Utility Cut Repair Techniques—Investigation of Improved Cut Repair Techniques to Reduce Settlement in Repaired Areas

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Final Report
December 2005

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16. Abstract

Pavement settlement occurring in and around utility cuts is a common problem, resulting in uneven pavement surfaces, annoyance to drivers, and ultimately, further maintenance. A survey of municipal authorities and field and laboratory investigations were conducted to identify the factors contributing to the settlement of utility cut restorations in pavement sections.

Survey responses were received from seven cities across Iowa and indicate that utility cut restorations often last less than two years. Observations made during site inspections showed that backfill material varies from one city to another, backfill lift thickness often exceeds 12 inches, and the backfill material is often placed at bulking moisture contents with no Quality control/Quality Assurance. Laboratory investigation of the backfill materials indicate that at the field moisture contents encountered, the backfill materials have collapse potentials up to 35%. Falling Weight Deflectometer (FWD) deflection data and elevation shots indicate that the maximum deflection in the pavement occurs in the area around the utility cut restoration. The FWD data indicate a zone of influence around the perimeter of the restoration extending two to three feet beyond the trench perimeter.

The research team proposes moisture control, the use of 65% relative density in a granular fill, and removing and compacting the native material near the ground surface around the trench. Test sections with geogrid reinforcement were also incorporated. The performance of inspected and proposed utility cuts needs to be monitored for at least two more years.

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

Final Report
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Principal Investigator
Vernon R. Schaefer, Professor
Department of Civil, Construction and Environmental Engineering, Iowa State University

Co-Principal Investigators
Muhannad T. Suleiman, Lecturer and Research Associate
Department of Civil, Construction and Environmental Engineering, Iowa State University

David J. White, Assistant Professor
Department of Civil, Construction and Environmental Engineering, Iowa State University

Colby Swan, Associate Professor
Department of Civil and Environmental Engineering, University of Iowa

Research Assistant
Kari Jensen

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A report from
Center for Transportation Research and Education
Iowa State University
2901 South Loop Drive, Suite 3100
Ames, IA 50010-8632
Phone: 515-294-8103
Fax: 515-294-0467
www.ctre.iastate.edu

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EXECUTIVE SUMMARY

Utility cuts are made in existing pavement sections to install a variety of underground conduits, including electric, water, and wastewater utilities, as well as drainage pipes under roadways. If the backfill material is not suitable for the site conditions or not properly installed, this material will begin to settle relative to the original pavement. Several cities in the United States and abroad spend millions of dollars each year on maintenance and repairs of utility cuts made in pavements (APWA 1997). This study was undertaken to improve utility cut construction practices in Iowa, thereby increasing the pavement life and reducing maintenance. This report includes (1) a detailed literature review, (2) a summary of the results of a utility cut survey sent to several cities in Iowa, (3) field observations of a utility cut construction techniques in Iowa, (4) characterization of compacted backfill materials using in situ measurements, and (5) characterization of backfill materials using laboratory investigation.

Relevant Literature

Utility cuts made in existing pavement sections to install various utilities under roadways not only disturb the original pavement, but also the base course and subgrade soils below the cut. Utility cuts in a roadway affect the performance of the existing pavement as settlement and/or heave occurs in the backfill materials of the restoration. Statistical data reported by the Department of Public Works (DPW) in San Francisco (1998) shows that the pavement condition and rating decreases as the number of utility cuts made increases. In fact, the Canada National Research Council indicates that excavations in pavements by utility companies reduce road life up to 50% (Tiewater 1997).

When a utility cut is made, the native material surrounding the perimeter of the trench is subjected to loss of lateral support. This leads to loss of material under the pavement and bulging of the soil on the trench sidewalls into the excavation. Subsequent refilling of the excavation does not necessarily restore the original strength of the soils in this weakened zone. The weakened zone around a utility cut excavation is called the "zone of influence."

Poor performance of pavements over and around utility trenches on local and state systems often causes unnecessary maintenance problems due to improper backfill placement (i.e., under compacted, too wet, too dry). The cost of repairing pavements as a result of poorly performing utility cut restorations can be avoided with an understanding of proper material selection and construction practices. This research aims to improve utility cut construction practices with the goal of increasing the pavement patch life at an affordable cost, and thereby reduce the maintenance of the repaired areas.

Backfill materials and compaction requirements should include gradation, moisture control, lift thicknesses, and compaction equipment. The majority of Departments of Transportation in the United States use a granular backfill material with an AASHTO classification of A-1 and A-3. Granular backfill requirements should be based on relative density with moisture control, and not on standard Proctor. Lift thicknesses should be less than or equal to 12 inches.

Quality Control and Quality Assurance (QC/QA) include using the nuclear gauge, Dynamic Core Penetrometer (DCP), and Clegg Hammer. State DOTs generally specify 90% to 95% of standard

Proctor density for all backfill materials; however, relative density should be used for granular backfill materials. APWA (1997) suggests that when using the DCP, if the penetrometer does not penetrate more than 3¼ in (129 mm) with a minimum of 11 drops, a compaction level of 90% is obtained. A minimum Clegg hammer value of 18 is recommended for proper compaction for pavement surfaces. All these values are used for general compaction requirements, and not necessarily in utility cut regions.

Controlled Low Strength Materials (CLSM) eliminates future settlement that may occur when using soil backfill materials and does not require the use of compaction equipment. However, it has a higher initial cost than conventional backfilling.

The use of trenchless technology can eliminate the impact a utility cut has on a roadway and lower traffic interruptions, requires a smaller construction crew, has less impact on businesses, decreases the noise, and has less air pollution. However, trenchless methods have the potential of forming sinkholes, may result in heaving, leaking of drilling fluid, and drilling tools puncturing the pavement surface and other underground facilities, and have a relatively higher cost.

Survey Results

The survey results indicate opinions based on city personnel from seven cities in Iowa: Ames, Cedar Rapids, Davenport, Des Moines, Dubuque, Waterloo, and Burlington. Discussions in this area include topics such as permit fees, extent of the problem, construction requirements, and quality control.

Using the statistical data provided by the city of Ames, January and December are the prominent months for water main breaks. This trend may be a result of frost loading, which could substantially increase vertical loads (i.e., up to twice the original load) on buried pipes, Moser (1990). The effect of frost on the stresses on buried pipes and the behavior of backfill materials under freeze-thaw conditions should be further investigated.

Many cities throughout Iowa require permits before an excavation can be made, however a fee is not assessed in all cases. Ames indicated that no fee is acessed; however a permit must be obtained. Other cities charge fees in excess of $200. A permit is a mechanism to track who conducted the work and when it occurred, and fees generally attempt to recoup administrative costs. By implementing and updating permit fees in accordance with the growth of the economy, future restorations will have less of an impact on funds that could be used in other areas.

Each city surveyed indicated that the current method of utility cut construction resulted in satisfactory results, and they all indicated that there was virtually no problem. However, these cuts were estimated to last less than two years, which is a relatively short period. The life of an undisturbed pavement can be approximately ten times this length. This may be a result of minimal documentation kept on utility cut maintenance and repairs, as well as a personal opinion of the definition of a poorly performing utility cut.

Construction requirements and materials used in the construction of a utility cut repair varied in each city. The material selection is based on regional availability, with each city using a different gradation and material. Burlington experienced many problems when using sand backfill, and now is the only city surveyed that consistently uses a flowable fill for utility cuts. Other cities in Iowa have used flowable fill under specific circumstances.

Although all surveyed cities use granular backfill materials, all used 90% to 95% standard Proctor requirements in their specifications. Quality control is minimal. Dubuque and Waterloo use the nuclear density gauge for monitoring compaction requirements. In some cases, however, an inspection program consisted of only visual inspection.

Construction Techniques

A typical observed excavation consisted of a pavement cut and excavation. The utility was then repaired, and the trench backfilled with imported material. Lift thicknesses generally ranged from 2 to 4 foot, with compaction sporadically throughout the fill using a vibrating plate on the end of a backhoe. In most cases, the method of obtaining compaction was based on experience, rather than a quality control program or device. Backfill materials were compacted using large compaction equipment, which was observed getting very close to the edge of the cut. This resulted in damage to pavement surfaces along the perimeter of the excavation.

The common practice of placing 2 to 4-foot (0.6 m to 1.2 m) thick lifts leads to difficulty in obtaining adequate compaction. Essentially, the material in the upper portion of the lift is compacted, however the vibration used to orient the soil particles into a more dense structure tends to decrease with depth.

Pavement surfacing was placed any time from immediately after the utility cut was constructed to up to two weeks later. It was observed that Des Moines was the only city that plated the unpaved utility cut until surfacing was available. Other cities typically use temporary surfacing of cold asphalt, granular material or a thin PCC layer.

It was often observed that saturated native materials were added to the excavation in an attempt to clean the utility cut area. With the addition of these materials, the potential for the formation of voids increases, therefore leading to potential settlement in the future. This is an undesirable practice in two respects. First, a saturated material is very weak and has low compaction properties; second, once a native material is disturbed, achieving its original density is extremely difficult, specifically in clay-type native materials. The use of native materials in an excavation also requires monitoring of the moisture content for optimum performance.

Ultimately, sites where construction was observed from the time of excavation to the backfilling of the trench, no density or moisture quality control was used to ensure compaction requirements were met.

Field Results

The backfill materials used in several utility cut sites were characterized using the following destructive and non-destructive devices: Nuclear Density Gauge, Dynamic Cone Penetrometer (DCP), Clegg Hammer, GeoGauge, and the Falling Weight Deflectometer (FWD).

The Nuclear Density Gauge generated dry density and moisture contents for each imported backfill material. These values were then used with laboratory data to calculate relative density values. The calculated relative density values indicate a dense to very dense compacted material in investigated utility cuts in both Davenport and Cedar Rapids. The backfill material used in Ames was placed at a medium dense state; however, the backfill material used in Des Moines was placed in a loose to very loose state.

The CBR values calculated using DCP test results were fairly consistent throughout the excavated area. CBR values were higher near the center of the excavated areas when compared to CBR values near the edge of the trench. These profiles indicate that smaller compaction equipment may be needed to achieve uniform compaction throughout the trench. By incorporating smaller compaction equipment, confined areas can be reached and compacted properly. This also decreased the impact that heavy equipment such as backhoes has on the zone of influence during compaction. This was observed in Cedar Rapids, where an asphalt pavement cracked.

DCP data obtained from native material indicate a trend of fewer blows required for 3.9-inch (10 cm) penetration. This is a result of the loss in lateral support during the excavation.

When plotting the number of blows required to penetrate 3.9 inches (10 cm) into the ground, the DCP profile showed a trend of high CBR values at approximately 1.5 feet from the top of the layer below the surface layer, as the surface layer is usually disturbed. Then the CBR values reduce with depth afterward, as the effect of compaction decreases with depth for large lift thicknesses. This reiterates the importance of lift thicknesses being less than or equal to 12 inches.

According to the available literature, a minimum Clegg Hammer Impact Value of 18 is needed for proper compaction beneath a pavement surface. However, when comparing all data obtained in the field, this value was not reached at any site.

The FWD results show larger deflection in the zone of influence, which indicates the softening of this zone as a result of the cut. FWD results also show a trend of higher stiffness near the center of tested trenches as was also observed using DCP results. When subjected to FWD loading, concrete pavements produced a smaller deflection compared to the asphalt and composite pavement materials. This may be a result of the dowel bars located in the concrete aiding in the distribution of loads. The Cedar Rapids data dramatically illustrates the damage that heavy compaction equipment causes on the pavement at the edge of the cut and on the zone of influence around the excavation when the cut is open.

Laboratory Results

The laboratory results were obtained from test methods, including sieve analysis, relative density, Standard Proctor, and collapse tests. These results were then used with the field data to further characterize the material properties.

The backfill material used in all observed cities, except Des Moines, had fines contents (percentage passing sieve No. 200) greater than the maximum limit allowed by Iowa DOT (i.e., 10%) for backfill material gradation. Furthermore, most of these materials were placed at or near the bulking moisture content, which increases the settlement (collapse) potential. Bulking is a capillary phenomena occurring in moist sands in which capillary menisci between soil particles hold the soil particles together in a honeycombed structure. This structure can collapse upon the addition of water.

Collapse tests indicate a high collapse potential of 36% for loosely placed limestone screenings, 9% for 3/8-inch material used in Ames, 8.5% for 3/4-inch material used in Cedar Rapids, and 24% for manufactured sand. The material specified in SUDAS (1½-inch clean stone) had a low collapse potential of 0.35%. The collapse potential increases as the percentage of sand particles increases. Each material has a different bulking moisture content, which should be avoided when placed.

The use of granular backfill materials may require watering the material in the trench to reduce settlement potential induced by moisture change. The addition of water 2%–4% above the bulking moisture content could be used in the field during construction to reduce future settlement potential due to water effects.

Backfill materials used in Cedar Rapids and Davenport, which are classified as SM and GC, respectively, with% of sand not exceeding 35%, achieved relative densities of dense to very dense without a significant amount of compaction.

Based on the relative density data, the backfill material used in Des Moines, which is classified as SP-SM with 88% sand, was difficult to achieve the required relative density. The material placed in the field was characterized as loose with relative density less than 35%.

Design charts were generated to indicate a specified target region of compaction for a material to obtain the required density for selected granular backfill materials. These charts could be used in the field as a quality control measure if soil density is measured Relative density of 65% is suggested as a minimum requirement of compaction. Based upon information in the literature and the results of the tests conducted herein, relative densities in the range of 65% and greater can be achieved in the field by watering granular materials with water immediately after placement.

Trial Trenches

After observing the construction techniques and field and laboratory investigation, six trenches were designed and proposed to the city of Ames for construction with the goal of minimizing future settlement. Settlement expected to result from collapse and low compaction effort used in the field was avoided by using the SUDAS Class I gradation backfill with 100% passing 1½ inch sieve and with a maximum passing sieve No. 4 of 10%. The research team also tried to avoid settlement using a structural geogrid to bridge over the excavated area, with 3/8-inch backfill material used in Ames with no moisture or compaction control. Three similar trenches were proposed using the two different backfill materials. These three trenches are as follows:

1) T-section using up to three-foot wide excavation around the perimeter of the cut and applying compaction to the surrounding native material in the cutback region.

2) A two to three-foot cutback and pavement removal, along with an excavation of two feet deep into the native material. This material will be replaced with imported backfill material.

3) A trench constructed the same as number 2 above with a structural geogrid placed on the bottom of the excavated area.

The cutback excavation incorporated into the last two trenches was placed in the cutback region two to three feet beneath the excavation for bridging purposes. A two- to three-foot (0.6 to 0.9 m) cutback depth was excavated to compensate for the majority of settlement that was found to occur in backfill at two feet (0.6 m) beneath the pavement surface, according to the literature review. Cross-sections of these proposed trenches are illustrated in Figure 93.

Recommendations

Based on the field observations, field measurements, and laboratory testing, the following recommendations are made:

1. Proper compaction is generally determined according to Standard Proctor compaction in most cities. However, the determination of compaction of granular compacted material is more properly determined using relative density. When determining compaction based on relative density, a target relative density value of 65% or greater is suggested as a minimum value to achieve a sufficiently dense compacted material.

2. It has been shown throughout this research that moisture is an important factor in utility cut restorations. It has also been shown that much of the granular backfill material placed is at or near the bulking moisture content. It is recommended that granular backfill for utility cut restorations be constructed at moisture contents exceeding the bulking moisture content region for the particular backfill used. This can be achieved by watering the the material onsite. The material as placed will then overcome the collapse potential that could be induced on the pavement patch as a result of infiltration or a rise in the groundwater table. Based on the results of the tests reported herein, granular backfill materials placed in this manner will achieve the recommended 65% relative density.

3. It was observed in the field studies that instrumentation and quality control were rarely used to ensure standards and proper construction procedures were being met. Due to regulatory concerns, the use of the nuclear density gage for density control into the future is considered unlikely. The DCP provides an alternative density control method; however, correlations between the DCP and dry density would need to be established for specific backfill materials.

