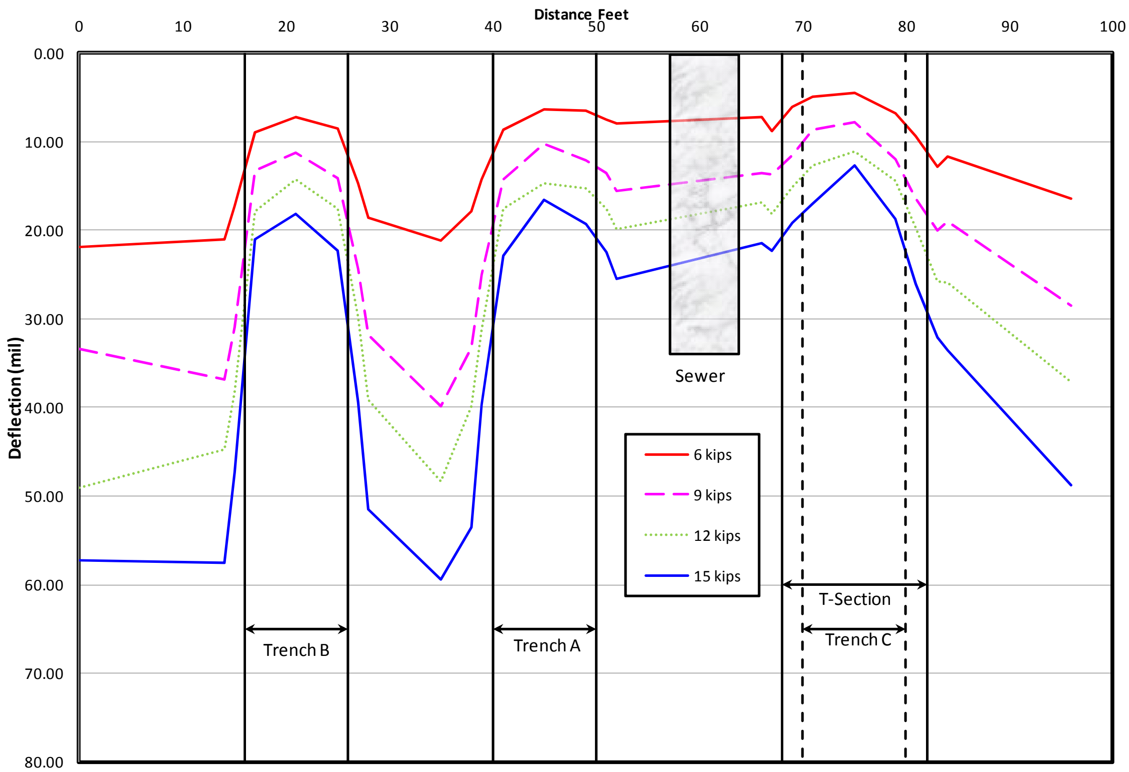


Figure 5.49. Falling weight deflectometer testing for Trenches BI, AI, and CI on November 5, 2007, with a temperature of 53.2ºF

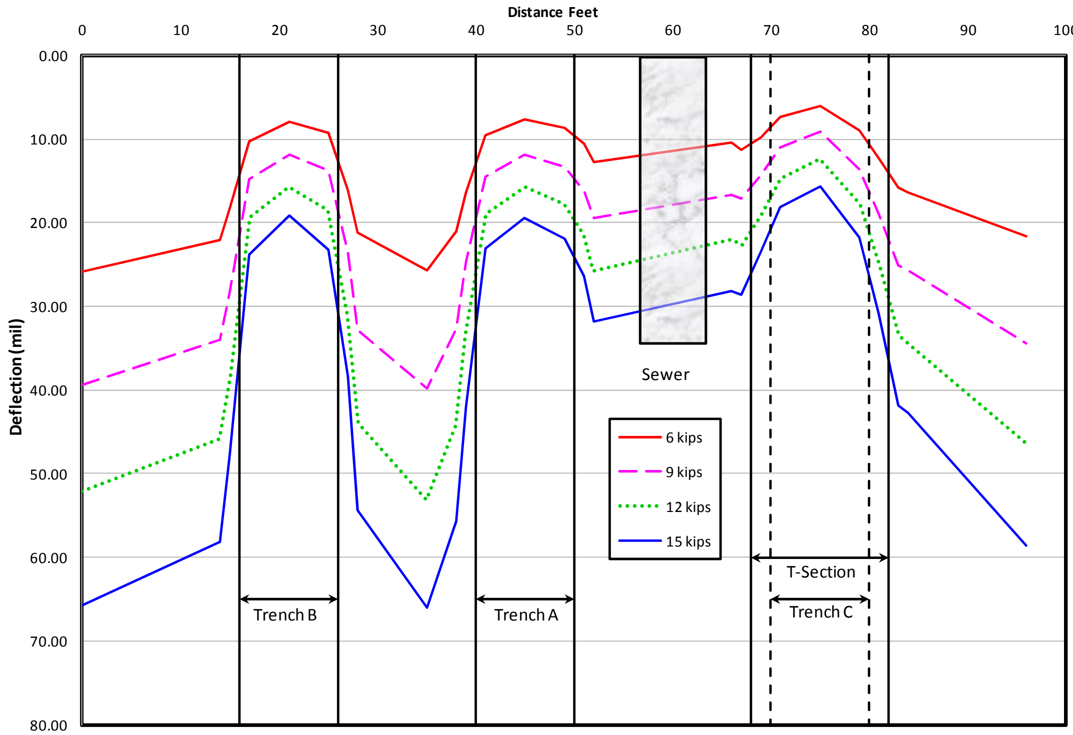


Figure 5.50. Falling weight deflectometer testing for Trenches BI, AI, and CI on June 6, 2008, with a temperature of 69.0ºF

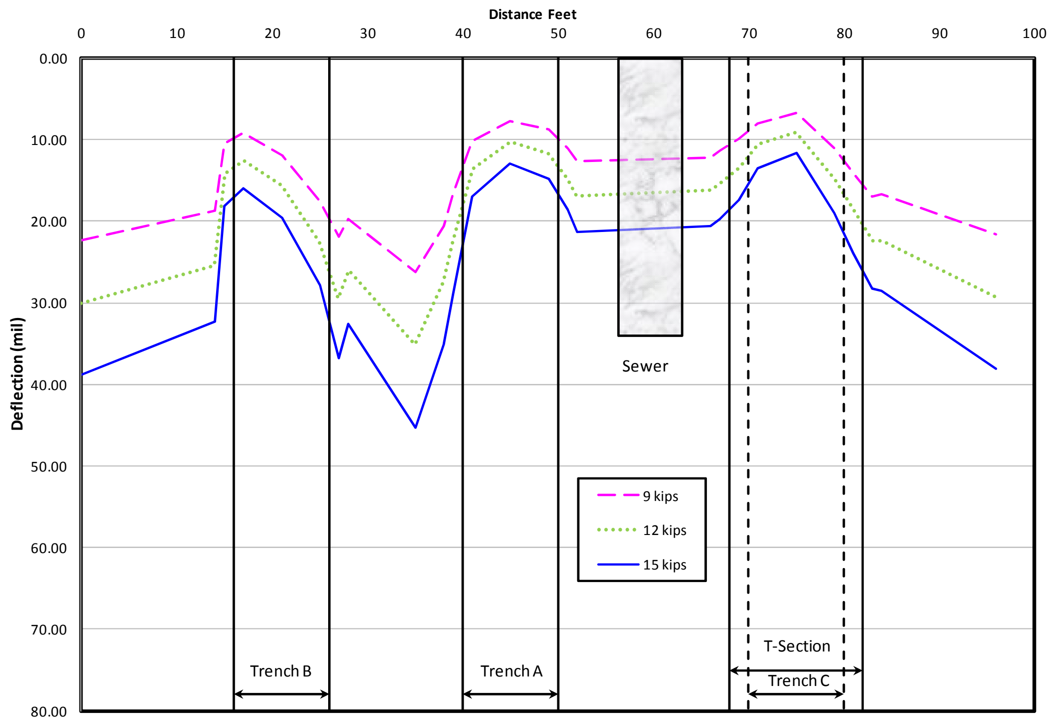


Figure 5.51. Falling weight deflectometer testing for Trenches BI, AI, and CI on March 25, 2009, with a temperature of 42.4ºF

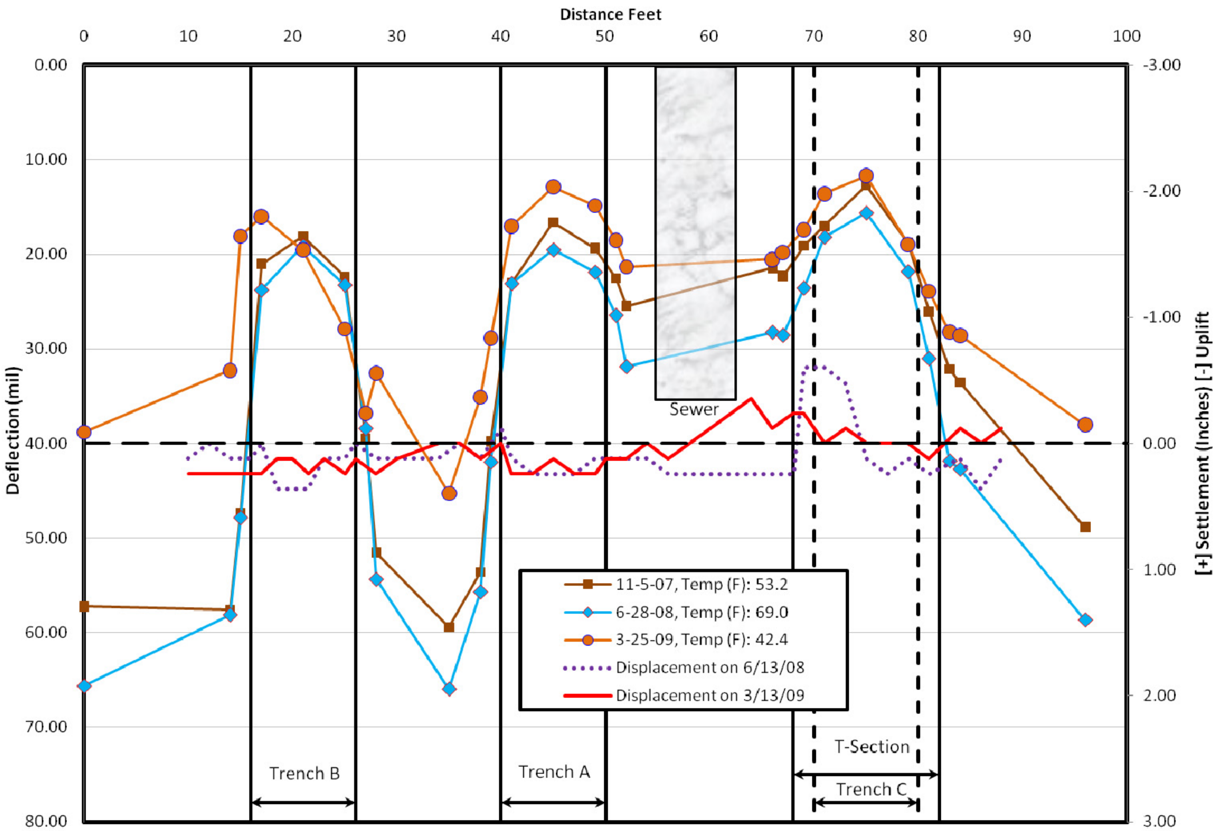


Figure 5.52. Overall FWD at 15-kip comparison with displacement superimposed

Instrumentation Results

The instrumented trenches on Kellogg Avenue were monitored from November 2007 to May 2009. The performance of the trenches was monitored with instrumentation.

Figures 5.53, 5.54, and 5.55 show the cross-sections of the trenches with the instrumentation installed on each layer.

An additional extensometer and moisture sensor were installed at the base of Trench AI; however, they were not working during monitoring for Trench AI. In Trench BI, there was an additional pressure cell installed on the side of the trench on the top lift. An extra pressure cell and moisture sensor were installed on the bottom of the T-section of Trench CI.

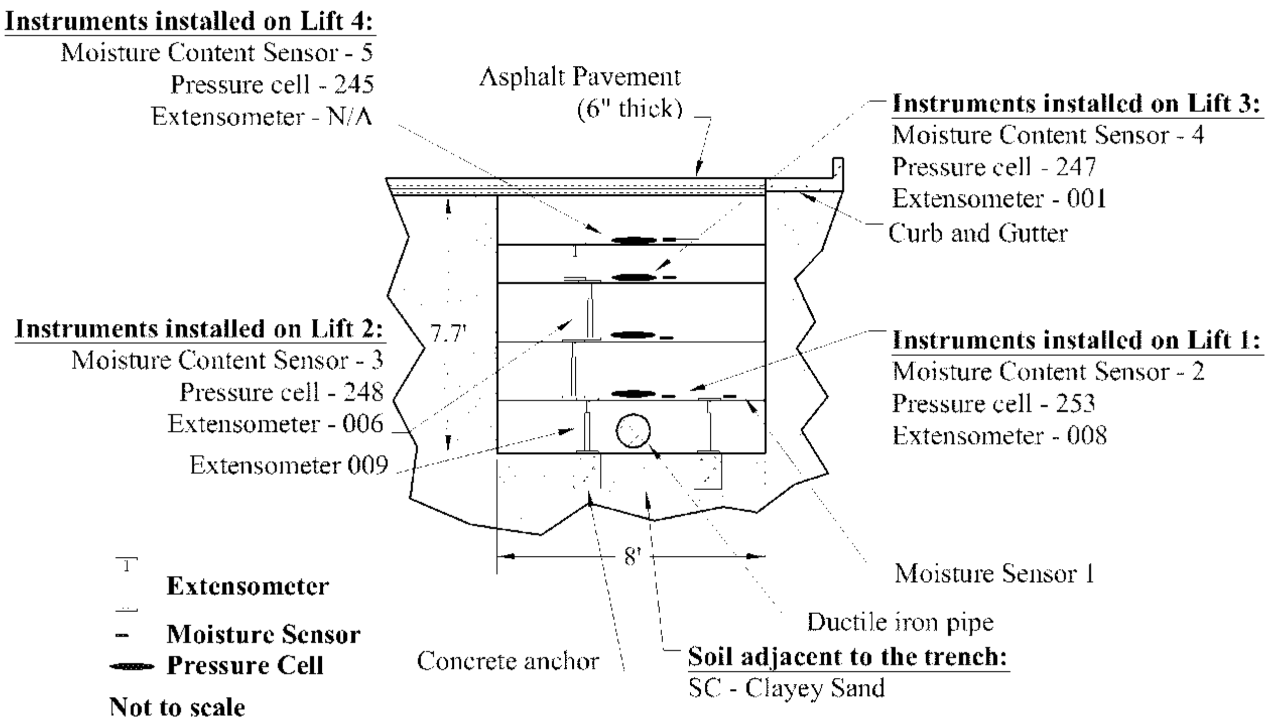


Figure 5.53. Cross-section of Trench AI showing the locations of the instrumentation

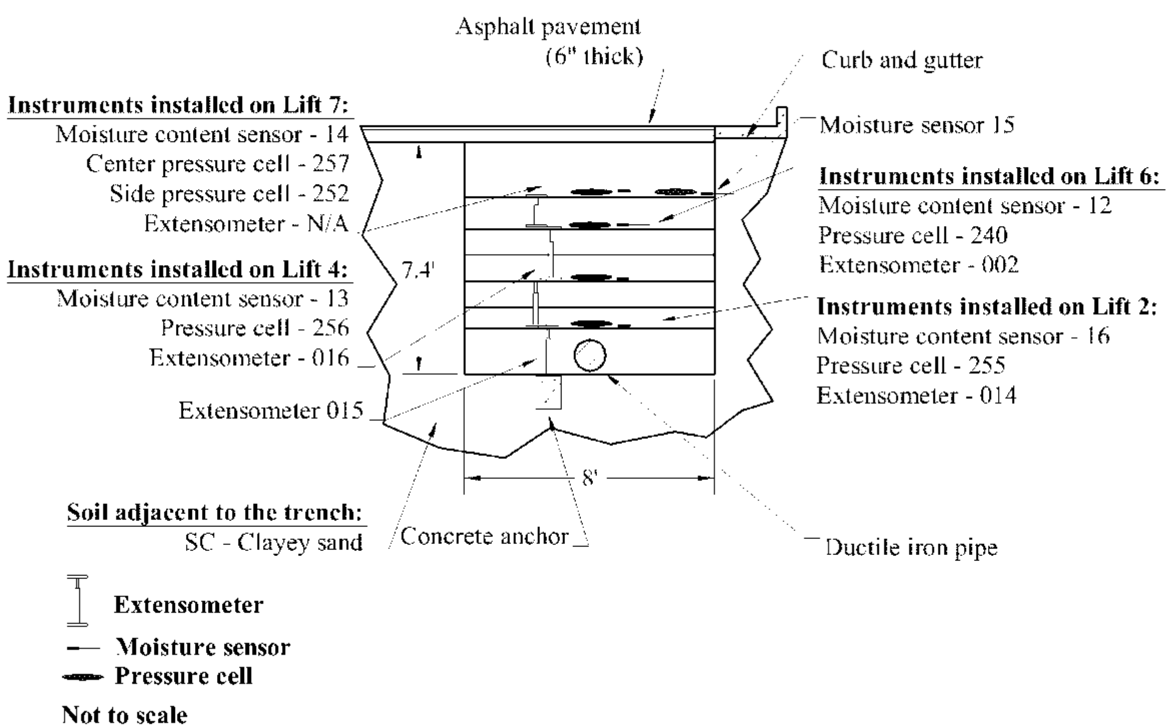


Figure 5.54. Cross-section of Trench BI showing the locations of the instrumentation