4. The zone of influence has been shown to be a critical factor in the construction of these utility trenches. To compensate for the zone of influence effects on utility cut restorations, it is recommended that a pavement cutback of two to three feet laterally beyond the limit of the trench excavation be constructed. The pavement cutback and excavated area should be recompacted before the pavement surfacing is placed. To compensate for the zone of influence and to provide bridging over the trench backfill materials it is recommended that T-sections be used in repairing utility cuts. Although monitoring is continuing on the T-sections installed in Ames, at this time it is recommended that T-sections consist of a cutback laterally three feet from the edge of the trench excavation and that particular attention be paid to the upper three feet of the recompacted material. This upper three-foot zone can be constructed of either granular

fill material or native cohesive materials, provided that proper moisture and density is achieved in the materials. Cohesive matierals placed in the upper three feet should be placed at a minimum of 95% of Standard Proctor density and within two percentage points of optimum water content.

Future Research

Continued research should monitor the performance of the constructed trial trenches. According to survey results and previous studies, a restored trench will begin to show signs of settlement as early as after two years. Therefore, to accurately determine the performance of the trenches, monitoring should continue for a minimum of two years.

It would be desirable to monitor the change in moisture content, the frost depth, and the stresses around the pipe in the utility cut region, as well as under the pavement in the cut region and the surrounding undisturbed pavement. This will help in understanding the mechanisms of pavement settlement, the difference in the response between backfill materials and native subgrade when subjected to freeze-thaw, and the changes of stresses on the pipe as a result of freezing.

INTRODUCTION

Utility cuts are made in completed pavement sections to install electric, water and wastewater utilities, as well as drainage pipes under roadways. Utility cuts are also made to repair existing utilities. Once a cut is made, a restoration is constructed, resulting in a patched surface on the pavement. Cuts not only disturb the original pavement, but also the base course and subgrade structure around the cut. Once a utility is repaired and in place, the cut is backfilled, compacted and surfaced. If the backfill material is not suitable for the site conditions or not properly installed, this material will begin to settle relative to the original pavement. According to the Department of Public Works City and County of San Francisco (1998), utility cuts have the greatest damaging impact on newly paved streets, and therefore reduce the roadway life of these new pavements considerably. In some cities, millions of dollars are spent each year on maintenance and repairs of utility cuts made in pavements (APWA 1997). With the continual growth and need for repair of utilities, this issue is becoming a larger problem and further studies are needed to reduce or prevent the resulting damage.

Problem Statement

Pavement settlement occurring in and around utility cuts is a common problem that draws significant resources for maintenance. Recently, a survey was conducted to identify factors that contribute to the settlement of utility cut restorations in pavement sections throughout Iowa. Survey responses were received from seven cities in Iowa, with responses indicating that the current methods of repair provide satisfactory results. However, the responses also stated that in most cases, utility cut repairs generally last two years or less before problems arise, leading to future maintenance and repair needs. To further investigate the problem, site visits were made to both define and observe factors contributing to a poorly performing restoration.

The amount of distress and damage resulting from a pavement cut may be subjective, since a majority of the survey results indicate a low percentage of utility cuts performing poorly. However, through city visits made throughout Iowa, the existence of poorly performing restorations is evident in several roadways. In many cases, differential settlement occurs and subsequently reduces the life of pavements in and around utility cuts. Two examples of differential settlement are documented below, one each in asphalt and concrete surfaced pavements.

In Ames, Iowa, a utility cut in an asphalt-surfaced pavement on the corner of Wilson Avenue and 16th Street resulted in noticeable settlement (see Figure 1). The trench is 14 feet (4.3 m) long and 25.8 feet (7.9 m) wide, with elevation shots taken on the centerline as shown in Figure 1. Figure 2 shows a cross-section of the elevation shots taken on the restoration and the noticeable settlement difference that has developed since construction of the patched utility cut. This figure illustrates the effect this restoration is having on the site, with considerable settlement occurring around the perimeter of the trench, as well as near the water main valve. The perimeter of the trench currently has a 1.1-inch (2.8 cm) elevation drop between the assumed trenching excavation limits and existing pavement, indicating significant settlement on the patched or reconstructed site.

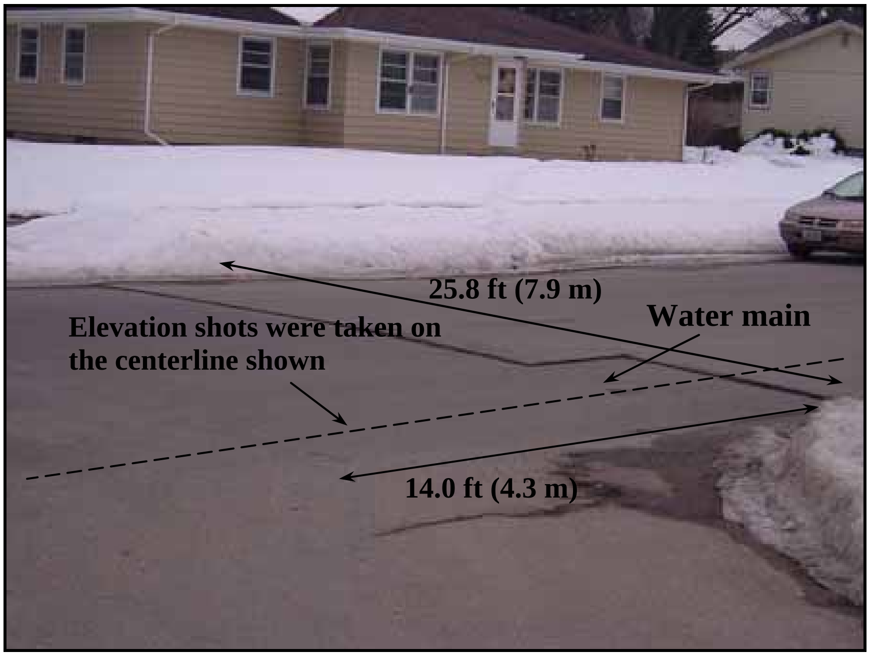


Figure 1. Poorly performing utility cut in asphalt pavement

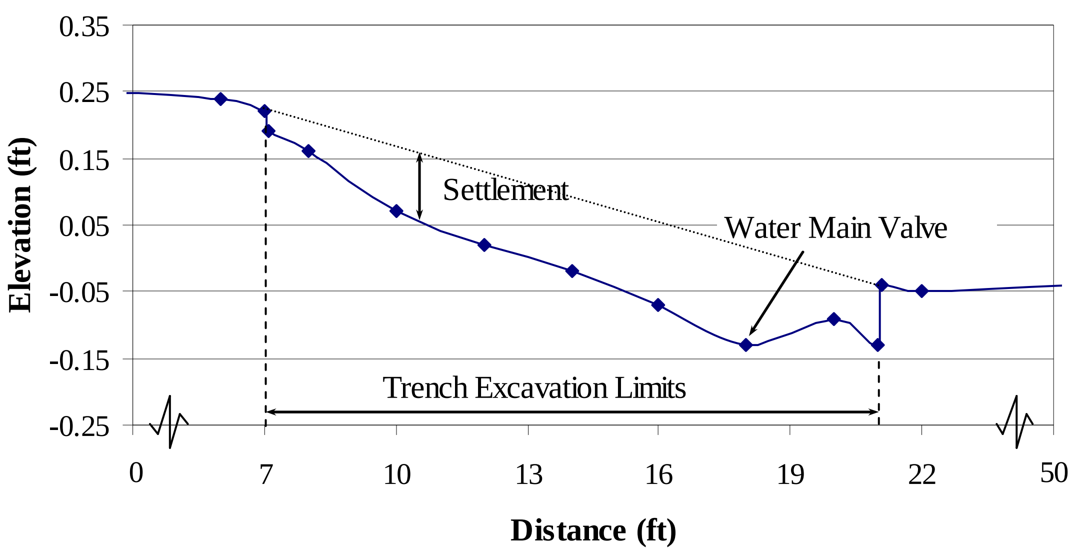


Figure 2. Settlement profile of poorly performing utility cut in asphalt pavement

In Cedar Rapids, Iowa, a poorly performing utility cut in concrete pavement was documented and evaluated as a result of visible settlement and damage occurring in and around the pavement cut. The utility cut shown in Figure 3 is located near the intersection 12th Street SW and 21st Avenue SW on 12th Street SW. The patch is 3.6 feet (1.1 m) long and 8.3 feet (2.5 m) wide, with elevation shots taken along the centerline of the trench, as shown in Figure 3. Elevation differences of 0.12 inches (0.30 cm) and 0.48 inches (1.22 cm) were measured along the edge of the assumed excavation limits of the utility cut (see Figure 4). With nearly 0.5 inches (1.3 cm) of difference in elevation, this amount of settlement was noticeable in a moving vehicle.

Utility cuts, specifically to repair water main breaks, are made throughout the year. Breaks that occur in the winter months are generally surfaced with a temporary cold patch installed until weather conditions improve for placing of a permanent pavement surface. Figure 5 shows an example of a utility cut constructed by a private contractor in the winter that has yet to receive a permanent asphalt surface. At the time this picture was taken, the patch was said to be three years old. With the deterioration of this temporary patch, visible map cracking can be observed in Figure 5.

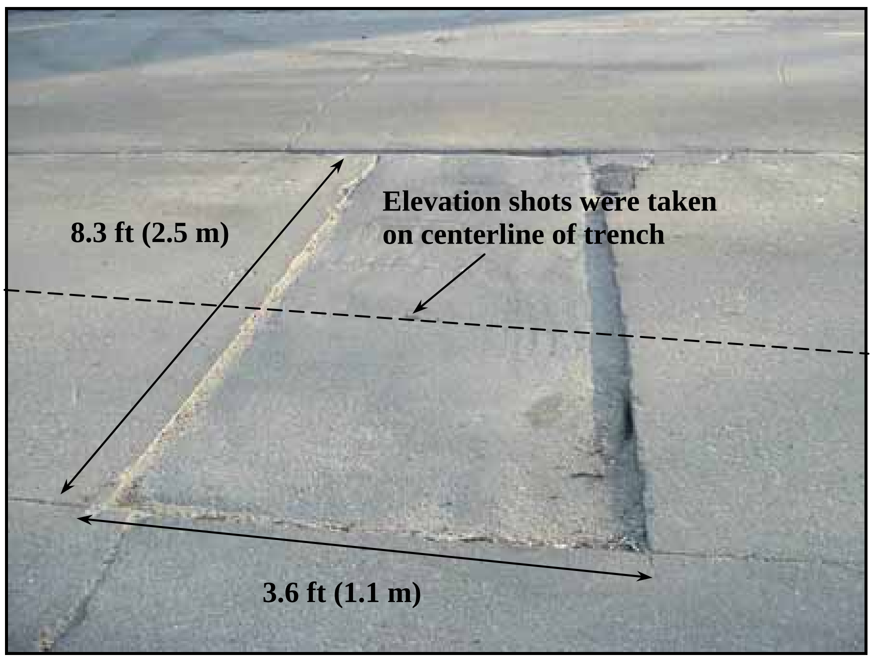


Figure 3. Poorly performing utility cut in concrete pavement

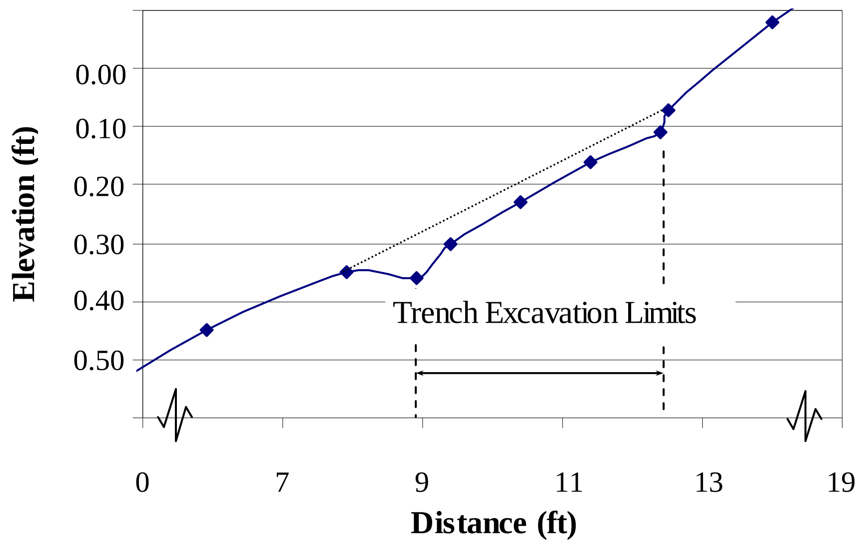


Figure 4. Settlement profile of poorly performing utility cut in concrete pavement

Figure 5. Temporary cold patch in Cedar Rapids, with an estimated age of three years

During the site visits, it was observed that in one city, utility cuts were repaired by placing asphalt near the edge of the concrete surfaced cut to compensate for the differential settlement. Applying this technique decreases the settlement impact felt by a driver; however it also decreases the aesthetic appearance of the existing roadway (see Figure 6).

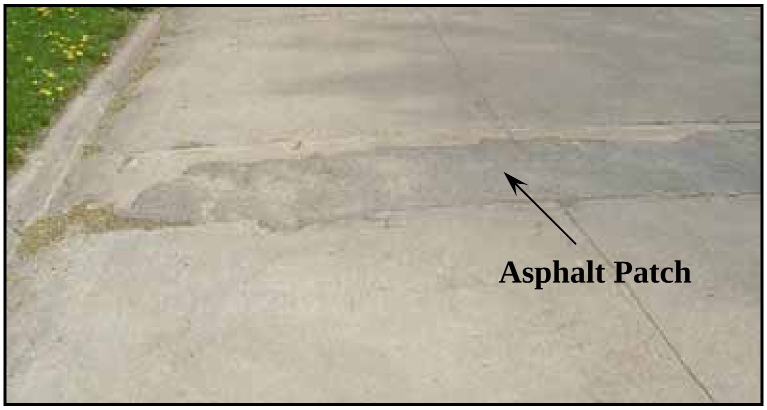


Figure 6. Asphalt patch on top of concrete patch to "repair" the settlement problem

Natural factors play a role in the performance of a utility cut. For example, during an excavation of a water main break, adverse conditions occur such as that shown in Figure 7. As a result of the break, material becomes saturated and weak and begins to slough off. This in turn forms large voids underneath the existing material surrounding the cut, making adequate compaction difficult. Other problems that may arise during the reconstruction of the trench include large lift thicknesses, improper compaction, and lack of moisture control.

Figure 7. Material sloughing off the edges of the trench

Utility cut settlement in both concrete and asphalt pavement was observed in several cities throughout Iowa. Observed problems include settlement both in and around the excavated area and pavement separation. Field visits and observations of in-service utility cuts noted above indicate that problems associated with these utility cuts do exist. This study's focus was based on cuts made in existing pavements; however, practices and recommendations found in this research can be applied to the installation of new utilities as well.

Research Objectives

Poor performance of pavements over and around utility trenches on local and state road systems often cause unnecessary maintenance problems due to improper backfill placement (i.e., under compacted, too wet, too dry). The cost of repairs resulting from poorly performed utility cut restoration can be avoided or reduced with an understanding of proper material selection and construction practices. Current utility cut and backfill practices vary widely across Iowa and result in a range of maintenance problems. The objective of this research is to improve utility cut construction practices, with the goal of increasing the pavement patch life at an affordable cost and thereby reduce maintenance of the repaired areas.

Research Methodology

This study is organized according to the research tasks conducted throughout this study. A literature review was initially completed to become familiar with current field practices as well as developing research in the area of utility cuts. A survey was distributed to several city officials in Iowa to define problems specific to Iowa. Site visits were made for observations and documentation of practices currently conducted in the field. Additional field testing was then completed to determine material compaction properties, as well as a nondestructive monitoring technique to determine pavement system performance. Samples of backfill material were obtained during the site visits for further laboratory analysis, and finally conclusions and recommendations were developed.

LITERATURE REVIEW

Introduction

Utilities, such as gas, water, telecommunications, and sanitary and storm sewers, require an excavation for the installation of the pipes or lines. The number of utilities placed underground continues to increase with the desire to hide utility lines for reasons such as aesthetics, factors contributed as a result of weather, and safety purposes (APWA 1997).

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, aesthetic value, and 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. Bodocsi 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%. Statistical data reported by the Department of Public Works in San Francisco (1998) show that the pavement condition rating decreases as the number of utility cuts made increases (see Figure 8). The rating system is based on conclusions from a panel of Department of Public Works staff and data from a Pavement Management and Mapping System developed for the city of San Francisco considering factors such as the pavement condition, age of pavement surfacing, street area, and the number of utility cuts (Department of Public Works in San Francisco 1998). For example, the pavement condition score for a newly constructed pavement is reduced from 85 to 64 as the number of utility cuts increase to ten or more for pavement less than five years old.

Poor performance of pavements around utility trenches on local streets and state highway systems often require maintenance due to improper backfill placement (i.e., improper backfill, under compacted, too dry, too wet). 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.

This literature review discusses various aspects and important factors of utility cut restoration and susceptibility to pavement deterioration. Factors that have been studied and discussed below include (1) causes of utility cut failures, (2) trench shapes and sizes, (3) backfill materials (traditional and non-traditional materials), (4) compaction methods and equipment, (5) quality control and quality assurance, (6) the economic impact of utility cuts, and (7) permit fees.

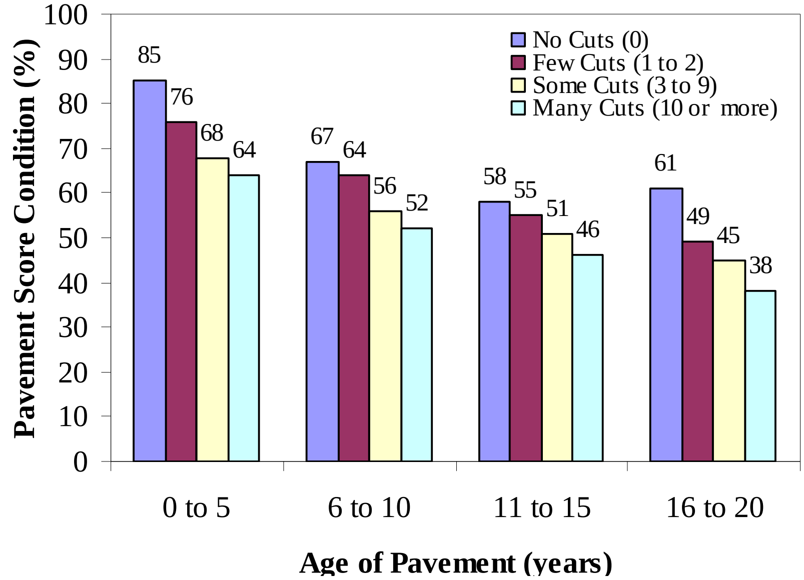


Figure 8. Utility cut effects on pavement condition (from the Department of Public Works City and County of San Francisco 1998)

Typical Utility Cut Patching Failures

Three typical pavement patch failures occur within the first year or two after the initial utility cut has been made and the pavement patch has been completed.

1. The pavement patch settles, resulting in vehicles hitting a low spot, as well as the collection of moisture, which can induce additional settlement. Typically, settlement is caused either by a combination of a poor compaction effort in natural soils or other backfill materials which have been or are exposed to wet or frozen conditions or the use of unsuitable backfill materials. A study conducted by Southern California Gas Company concluded that the top 2 feet (0.6 meters) of a backfilled excavation experiences the most settlement in a trench (APWA 1997).
2. The pavement patch rises forming a "hump" over the utility cut area, particularly in winter freeze/thaw conditions due to frost action. Frost action requires three factors: (1) soils susceptible to frost (i.e., silty soils), (2) a high water table, and (3) freezing temperatures (Monahan 1994). These factors all contribute to pavement heaving in that cold temperatures are needed for the development of the frost line, which in turn penetrates the subgrade forming ice lenses with moisture in the soil. These ice lenses continue to grow due to capillary rise and ground water table fluctuation, therefore increasing the size of ice lenses and forming visible heave on pavements (Spangler and Handy 1982).
3. The pavement adjacent to the utility patch starts settling and fails, leading—in time—the patch itself to fail. This condition normally results when the natural soil adjacent to the utility trench and the overlying pavement section has been weakened by the utility excavation, as shown in Figure 9. This weakened zone around the utility cut excavation

is called the “zone of influence” and extends up to 3 feet (1 m) laterally around the trench perimeter (The Department of Public Works City and County of San Francisco 1998).

The causes of the three types of failures discussed above depend on factors such as quality and type of restoration adopted, backfill materials used and their compaction, and the age and condition of the existing pavement before restoration. Ghataora and Alobaidi (2000) concluded from Falling Weight Deflectometer deflection data, that certain areas of a utility cut have a greater amount of settlement than others. For example, longitudinal trenches with a granular backfill material settled more at the edge than in the middle. Futhermore, trenches with transverse cuts show a majority of the settlement occurring in the wheel paths rather than edges. Both longitudinal and transverse cuts showed the greatest amount of settlement occurring in the first two months after the repair.

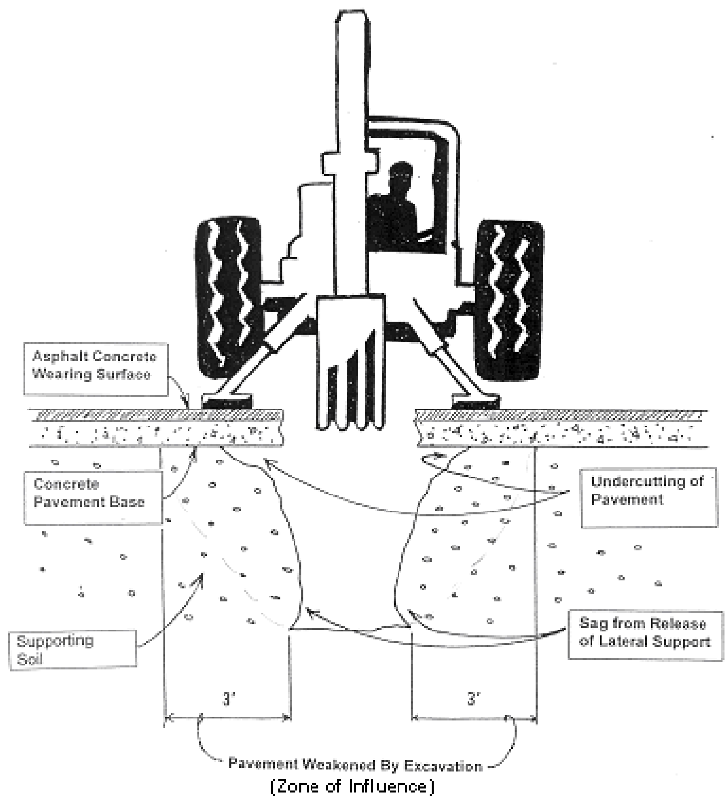


Figure 9. Overstressing of the pavement and natural materials adjacent to the trench (modified from the Department of Public Works City and County of San Francisco 1998)

Certain improvements of various practices may prevent settlement from occurring as quickly in utility trenches; however, a discussion of current practices conducted is necessary first.

Current practices

A number of studies have been conducted on utility cut repair techniques in a variety of states. Research has been conducted at universities and agencies to improve backfill and trenching techniques. In this section, trench and trenchless excavations, the zone of influence, backfill materials, compaction requirements and quality control and quality assurance are further discussed.

Trench and Trenchless Excavations

The size of an excavation depends on (1) pipe diameter, (2) compaction requirements, and (3) the type of backfill material chosen. The excavation size of a trench can vary from very narrow and confined, to wide and open spaces. Generally, as the trench width increases, the project cost will increase as well. This cost increase may be a result of added labor, materials, and/or equipment needed for construction. A trench that is too narrow, however, may result in poor compaction due to the confinement and mobility restrictions of compaction equipment such as backhoes. Small pipe diameters generally result in a minimum trench width equivalent to the smallest bucket size that a contractor can use to dig a trench. The maximum width value is determined by measurements corresponding to the bottom of the trench and if applicable, the area including sheeting and bracing (Polk County Public Works 1999). The depth of a trench depends on factors such as location and slope needed for pipe installation or repair.

Trenching excavations can be eliminated for new utilities by using trenchless technology. However, this method may eventually require an additional smaller trench to be constructed for connection to the existing pipeline and therefore is not a completely trenchless method (Department of Public Works City and County of San Francisco 1998). Khogali and Mohamed (1999) note that a significant advantage of trenchless technology is that there is very little disturbance to traffic flow. Iseley and Gokhale (1997) add that in addition to minimal traffic disturbance, trenchless technology generally does not require a large construction crew, has less of an impact on businesses, decreases in noise, has less air pollution, as well as less material to haul away. Iseley and Gokhale (1997) indicated that in a survey given to several DOTs, trenchless methods had the potential for the formation of sinkholes, heaving, leaking of drilling fluid, and drilling tools puncturing the pavement surface, all occurring as a result of trenchless technology. Trenchless methods have also been known to damage existing underground utilities (APWA 1997 and Department of Public Works City and County of San Francisco 1998).

Effect of the Zone of Influence

The zone of influence, illustrated in Figure 9, plays a critical role in road deterioration around utility cuts. Traffic loads produce a greater deflection in this critical area as a result of a decreased amount of support from the soil surrounding the excavation perimeter and therefore inducing early pavement deterioration (Arudi et al. 2000). A study conducted in Kansas City, Missouri concluded that in two years, the structural capacity around the perimeter of the trench decreased 50% to 65%, with respect to the central region of the trench (APWA 1997). To determine the extent of this zone of influence, non-destructive deflection tests have been performed. Peters (2002) reported considerable strength reduction along the perimeter of utility cut excavations, as a result of non-destructive deflection testing. Peters (2002) stated that 23 of 24 trenches studied in Salt Lake City, Utah showed a large amount of strength loss within the zone of influence. To reconstruct the soil strength and stiffness within this zone, a T-section, where pavement is cut back two to three feet adjacent to the trenched area, is constructed. Figures 10 and 11 illustrate the dimensional requirements of the T-section cross-section used in Salt Lake City, Utah (Peters 2002). Washington DOT (WSDOT) uses a 2-foot (0.61 m) cutback, unless the trench is located in a confined area where this distance is not feasible (www.wsdot.wa.gov).

When using a controlled density fill (i.e., flowable fill), a cutback should be a maximum of 1 foot (0.31 m) on each side of the trench according to WSDOT (www.wsdot.wa.gov). Bodocsi (1995) states that after analyzing several trenches in Cincinnati, Ohio, a typical trench size of 5 feet (1.5 m) long by 4 feet (1.2 m) wide, had a zone of influence area extending 3 feet (0.91 m) on all sides of the trench for asphalt and macadam pavements. APWA (1997) indicated very little damage occurred in 9-inch-thick (22.9 cm) concrete pavements, except when the trench was constructed near a curb or slab edge. Figure 12 illustrates typical T-sections showing minimum widths and depths recommended by APWA (1997). By constructing a T-section, stresses imposed on the pavement may decrease by incorporating undisturbed soil from around the excavation and in turn adding extra support to the pavement patch (APWA 1997). If a T-section or cutback is constructed, a study in California suggests conducting the cutback after the trench has been backfilled (Department of Public Works City and County of San Francisco 1998). This may reduce the amount of stress release incorporated with an open trench. Table 1 compares various city and state cutback distances.

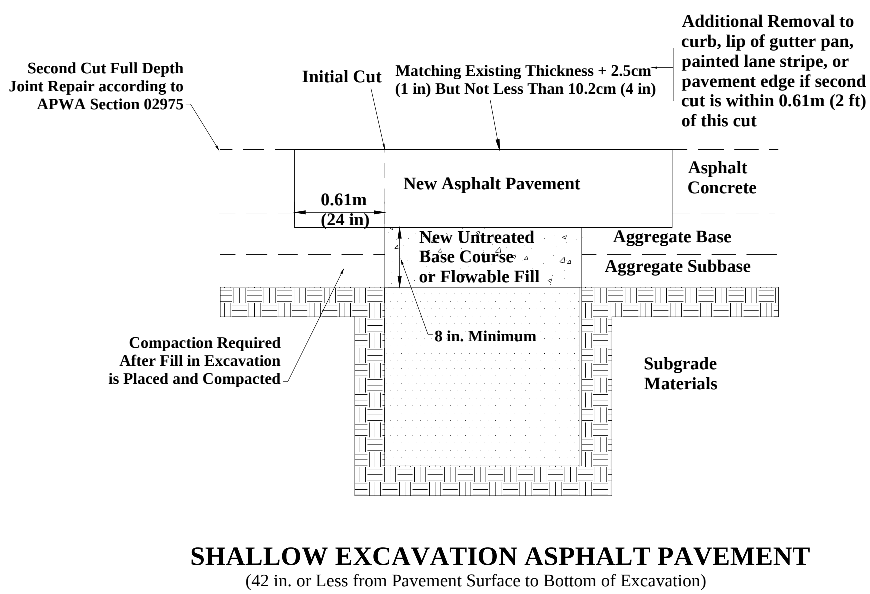


Figure 10. Salt Lake City T-section cross section for a shallow excavation (Peters 2002)

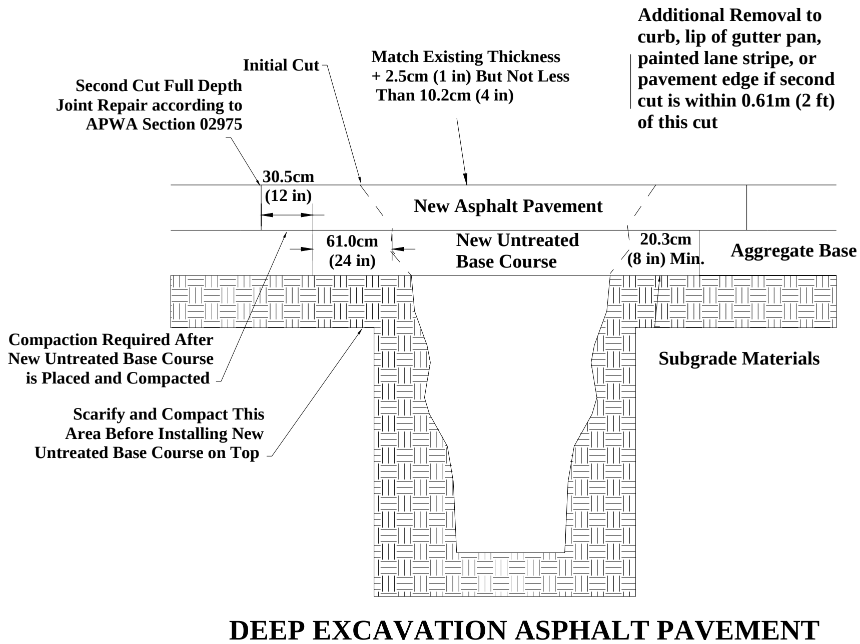


Figure 11. Salt Lake City T-section cross section for a deep excavation (Peters 2002)