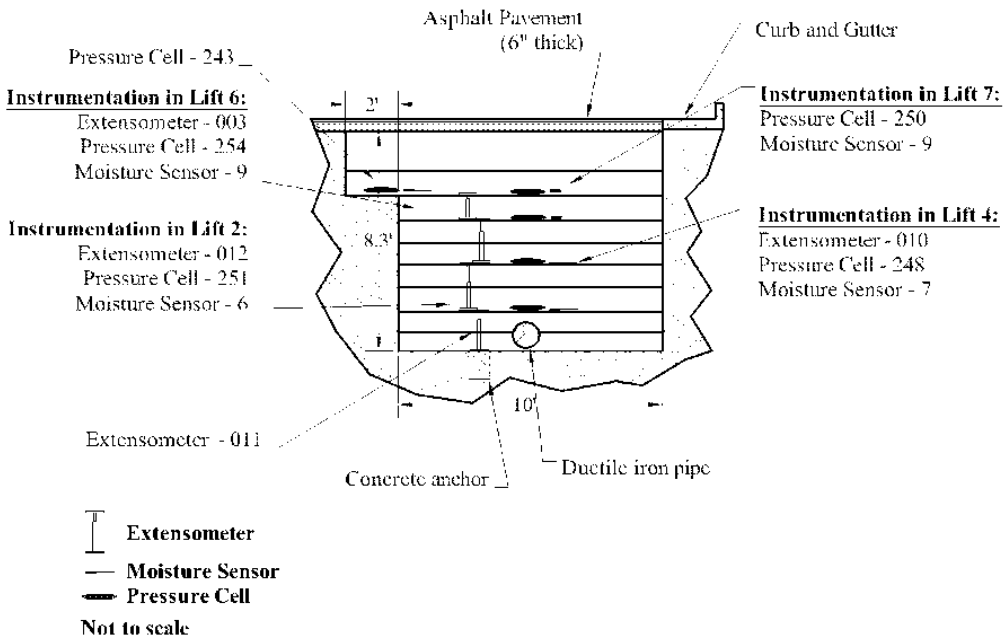


Figure 5.55. Cross-section of Trench CI showing the locations of instrumentation

All instrument readings began on November 4, 2007, except for the extensometers, which began on September 25, 2007. The delay from the time of installation to the beginning of data collection occurred because of several factors, including missing multiplexers, wiring difficulties, and inappropriate code to scan all equipment.

The flat line gap in the instrumentation data from December 7, 2007, to December 13, 2007, occurred because the data on December 20, 2007, was downloaded and saved directly over the file from the previous week. The gap in the March data was the result of the same problem.

To compare temperature and pressure as a function of depth, twelve dates were selected for comparison. Table 5.15 shows the justification for the selection of each date.

Table 5.15. Dates used for comparing temperatures and pressures as a function of depth

Date	Justification
11/14/2007	Construction was completed, all instruments were providing read-outs, and no freezing had occurred in the trench.
12/25/2007	The ambient air temperature was below freezing, and temperatures were measured in the soil adjacent to the trench. The temperatures in the trenches were just above freezing.
01/25/2008	The temperatures and pressures in the trench reached the lowest values.
02/10/2008	Temperatures in the trench briefly increased towards freezing and there was an increase in pressures in the trench.
02/21/2008	Temperatures in the trench began to decrease again and pressures decreased in all the trenches.
03/10/2008	Temperatures in the trench began to rise above freezing, and the pressures in the trench were increasing.
05/14/2008	All temperatures measured in the trench were above freezing, and the pressures had stabilized.
08/05/2008	Temperatures in trenches reached maximum.
12/22/2008	The ambient air temperature was below freezing, temperatures were measured in the soil adjacent to the trench, and the temperatures in the trenches were just above freezing.
01/16/2009	The temperatures and pressures in the trench reached the lowest values.
03/04/2009	Temperatures in the trench began to rise above freezing, and the pressures in the trench were increasing.
05/08/2009	All temperatures measured in the trench were above freezing, and the pressures had stabilized.

Temperature Readings in Boring 1

Figure 5.56 shows the readout from the temperature sensors installed in Boring 1 located north of Trench CI (see Figure 5.13). This figure illustrates the delay between the changes in average ambient air temperature versus the temperature in the soil. The upper region of soil experienced larger changes in temperature in response to changes in the ambient air temperature than soils in deeper regions of the trench. The temperature in the soil adjacent to the trench decreased until late February and then began to increase again. Freezing temperatures were measured to a depth of 2.45 feet during the winter of 2007/2008. The uppermost stratum of soil experienced freezing temperatures from December16, 2007, to March 13, 2008, to a depth of 2.45 feet, and January 27, 2008, to January 30, 2008, to a depth of 2.45 feet. During spring warming, the upper layers of soil responded sooner to rising air temperatures than deeper regions of the trench. The deeper soils in the trench experienced smaller changes in temperature over the winter season. The winter of 2008/2009 experienced almost identical results, as shown by the Figure 5.56.

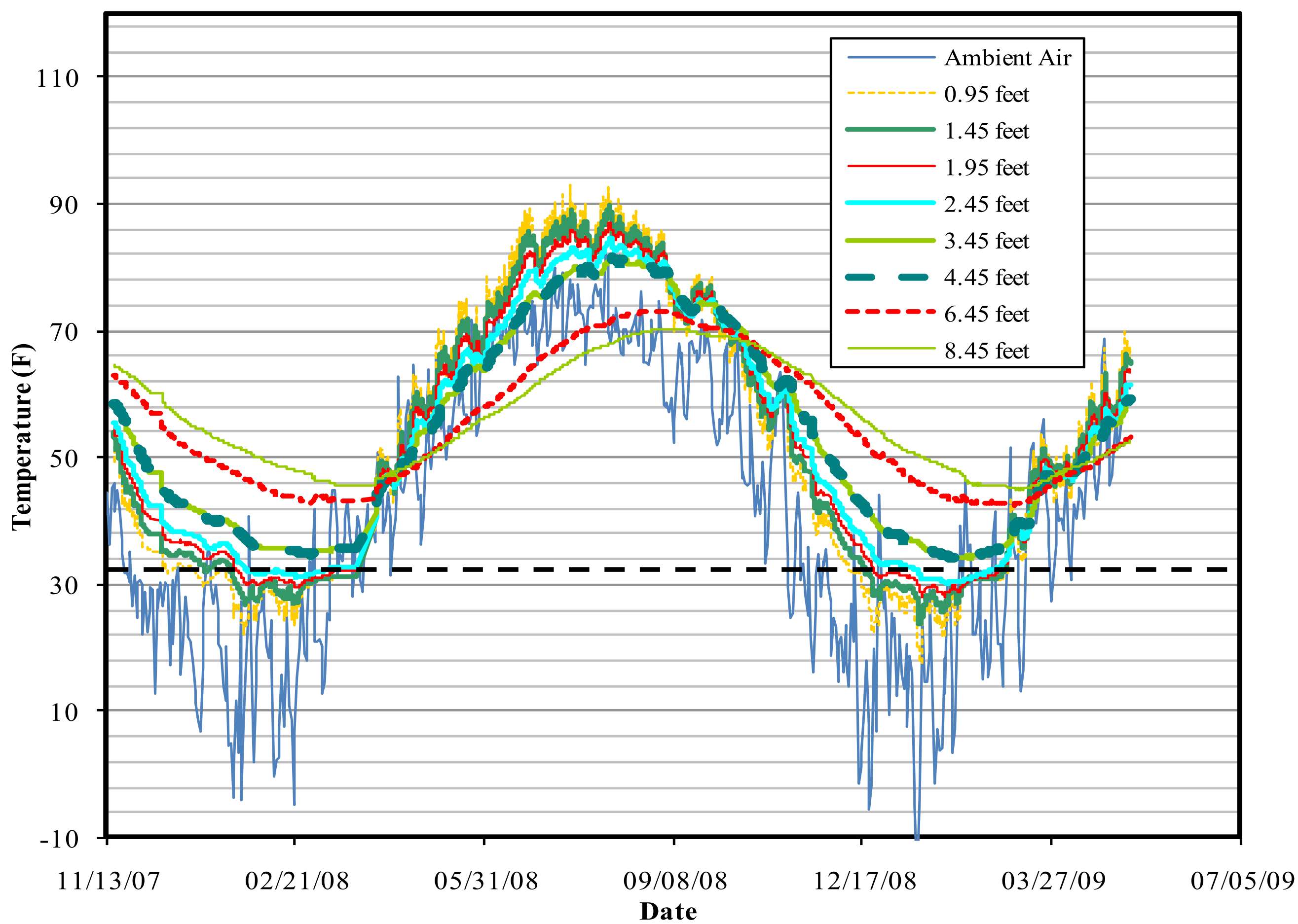


Figure 5.56. Variation of temperature over time at various depths below pavement surface

Figure 5.57 shows the temperature profiles for the twelve selected dates from Table 5.15. The profiles of the temperatures varied linearly through the profile as the ambient air temperature decreased until January 30, 2008. After January 30, 2008, the upper layer of the trench was affected by the increases in ambient air temperatures. During this time, the lower region of the soil profile continued to decrease in temperature. Notice that for the summer dates (5/14/08, 8/05/08, and 5/08/09), the temperature profiles have a small range.

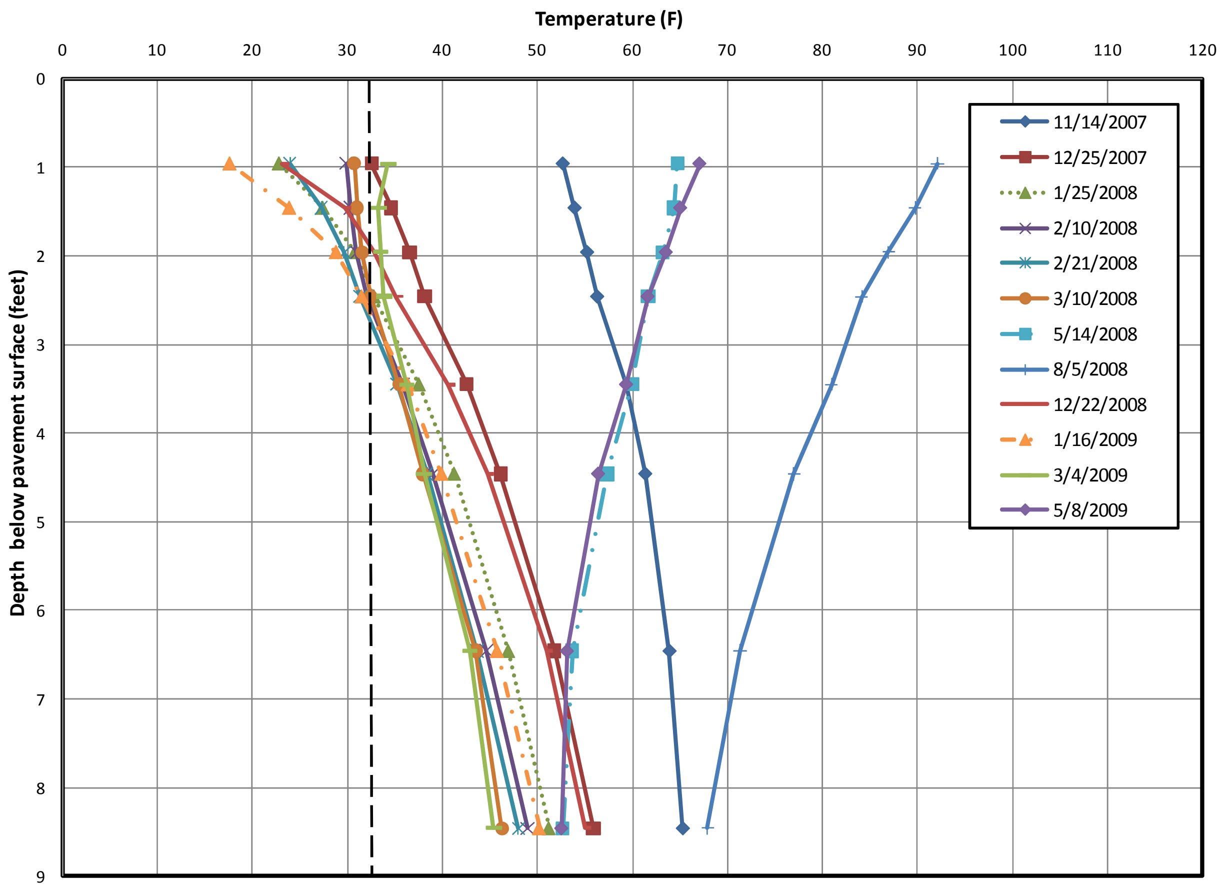


Figure 5.57. Temperature profiles in subgrade soil adjacent to trenches

Instrumented Trench AI

The temperature readings for Trench AI are plotted in Figure 5.58. Figure 5.59 shows the temperature profiles in Trench AI for the dates selected in Table 5.15. The temperatures in the trench followed a pattern similar to the temperature sensors in the soil adjacent to the trench. The figure shows that freezing temperatures in the trench were to a depth of 2.7 feet from January 25, 2008, to February 5, 2008, and from February 29, 2008, to March 13, 2008. Freezing temperatures were also at a depth of 1.6 feet in the trench on January 4, 2008, and March 13, 2008. The trench experienced freezing temperatures to a depth greater than the subgrade soil adjacent to the trench. The thermal conductivity of this trench was higher because of the voids in the backfill.

Figure 5.60 plots the pressure readings for Trench AI. This figure shows that the top pressure cell did not experience decreased and increased pressure, as did the upper pressure cells in Trench BI. However, the pressure cell placed at a depth of about 2.7 feet followed a similar pattern as Trench BI. In Trench AI, the backfill was placed at about 4% relative density. The lower relative density correlates to larger voids in the backfill.

The larger voids caused two things to happen. First, the larger voids in the backfill allowed the ice lens to form without displacing the backfill or causing it to stiffen. Second, the larger voids prevented water from moving to the frozen fringe. These two factors caused the upper lift of the trench to have relative constant pressures through the winter. The constant pressure was contrary to Moser (1990) about pressures in a frozen trench. However, the soil was frozen or just above the frozen fringe during most of the winter. Once the region was above the frozen fringe, no ice lens could form. The pressure cell was then encapsulated by frozen soils. As the frozen fringe migrated downward, there was sufficient void space that the expansion of the soil below did not cause the pressure to increase at the top. This was also reflected in the lack of movement in the upper extensometer.

The pressure cell at elevation 2.7 feet below the surface did experience the fluctuation in pressure like those seen in Trench BI. This was caused by the temperature in this region being at or just below the freezing point for several weeks. When the temperature is close to freezing, more ice lens will form. Because the fringe remained below the pressure cell at 1.6 feet but above or at the pressure cell at 2.7 feet for several weeks, larger ice lens in this region could have formed (see Figure 5.61). This resulted in the compacted soil between 1.6 feet and 2.7 feet to become stiffer than the backfill above it and the backfill covering the instrumentation. The pressure cells at 4.5 feet and 6.2 feet also experienced a decrease in pressure because of the frozen fringe being below the uppermost pressure cell. The frozen fringe was in the region of the pressure cell at 2.7 feet. The pressure fluctuations at a depth of 2.7 feet can be seen in Figure 5.62.

Figure 5.63 shows the moisture contents at various depths in Trench AI. Similar to Trench BI, the moisture contents measured during the field testing were within the error bars of the moisture sensors at depths of 1.6 feet and 2.7 feet. The lowest moisture sensors had a higher reading than the field-testing point because the backfill was free draining and the surrounding soil was cohesive and had a lower permeability rate, so the moisture pooled at the bottom of the trench. The moisture sensor at the bottom of the trench was above 68% saturation for the duration of monitoring. This indicated that water was freely flowing to the bottom of the trench. During the spring, the moisture sensors recorded rapid fluctuation in the moisture content of the soil. These fluctuations in moisture content correlated to precipitation events in Ames, Iowa.

Data from Trenches BI and AI did not indicate similar fluctuations during the spring. The moisture sensors at depths of 2.7 feet and the moisture sensor at a depth of 1.6 feet did not have these fluctuations in moisture content associated with precipitation. It was hypothesized that the drain tiles that were broken and patched with packaged concrete were acting as conduits to draw water directly into the trench at depths of 2.7 feet and 4.5 feet.

The settlement profiles and measurements are shown in Figures 5.64 and 5.65. The maximum settlement in the trench was 0.67 inches. This settlement occurred after rain infiltrated the trench from around the temporary patch. During the winter as the frozen fringe penetrated to depths of between 1.6 feet and 2.7 feet, the extensometer spanning these depths experienced uplift. This showed that an ice lens was forming between the two pressure cells. The net movement upward from the frost heave was not sufficient to cause the trench to be in an uplifted state. The elevation survey measured uplift at this trench. However, the extensometers did not account for the movements of the uppermost lift of backfill.

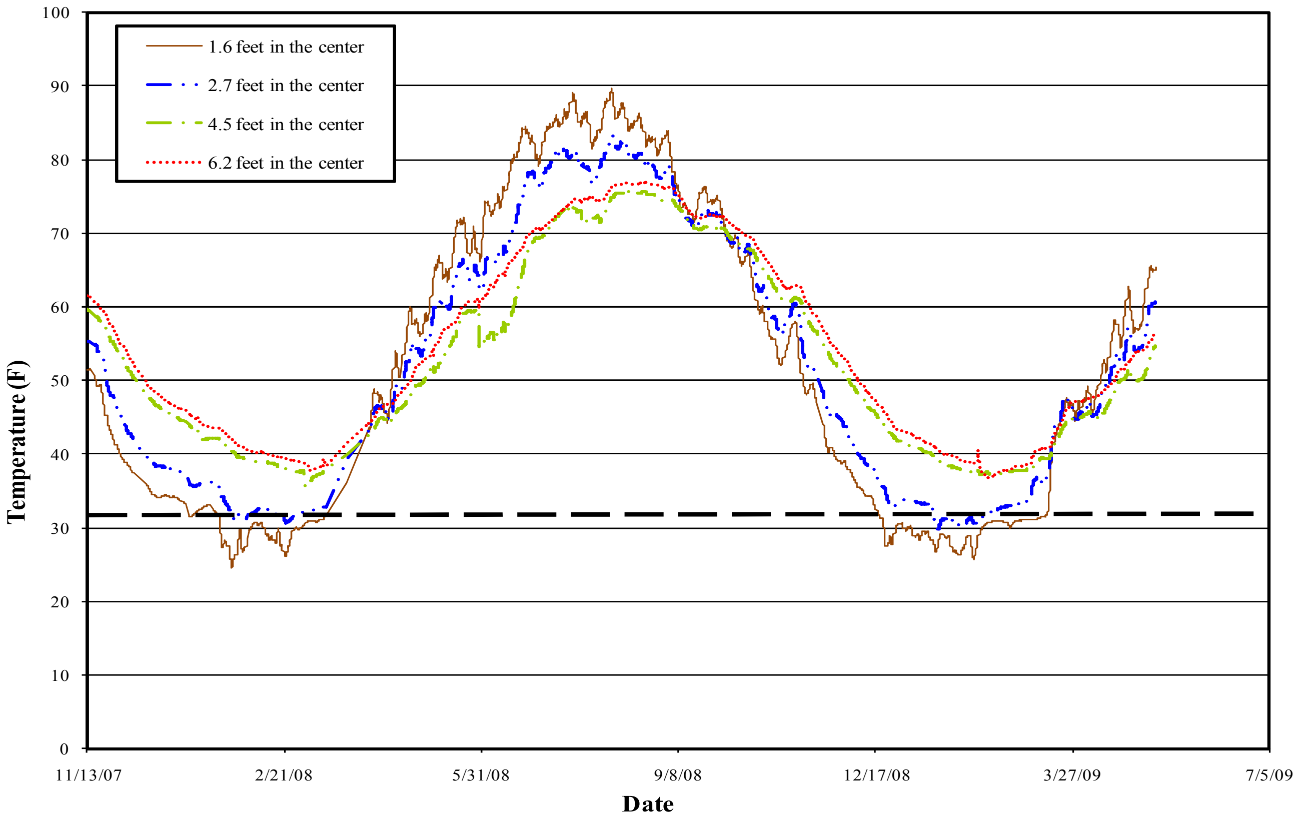


Figure 5.58. Temperature in Trench AI

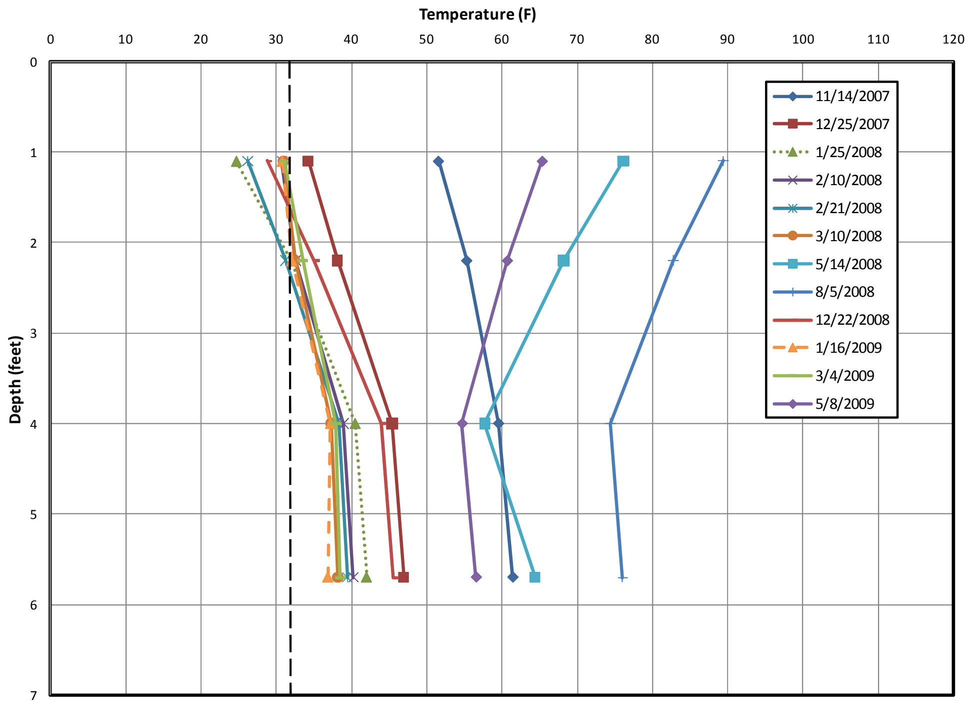


Figure 5.59. Temperature profiles for selected dates for Trench AI

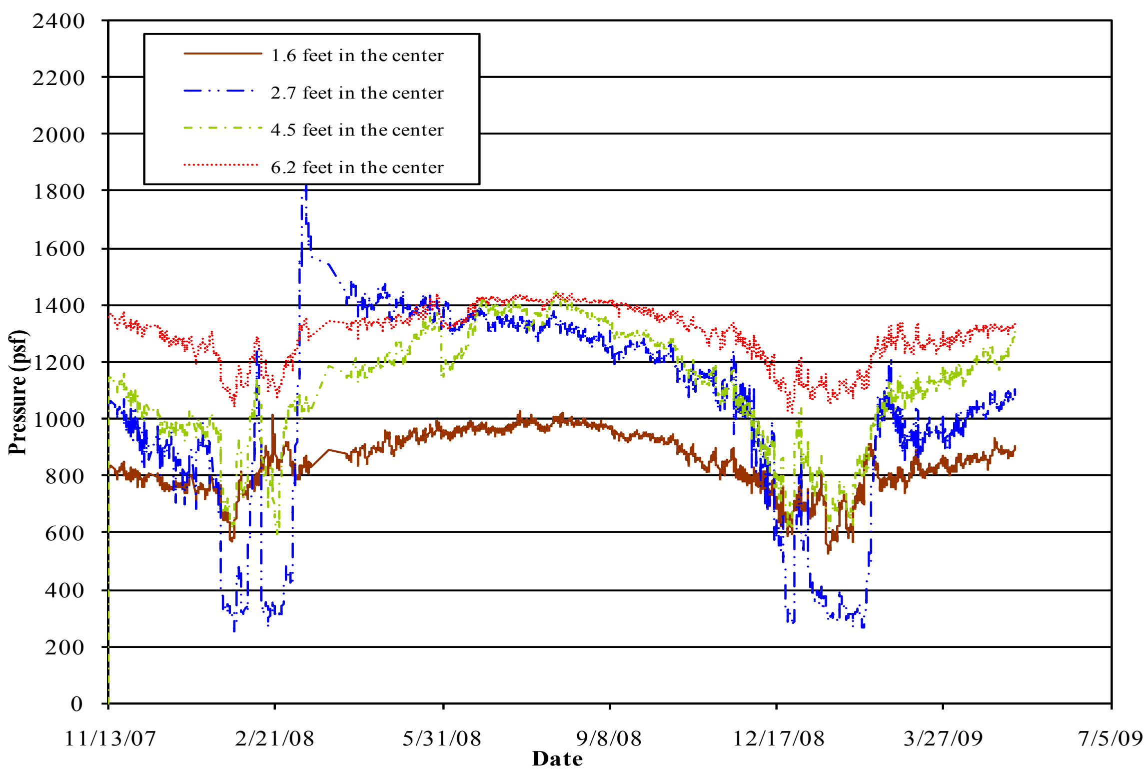


Figure 5.60. Pressures measured in Trench AI

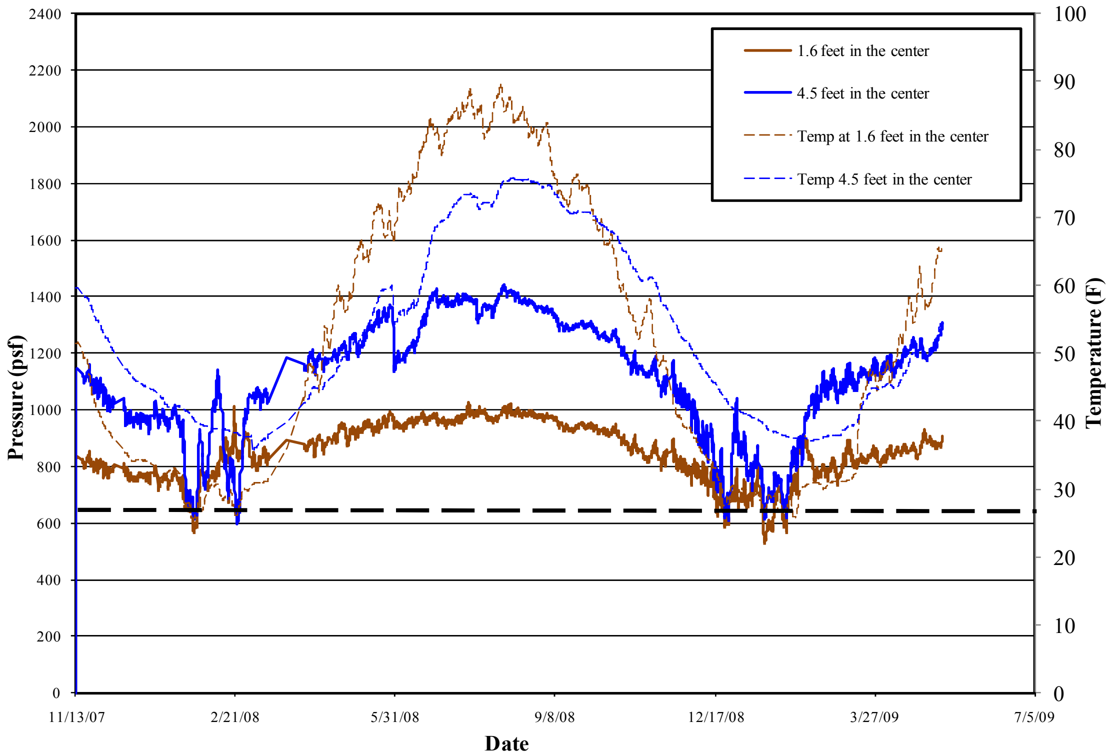


Figure 5.61. Comparison of temperatures and pressures in the upper two lifts of Trench AI