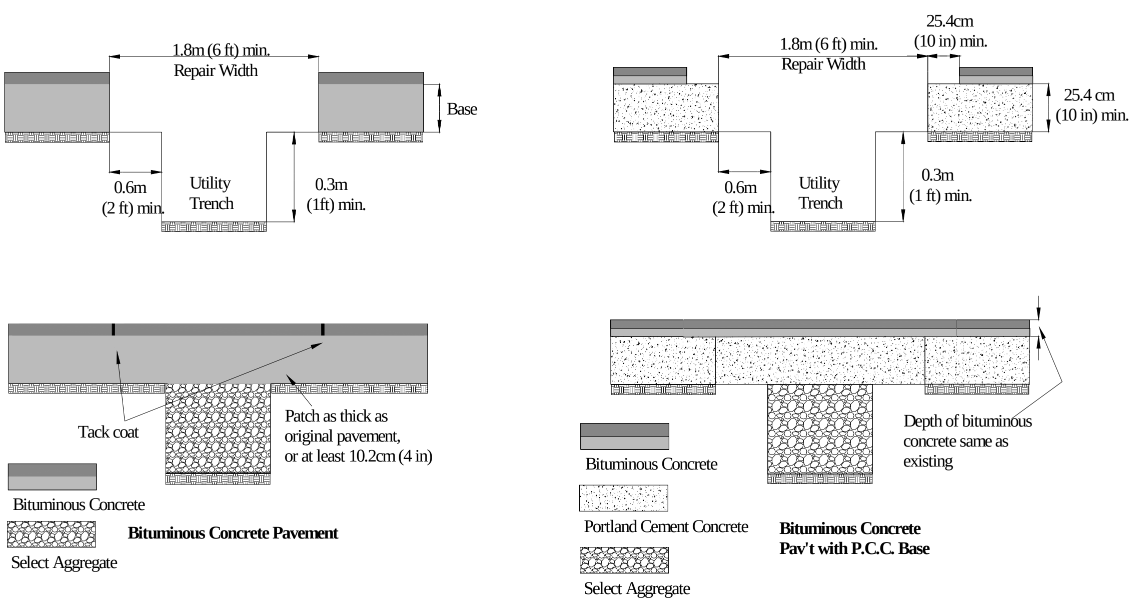


Figure 12. T-section cross sections (APWA 1997)

Table 1. T-section cutback comparison
(Peters 2002, www.wsdot.wa.gov, and Bodocsi 1995)

State/City	Cutback distance from perimeter per trench side in feet (meters)
Salt Lake City, Utah	2 to 3 feet (0.61m to 0.91m)
Washington State (granular)	2 feet (0.61m)
Washington State (flowable)	1 foot (0.30m)
Ohio	3 feet (0.91m)

APWA (1997) reports that some cities are constructing larger cutbacks extending to a centerline or gutter pan of a street, and therefore providing a smooth transition from undisturbed to disturbed pavement sections. Cities such as Seattle and Indianapolis require this type of cutback in order to prevent weak pavement areas forming in smaller patches (APWA 1997). Peters (2002), in a study conducted in Salt Lake City, concluded that when a patch is within 2 feet (0.61 m) of another patch on a road, pavement should be removed to the curb, gutter, striping line or other utility cut on asphalt pavements.

Other cities have indicated similar requirements. For example, in a 15-foot section (4.57 m), if a minimum of three patches are made, the entire section must be removed in Worcester, Massachusetts and Chicago, Illinois requires no pavement disturbance within 16 feet (4.88 m) of two patches (APWA 1997). When several trenches in Ohio are excavated in close proximity to each other, Bodocsi et al. (1995) suggests a distance of 7.5 feet (2.29 m) between trenches to compensate for the zone of influence.

Backfill Materials

The type of trench backfill material (i.e., cohesive vs. noncohesive) selected for a restoration can impact future settlement. Cohesive clay type backfill materials require moisture control to reach maximum density, worker experience, extensive compaction monitoring, and can be difficult to compact, specifically in tight trenches (APWA 1997). APWA (1997) indicates that a study conducted in California monitored 67 trenches where backfill material consisted of native material. Of the 67 trenches monitored, only four trenches, consisting of granular native materials, reported no settlement (APWA 1997). A conclusion was made that granular native materials with a high compacted density may be suitable as a backfill material (APWA 1997).

For many reasons such as those stated above, generally cohesionless granular materials are used as backfill material in trenches, as opposed to native cohesive clay soils. Furthermore, granular materials can be compacted more easily (APWA 1997). A well-graded granular material containing nonplastic fines has the ability to produce a high density in the field, as a result of these fines filling areas where air voids and water would have existed (Monahan 1994). However, the presence of many fines can result in poor drainage and lead to poor compaction and frost action (Monohan 1994). According to Table 2, a well graded, gravel-sand mixture with little or no fines is most suitable for compacted fills in roadways, with and without frost heave potential.

Jayawickrama et al. (2000) states that many State Departments of Transportation (DOTs) require granular material that classifies as an A-1 or A-3 according to AASHTO M145 (see Table 3). Iowa DOT suggests 100% passing the 75 mm (3-inch) sieve, 20% to 100% passing the 2.36 mm (#8), and 0% to 10% passing the 0.075 mm (#200) sieve. ASTM D 2321-89 provides a standard for thermoplastic pipe installation and Table 4 summarizes the properties of the aggregate material recommended by ASTM D 2321-89. This table shows that material classified as Class I and II according to ASTM D 2321-00 are all non plastic, cohesionless materials.

The Statewide Urban Design Standards (SUDAS) of Iowa recently recommended a new storm sewer and sanitary sewer Class I gradation for bedding and backfill, approving use of materials such as gravel, crushed Portland Cement Concrete, or crushed stone material. The gradation consists of 100% passing sieve 1.5 inch (37.5 mm), 95% to 100% passing the 1-inch (25 mm) sieve, 25% to 60% for the 0.5 inch (12.5 mm) sieve, and 0% to 10% for #4 (4.75 mm) sieve; as opposed to the old gradation, where 100% passing sieve 1.5-inch (37.5 mm), 95% to 100% passing the 1.0 inch (25 mm) sieve, 35% to 70% for the 0.75 inch (19.0 mm) sieve, 25% to 50% for the 0.5-inch (12.5 mm) sieve, 10% to 30% for the 3/8-inch (9.5 mm) sieve, and 0% to 5% for #4 (4.75 mm) sieve (SUDAS 2003) (see Table 5). This change was based on the need to obtain a gradation that limestone producers can make readily available across Iowa.

Table 2. Relative desirability of soils as compacted fill (modified from NAVFAC 1986)

Group Symbol	Soil Type	Relative Desirability for Various Uses (No. 1 is Considered the Best, No. 14 Least Desirable)		
		Roadways		
		Fills		
		Frost Heave Not Possible	Frost Heave Possible	Surfacing
GW	Well graded gravels, gravel-sand mixtures, little or no fines	1	1	3
GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	3	3	-
GM	Silty gravels, poorly graded gravel-sand-silt mixtures	4	9	5
GC	Clayey gravels, poorly graded gravel-sand-clay mixtures	5	5	1
SW	Well graded clean sands, gravelly sands, little or no fines	2	2	4
SP	Poorly graded sands, gravelly-sands, little or no fines	6	4	-
SM	Silty sands, poorly graded sand-silt mix	6	10	6
SC	Clayey sands, poorly graded sand-clay-mix	7	6	2
ML	Inorganic silts and vary fine sands, rock flour, silty or clayey fine sands with slight plasticity	10	11	-
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	9	7	7
OL	Organic silts and organic silt-clays, low plasticity	11	12	-
MH	Inorganic silts, micacaous or diatomaceous fine sandy or silty soils, elastic silts	12	13	-
CH	Inorganic clays of high plasticity, fat clays	13	8	-
OH	Organic clays of medium high plasticity	14	14	-

- Not appropriate for this type of use

Table 3. Classification of soils and soil-aggregate mixtures (modified from AASHTO M145-91)

General Classification	Granular Materials (35% or Less Passing sieve #200)						
	A-1			**A-2**			
Group Classification	**A-1-a**	**A-1-b**	**A-3**	**A-2-4**	**A-2-5**	**A-2-6**	**A-2-7**
Sieve analysis,% passing	--						
2.00 mm (No. 10)	50 max	--	--	--	--	--	--
0.425 mm (No. 40)	30 max	50 max	51 min	--	--	--	--
75 μm (No. 200)	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	Excellent to Good						

Table 4. Classes I and II of ASTM backfill material specifications (Jayawickrama et al. 2000)

Soil Class	Soil Class	Soil Group Symbol D2487	Description	Percent Passing Sieve Sizes			Atterberg Limit		Coefficients	
				1 1/2 in. (40mm)	No. 4 (4.75mm)	No. 200 (0.075mm)	LL	PI	Uniformity Cu	Curvature Cc
IA	Manufactured Aggregates, open-graded, clean	None	Angular, crushed stone or rock, crushed gravel, broken coral, crushed slag, cinders or shells; large void content, contain little or no fines	100%	≤10%	<5%	Non Plastic			
IB	Manufactured, Processed Aggregates, dense-graded, clean	None	Angular, crushed stone (or other Class IA materials) and stone/sand mixtures with gradations selected to minimize migration of adjacent soils; contain little or no fines	100%	≤50%	<5%	Non Plastic			
Class II	Coarse-Grained Soils, clean	GW	Well-graded gravels and gravel-sand mixtures; little or no fines	100%	<50% of "Coarse Fraction"	<5%	Non Plastic		>4	1 to 3
		GP	Poorly-graded gravels and gravel-sand mixtures; little or no fines						<4	<1 or >3
		SW	Well-graded sands and gravelly sands; little o no fines		>50% of "Coarse Fraction"				>6	1 to 3
		SP	Poorly-graded sands a gravelly sands; little o no fines						<6	<1 or >3
	Coarse-Grained Soils, borderline clean to w/fines	e.g. GW-GC, SP-SM	Sands and gravels which are borderline between clean and with fines	100%	Varies	5% to 12%	Non Plastic		Same as for GW, GP, SW and SP	

Table 5. Iowa DOT and SUDAS gradations

		Iowa DOT Backfill		SUDAS Specification	
		Gradation-Percent Passing			
Pipe Size		Granular Backfill		Class 1	
Sieve No.	Sieve Size (mm)	UL	LL	UL	LL
3"	75	100	100	-	-
1 1/2"	37.5	-	-	100	100
1"	25	-	-	100	95
1/2"	12.5	-	-	60	25
#4	4.75	-	-	10	0
#8	2.36	100	20	-	-
#200	0.075	10	0	-	-
		UL= Upper Limit	LL= Lower Limit		

Backfill Lift Thicknesses

Backfill lift thicknesses are critical in achieving a well-constructed utility cut. For trenches, Monahan (1994) suggests the use of 3 to 6 inch thick (8 cm to 15 cm) lifts. APWA (1997) indicates that the thickness of backfill lifts generally range from four inches (10 cm) to 12 inches (31 cm), with 6 inches (15 cm) being the most common depth, and 12 inches (31 cm) the next most common. Generally the deeper the backfill lift, the more difficult it is to compact properly. Minnesota DOT, California DOT and Ohio DOT, lift specification for pipe culverts, indicates that loose lifts should not exceed 8 inches (20 cm). Washington DOT specifies placing material in six-inch (15 cm) lifts. Iowa, Florida, and Illinois use compacted lifts of 6 inches (15 cm). However, Florida states that in the top zone (area near the surface), 12 inches (31 cm) may be used if proof of proper density can be obtained.

Figure 13 shows a typical trench section for granular backfill in Iowa, according to SUDAS specifications of Iowa. This figure illustrates the various lifts of backfill material required by the standard. In Figure 13, the trench width at the top is represented as 8d, and the bottom width of EW (excavation width). SUDAS recommends lift thicknesses of 6 inches (15 cm) in the haunch support, primary and secondary backfill areas. The final trench backfill should be placed in loose lifts of no greater than 12 inches (31 cm).

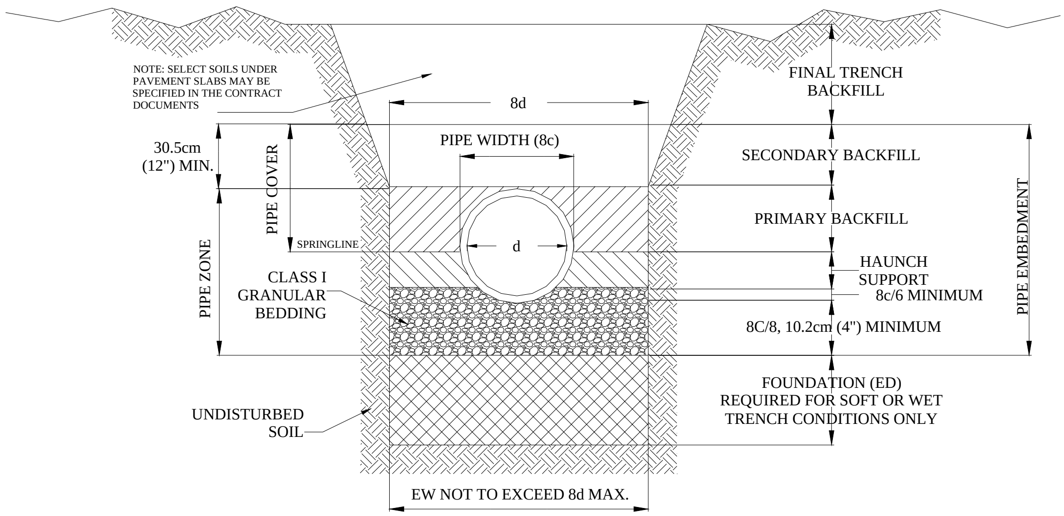


Figure 13. Typical trench cross section (SUDAS 2004)

Hancor Inc. (2000) suggests that backfill around thermoplastic pipes be placed in layers of 4 inches (10 cm) to 6 inches (15 cm) in the haunching area to support the pipe from traffic loads. The initial backfill is placed on top of the haunching layer up to at least 6 inches (15 cm) above the top outside diameter of the pipe. The initial backfill helps in distributing the load and in restraining movement of the pipe. The final backfill layer should be a minimum of 4 inches (15 cm) for pipe diameters of 4 inches (10 cm) to 48 inches (122 cm) and for pipe with diameters between 54 inches (137 cm) and 60 inches (152 cm), a minimum final backfill depth of 12 inches (31 cm) is recommended extending from the initial backfill layer to the surface. Generally native material excavated from the trench, would be sufficient for use in the final backfill layer. Figure 14 illustrates these different backfill layers.

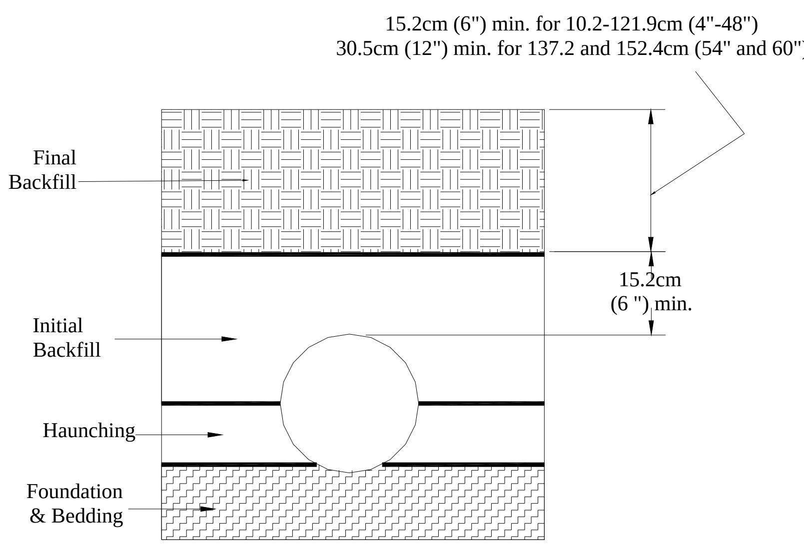


Figure 14. Typical backfill cross section for thermoplastic pipes (Hancor Inc. 2000)

Compaction Methods

Soil compaction is another key factor in the construction of a quality utility cut and is defined as "the expulsion of air from the soil mass" (Monahan 1994). As Holtz and Kovacs (1981) explain, "the objective of compaction is to stabilize soils and improve their engineering behavior." NAVFAC (1986) describes compaction as a method of lowering permeability, frost penetration, and settlement, as well as increasing material strength and controlling expansion ability. Four significant factors affect compaction of a material: (1) dry density, (2) moisture, (3) compaction equipment, and (4) soil properties (Holtz and Kovacs 1981). NAVFAC (1986) has generated a table of typical values of properties such as dry density, optimum moisture content, permeability, CBR values and subgrade modulus for a variety of soil types which all contribute to or define proper compaction (see Table 6).