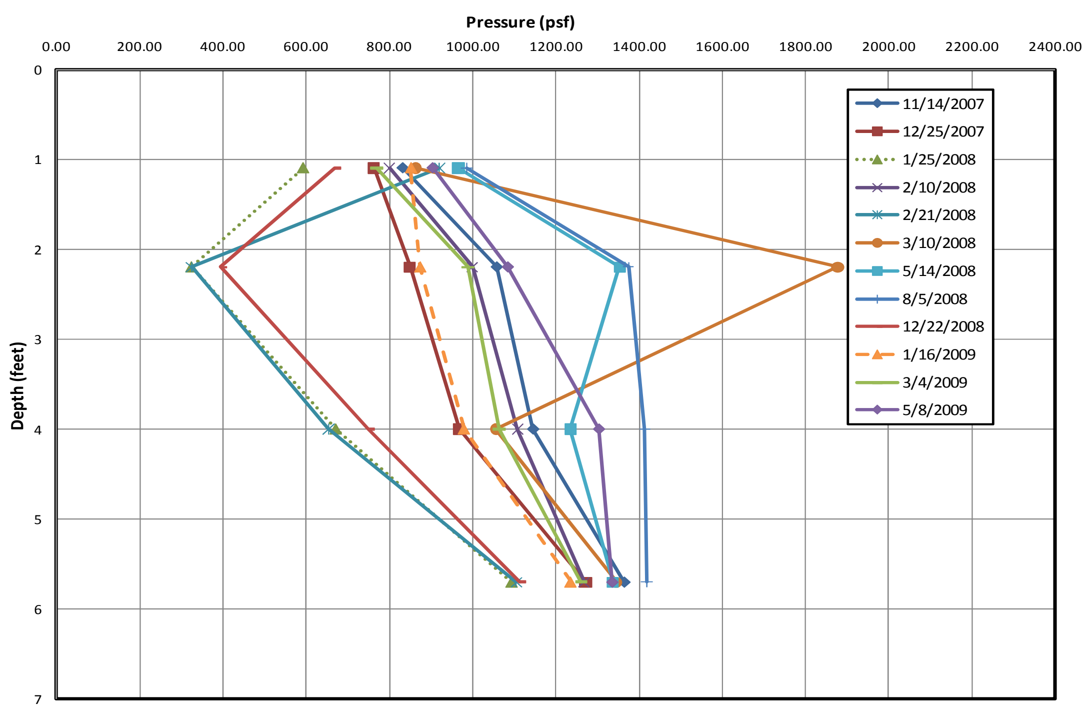


Figure 5.62. Pressure profiles for selected dates for Trench AI

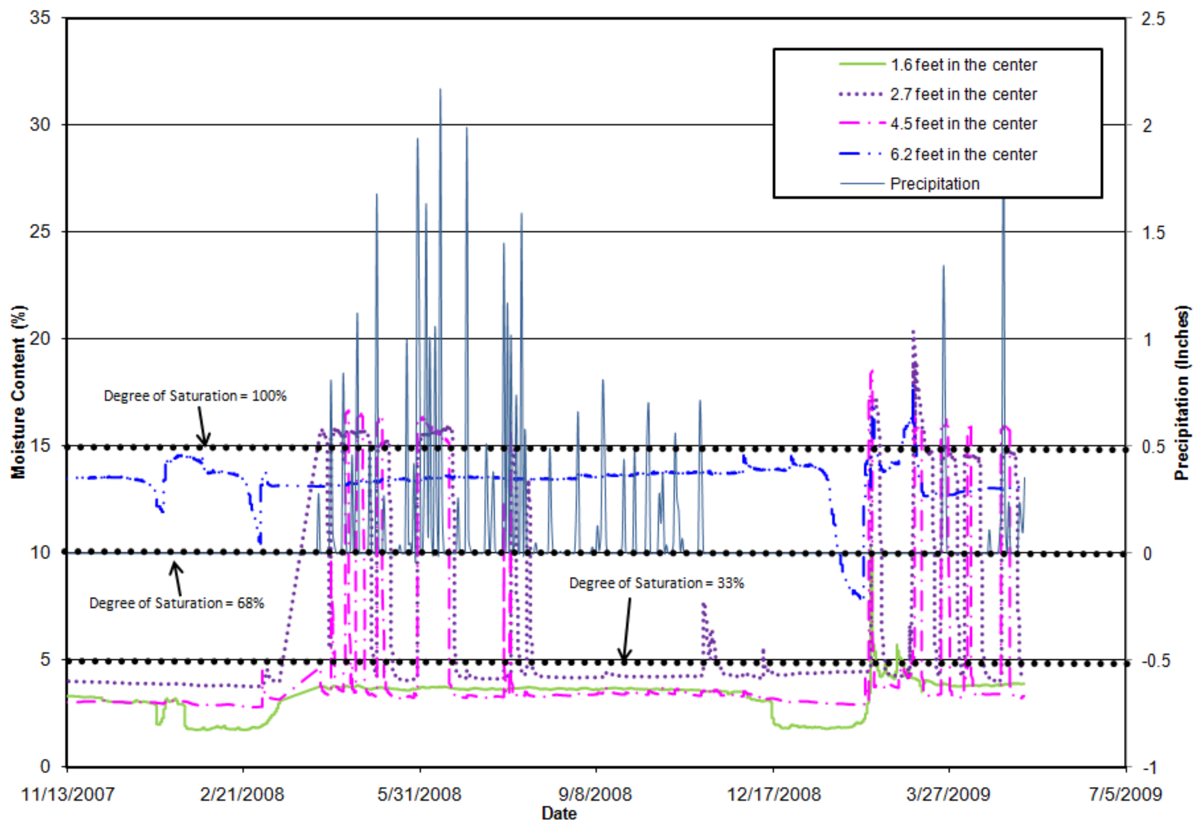


Figure 5.63. Moisture content in Trench AI

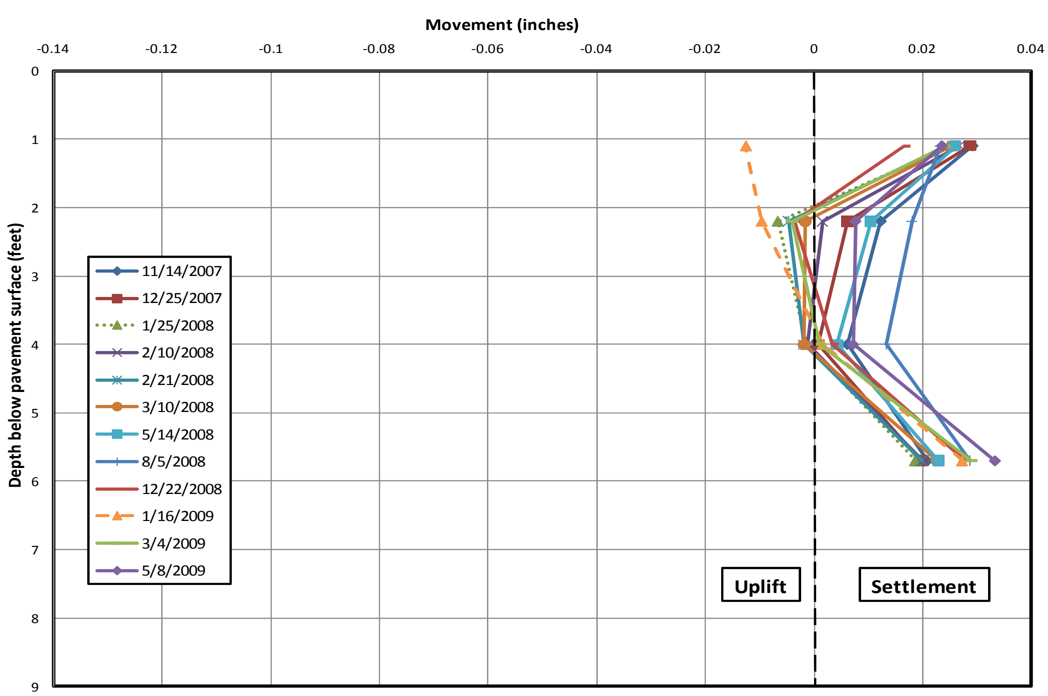


Figure 5.64. Settlement profiles for selected dates for Trench AI

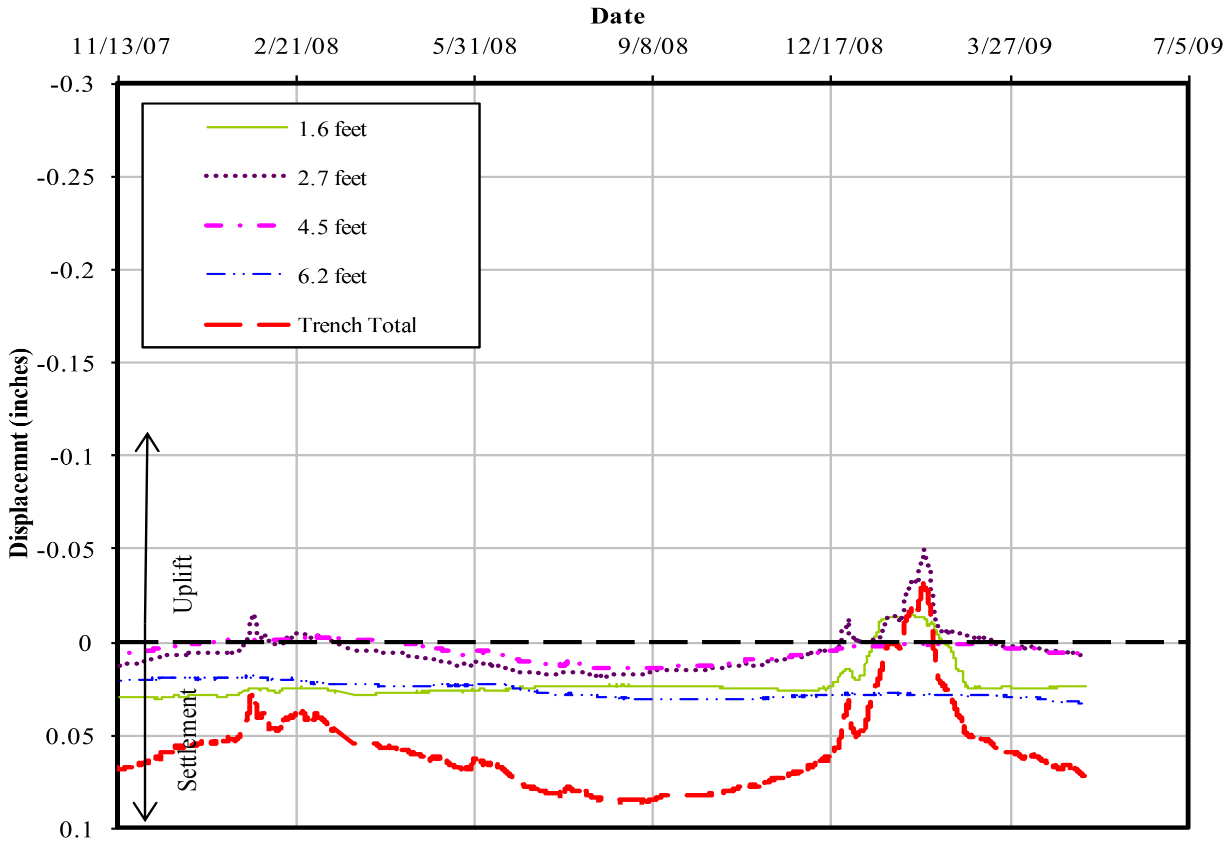


Figure 5.65. Extensometer readings for Trench AI

Instrumented Trench BI

Temperature Measurements

The temperature readings from Trench BI are plotted in Figure 5.66. The temperatures in the trench follow a similar pattern as the temperature sensors in the soil adjacent to the trench. The figure shows freezing temperatures in the trench reached a depth of 1.6 feet (the top lift) on the side of the trench between January 1, 2008, and March 13, 2008, and to the same depth in the center of the trench from January 20, 2008, to March 3, 2008. The trench did not experience freezing temperatures to the same depth as measured in Boring 1. Soil temperature is a function of the soil's thermal conductivity. Granular materials have lower thermal conductivities than cohesive soils.

The temperature measured at the side of the trench was constantly lower by various magnitudes during the winter. This was the result of the granular soil at the side of the trench being adjacent to a heat sink (i.e., the cohesive soil adjacent to the trench). The colder cohesive soils adjacent to the trench caused a thermal gradient to occur between the center of the trench and the side of the trench.

The temperature measured at the bottom of the trench (at depths of about 6.1 feet and 4.4 feet) were similar. This was different than what would have been typically expected (linear varying temperatures with depth through the same soil media). The relative densities measured at these elevations in Trench BI were uniform. The pressure cell at a depth of 4.4 feet possibly did not provid accurate temperatures and pressure measurements.

Figure 5.67 shows the temperature profiles for the trench as a function of depth for the selected dates in Table 5.15. The temperature profiles in the trench show that as the ambient air temperature was decreasing, the backfill decreased in temperature uniformly until January 30, 2008. After January 30, 2008, the lower region of the trench continued to decrease steadily, while the upper region of the trench fluctuated with the ambient air temperature. This is similar to the trend observed in the soil adjacent to the trench. However, the temperature profiles were not linear. The nonlinearity of the profiles was caused by the pressure cell at a depth of 4.4 feet, which may not have been working properly. During the spring, the upper region of the trench experienced increased temperature and the temperature was above the temperature measured after installation on November 14, 2007.

Pressure Measurements

Figure 5.68 plots the pressure measurements in Trench BI. As expected, the highest pressure was at the bottom of the trench. At a depth of 4.4 feet, the pressure measurements were similar to measurements at depths of 2.7 feet. These readings were not expected because, typically, pressures vary linearly as a function of depth. However, the pressure sensor at a depth of about 4.4 feet may not have been functioning properly.

At a depth of 1.6 feet, a pressure cell was installed in the center of the trench and at the side of the trench. During the winter, the pressure cell on the side of the trench recorded pressures that were lower than pressures in the center of the trench. This is a result of friction between the backfill of the trench and surrounding soil.

During the winter, the difference in pressures between the side of the trench and the center of the trench was the result of adjacent soil freezing to a deeper depth than the backfill in the trench. Using linear interpolation in Figure 5.59, freezing temperatures were experienced to a depth of 2.5 feet in the adjacent soil and 1.9 feet in the trench. Because there was no FWD testing done during the winter, it is hypothesized that soil adjacent to the trench during the winter became stiffer than the backfill. This was the reverse of the stiffness measured during the fall 2007 FWD testing.

Figure 5.69 compares the pressures measured in the top two lifts of instrumentation with the temperatures measured by the pressure cells on the side of the trench and in the center of the trench. During the winter, all the pressure cells experienced a reduction in the pressures during freezing. The pressure cells were installed with a lens of clean sand. The clean sand, according to the literature review, was less susceptible to frost action than the 3/8-inch minus limestone. In addition to the sand lens, the 3/8-inch minus backfill was not compacted until 2 feet above the instrument. As shown by the CBR tests on the six recommended trenches (Chapter 4), when the lift thickness exceeded 12 inches, the compaction was no longer effective. When the noncompacted backfill was freezing, the larger void space allowed the ice lenses to form without displacing the surrounding particles. Because of the sand and the loosely placed backfill, the compacted 3/8-inch minus limestone surrounding the instruments pulled water up to the freezing fringe (shown in Figure 2.11) more effectively and formed ice lenses to a deeper depth than where the instruments were located. The result was that the backfill surrounding the instrumentation was stiffer than the backfill covering the instrumentation. As loads (overburden and heave) were applied using the beam on elastic foundation theory, the stiffer surrounding backfill carried the load. This accounts for the lower pressures experienced during freezing and until the first thaw on January 29, 2008 (date varies slightly by instrument).

In January, the pressure decreased as freezing temperatures penetrated deeper into the trench. However, from about January 29, 2008, to February 12, 2008, the pressure cells in the top two lifts experienced an increase in pressure. Figure 5.70 shows the pressure profiles for various dates selected in Table 5.15. During this period, the trench experienced freezing temperatures; however, for the top lift, the temperatures rose above freezing at the center of the trench and approached freezing along the side of the trench. The lower temperature measured at the side of the trench was a result of the soil adjacent to the trench being colder than the soil toward the middle of the trench. As the backfills above the pressure cell warmed and the ice lens receded, the stiffness of the backfill adjacent to the instruments decreased. As the stiffness of the backfill adjacent to and above the instrument became similar to the stiffness of the material surrounding the instrument, the pressure cells experienced pressures that were similar to when they were initially installed. In addition, during this time the pressure cell on the side of the trench did not experience the same magnitude of increase in pressure because the soil adjacent to the trench was still frozen, even though the frozen fringe was migrating upward.

Around February 12, 2008, the temperatures in the trench began to decrease again. As the temperature of the soil decreased, the frozen fringe migrated downward. In addition, the backfill surrounding the instruments became stiffer relative to the backfill above the pressure cells and the loads were distributed around the pressure cells.

These pressure cell results do not support Moser’s (1990) hypothesis that pressures on underground utilities can double during the winter. However, the method of installing the instruments could affect the pressure results.

Figure 5.71 shows the moisture sensor readings. Plotted on the chart are the nuclear density test results from the field testing during construction. The field-testing results are plotted as the first moisture content in the figure. The difference between the moisture sensors and the field measures was small and could also be affected by the nuclear density gauge’s accuracy and time delay from installation and the first reading (six days). The moisture sensor located in the soil adjacent to the trench recorded higher moisture contents than the moisture sensor in the center of the trench. This was expected because the soil adjacent to the trench was cohesive and therefore did not drain as quickly as the granular backfill. The moisture sensor at a depth of 6.1 feet had readings that were above 100% saturation. This indicates that water was freely flowing and pooling at the bottom of the trench.

After the first day of readings, the moisture sensors were offline until November 14, 2008. Valid readings continued until early to mid-December. During this time, the temperature fell below 50°F in the backfill and surrounding soils. This lasted until April. The data collected when the instruments were operating out of designed parameters were lighter than when there were data points. However, even though the instruments were operating out of design specifications, it can be seen that the moisture sensors in the top center (1.6 feet) of the trench and in the soil adjacent to the trench both showed moisture content decrease. The decrease in the moisture content was the result of frost formation and water being drawn upward away from the sensors.

Figures 5.72 and 5.73 plot the settlement profile and measurement data from Trench BI. When the extensometers were initially installed, the elevation was assigned a reference value of zero. After installation, the trench experienced a maximum settlement of 0.04 inches. This initial settlement was the result of rain infiltrating into the trench from around the temporary patch. The initially measured total settlement decreased as the soil froze and ice lens formed and pushed the backfill upward. Data indicate that the trench remained in an uplifted condition from January 23, 2008, to April 8, 2008. During the winter months, the extensometers measured a total uplift of 0.13 inches from February 4, 2008, to March 4, 2008. This movement was controlled by the top lift. The larger rate of uplift during this time was the result of the temperature of the backfill being closer to the freezing point. Temperatures closer to the freezing point increased the backfill's ability to draw water upward to form an ice lens. This increased the amount of lensing in the upper lift of the trench, resulting in larger uplifts. This uplift was reflected in the elevation survey performed on March 19, 2008. As the soil thawed, the backfill in the trench began to settle again.

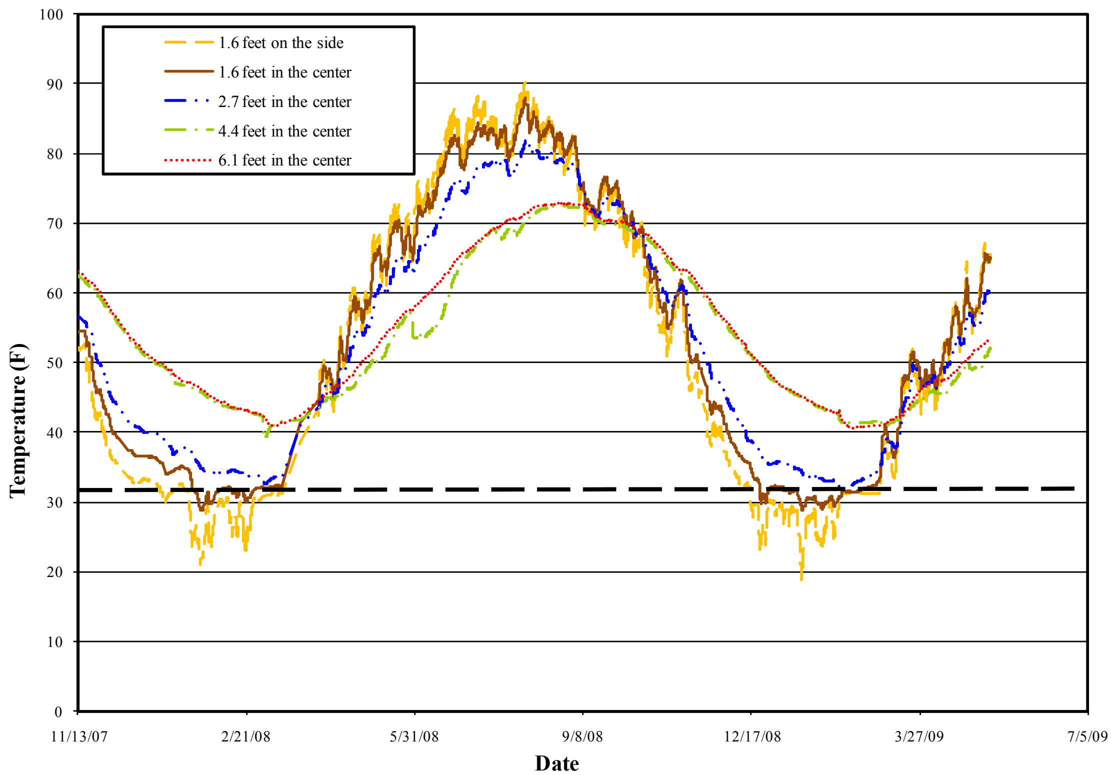


Figure 5.66. Temperature in Trench BI

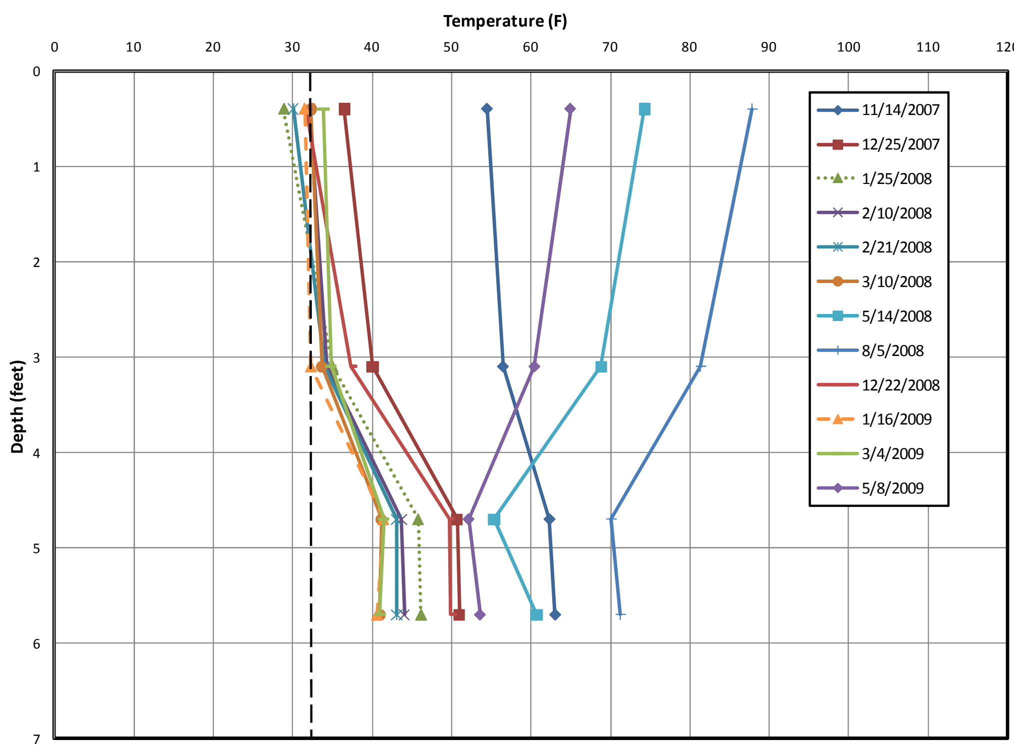


Figure 5.67. Temperature profiles for selected dates of Trench BI

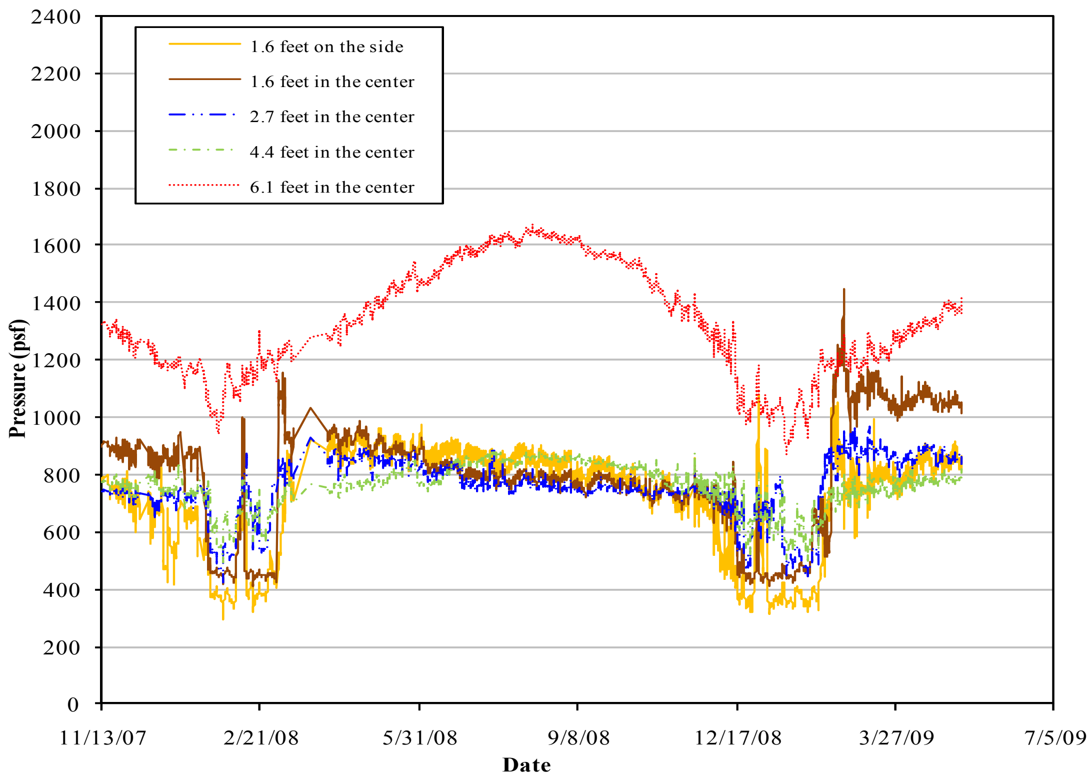


Figure 5.68. Pressure readings for Trench BI

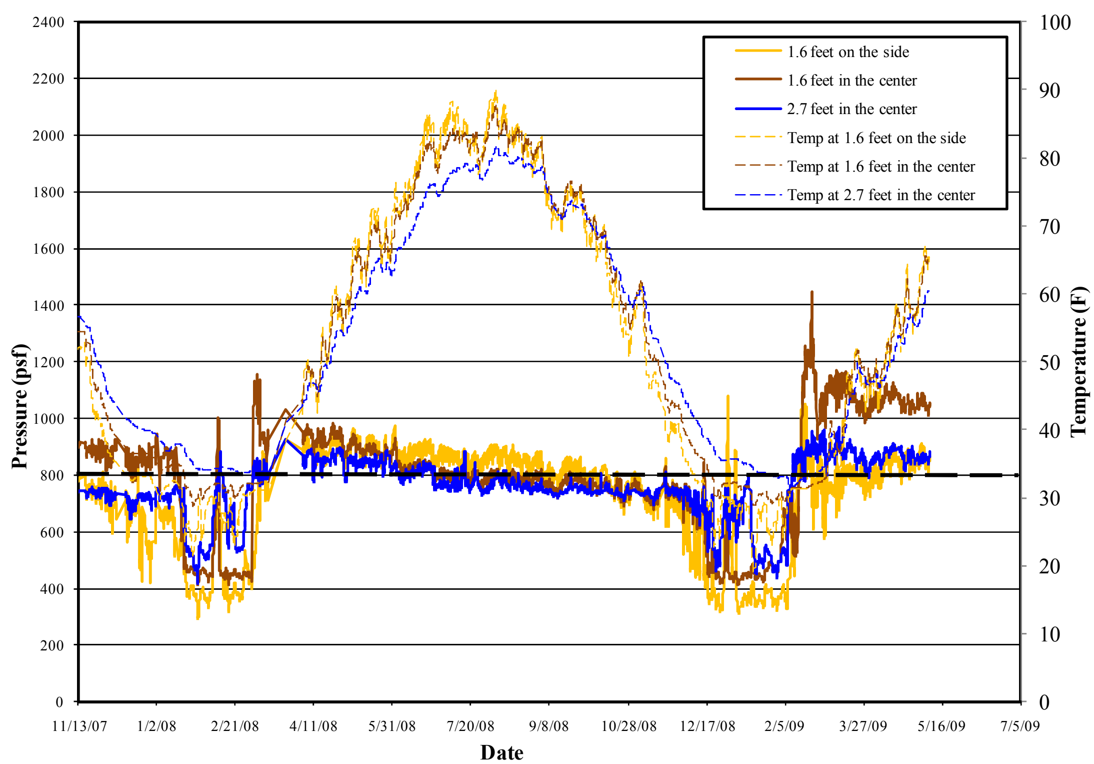


Figure 5.69. Temperature and pressure comparison for the top lift of Trench BI

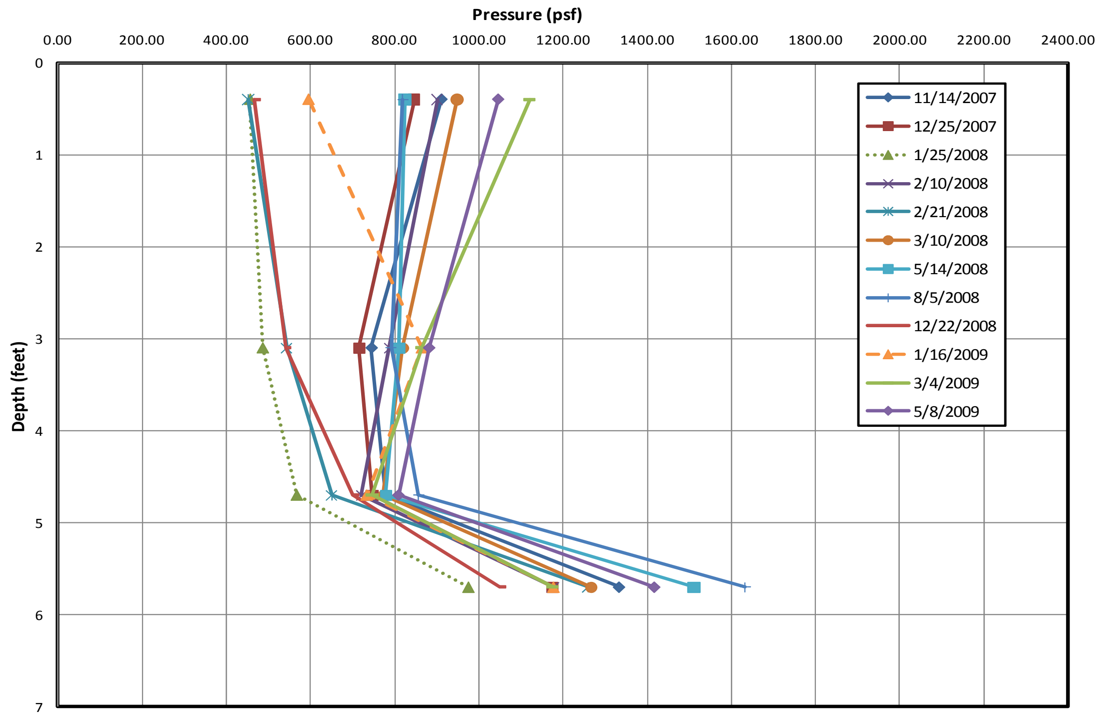


Figure 5.70. Pressure profiles for selected dates for Trench BI

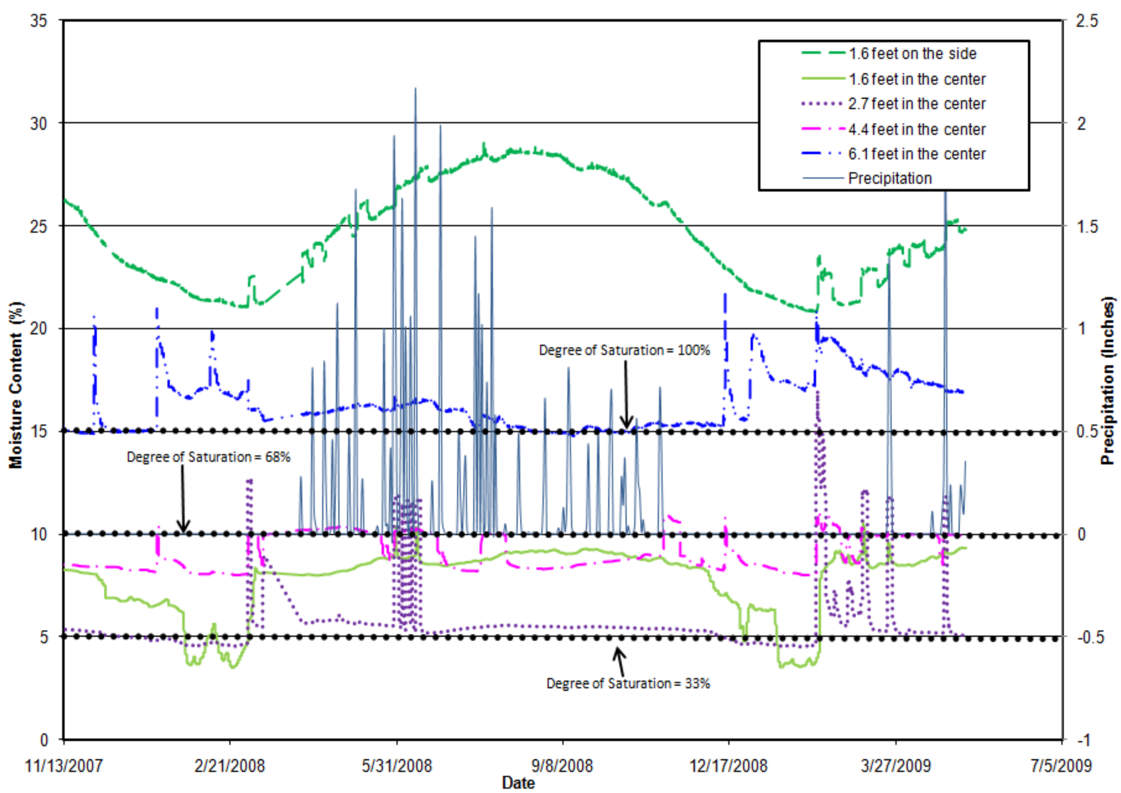


Figure 5.71. Moisture content in Trench BI

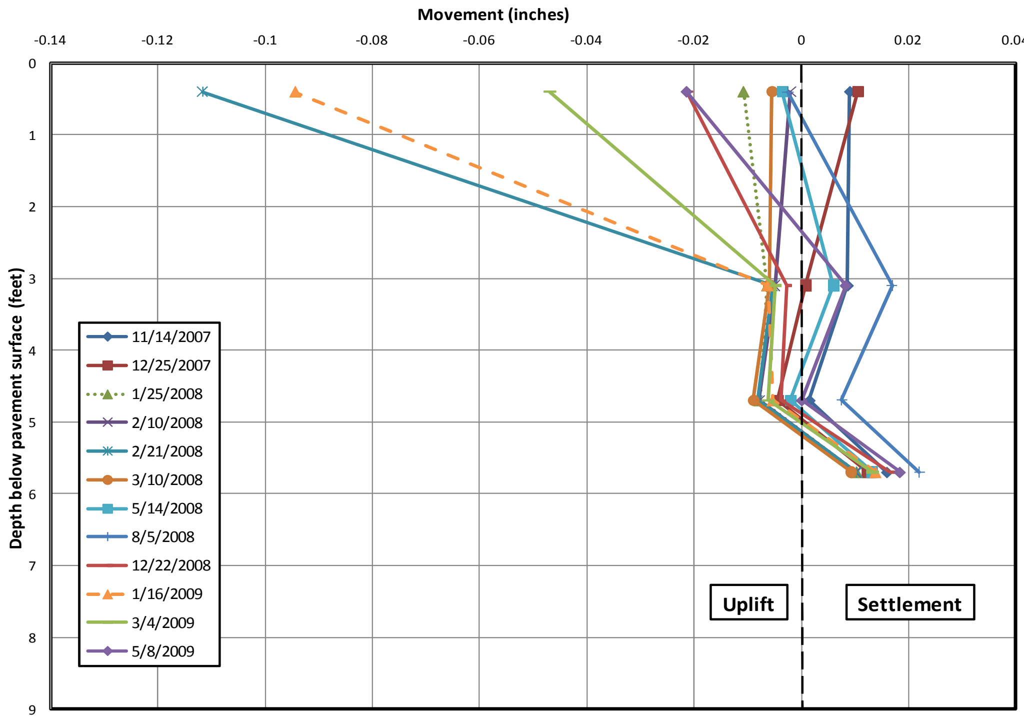


Figure 5.72. Settlement profiles for selected dates for Trench BI

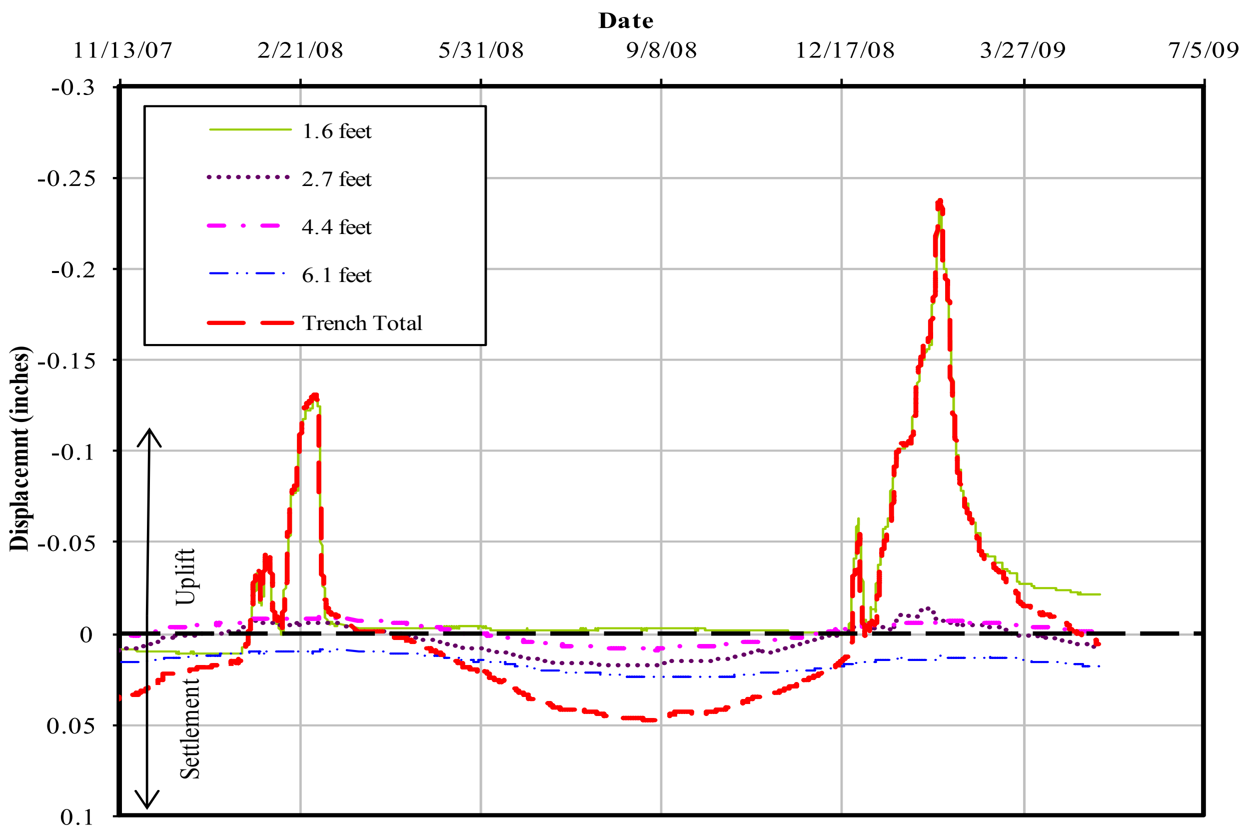


Figure 5.73. Extensometer readings for Trench BI

Instrumented Trench CI

The temperature readings for Trench CI are plotted in Figure 5.74. Figure 5.75 shows the temperature profile in Trench CI for the dates selected in Table 5.15. The temperatures in the trench followed a similar pattern as the temperature sensors in the soil adjacent to the trench. The figure shows that freezing temperatures in the trench went down to a depth of 1.37 feet from January 4, 2008, to March 13, 2008, for the sensors both on the side and in the center of the trench. However, based on the temperature profiles, freezing temperatures may have penetrated deeper into the trench (up to 2.8 feet below the surface). The trench experienced freezing temperatures to a depth less than the subgrade soil adjacent to the trench. The temperature profiles were linear, which was expected. The temperatures measured in the T-section and in the center of the trench were similar. This indicates that the pressure cell in the T-section was far enough away from the edge of the trench so the cohesive soil did not affect its readings.