A majority of compaction specifications base compactive effort on Proctor results, which is appropriate for cohesive materials (Monahan 1994). However, standard Proctor is not recommended for use as a compaction requirement in granular soils because of the inability to obtain a clear relationship between moisture and density (Amini 2003). Spangler and Handy (1982) explain that the use of relative density, rather than standard Proctor, is necessary to achieve proper compaction in granular materials because of the ability to obtain correct density characteristics and as opposed to underestimated values. Figure 15 illustrates the comparison of relative density values and Proctor tests for a cohesionless material and Table 7 defines material compaction classifications based on relative density values.

Table 6. Typical properties of compacted soils (modified from NAVFAC 1986)

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, percent	Typical Coefficient of Permeability ft/min	Range of CBR values	Range of Subgrade Modulus, k lb/cu inches
GW	Well graded clean gravels, gravel-	125-135	11-8	5 x 10^{-2}	40-80	300-500
	sand mixtures					
GP	Poorly graded clean gravels, gravel	115-125	14-11	10^{-1}	30-60	250-400
	sand mixtures					
GM	Silty gravels, poorly graded gravel-	120-135	12-8	$>10^{-6}$	20-60	100-400
	sand-silt					
GC	Clayey gravels, poorly graded	115-130	14-9	$>10^{-7}$	20-40	100-300
	gravel-sand-clay					
SW	Well graded clean sands,	110-130	16-9	$>10^{-3}$	20-40	200-300
	gravelly sands					
SP	Poorly graded clean sands, sand-	100-120	21-12	$>10^{-3}$	10-40	200-300
	gravel mix					
SM	Silty sands, poorly graded sand-	110-125	16-11	5 x 10^{-5}	10-40	100-300
	silt mix					
SM-SC	Sand-silt clay mix with slightly	110-125	15-11	2 x 10^{-6}	5-30	100-300
	plastic fines					
SC	Clayey sands, poorly graded sand-	105-125	19-11	5 x $>10^{-7}$	5-20	100-300
	clay-mix					
ML	Inorganic silts and clayey silts	95-120	24-12	$>10^{-5}$	15 or less	100-200
ML-CL	Mixture of inorganic silt and clay	100-120	22-12	5 x $>10^{-7}$	……	
CL	Inorganic clays of low to medium	95-120	24-12	$>10^{-7}$	15 or less	50-200
	plasticity					
OL	Organic silts and silt-clays, low	80-100	33-21	……	5 or less	50-100
	plasticity					
MH	Inorganic clayey silts, elastic silts	70-95	40-24	5 x $>10^{-7}$	10 or less	50-100
CH	Inorganic clays of high plasticity	75-105	36-19	$>10^{-7}$	15 or less	50-150
OH	Organic clays and silty clays	65-100	45-21	……	5 or less	25-100
Notes:	1. All properties are for condition of "Standard Proctor" maximum density, except					
	values of k and CBR which are for "modified Proctor" maximum density.					
	2. Typical strength characteristics are for effective strength envelopes and are					
	obtained from USBR data.					

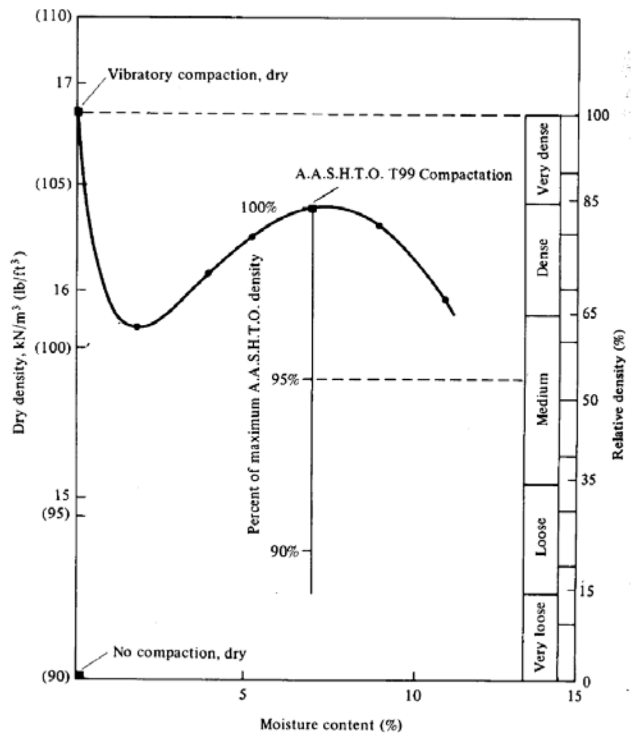


Figure 15. Relative density vs. AASHTO T99 compaction (Spangler and Handy 1982)

Table 7. Relative density classifications (Budhu 2000)

	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

As Figure 15 illustrates, at low moisture contents, granular materials decrease in density, resulting in a concave up density-moisture curve because of a high capillary tensile force between soil particles. However, at this moisture content the soil is very stiff. Once the moisture increases, the soil settles rapidly because of the reduction of capillary tensile forces between soil particles. Spangler and Handy (1982) and Holtz and Kovacs (1981) describe the bulking phenomenon that occurs in granular materials. Bulking is a capillary phenomena occurring in moist sands in which capillary menisci between soil particles hold the soil particles together in a honeycombed structure. This structure can collapse upon the addition of water. Spangler and Handy (1982) explain that the addition of a small amount of water to dry sand, between about 6% to 8%, results in the formation of capillary rings at particle contact. The result is an increase in volume due to an open structure or bulking effect, of up to 25% (Spangler and Handy 1982). This capillary tension maintains the bulking effect until destroyed by the addition of water. Essentially, flooding this type of material will eliminate the bulking effect, but may lead to difficulty in obtaining proper compaction of a material

(Spangler and Handy 1982). Holtz and Kovacs (1981) also indicate that although flooding a granular material induces collapse, flooding the fill can ultimately result in a low relative density because of the excess moisture present and in turn, result in a poor foundation material. When a material is saturated, additional water is added without elimination of air, therefore decreasing the density (Monahan 1994). APWA (1997) indicates that in many cases water compaction (i.e., flooding a material under water and its own weight) of soils, results in natural density values of 85% to 90% compared to a compaction requirement of 95% density.

Laboratory test results and numerical analyses results have been conducted on granular materials and were found to produce similar results in regard to this bulking phenomenon (Gili and Alonso 2002). Gili and Alonso (2002) state that water tension forming between particles stabilizes particles in a loaded chain defined as internal tensioning. This tension therefore provides the stability for preventing a collapse. In the case of roadways, water may be induced to subgrade materials after construction of the trench as a result of factors such as infiltration or seasonal variations in the groundwater table and therefore decreasing the stability of the internal tension. The bulking moisture content region is a critical factor in the settlement of granular materials.

Despite the argument presented above, a majority of compaction standards are according to standard or modified Proctor. Generally, compaction of 95% maximum dry density using standard Proctor is required for backfill materials (APWA 1997). NAVFAC (1986) requires achieving 90% of maximum density using modified proctor and a maximum layer thickness of 8 inches (20 cm) (see Table 8). As Sowers (1979) indicates in Table 9, based on experience, materials have a variety of representative percentage of maximum standard Proctor values needed to achieve good compaction. This table indicates that for a majority of classified materials, beneath the pavement to 3 feet (1 m) below the subgrade, compaction ranging from 97% to 100% standard Proctor is required, and material exceeding 3 feet (1 m) should have a compaction of 94% to 97% required standard Proctor to achieve good compaction.

Table 8. Compaction requirements (modified from NAVFAC 1986)

Fill Utilized for:	Required Density, % of Modified Proctor	Tolerable Range of Moisture About Optimum, (percent)	Maximum Permissible Lift Thickness, Compacted (inches)	Special Requirements
Backfill in pipe or utility trenches	90	-2 to +2	8(+)	Material excavated from trench generally is suitable for backfill if it does not contain organic matter or refuse. If backfill is fine grained, a cradle for the pipe is formed in natural soil and backfill placed by tamping to provide the proper bedding. Where free draining sand and gravel is utilized, the trench bottom may be finished flat and the granular material placed saturated under and around the pipe and compacted by vibration.

Notes: 1. Density and moisture content refer to “Modified Proctor” test values (ASTM D1557)
2. Generally, a fill compacted dry of OMC will have higher strength and a lower compressibility even after saturation.

Table 9. Compaction characteristics (modified from Sowers 1979)

Class	Compaction Characteristics	Maximum Dry Density (tons/m^3)	Value as Temporary Pavement With Bituminous Treatment	Required Compaction % of Standard Proctor Maximum		
				Class 1	Class 2	Class 3
GW	Good: tractor, rubber-tired, steel wheel, or vibratory roller	2.00-2.16	Excellent	97	94	90
GP	Good: tractor, rubber-tired, steel wheel, or vibratory roller	1.84-2.00	Fair	97	94	90
GM	Good: rubber-tired or light sheepsfoot roller	1.92-2.16	Poor to fair	98	94	90
GC	Good to fair: rubber-tired or sheepsfoot roller	1.84-2.08	Excellent	98	94	90
SW	Good: tractor, rubber-tired or vibratoryroller	1.76-2.08	Good	97	95	91
SP	Good: tractor, rubber-tired or vibratory roller	1.60-1.92	Poor to fair	98	95	91
SM	Good: rubber-tired or sheepsfoot roller	1.76-2.00	Poor to fair	98	95	91
SC	Good to fair: rubber-tired or sheepsfoot roller	1.68-2.00	Excellent	99	96	92
ML	Good to poor: rubber-tired or sheepsfoot roller	1.52-1.92	Poor	100	96	92
CL	Good to fair: sheepsfoot or rubber-tired roller	1.52-1.92	Poor	100	96	92
OL	Fair to poor: sheepsfoot or rubber-tired roller	1.28-1.60		--	96	93
MH	Fair to poor: sheepsfoot or rubber-tired roller	1.20-1.60	Very poor	--	97	93
CH	Fair to poor: sheepsfoot roller	1.28-1.68	Not suitable	--	--	93
OH	Fair to poor: sheepsfoot roller	1.12-1.60	Not suitable	--	97	93
Pt	Not suitable		Not suitable	--	--	--
Class 1 Upper 1m (3 feet) of subgrade under pavements						
Class 2 Deeper parts (to 10 m (30 feet)) of fills under pavements						
Class 3 All other fills requiring some degree of strength or compressibility						

In Iowa, SUDAS requires the final trench backfill materials to be compacted to 95% of maximum standard Proctor and the bedding region 90% standard Proctor density. In the primary and secondary layers, Class II (USCS soils classified as GW, GP, SW, and SP, non-plastic and passing 1.5-inch (37.5 mm) sieve should have compaction of 90% standard Proctor and Class III (USCS soils classified as GM, GC, SM, and SC) and IVA (fine grained inorganic soils that are fine grained) compaction of 95% standard Proctor (see Figure 13).

State DOTs compaction specifications for backfills are as follows. Ohio’s structural backfill should be compacted to 96% maximum dry density (www.dot.state.oh.us). Iowa DOT requires 95% standard Proctor for backfill compaction (www.erl.dot.state.ia.us). Florida follows specifications determined by AASHTO T99, method C, where a minimum density of 100% maximum standard density should be obtained (www.dot.state.fl.us). However, for metal and plastic pipes, the cover zone (area around the pipe) to be at least 95% maximum density (www.dot.state.fl.us). California DOT requires a relative compaction of at least 95%

(www.dot.ca.gov). Washington DOT suggests that material which is placed above the pipe zone, be compacted to 95% maximum density (www.wsdot.wa.gov). The pipe zone should be compacted to 90% maximum density (www.wsdot.wa.gov). Table 10 compares these various state compaction requirements.

Table 10. Compaction requirements by state

State	Required Compaction
Florida	M inches of 100% maximum density
Ohio	96% Maximum dry density
California	Minimum 95% relative compaction
Washington	95% Maximum dry density
Iowa	Minimum density of 95%

A study conducted for SoCalGas showed that material compacted at 90% modified Proctor, had settlement ranging from 0 to 1/8 inch, whereas material compacted below 90% modified Proctor, showed settlement up to and exceeding 1/2 inch (APWA 1997). Therefore the study concluded that backfill material compacted at 90% modified Proctor or greater, show little or no settlement. Further studies conducted by Dames and Moore, Inc. for SoCalGas, indicated that a pneumatic rammer should compact a material for seven seconds, every square foot for every four-inch-thick (10.2 cm) lift, in order to obtain a 90% modified Proctor correlation (APWA 1997).

Compaction Equipment

Using the correct equipment for a project is important for achieving correct levels of specified compaction. The type of equipment used for a project may depend on factors such as the type of material, amount of compaction needed, amount of moisture the material contains, and availability of compaction equipment. APWA (1997) lists three types of compactors used for backfilling trenches: (1) ramming, (2) static, and (3) vibratory. The vibratory method provides a more consistent compaction, but a limited amount of vibration should be used because excessive vibration can reverse its effect by loosening the soil (APWA 1997). Jayawickrama et al. (2000) reported different types of compaction equipment used around plastic pipes. The compaction equipment studied included (1) impact hammer, () vibratory plate compactor, and (3) compressed air tamper (see Figure 16). The vibrating plate is best used for granular materials because of its ability to lower friction between sand and gravel, therefore allowing both the machine and material weight to aid in compaction (Jayawickrama et al. 2000).

Monahan (1994) also recommends a vibratory source for non-plastic materials, as well as the use of handheld tampers in trenched areas. The handheld tampers allow better compaction of material in confined areas (Monahan 1994). For thermoplastic pipes, the haunching layer requires careful compaction practices and small equipment such as hand held tampers weighing no more than 20 pounds and a tamper base with a maximum of 6 inches by 6 inches (15 cm x 15 cm) to be used (Hancor Inc. 2000). Backfill material with cohesive and clay materials should use a rammer for compaction, reducing the amount of air in the material, therefore allowing good compaction. For non-cohesive fills a vibrating compactor may be useful and can be used near a pipe, assuming it is light weight (Hancor Inc. 2000). Figure 17 provides guidelines for the selection of compaction equipment in various mixtures of clay and sand materials for use with thermoplastic pipes.