Figure 5.76 shows the pressures in Trench CI. Figure 5.77 shows the pressures compared with the temperatures measured in the trench for the top two lifts. The pressure cells in the middle of the trench experienced the same pattern of pressure increases and decreases as Trench BI. However, the pressure cell in the T-section had inverse pressure readings during the winter. This resulted because of the T-section being cut into the cohesive material adjacent to the trench. The cohesive material below the T-section had a lower permeability than the granular backfill. This prevented the upward movement of water in this area, which in turn prevented frost from forming and the surrounding soil from stiffening. When comparing the pressure cell in the T-section to the pressure cell in the center of the trench, the pressure cell in the T-section had lower pressure readings than the pressure cell in the center of the trench. This difference was not as large as the difference between pressures in Trench BI. The smaller pressure difference was the result of the pressure cell being placed further away from the edge of the trench than the pressure cell in Trench BI.

Figure 5.78 shows the pressure profiles for the dates selected in Table 5.15. This figure illustrates that during February, when the frozen fringe was shifting vertically in the trench, the pressure profiles were not linear. The upper regions of the trench were affected more than lower regions of Trench CI. This indicates that even though there was a spike in the pressures at the surface, the effect of the spike decreased with depth. This was contrary to Moser (1990) about the pressure increasing on utility cuts because of frost heave.

Figure 5.79 shows the reading from the moisture sensors. The field-testing results are shown as dots at the beginning of the monitoring period. The difference between the moisture sensor readings was the result of instrument error (moisture sensor and nuclear density), infiltration of rain around the temporary patch, and time delay between field-testing and initial readings. As in Trenches AI and BI, the deepest moisture sensor had the highest values. This indicates that water was freely flowing through the trench and pooling at the bottom of the trench. During the spring, the moisture sensors did not measure variations in the trench despite the rain events in early spring in Ames.

Figures 5.80 and 5.81 plot the settlement profile and measurement data in Trench CI. The initial settlement in Trench CI was smaller than in the other trenches because less water infiltrated into the trench around the temporary patch. The maximum settlement was 0.02 inches. The maximum uplift in the trench was 0.11 inches. The uplift in the trench was controlled by the upper two extensometers extending from depths of 5.5 feet to 2.8 feet. This indicated that the frozen fringe extended below a depth of 3.8 feet and the frozen fringe caused heave on the extensometer, spanning from 5.5 feet to 3.8 feet.

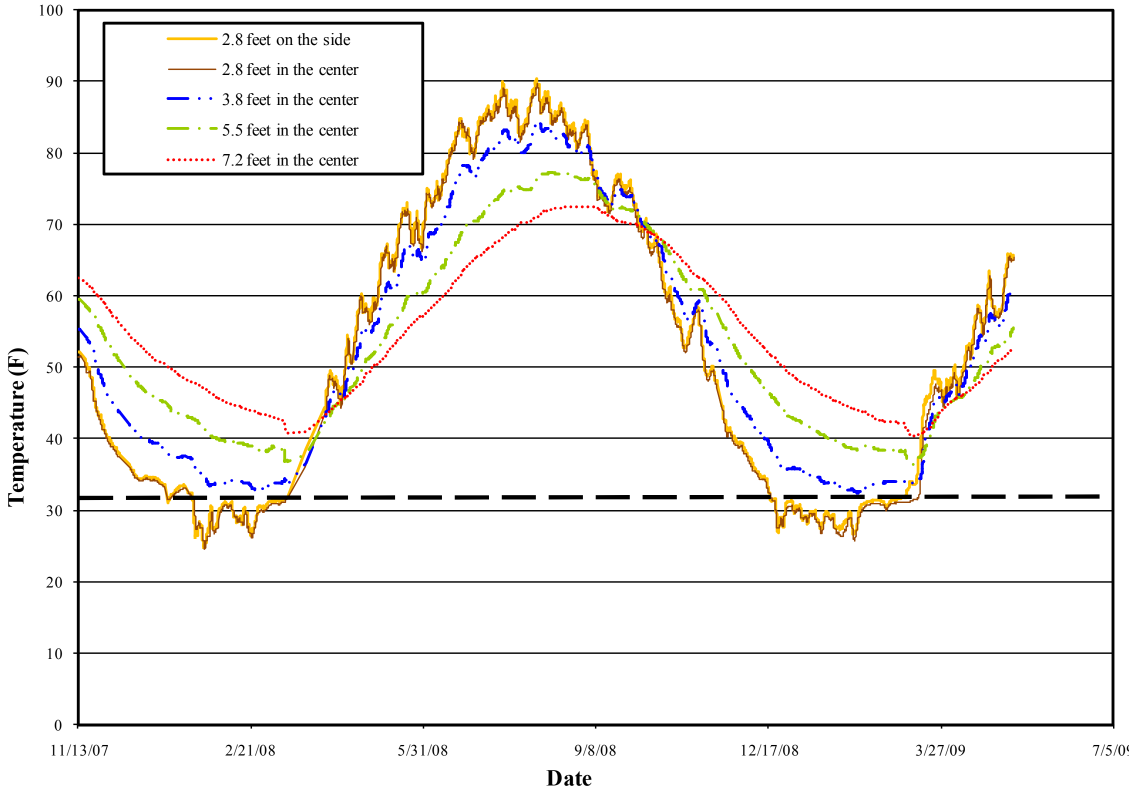


Figure 5.74. Temperatures in Trench CI at various depths

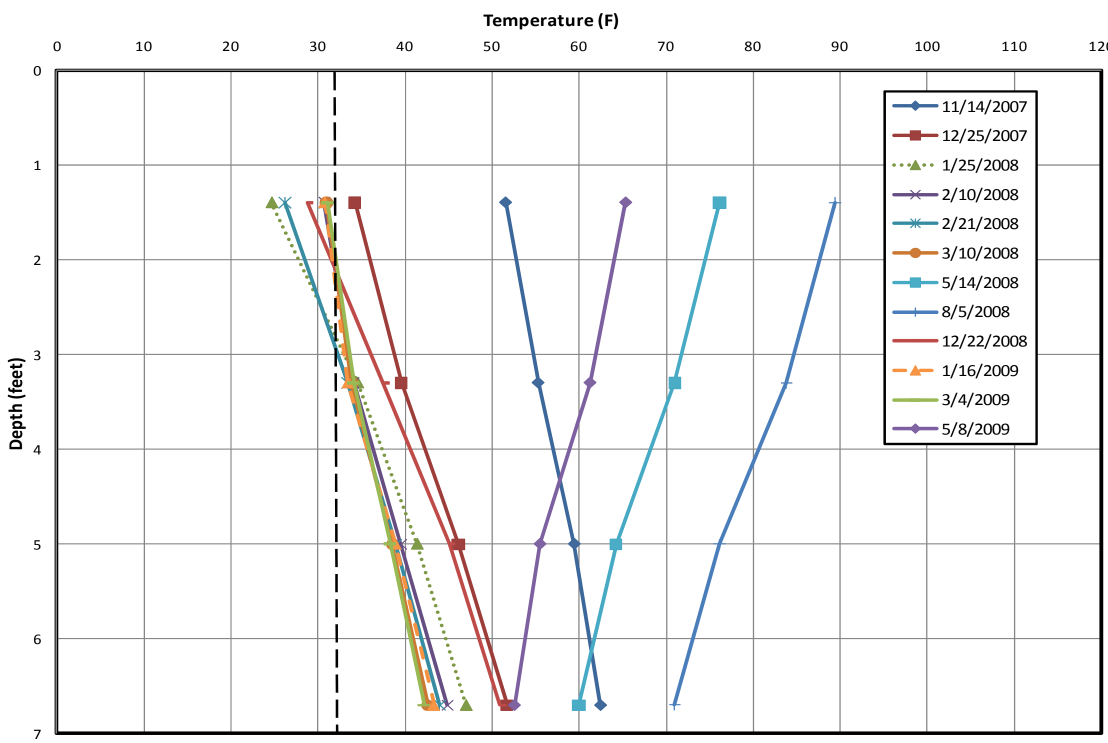


Figure 5.75. Temperature profiles for selected dates for Trench CI

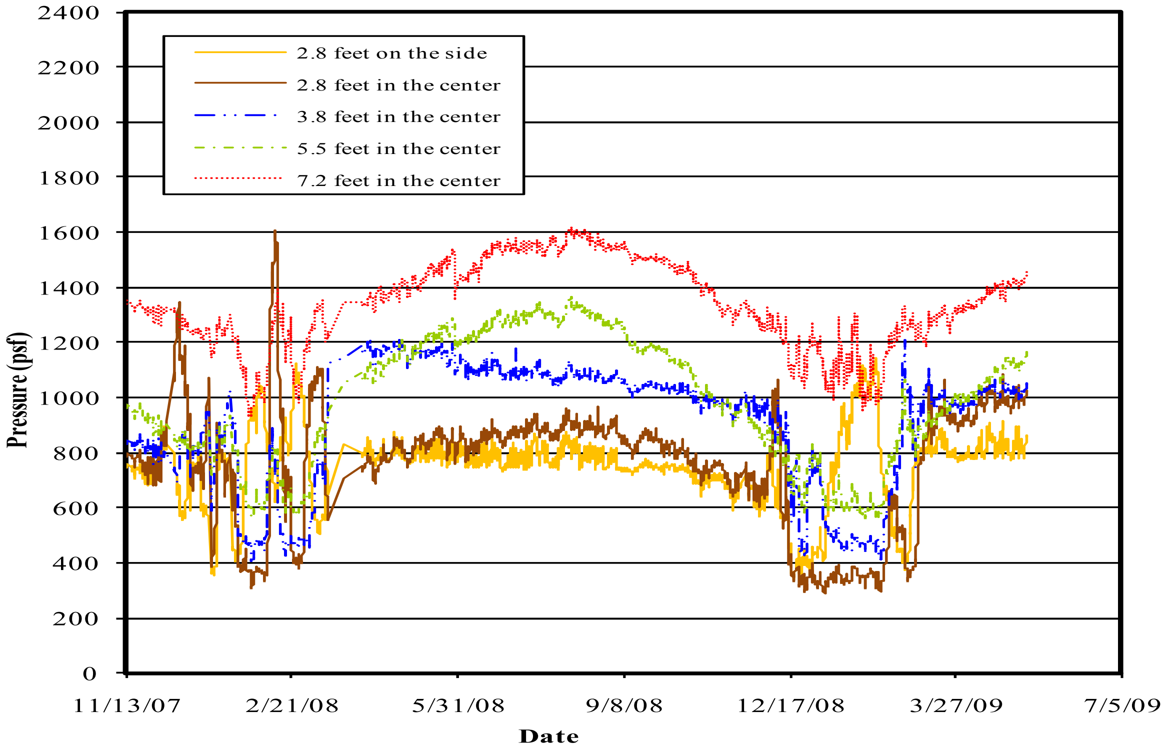


Figure 5.76. Pressures in Trench CI

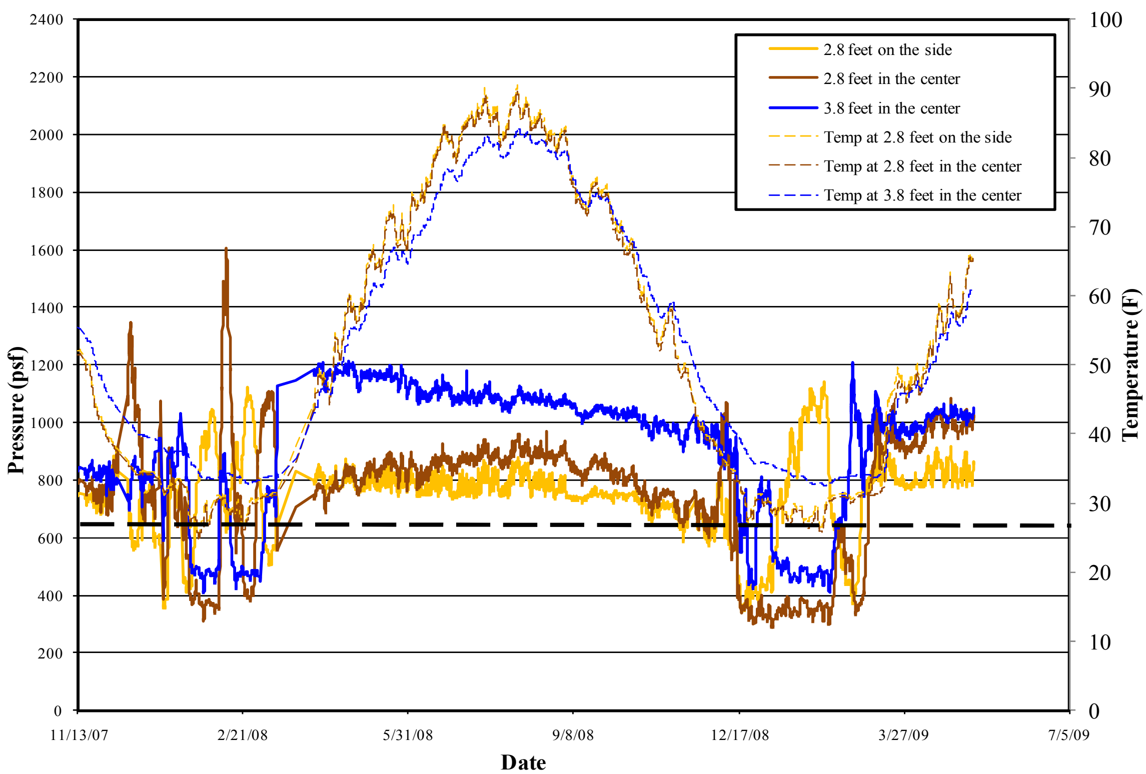


Figure 5.77. Pressure and temperature readings for the top two lifts in Trench CI

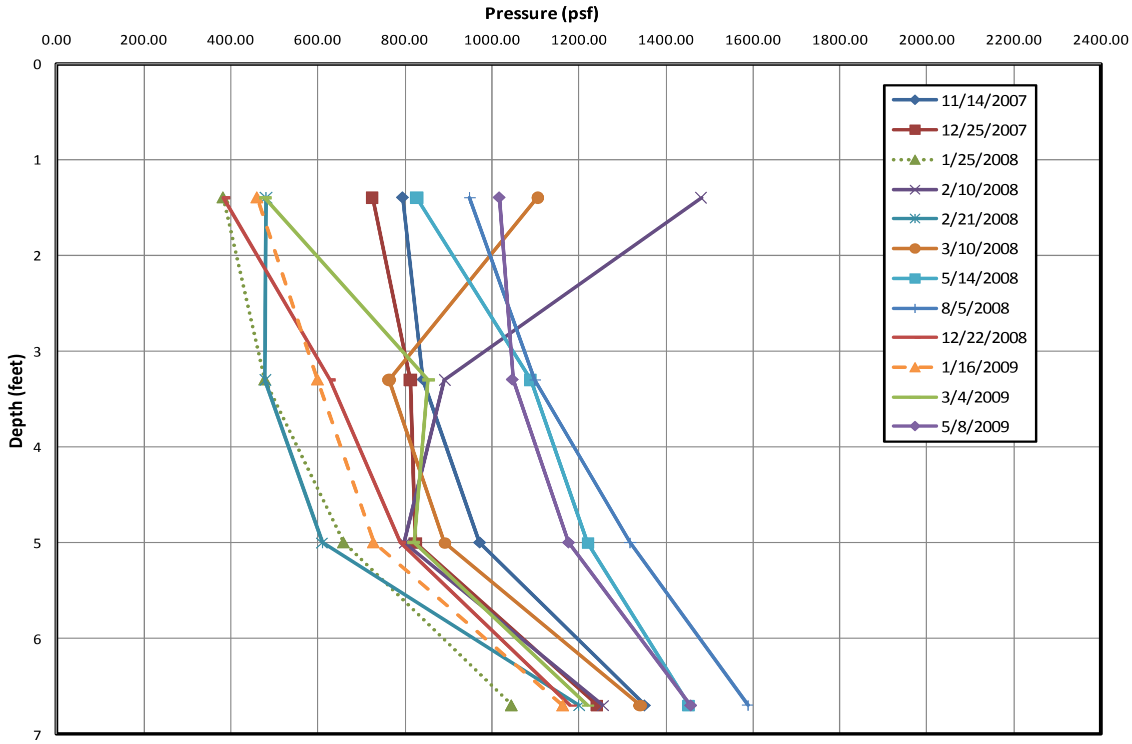


Figure 5.78. Pressure profiles for selected dates for Trench CI

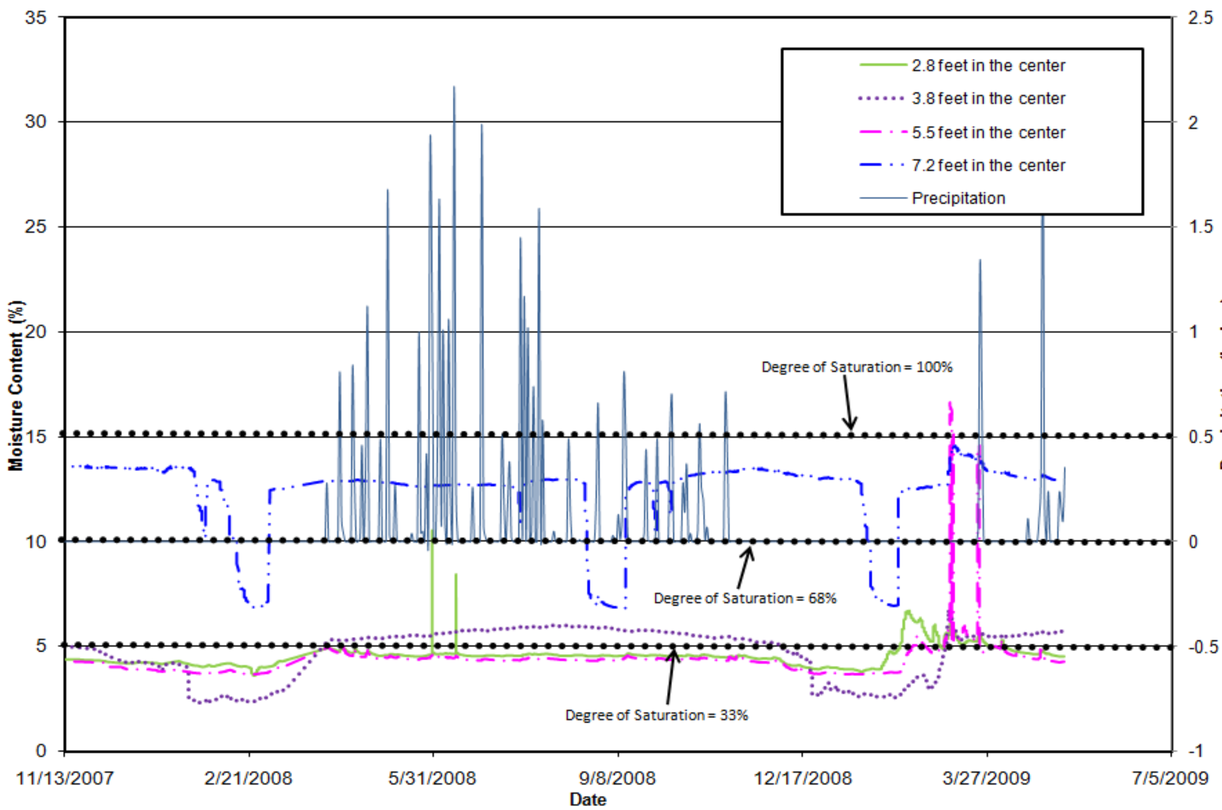


Figure 5.79. Moisture data for Trench CI and the field-testing results

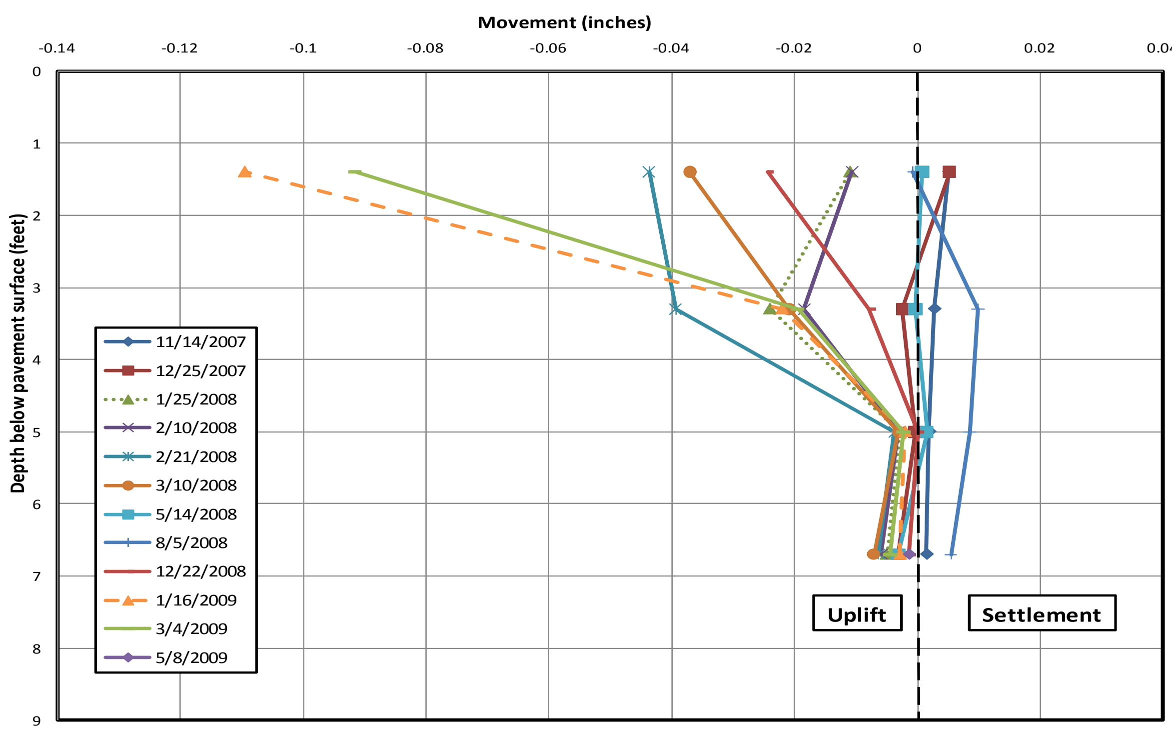


Figure 5.80. Settlement profiles for selected dates for Trench CI

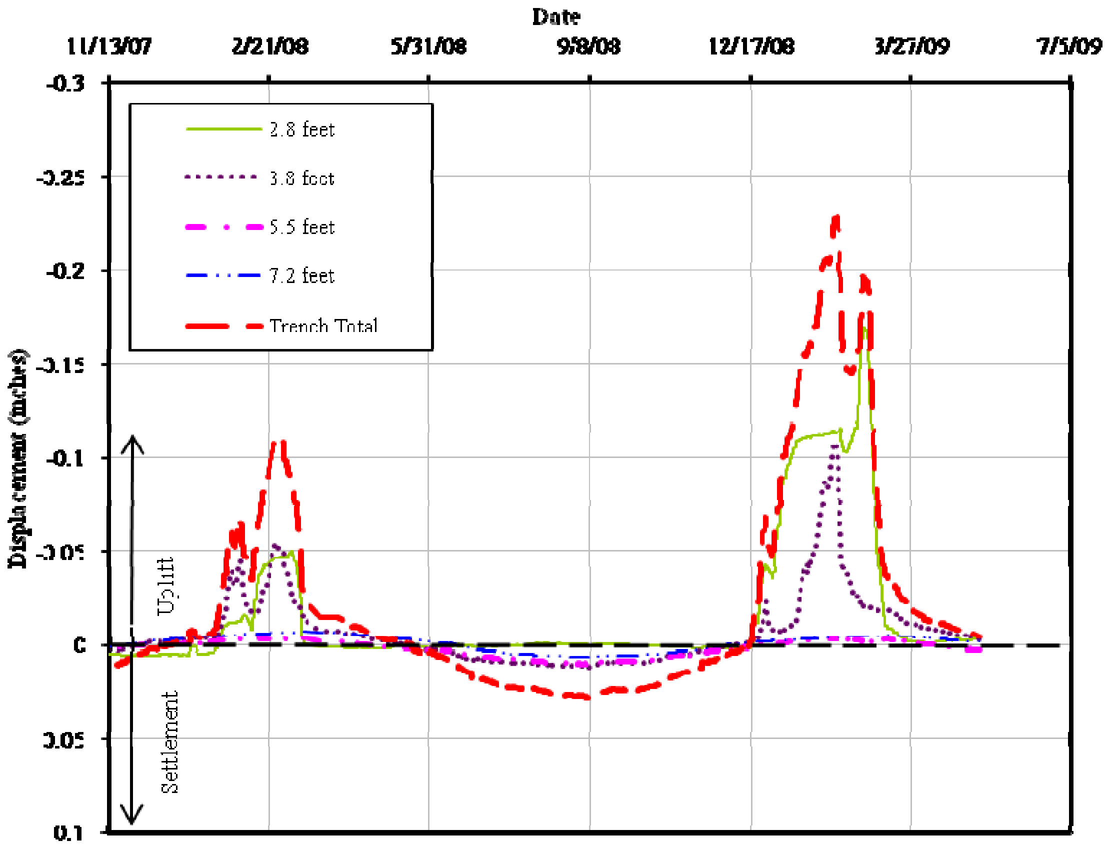


Figure 5.81. Extensometer reading for Trench CI

Comparison of Instrumented Trenches

Figure 5.82 shows the pressures in the center of the top lifts of the trenches. Trench CI experienced the largest pressure change when the frozen fringe migrated downward in February. In Trench AI, the upper lifts did not show the same pressure changes as Trenches BI and CI because instrumentation was above the frozen fringe for the duration of the winter. During nonfreezing temperatures, the readings were similar.

Figure 5.83 shows the pressure readings from the side of Trench BI and the T-section in Trench CI. This shows in the T-section the pressures were not affected by the stiffening of the cohesive materials adjacent to the trench, unlike the pressure cell in Trench BI.

Figure 5.84 shows the total settlements for all three trenches. Trench BI had the largest uplifts during the winter. This was a result of Trench BI being placed at the highest relative density. Because the backfill had silts in it, the higher relative density resulted in smaller voids that were more susceptible to frost heave. Trench AI had the greatest settlements. This was the result of the backfill being at the lowest relative density. This caused the backfill to collapse when water infiltrated into the trench.

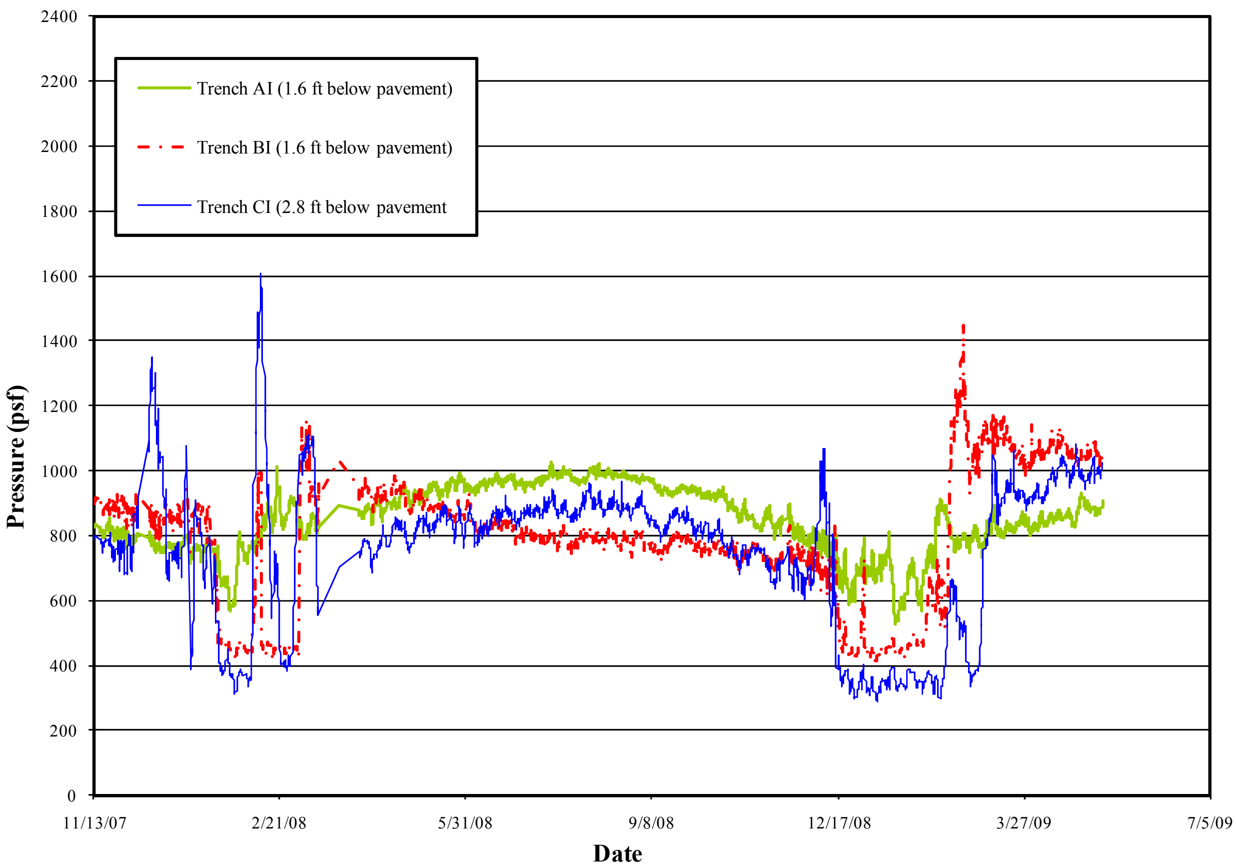


Figure 5.82. Pressures in the center of the top lift with instrumentation of the trenches

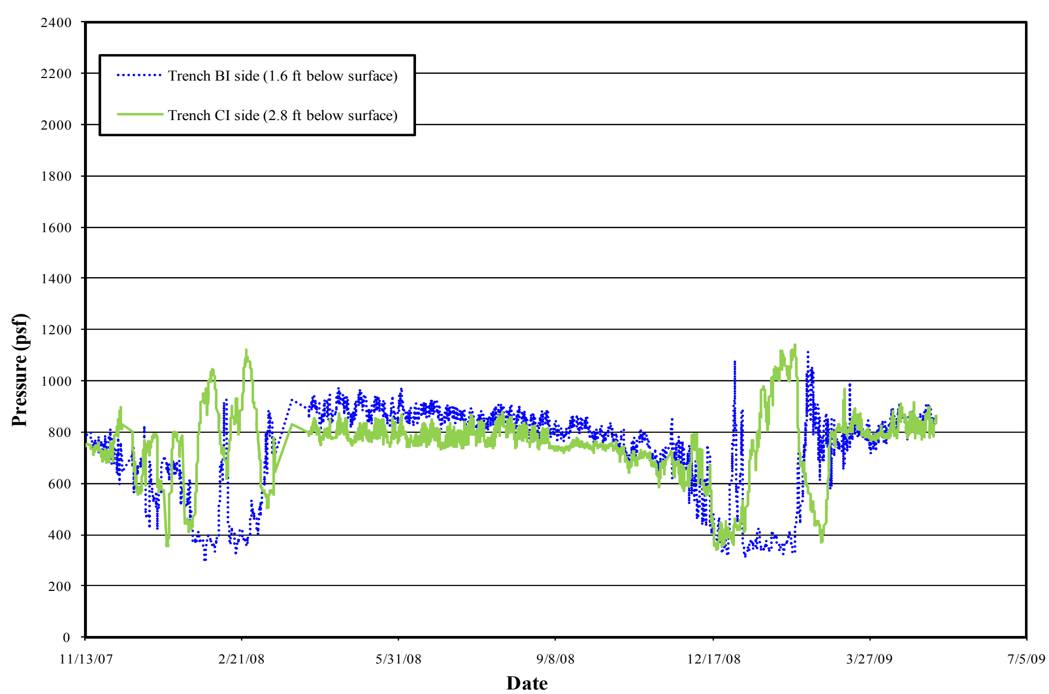


Figure 5.83. Pressures in the side of Trenches BI and CI for the upper-instrumented lifts