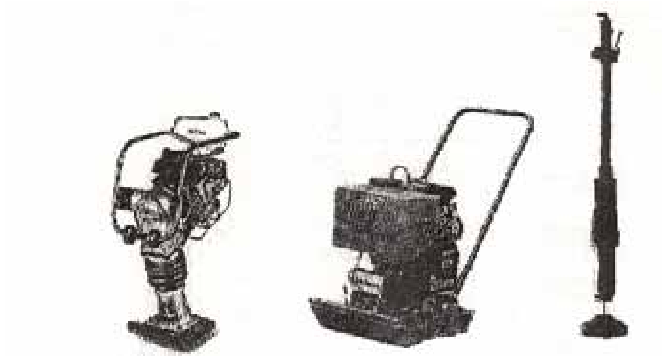


Figure 16. Compaction equipment from left to right: impact rammer, vibratory plate, and compressed-air tamper (Jayawickrama et al. 2000)

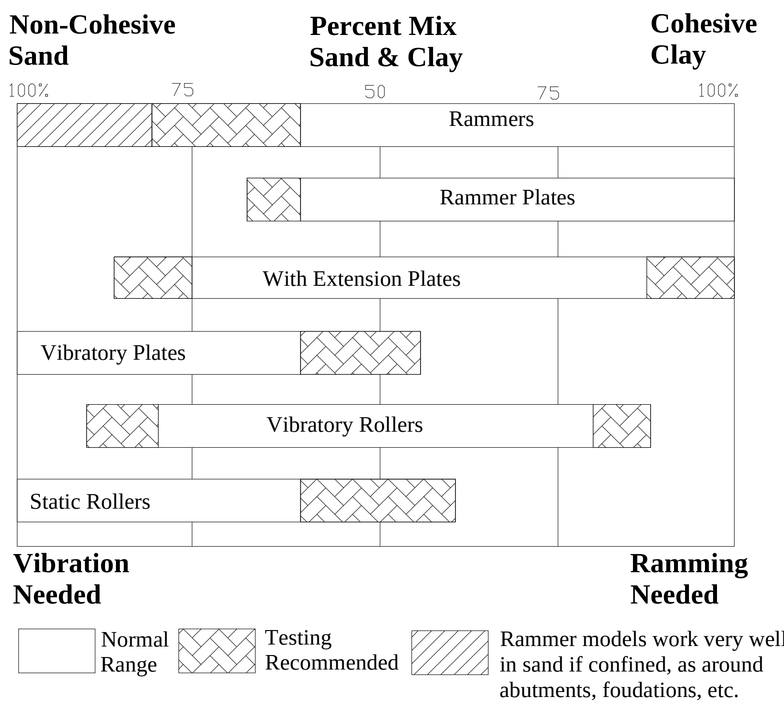


Figure 17. Guide to compaction equipment (Hancor Inc. 2000)

Non-traditional backfill

As previously mentioned, cementitious materials have been used as a method of filling many utility cut trenches. Henn (2003) mentions that Controlled Low Strength Materials (CLSM) are referred to by names including flowable fill, controlled density fill, unshrinkable fill, flowable mortar, fly ash slurry, flowable fly ash, soil-cement slurry, plastic soil-cement, and K-Krete. CLSM is considered a successful method of fill by several agencies. For example, after severe settlement problems occurred in 1988 with soil backfill material, the city of Peoria, IL began requiring the use of CLSM for trench backfilling (ACI 1994). The city of Peoria was convinced of the use of CLSM after several tests were conducted (ACI 1994). Outcomes of the tests conducted showed that the material needed only two to three hours to set, shrinkage cracks were minimal, and surfacing the patch could be completed within three to four hours (ACI 1994). In Metropolitan Toronto, CLSM is also the recommended backfill for trenches (Zhan 1997).

A CLSM mix consists of materials such as sand, fly ash, cement, water, and air entrainment. The Iowa DOT specification uses 100 lb/yd^3 of cement, 300 lb/yd^3 fly ash, 2600 lb/yd^3 fine aggregate, and about 585 lb/yd^3 water (ACI 1994). The cement acts as a binder and impacts cohesion and strength; fly ash can increase strength and flowability, but can also lower permeability, bleeding, and shrinkage properties of the mix; and aggregate (i.e., sands) impact strength and flowability of the mix (ACI 1994). Gassman et al. (2001) states common characteristics of constituents in a mix design: (1) an increase in water content increases flowability and mix time and decreases strength, and (2) an increase in water to cement ratio (w/c) decreases the compressive strength. Gassman et al. (2001) concluded through studies that by increasing the mixing time of CLSM past thirty minutes, setting time increases and unconfined compressive strength and flowability decreases.

CLSM can reach a self compacted compressive strength of 1200 psi (8268 kN/m^2), with an ideal strength around 50 to 100 psi (7 to 15 kN/m^2) to be obtained in trenches where future excavation may be required (APWA 1997). Mixes containing sand and fly ash can be excavated with compressive strengths reaching 300 psi (44 kN/m^2)(ACI 1994). ACI (1994) also mentions that a fill with a compressive strength of 50 to 100 psi (7 to 15 kN/m^2) is equivalent to an allowable bearing pressure of a well compacted soil.

CLSM has many advantages, including (1) strength and durability, (2) ability to be excavated in the future, assuming the mix design was designed correctly, (3) little required field inspection, (4) minimal traffic delay, (5) elimination of settlement once the mix has cured, (6) lower excavation costs as a result of the self compacting properties of CLSM (i.e., no compaction equipment needed and therefore construction of narrow trenches), and (7) year round usage (ACI 1994). CLSM greatest advantage is that it does not require any compaction equipment due to its ability to self-compact, therefore lowering the cost of equipment (ACI 1994 and Gassman et al. 2001). Kepler (1986) states that in trench areas where limited space is available for mechanical compaction, cement mortar may be advantageous (Ghataora and Alobaidi 2000).

There are several disadvantages to using CLSM as a backfill material, including (1) potential for long-term delays in construction procedures due to setting time needed as a result of mixing (Gassman et al. 2001), (2) potential for pipes to float, since it is a flowable material (Jayawickrama et al. 2000); however, this can be avoided by placing CLSM in lifts and therefore reducing the uplift load CLSM applies to pipes (ACI 1994), (3) initial costs for using a CLSM material is high than if using a granular material to fill a trench

(Jayawickrama et al. 2000), and (4) future excavation of the trench can be difficult and time consuming if a compressive strength is too high (Ghataora and Alobaidi 2000).

Different cementitious materials including foamed concrete, lean concrete, cement/ash mortar (flowable fly ash), and Lytag/cement were used in trial trenches as backfill materials in a study on flowable fills (Kepler 1986; and Peindl et al. 1992). Advantages and disadvantages of using each material are summarized in Ghataora and Alobaidi (2000). For example, foamed concrete has advantages such as its ability to self compact. However, foam concrete is expensive, backfilling operations can be difficult if the trench is located on a slope, and it may take longer for the material to set and the site to reopen. Lean concrete, a material with a low amount of cement, reduces stresses on PVC piping as opposed to a granular fill, but it is more expensive and does not resist frost as well as foamed concrete. Peindl et al. (1992) tested cement/ash mortar using pulverized fuel ash (PFA), cement, superplasticizer, and water with results showing very little settlement. It was noted that pipes had very little strain contributed to them, little maintenance was required in the future, and this method was also inexpensive.

Washington DOT uses control density fill (CDF) in a portion of the backfill. Figure 18 shows a typical cross section of Washington DOT (WSDOT) utility cdf backfilled trench for asphalt roadway. As seen in the figure, a minimum of three feet (0.91m) of CDF is required and granular material located beneath extends to the floor of the trench. The trench width that is noted in the figure should be applied only when the excavation allows.

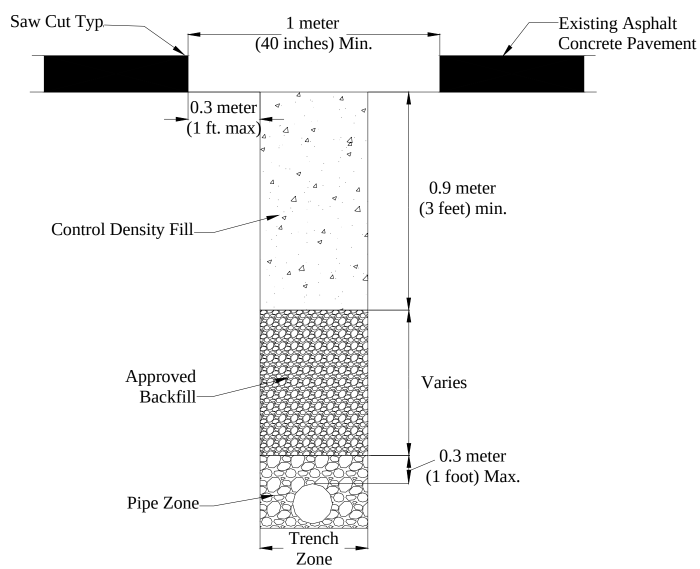


Figure 18. Typical trench from WSDOT cross section using cdf as backfill material (WSDOT)

The mix design of a flowable fill will determine the ease at which potential future excavations can occur. Ghataora and Alobaidi (2000) found that removing granular and cementitious material for future repair needs, ranged from ten to thirty minutes (see Table

11). The mixture of PFA:sand:cement may have had a shorter excavation time if an accelerator and lower amount of cement was used in the mix (Ghataora and Alobaidi 2000).

Table 11. Removal of trenching material (Ghataora and Alobaidi 2000)

Trench Type	Time Required to Excavate (minutes)	Operatives' Comments
Granular Type I	10	Material needed loosening and was easy to excavate
Lytag:cement	13	As above, but it broke in larger pieces and it was therefore easy to clear out the trench
pfa:sand:cement	30	Difficult to loosen but easy to clear trench once loosened

Summary of Utility Cut Practices Used by Agencies

Two major studies discussed in detail above have established good standards of practice for use in the field. These practices have been found to be advantageous to these agencies. Southern California Gas Company (SoCalGas) devised the following compaction procedure. For each lift the moisture content should be tested for (1) compliance with the optimum moisture content and (2) the amount of time for a lift to be compacted; then (3) compaction is performed from the outer region of the trench towards the center to eliminate excess soil on the edges and to form a connection with the trench walls and soil (APWA 1997). Also, when using native material, compaction density should be tested on the excavated soil for compliance (APWA 1997). Last, SoCalGas recommends that the backfill be compacted to 90% or more of the maximum density, with the backfill consisting of mostly sand or silty soil (APWA 1997). Studies conducted by SoCalGas indicate that a moisture meter used for potted plants provides a good estimated moisture content measurement for compaction at optimum, with readings indicating “appropriate”, “too wet”, and “too dry”(APWA 1997). Another advantage to the moisture meter is the ability to determine moisture contents with the use of devices such as the Dynamic Cone Penetrometer, which are unable to measure moisture (APWA 1997).

After completion of the study conducted in Salt Lake City, Utah, a new method for backfilling utility trenches was devised (Peters 2002). They now require base course and backfill material used in trenches, with compaction of 95% modified proctor density (APWA Section 02324). The zone of influence is then compacted with the backfill material in the excavated region as noted in Figures 10 and 11 (Peters 2002). A minimum of eight inch (20 cm) thick base course should be used, along with one inch (3 cm) of asphalt plus any additional asphalt, minimum of four inches (10 cm), to reach the existing pavement. Asphalt should be placed in three-inch (8 cm) lifts and compacted to 96% laboratory density (Peters 2002). For asphalt pavements, the tack coat should cover all vertical surfaces where the trench has been cut. If a crack were to form in the T-section, it should be repaired according to APWA Section 02975 (Peters 2002). Furthermore, Salt Lake City, Utah suggests that flowable fill (e.g. CLSM) with a 28-day compressive strength of 60 psi be used in confined trenching areas (Peters 2002). However, the material should be allowed to cure to the initial set before untreated base course or asphalt pavement is added (Peters 2002).

Quality Control/ Quality Assurance (QC/QA)

Quality Control and Quality Assurance may be one of most important factors in a successful trench. APWA (1997) stated that a permit program is only as good as its enforcement and recommends that inspection take place when work is in progress, at the completion of the project, and about one year from completion assuming that there is a warranty on the patch ending after one year.

New technology in reaching specified backfill compaction standards is involving the use of Dynamic Cone Penetrometer (DCP). The DCP originated in 1956 in South Africa and has now been brought to the United States and adopted for use in many projects (Amini 2003). Amini (2003) states advantages and disadvantages to the DCP, including advantages of (1) potential use as a quality control device and correlations to be made with CBR; (2) it is relatively inexpensive, fast, and easy to use; and (3) no significant training is required for the use of the instrument. Disadvantages include (1) results are not consistent with large well graded granular material, (2) aggregate greater than 2 inches (5.1 cm) may produce variable results, and (3) strength correlations may be effective for a specific material only.

The DCP was used by Jayawickrama *et al.* (2000) to compare the compaction results of four different backfills and three different compaction machines. They concluded that DCP values depend greatly on the depth of the test. In other words, at great depths, higher blow counts were achieved. This was determined by defining the DCP blow count as the number of blows needed to penetrate 10 cm into the material being tested (Jayawickrama et al. 2000). Jayawickrama et al. (2000) contributed this effect to confining pressure.

The DCP test was adopted by the Minnesota Department of Transportation (MnDOT) as a Quality Control device for determination of proper compaction in pavement edge drained trenches and compaction of layers when granular base course is used (Burnham 1997). The trial version of the DCP QC for base course procedure was as follows. The DCP is placed on an undisturbed area. If the DCP penetrated with its own weight more than 0.80 inches (20 mm), a new testing area is to be located about 11.8 inches (300 mm) away. If more than 0.80 inches (20mm) is still a result, then the test fails and more compaction is needed. The MnDOT sand cone density test, a version of AASHTO T191, must confirm soil failure. If material penetrates less than 0.80 inches (20mm) the DCP test can continue. Initial reading is read and then the hammer is dropped 4 times and a final reading is read. The final reading minus the initial reading is divided by four (the number of drops). If this value is 0.75 inches (19 mm) or lower, the site passes the test (Burnham 1997). Burnham (1997) suggests testing a silty/clay material DPI (DCP's penetration index) should be less than or equal to 1 inch/blow (25mm/blow) and is confirmed by the use of Army Corp of Engineers DCP-CBR formula and correlation. Table 12 indicates typical CBR values for USCS classified soils.

SoCalGas uses the DCP as a quality control device to measure proper compaction; however, no standards were specified (APWA 1997). APWA (1997) suggests that when using the DCP, if the penetrometer does not penetrate more than 3.25 inches (129 mm) above the rod with a minimum of 11 drops, a compaction level of 90% is obtained. The Clegg hammer also uses correlations for material strength. According to Ghataora and Alobaidi (2000), a minimum Clegg hammer value of eighteen is needed in proper compaction for pavement surfacing.

Another quality control device that has been used in the field is the nuclear gauge. The nuclear gauge can be used to check density of a backfill material, although it can be expensive (Peters 2002). Another disadvantage to the nuclear gauge is that it emits radiation and therefore requires certification for its use. Salt Lake City is using the nuclear gauge, as well as a variety of other quality control techniques such as inspecting projects during construction and making sure that the zone of influence is properly constructed. San Francisco also uses the nuclear gauge and sand cone method C when determining compaction properties (APWA 1997).

Table 12. Typical CBR values for USCS classified soils (Rollings and Rollings 1996)

Description of Material	CBR (%)
Classification by Unified Soil Classification	
GW: gravel or sandy gravel	60 to 80
GP: gravel or sandy gravel	35 to 60
GM: silty gravel or silty, sandy gravel	40 to 80
GC: clayey gravel or sandy, clayey gravel	20 to 40
SW: sand or gravelly sand	20 to 50
SP: sand or gravelly sand	10 to 25
SM: silty sand	20 to 40
SC: clayey sand	10 to 20
CL: lean clays, sandy clays, gravelly clays	5 to 15
ML: silts, sandy silts, diatomaceous soils	5 to 15
OL: organice silts, lean organic clays	4 to 8
CH: fat clays	3 to 5
MH: plastic silts, micaceous clays or diatomaceous soils	4 to 8
OH: fat organic clay	3 to 5
PT: peat and highly organic soils	< 1

From reviewing current practices, it has been noted that a variety of stages in the construction of a utility cut are critical and if not performed correctly can have effects that may cause a poorly performing restoration in the future. The effects of poorly constructed utility cuts have a large impact on the economics of a community. The following sections further discuss the economic impact on a city, as well as permit fees that could compensate for economic losses.

Economic Impact of Utility Cuts

The economic impact that utility cuts pose on a city is evident with the continual need for a number of utility repairs each year. Khogali and El Hussien (1999) report that more than 250,000 utility cut restorations a year were made in New York City streets. American Public Works Association (APWA 1997) reported that a study conducted in Burlington, Vermont found that an overlay of 0.75 to 1.5 inches (1.9 cm to 3.8 cm) was needed to compensate for weakened pavement resulting from a cut. With additional materials and maintenance needed, these utility cut patches resulted in an estimated added cost of $522,000 per year (APWA

1997). Cincinnati, Ohio spent an additional $2,000,000 per year for utility cuts made in asphalt pavements, and Los Angeles, California spent $16.4 million a year on overlays to compensate for maintenance repairs of these cuts (APWA 1997). Internationally, Jones (1999) reported that utility cut restorations are the second major cause of traffic disruption in the United Kingdom, with an estimated cost of $13 billion dollars, while in Toronto an additional $3 million was used annually for maintaining poor utility cut restorations (Arudi et al. 2000).