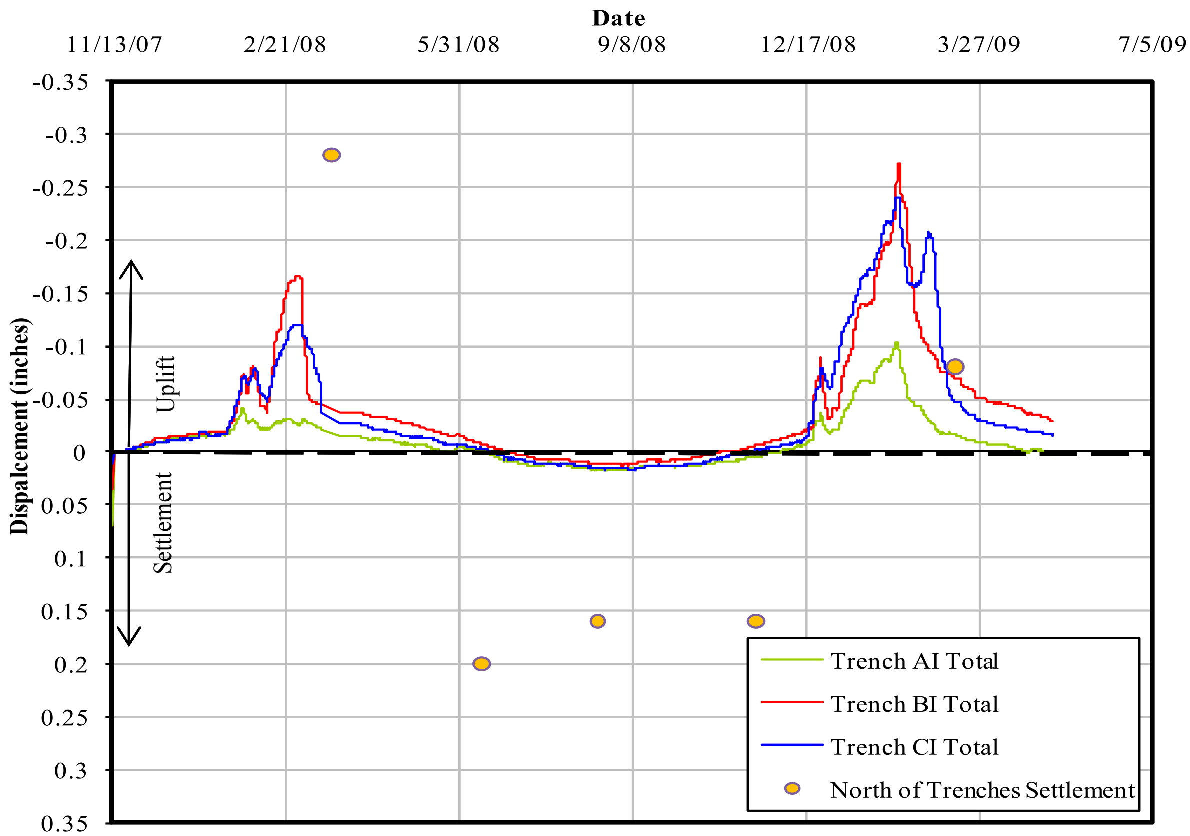


Figure 5.84. Total settlements measured in each trench

Summary of Results and Discussion

Following are the findings from the three instrumented trenches:

- After the trenches were constructed, water infiltrated into the trenches around the temporary patch, causing the backfill to collapse. The largest collapse occurred in Trench AI, where the backfill was placed at the lowest relative density and at a moisture content within the range of bulking moisture content. Trench BI had the second-largest collapse even though it had a higher relative density than Trench CI. This occurred because during the rain event, water drained around Trench 3 rather than over the trench, like in Trench BI. Also, the collapse in Trench CI was smaller than that in Trench BI because as the water entered the utility cut, the T-section helped to keep the water from affecting the center of the trench where the extensometers were located.
- The 3/8-inch minus is susceptible to collapse behavior as shown in the laboratory-test results and the results from the instrumented trenches. It is also susceptible to frost heave, as shown by the heave in Trenches BI and CI during the winter.
- The measured frost heaves occurred where the frozen fringe was located. In Trench AI, the frozen fringe was located below the uppermost pressure cell and at the bottom of the uppermost extensometer. This resulted in the pressure cell and the

extensometer moving as a unit within the upper region of frozen backfill and therefore not measuring changes in pressures and uplift movements.

- When frost was present in the trenches, the pressures in the top of the trenches increased when the frozen fringe migrated downward in February. The increase in pressures was measured deeper into the trenches. However, this pressure increase did not double the pressure in the bottom of the trench as Moser (1990) stated.
- When there was frost in the trenches, the extensometers measured uplift in the trenches. In Trenches BI and CI, the maximum uplift was also experienced. Trench AI experienced uplift in the second lift from the surface; however, its overall settlement was still downward. In Trench AI, the greatest increase in pressure during the winter months was experienced in the second lift from the surface.
- After the trench construction, water infiltrated into the trenches around the temporary patch and caused initial settlements. Trench AI, which was placed at the lowest relative density, experienced the greatest settlements when the water infiltrated. Trenches BI and CI experienced uplift during the winter months. Trench CI, which was placed at the highest dry unit weights, experienced the smallest settlements and uplifts.

Conclusions

Based on the field-testing results, continued monitoring, and measurements with the instruments, the following conclusions can be drawn:

- 3/8-inch minus exhibits collapse behavior when wetted, as seen when water infiltrated around the temporary patch into the trenches; is frost susceptible; and undergoes heave during freezing conditions. Backfill of 3/8-inch minus limestone is not an acceptable backfill material.
- Trench CI performed the best. Placing the backfill with moisture control decreased the settlement and uplift. The FWD testing showed that Trench CI provided the stiffest response. Constructing the T-section reduced the effects of the zone of influence next to the trench.
- Temperature sensors installed in the soil adjacent to the trench confirmed that cohesive soils have higher thermal conductivity than granular soils used in trenches. This causes pressure and temperature gradients in the trenches from the edges to the center.
- Pressure measured in the trenches did not confirm Moser's theory that frost heave causes the pressures in trenches to double. Rather, as the frozen fringe migrated downward, the voids above the frozen fringe were sufficient to prevent the buildup of pressures. However, during the next frost cycle, the trenches will have undergone more settlement, the void spaces will be smaller, and pressure could increase. The pressure readings in the trench during the winter could have been affected by the installation process.
- The extensometers and pressure cells were able to detect the movement of the frozen fringe within the trenches; however, the exact location of the fringe could not be determined with the current instrumentation arrangement.

- The moisture sensors in the trenches were operating below the design operation temperatures during most of the monitoring. The moisture sensors did show that the cohesive soils adjacent to the trench had higher moisture contents than the granular backfill. The sensors also showed that water was pooling in the bottom of the trenches.

CHAPTER 6. SUMMARY AND CONCLUSIONS

Background

The common procedure of installing utilities, such as gas, water, telecommunications, and sanitary and storm sewers, requires an excavation to install the pipes or lines. Utility cut restoration has a significant effect on pavement performance. It is often observed that the pavement within and around utility cuts fails prematurely, increasing maintenance costs. For instance, early distress in a pavement may result in the formation of cracks where water can enter the base course, in turn leading to deterioration of the pavement (Peters 2002). The resulting effect has a direct influence on the pavement integrity, life, and aesthetic value, as well as driver safety (Arudi et al. 2000). The magnitude of the effect depends upon the pavement patching procedures, backfill material condition, climate, traffic, and pavement condition at the time of patching. Bodosci et al. (1995) noted that new pavement should last between 15 and 20 years; however, once a cut is made, the pavement life is reduced to about 8 years. Furthermore, Tiewater (1997) indicates that several cuts in a roadway can lower the road life by 50%.

Poor performance of pavements around utility trenches on local streets and state highway systems often causes maintenance due to improper backfill placement (i.e., improper backfill, undercompacted, too dry, too wet, etc.). The cost of repairing poorly constructed pavements can be reduced with an understanding of proper material selection and construction practices. Current utility cut and backfill practices vary widely across Iowa, which results in a range of maintenance issues.

To address these concerns, InTrans (formerly the Center for Transportation Research and Education) at ISU, along with the Iowa DOT, began a multiyear investigation into utility cut restoration failures. The Iowa Highway Research Board funded two phases of this investigation. “Investigation of Improved Utility Cut Repair Techniques to Reduce Settlement in Repaired Area” (Phase I) was an initial investigation into utility cut restoration failures to document the occurrence and frequency of failures and to determine the failure mechanisms. Phase II continued the Phase I investigations of failure mechanisms and the monitoring of utility cut restorations at various locations around Iowa. In addition, Phase II implemented the Phase I recommendations for construction and monitoring of several new utility cut trenches and investigated trench settlement using instrumentation in trenches.

Three activities took place during Phase I to evaluate the construction and performance of utility trench restorations: (1) a survey was conducted to document construction practices used in utility trench restorations in several cities in Iowa; (2) laboratory tests were performed on backfill to determine its engineering properties; and (3) trench restoration performance was monitored using FWD testing. Survey results indicated that many restored utility cut restorations fail in less than two years. Field and laboratory tests of backfill indicated inadequate compaction, moisture content, and density of the backfill are factors that contribute to utility cut trench restoration failures. Falling weight deflectometer tests indicated weakened subgrade soil around the utility cut trench restorations. This weakened soil is known as the “zone of influence.”

Based on the results of Phase I, a three-part research project, "Utility Cut Repair Techniques—Investigation of Improved Utility Cut Repair Techniques to Reduce Settlement in Repaired Areas, Phase II", was initiated to further investigate the influences of compaction, moisture content and density of the backfill, and zone of influence on the performance of utility cut trench restorations. These research areas include: (1) continued monitoring of the utility cut restorations constructed during Phase I for two additional years; (2) the construction of new trenches using six recommended practices; and (3) instrumentation of three new trenches to understand the mechanisms of trench backfill settlement and load distribution.

The objectives of Phase II were to

- Correlate the long-term performance of trench restorations with the in-situ properties of the backfill during construction and the engineering properties in laboratory testing.
- Continue the monitoring of the utility cut restorations constructed during Phase I.
- Construct recommended trenches and monitor their long-term performance.
- Research and identify the principles of trench subsurface settlement and load distribution in utility cut restoration areas using the three instrumented trenches.
- Update the recommendations made during Phase I and recommend the best practices for utility cut restoration repair techniques for the SUDAS Program.

Research Plan

To achieve the objectives of the project, four tasks were outlined. First, monitoring of trenches constructed during Phase I was continued. Second, six trenches were constructed using different practices. Third, three instrumented trenches were constructed to compare the performance trenches constructed using different construction practices. Fourth, data collected from the first three tasks was evaluated and summarized.

Task 1: Continued Monitoring of Trenches Documented during Phase I

During Phase I, the construction practices of four utility cut restorations were documented across Iowa. While the trenches were being constructed, the top lift was tested using nuclear density tests and DCP. After the restorations were completed, elevation surveys were performed; however, only three of the trenches were tested using FWD testing during Phase I. These three trenches were monitored during Phase II with elevation survey and FWD testing.

Task 2: Construct the Five Remaining Trenches Proposed in Phase I and Monitor All Six Trenches Using FWD

One of the six recommended trenches was constructed in Phase I; the remaining five utility cut restorations were constructed during Phase II. These utility cut trench restorations, along with the one constructed during Phase I, were monitored using FWD tests.

The engineering properties of the soil removed from the trenches and the backfill materials were measured in the field and in laboratory testing. Laboratory testing included sieve analysis, hydrometer, Standard Proctor tests, and relative minimum and maximum density tests. During the construction of the proposed trenches field tests, nuclear density, DCP, and/or Clegg Hammer tests were used to evaluate the in-situ properties of the backfill material. The results from the field and laboratory investigations were used to evaluate how the in-situ properties of the backfill material affect the long-term performance of the utility cut restoration.

Task 3: Instrument and Monitor Three Utility Cut Trenches

To better understand the principles of settlement and load distribution in and around the utility cut restoration area, three trenches were instrumented. The trenches, located on Kellogg Avenue in Ames, Iowa, were constructed using

- A shallow trench (less than 8 feet deep) employing the City of Ames current construction practices. The current construction practices included lifts of 2 feet, granular backfill with minimum moisture, and density control (Trench AI).
- A shallow vertical walled trench with granular backfill, lift thickness less than 12 inches, moisture control, and relative density of 65% or more (Trench BI).
- A shallow T-section trench with granular backfill, lift thickness less than 12 inches, moisture control, and relative density of 65% or more (Trench CI).

The instrumentation monitored settlement with extensometers, overburden pressure with pressure cells, moisture content of the backfill material with moisture sensors, and temperature using temperature sensors. These trenches were monitored from fall 2007 to spring 2009 (20 months)

Task 4: Data Evaluation

Data collected from the tasks above were analyzed. The field- and laboratory-testing data from Tasks 1 and 2 were compared with the settlement and FWD measurements. From this comparison, the goal was to determine if the recommended construction practices improved the performance of the utility cuts. The data output from the instrumentation in Task 3 was compared to determine the specific mechanisms that cause deterioration of the utility cut patches.

Recommendations and Conclusions

Based on the monitoring of the trenches constructed during Phase I, the six recommended trenches during Phase II, and the three instrumented trenches constructed during Phase II, the conclusions and recommendations in the following lists can be made.

Material Selection

- Three-eighth-inch minus backfill is not an acceptable backfill material because it exhibits collapse behavior when wetted, as seen when water infiltrated around the temporary patch into instrumented Trench AI. It is also frost susceptible and undergoes heave during freezing conditions as shown in the instrumented trenches placed with moisture control and proper compaction techniques.
- Soils containing silt-sized particles are most susceptible to frost heave.
- One-inch clean limestone or other clean backfill with limited fines do not experience collapse and are least susceptible to frost-heave. The use of 1-inch clean limestone improved the performance of the trenches. It stiffened the response of the trench in FWD testing, and the settlement within the trench is less than in trenches constructed with 3/8-inch minus limestone.
- Soils excavated from the trenches could be mixed with granular backfill if laboratory tests indicated the range of moisture content and densities that the material need to be placed at and appropriate quality control measures were used.

Construction Practices

- The use of a concrete patch with dowels performed the best over the long term. This was documented with the utility cut constructed in Des Moines.
- It is recommended to remove at least two feet of pavement around the perimeter and to compact the soil if a T-section is not constructed. This is supported by the results in Trench A.
- When comparing trenches constructed with 1-inch clean (Trenches D, E, and F), the trench constructed with highest relative density shows the smallest settlement and highest uplift movement. The trench with the lowest relative density shows the highest settlement.
- When comparing the FWD tests performed on Trenches D, E, and F, the trenches with T-sections (Trenches E and F) showed reduced measured deflections within the zone of influence. The T-section may have caused a shift of the zone of influence.
- Instrumented trenches show higher settlement in the trench constructed using the City of Ames specifications (i.e., lower relative density), which was constructed with lift thicknesses of 2 feet with no moisture or compaction control.
- Instrumented trenches with construction control showed lower settlement and higher uplift movement when compared to the city practice trench. However, trenches with construction control show uplift movement closer to the surrounding soil than the city practice trench.
- Trench BI showed higher deflections than Trenches AI and CI, measured using the FWD test results. This trench corresponds to the CBR values estimated from DCP test results (i.e., higher CBR corresponded to smaller FWD deflections).
- Monitoring recommended and instrumented trenches shows that smaller settlement was measured in trenches with higher relative densities, and smaller FWD deflections corresponded to higher CBR values estimated from the DCP test.
- The T-section could be modified to use walls that are beveled outward to facilitate

compaction of backfill. Beveled edges may reduce the amount of disturbance to the surrounding soil and eliminate the vertical excavation; however, it may make compacting the backfill at the edges difficult. This is expected to prevent the zone of influence from migrating outside of the T-section.

- Construction equipment should be kept away from the edges of the trench. Falling weight deflectometer testing on the Cedar Rapids trench showed that damage caused by equipment during construction had a long-term impact on the performance of the trench.
- The use of geogrid in the trenches did not improve the performance of the trenches compared to the trenches constructed without the geogrid for the trenches using 3/8-inch minus limestone. For trenches constructed with 1-inch clean backfill material, Trench F (T-section with geogrid) showed smaller settlement than Trench E (T-section without geogrid), but Trench F also was compacted at higher relative density. The geogrid appears to have stiffened the response of the trench based on FWD testing.
- In 2008, temperature profiles in Trenches AI and BI showed freezing temperatures as deep as 2.7 feet in the city practice trench (AI) and 1.7 feet in the controlled construction trench. Trench CI (with T-section) showed a deeper freezing temperature, which could be the effect of the native soil around the T-section. The freezing temperature in the native soil was about 2.5 feet.
- In 2008, pressures corresponded to the depth of freezing. Temperatures showed a reduction during winter, which could be attributed to heaving of the layer above.
- In 2008, settlement profiles in different trenches showed that most movement happened in the top layer, except the trenches with T-sections, which could be related to the deeper temperature change observed in this trench.

Quality Management

- Quality control measures should be implemented in the field to ensure that compaction requirements are met. This includes achieving at least medium-to-dense relative density with moisture contents above the bulking moisture content for cohesionless soils and above 95% of Standard Proctor and +/-2% of optimum moisture content for cohesive soils.
- An educational program should be established to educate city maintenance crews on the importance of proper construction practices. Based on the experience with the City of Ames, a program including demonstrations will help solidify the importance of moisture control during the construction of trenches.

Future Research Needs

- To reduce work associated with T-section and improve the zone of influence, construct trenches with a beveled cross-section at the top to facilitate the compaction of the backfill at the perimeter of the trench.
- Continue FWD testing on the trenches.
- Continue to monitor the settlement of the trenches.

- Continue to monitor the instrumented trenches.
- Pavement surface roughness using the IRI data to assess the ride quality in utility cut areas needs to be studied and investigated across the state of Iowa to help in maintenance and improve ride quality in utility cut.

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