Permit Fees

Several jurisdictions have developed their own fee system after recognizing the effects of utility cuts on pavement performance. In some cases, the utility company is charged a fixed amount for every inspection. APWA (1997) indicates that an inspection program should consist of ensuring that permit and construction requirements are met. Most cities require a permit to be obtained before a cut can be made for a utility. The permit generally covers information such as administration, inspection, and fees dependent on the size of the cut (APWA 1997). Inspection fees, opening fees, and loss of structural integrity fees are being adapted in an attempt to compensate for future maintenance costs (Arudi et al. 2000). The purpose of the structural integrity fee is to require contractors to pay a fee to cover repairs that are expected in the future, based on the amount of damage that is foreseen (Tiewater 1997). Cincinnati conducted a study where a Microsoft Windows based program, UCMS version 1.0, was developed to assist in the evaluation of costs and performance of pavements, as well as using the information as an assessment for future maintenance and repairs (Arudi et al. 2000).

In some cases, future maintenance costs could be minimized by implementing a strong inspection program aimed at assuring that the permit standards are met (APWA 1997). Table 13 illustrates the number of cuts made each year in several cities and fees received from the cut; however in many cases these fees do not provide enough financial assistance to maintain a poorly performing patch in the future (Arudi et al. 2000). Arudi et al. (2000) suggests two factors which need to be considered when evaluating fees: (1) amount of damage, and (2) costs needed for rehabilitation. Cincinnati has several base fees consisting of a $15 administration fee for each permit, plus an additional $35 inspection fee for excavations up to 2.0 yd^2 (1.7 m^2) and for larger excavations, an additional $3 beng assessed for every 1.0 yd^2 (0.84 m^2) (Arudi et al. 2000). Arudi et al. (2000) adds that in Cincinnati, additional fees such as $1 for every 1.0 yd^2 (0.84 m^2) be assessed for loss of pavement strength, as well as a $10 street opening fee for each permit obtained.

Table 13. Annual number of utility cuts and permit fee revenues (modified from Arudi et al. 2000)

Jurisdiction	Annual Utility Cuts	Permit fee Revenues	Comments
Billings, MT	650-730	$49,900	
Boston, MA	25,000-30,000		
Cincinnati, OH	10,000	$800,000	
Chicago, IL	180,000	$2,500,000	
Ft. Collins, CO	500	$37,000	$65/Permit
Fresno, CA	4,500		
Mesa, AZ	800		$50 minimum
Oakland, CA	5,000		$53/hour inspection
Pasadena, CA	1,800		Random checks
Redmond, WA	500-1,000		$230/permit
Sacramento, CA			Full recovery fee
San Francisco, CA	14,000	$700,000	

Summary of Findings from the Literature Review

- Backfill materials and compaction requirements should include gradation, moisture control, lift thicknesses, and compaction equipment.
- The majority of DOTs in the United States use a granular backfill material with an AASHTO classification of A-1 and A-3.
- Granular backfill requirements should be based on relative density with moisture control and not on standard Proctor as reported by many state DOTs.
- Lift thicknesses vary between 4 and 12 inches, with 6 inches most commonly used by state DOTs.
- A majority of the settlement occurring in utility cuts occurs in the top 2 feet of an excavation.
- Softening of subgrade soils around the utility cut area within the zone of influence has been found to lower the structural capacity along the perimeter of a trench by 50% to 65% in two years.
- Correction for the zone of influence can be obtained with a pavement cutback of two to three feet removed and filled with compacted native soil or backfill materials. T-sections, and other similar engineered cross sections, have been used successfully to mitigate the zone of influence effects.
- Cutbacks are found to perform best when conducted after backfill has been compacted into the trench.
- Alternative field testing methods such as the DCP and Clegg Hammer have been used to monitor compaction.

- Controlled Low Strength Materials (CLSM) eliminates future settlement that may occur when using a granular material and does not require the use of compaction equipment; however, it has a higher initial cost that conventional backfilling.
- Flowable fills are advantageous in confined areas, with strengths ranging from about 50 to 100 psi needed for potential future excavations.
- Trenchless technology can eliminate the impact a cut has on a roadway and lower traffic interruptions, requires small number of construction crew, has less impact on businesses, decreases the noise, and has less air pollution. However, trenchless methods had the potential of forming sinkholes, may result in heaving, leaking of drilling fluid, and drilling tools puncturing the pavement surface.
- Utility cuts in a roadways result in an estimated decrease of pavement life up to 50%.
- Many cities in the United States and abroad reported spending millions of dollars on the maintenance and repair of utility cuts.
- Many cities reported using several fees to cover the cost of pavement maintenance in utility cut regions.

UTILITY CUT SURVEY RESULTS

A survey on utility cut standards and performance was devised to determine problem areas that city personnel observe. The prepared survey is shown in Appendix A. The survey was sent to major cities across Iowa and responses were received from Ames, Cedar Rapids, Davenport, Des Moines, Dubuque, Waterloo, and Burlington. Figure 19 shows the cities represented in this survey study. These surveys were compiled to compare city standards and practices.

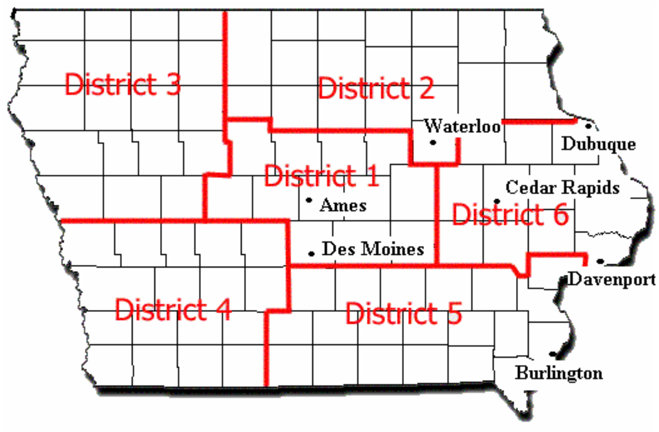


Figure 19. Survey responses from various Iowa cities (modified from www.dot.state.ia.us/tranreg.htm)

Weather can influence the occurrence of utility breaks because of the temperature fluctuations affecting soil behavior. In the survey, an inquiry was made on the time of year a majority of utility breaks occur and the number of breaks occurring annually. There was a large variation in responses from city to city in the seasonal occurrence of utility breaks.

Davenport stated that spring and late fall were predominant seasons for utility breaks to occur, with the number of utility cuts about 800 annually. In Cedar Rapids, utility breaks were stated to be most prominent in the winter and spring with 75 to 80 breaks a year. Dubuque stated the greatest number of breaks were thought to occur in the winter with 50 to 60 breaks, and Waterloo agreed with winter being the predominate season for occurring breaks, with 187 street excavations completed from July 1, 2003 to June 30, 2004.

Des Moines estimated 1500 utility cuts a year, with no specific season having more than another. Data received from Ames shows that a majority of past breaks have occurred in the winter months. Figure 20 shows the monthly distribution of breaks occurring in Ames since the year 2000. It was noted that there may have been more than one break on a site. The year 2003 had significantly more breaks occurring because of the need for a new water tower on the West side of Ames in July due to capacity demand, therefore increasing the pressure on the existing pipes. As a result, the data from July to October 2003 shows a higher number of breaks. Overall, the months of January and December have a majority of the natural breaks occurring, whereas utiliry cuts occurring in May and throughout the summer is thought to be a result of the beginning of the construction season. This trend may be a result of frost loading which could substantially increase vertical loads (i.e., up to twice the original load) on buried pipes, Moser (1990).

After compiling the data received from the city of Ames, the year 2000 had 29 breaks, 2001 had 23 breaks, 2002 had 24 breaks, 2003 had 71 breaks, and 2004 had 21 breaks total. Including all data, except the year 2003, an average of 24 water main breaks occur in Ames annually.

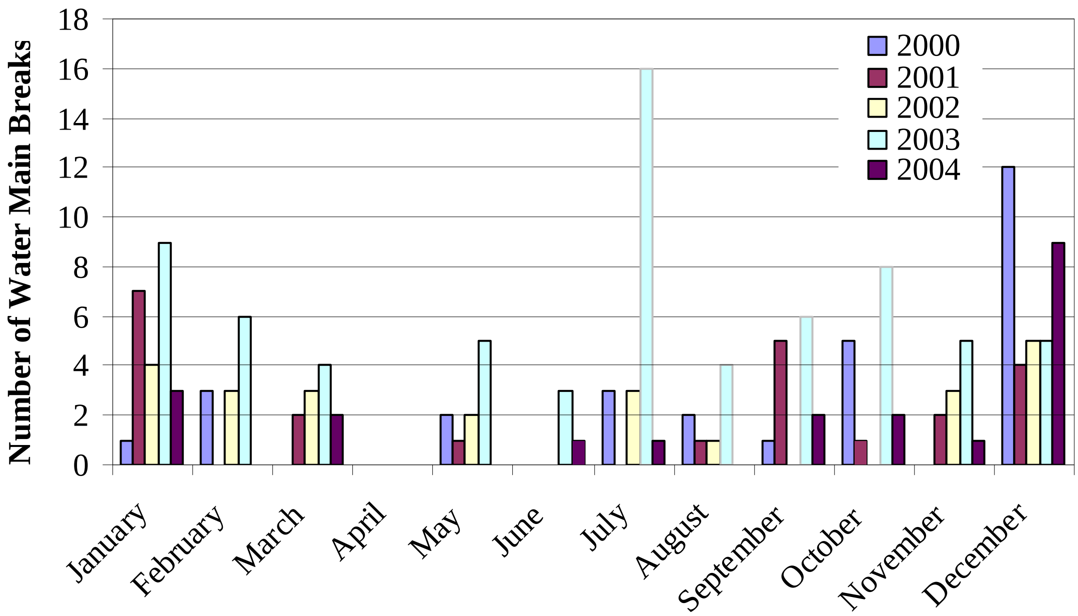


Figure 20. Monthly distribution of water main breaks in Ames, IA (Ames Street Department database)

As stated in the literature review, if a trench is constructed properly, pavements should last for fifteen to twenty years. However, the surveyed cities estimated utility cut patch life anywhere from five years to as little as one week, before the need for maintenance of the patch. The city of Davenport reported that typically a patch will last five years, Ames reported two years, while Cedar Rapids stated that patches last two to three months and Dubuque reported that one patch lasted only one week. In the survey no distinction was made between temporary patches and permanent patches. It seems likely that the reported short life of patches is for temporary patches.

Many cities throughout Iowa do not document the number of trenches that are performing poorly. Therefore, the values obtained from the survey may reflect a low number of poorly constructed trenches in a given city. The city of Davenport estimated that about 30% to 40% of trenches constructed have performed poorly, while Waterloo estimated about 10%. Dubuque, Ames, Des Moines, and Cedar Rapids all reported a very low percentage of poorly performing trenches. For example, Cedar Rapids estimated about 5%, Dubuque reported about 3%, Ames about 5%, and Des Moines stated less than 1%. Since converting to K-Crete, Burlington stated minimal problems with trench performance.

The personnel that completed the survey stated a variety of potential causes for utility trenches performing poorly. Davenport stated that poor performance may be due to improper bedding and backfill operations. Dubuque stated that trenches perform poorly when constructed in the winter and under adverse conditions. Ames and Burlington believe the major problems in a trench are due to poor compaction and the use of improper backfill materials, while Waterloo and Des Moines both agreed that poor compaction is of large concern in trench performance. Cedar Rapids believes that problems arise with the use of native materials with high moisture contents and trenches constructed in confined areas where compaction is difficult.

Most cities believe they have a satisfactory procedure for trenching, with only three cities recently changing their methods of repairs. These three cities include Davenport, Des Moines, and Burlington. As of July 2004, Davenport the city will no longer be providing excavation and surfacing services, rather these services will be contracted out. About a year and a half ago, Des Moines public works group changed to using full depth saw cuts and manufactured sand, and allowing plumbers to backfill their own excavations. Burlington also changed to K-Crete about eight years ago and state that significant improvements on the trench quality occurred after switching from backfill sand to K-Crete, a 500 psi mix.

Of the seven cities in Iowa that responded to the survey, all stated that a standard method of repair was used for utility cuts and all cities agreed that satisfactory results were obtained after construction. However, lack of documentation may have had an influence in these positive responses.

Imported and native backfill materials vary from each city based on regional availability of material. Davenport uses native material, select material containing no organics, Class A crushed stone, and material passing 3/4 inch, which is generally used when native material is not available. Sand is generally not allowed because of settlement problems trenches have experienced in the past. Dubuque uses a limestone crusher dust as an imported backfill, as well as native material. Ames uses native material, flowable fill and 3/8-inch minus limestone chips, which is most commonly used. Waterloo states that they use native material or material similar to the soil surrounding the trench. There were no specific materials mentioned. Des Moines uses native material, manufactured sand, and 50 psi K-Crete as backfill material. Burlington uses a granular base under and over pipe lines, and then a 500 lb. K-Crete mix (flowable fill) is used above the base material. Cedar Rapids uses granular material under streets and driveways.

The compaction requirement that a city requires is an important aspect in proper construction of a trench. When asked about the type of compaction required for use in each city, a variety of answers were obtained. Several cities responded with Proctor standards and others just noted the compaction equipment currently used. Dubuque, Waterloo, and Des Moines specify that backfill material should be compacted to at least 95% standard Proctor density; however, Davenport states 90% standard Proctor should be used to eighteen inches below finished subgrade and 95% above this region. The type of equipment used for compaction is generally a mechanical tamper, specifically in most cases a vibratory plate attached to the back of a backhoe.

During winter months, surfacing the trenching area with an appropriate pavement becomes difficult since hot mix plants are closed and there is difficulty in placing concrete. In such cases, a temporary pavement is used until the spring when permanent pavement can be placed. All of the cities that responded use temporary pavement for cuts made in the winter. Davenport uses cold-mix asphalt in the winter and requires replacement of the temporary pavement in the spring. In Dubuque, the pavement is covered from November to May with three inches of cold mix asphalt and replaced when hot mix asphalt becomes available. The city of Ames uses six inches of cold mix asphalt, four inches of concrete, or twelve inches of asphalt millings for temporary surfacing. This temporary pavement should be replaced with a permanent patch within six months. Des Moines uses a temporary pavement during the winter, which is constructed using PCC. The permanent pavement is made as soon as weather permits. In Burlington, on overlay streets, a cold mix is used and on concrete pavements, a road rock is used until suitable conditions exist to permit concrete surfacing. Cedar Rapids uses a cold patch mix from the Iowa DOT to surface patches in the winter months and it is then removed in the spring.

Each city that responded to the survey has an in-house repair crew for utility cuts if needed. However, as of July 1 2004, the in-house crew in Davenport will be eliminated as a result of budget considerations. These in-house crews do not necessarily complete the excavation and compaction process but, rather, they complete the surfacing of the excavation.

Quality control and quality assurance is of great importance in the proper construction of trenches. The five cities of Davenport, Dubuque, Waterloo, Des Moines, and Burlington stated that they have a quality control procedure that is used. However, Ames and Cedar Rapids do not currently have quality control requirements. Waterloo specifies quality control only on street reconstruction projects. Both Dubuque and Waterloo use the nuclear gauge to determine proper compaction. Davenport inspects the various sites, with some sites being guaranteed by franchise agreements. Des Moines uses a four-year performance and maintenance bond and Burlington stated that they make an effort to require a permit to work in the right of way by bonded contract, but they have no current inspection.

Requiring permit fees for utility cuts can help in alleviating the expenses that result from future maintenance of utility cuts. The city of Ames requires a permit to be obtained, however there is no fee assessed. Des Moines requires a permit fee and a four year performance and maintenance bond. The excavation fee consists of a $20 administration fee plus additional charges, such as a disruptive cost component dependent on the type of street and hours worked (principal arterial: $\$0.20/ft^2$; minor arterial:$\$0.15/ft^2$; collector: $\$0.10/ft^2$; and residential: $\$0.05/ft^2$) and an inspection cost component of $\$0.35/ft^2$. Davenport has changed its utility cut fees from $10-$15 to anywhere from $225-$1000, depending on the site and situation. This cost increase was due to the elimination of the city performing utility cuts. When the city of Dubuque surfaces a utility restoration, a minimum fee of $15 plus an inspection fee of $\$0.75/ft^2$ for asphalt, concrete, and concrete with an asphalt overlay pavements, as well as a pavement fee of $\$4/ft^2$ for asphalt pavements and $\$5/ft^2$ for concrete and concrete and asphalt overlay pavements is assessed, however no further permit fees were mentioned. Waterloo uses a computer program, EXCAVATE Version 2001, to calculate fees. Waterloo has a $10 permit fee and a $50 mobilization fee, plus additional fees, depending on the amount and type of surfacing material used for the excavation repair.

Summary of Findings from the Utility Cut Survey

Seven cities across Iowa responded to the survey sent by the research team: Ames, Cedar Rapids, Davenport, Des Moines, Dubuque, Waterloo, and Burlington.

- Using the statistical data provided by the city of Ames, January and December are the prominent months for water main breaks. This trend may be a result of frost loading which could substantially increase vertical loads (i.e., up to twice the original load) on buried pipes, Moser (1990).
- Each city follows its own adopted method of repair practice.
- All of the cities report that their standard of practice provides satisfactory results; however, almost all stated that utility cut restorations last for two years or less.
- Across the seven cities, a variety of materials is being used as backfill material, and are chosen according to regional availability. These materials include native material, select material containing no organics, Class A crushed stone, material passing 3/4 inch, limestone crusher dust, flowable fills, 3/8-inch limestone chips, and manufactured sand.
- Burlington is the only city surveyed using a flowable fill as its primary backfill and indicates its use is providing good results. The other cities use flowable fill in specific applications.
- Dubuque, Waterloo, and Des Moines require 95% standard Proctor compaction. Davenport requires 90% standard Proctor to eighteen inches below finish grade and 95% above that region.
- Inspection in most cases is visual and not by the use QC/CA measurements.
- Many cities throughout Iowa require the use of permits before initiating an excavation, however, a fee is not assessed in all cases. The permit serves as a mechanism to track who conducted the work and when, and fees are generally an attempt to recoup administrative costs.

UTILITY CUT CONSTRUCTION TECHNIQUES

Several cities in Iowa were visited for further documentation of current construction practices and to conduct field tests on compacted backfill material. The selected cities are Ames, Cedar Rapids, Council Bluffs, Davenport, Des Moines, Dubuque, and Waterloo (see Figure 21).

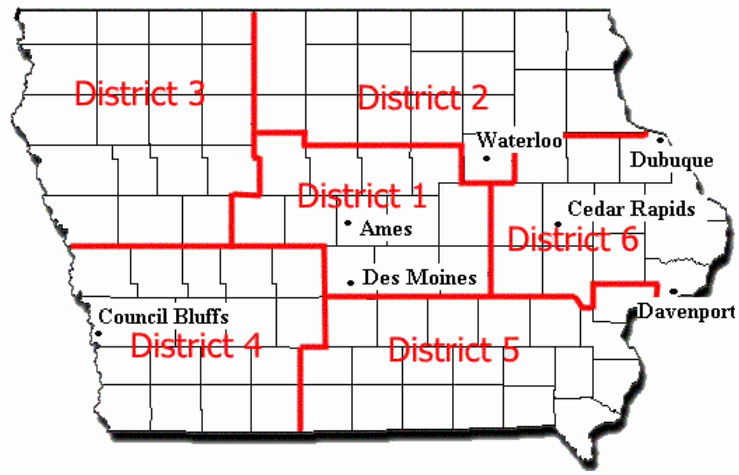


Figure 21. District map of Iowa (modified from www.dot.state.ia.us/tranreg.htm)

Field Observations of Iowa Practices

A variety of construction practices and materials used were observed during this study. Generally, an imported backfill material is selected based on regional availability, leading to a variety of materials being used throughout the state of Iowa. It was observed that in many cases, lift thicknesses greater than three feet (1 m) were used, resulting in poor compaction and potential settlement problems in the future. The following sites have been tested extensively in the field, with the restoration locations shown in Figure 22.

1. 20th Street & Hayes Avenue in Ames, IA.
2. Miami Drive & Sherman Avenue in Cedar Rapids, IA.
3. Iowa Street & East 4th Street in Davenport, IA.
4. East Grand Avenue & East 28th Street in Des Moines, IA.

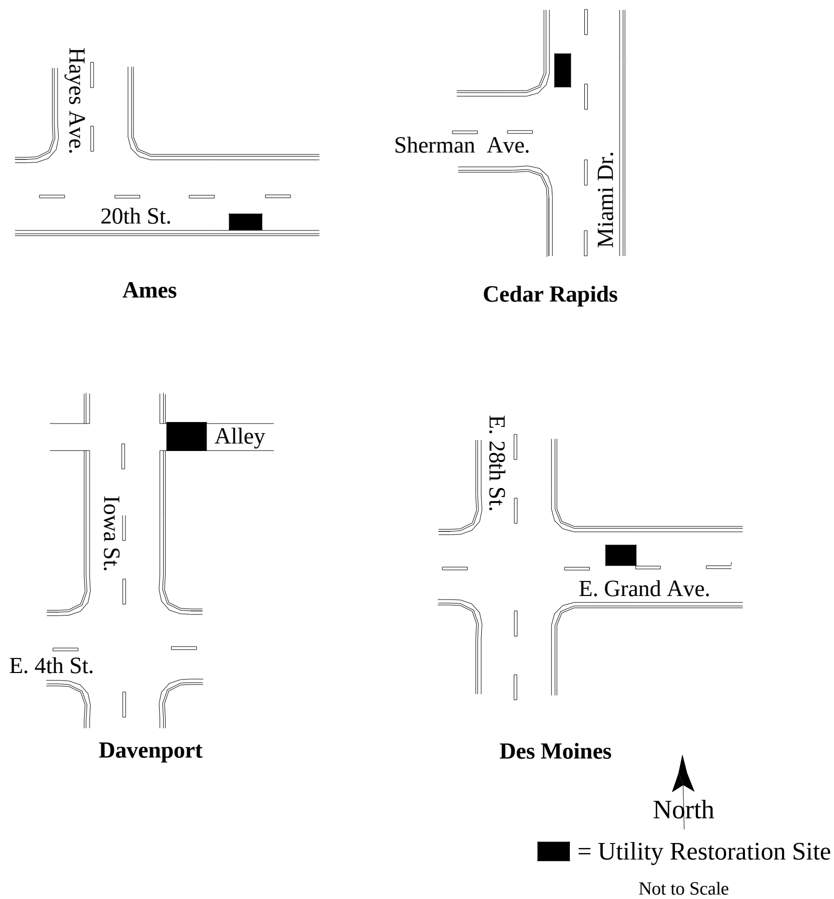


Figure 22. Iowa utility restoration site locations

Due to the unstructured occurrences of utility breaks and the traveling distance needed to reach a site, several visits became observational because of a need for immediate repair. The sites in Dubuque, Waterloo and Council Bluffs were visited and documented, but extensive testing was not conducted.

Ames: 20th Street & Hayes Avenue

The restoration of a water main break on 20th Street south of Hayes Avenue occurred on October 18, 2004. The excavation and construction of this trench was completed by the city of Ames. The trench is sixteen feet (5 m) long, six feet (2 m) wide and about ten feet (3 m) deep, excluding the cutback region. Figure 23 shows the excavation of the trench as it approaches the broken water main and illustrates the tough working conditions that exist. A dewatering device was used to pump excess water from the break into inlets on the street. This was done by immersing the pump, which was placed in the backhoe bucket, into the trench, and pumping soiled water through a hose into the street. As the excavation proceeded and additional water was removed during the dewatering process, saturated material along the perimeter of the trench began to slough off (see Figure 24). Cavities continued to develop around the perimeter of the trench until the broken water main was reached.

Figure 23. Trench excavation

Figure 24. Material sloughing off in Ames site

The water main break was reached at a depth of about ten feet (3 m) into the trench. After the trench was dewatered, construction crew members were able to repair the break. Figure 25 shows the break in the pipe as water sprayed out, and Figure 26 shows the shoring box being placed into the excavation. This shoring box acts as a support brace from the surrounding soil, which protects workers from material caving in during break repairs. The break was repaired with a pipe sleeve and the shoring box removed.

Figure 25. Ames water main break

Figure 26. Shoring box placed into trench

Approximately four feet (1.2 m) of one-inch limestone was placed as a bedding and backfill material up to two feet (0.6 m) above the pipe crown. This bedding material was compacted in two-foot (0.6 m) layers with a vibrating plate. Figure 27 shows the bedding material being dumped into the trench. A material referred to as 3/8 minus was then used as a backfill material extending from the top of the bedding material to the surface of the excavated area. This material was also compacted in about two-foot (0.6 m) layers with a vibrating plate. Figure 28 shows the compaction of the 3/8 minus material using the vibrating plate connected to the end of the backhoe. As a result of the construction, saturated material and debris from the excavation and surrounding area were shoveled into the trench during the backfilling process (see Figure 29).

Figure 27. Bedding material dumped into trench

Figure 28. Compaction of backfill material

Figure 29. Saturated material shoveled into the trench

The final lift was compacted with excess material on the top and leveled off with the backhoe bucket. The final compaction was completed by rolling over the constructed trench with the backhoe. Figure 30 shows the completed utility cut. The utility cut was then left open and unpaved for about two weeks, allowing traffic to further compact the material.

After the two week period, the pavement was cut back and removed to about 2.5 feet (0.8 m) from the edges. This pavement removal, in most cases, was standard in Ames because during the initial excavation, pavement was broken up with the backhoe bucket, leading to a non-uniform edge. Therefore, this cut in the pavement was made because of a need for straight edges in surfacing the trench. Once the cut was made, excess pavement was removed and hauled away. As stated before, the purpose of leaving the trench unpaved for two weeks was to reduce future settlement, however during this pavement removal process conducted by the backhoe, backfill material was disturbed and loosened (see Figure 31). After this backfill material disturbance, no additional compaction equipment was brought in to compact this area. Instead, the backhoe leveled off excess material and then completed several passes with the weight of the backhoe and patting the material with the backhoe bucket as a method of compaction (see Figure 32). Figure 33 shows the completed trench in Ames.

Figure 30. Utility cut left open for two weeks

Figure 31. Pavement removal

Figure 32. Backhoe bucket compaction

Figure 33. Ames site completed

Ames: 16th Street & Marston Avenue (Winter Break)

This winter water main break in Ames occurred near the intersection of Marston Avenue and 16th Street on February 7, 2005, where water was temporarily turned off. The excavation on the site was 7.5 feet (2.3 m) long, and 8.5 feet (2.6 m) wide, with a depth of 6 feet (1.8 m). This utility cut was constructed by the city of Ames, in pavement consisting of 10 inches (0.3 m) of asphalt. The removal of pavement from the trench can be seen in Figure 34.

Figure 34. Pavement removal from Ames winter break site

After pavement was removed from the surface, dewatering of the trench began. The trench was dewatered before excavation began (see Figure 35). While the water level was lowering, saturated material was excavated from the trench. Figure 36 shows the saturated material being excavated and the damage that has resulted to the surrounding pavement. Once the break was located the pipe was cut and repaired. Backfill material, which consisted of the SUDAS specification and 1.5-inch limestone, as a bedding material, was then added to the trench. The SUDAS backfill material segregated in the dump truck, therefore coarse material was placed near the center of the excavated trench and fines on the top. Figure 37 shows the backfill material being dumped and placed into the trench. Again, near the end of construction, saturated material was incorporated into the trench to clean the area up (see Figure 38). The completed unpaved trench is shown in Figure 39. The following day the trench had an asphalt cold patch placed on it until spring when the asphalt plant reopens.

Figure 35. Dewatering the trench

Figure 36. Saturated material being excavated

(a) Pushing in backfill material **(b) Dumping backfill material**

Figure 37. Addition of SUDAS backfill specification

(a) Cleaning excess material into the trench **(b) Saturated backfill material**

Figure 38. Incorporating surrounding material into the trench

Figure 39. Trench ready for cold patch

Cedar Rapids: Miami Drive & Sherman Avenue

Excavation and construction of this leaking valve restoration in Cedar Rapids began on July 14, 2004. This trench was located on the corner of Miami Drive and Sherman Avenue, resulting in a trench size of 8 feet wide (2.4 m), 12 feet long (3.7 m) and about 10 feet deep (3.0 m). This trench was excavated, repaired, and backfilled by the City of Cedar Rapids water and street department.

A standard vertical cut was made in the pavement and excavation of the native material began. At the completion of the excavation a shoring box was placed into the trench (see Figure 40). The leaking valve was repaired and a 2" x 4" block of wood and concrete block was placed beneath the pipe for support. The pipe was also wrapped with black plastic wrap for protection against corrosion.

Figure 40. Shoring box in place

A 1-inch clean material was then used as a bedding material around the pipe, where this material was worked around the pipe and block with a shovel. A recycled crushed concrete backfill material classified with particle sizes 3/4-inch or less was imported from the landfill where it has been reclaimed from previous concrete pavement excavations. This site was backfilled with two lifts of material dumped about three to four feet (0.9 m to 13.1 m) deep each, and was tamped with a vibrating plate for about three to four seconds in no specific compaction pattern. Figure 41 shows a lift of the material being tamped in place. The pavement surface consists of 6 inches of concrete and 2 inches of asphalt overlay. As a result of the backhoe rolling over the edge of the trench during the backfill compaction process, the surrounding composite pavement was damaged (see cracked pavement in Figure 42).

Figure 41. Backfill compacted into trench

Figure 42. Visible pavement damage on utility edge

Davenport: Iowa Street & E. 4th Street

Excavation and construction of a water main began on June 2, 2004, which was completed by the city of Davenport. The site tested was in the downtown area in an alley/street intersection near Iowa and 4th St. The trench is approximately 15 feet (4.6 m) wide, 13 feet (4.0 m) long and 10 feet (3.0 m) deep.

Imported backfill material consisted of 1.5-inch limestone as a bedding material and a backfill material with a maximum of 0.75-inch minus limestone used above the bedding material. During the backfilling process, significantly large lifts were noted in the compaction process (see Figure 43). According to Davenport's specification, lifts should be placed in no more than 6-inch lifts (15.2 cm). However, the material was placed in approximately 4-foot lifts (1.2 m), which would be excessive for good compaction.

As a result of the cut, large cavities formed beneath the surrounding pavement of the trench. Figure 44 shows the large cavities and the attempt to compact this hard to reach area. These confined cavities underneath the pavement made compaction difficult using the vibrating plate on the end of a backhoe.

Figure 43. Large backfill lift being placed

Figure 44. Large cavities forming beneath pavement

Des Moines: E. 28th Street & E. Grand Avenue

Excavation and construction of the trench began on June 30, 2004. The sewer main break repair was completed by a contractor in Des Moines and is located east of E. 28th Street and Grand Avenue. This site was excavated, filled, and plated the day before the research team arrived, therefore documentation of the construction procedures were not made. The site was covered with a metal plate since surfacing was unable to be placed the following day. When the plate was removed, the manufactured sand (crushed limestone) used as backfill material had begun to settle along the trench edges, likely as a result of traffic vibrations (see Figure 45). Therefore, the concrete pavement was cut back to compensate for these cavities (see Figure 46).

Figure 45. Backfill material caving in on trench edges

Figure 46. Concrete pavement cut being made

Figure 47. Adding additional manmade sand to the trench

The pavement consisted of 8 inches (20.3 cm) of concrete with mechanical connection (i.e., dowel bars) used in both the longitudinal and transverse direction. Figure 48 shows the spacing of holes being drilled for the dowel bars. Dowel bars were placed in the drilled holes and concrete was brought in and poured in place. Figure 49 shows the concrete being placed. After the concrete setup, a joint was cut in the patch to match the surrounding joint spacings on the pavement. Figure 50 shows the completed trench.

Figure 48. Drilling spacings for dowel bars

Figure 49. Concrete placement in Des Moines

Figure 50. Completed surface in Des Moines

Several city visits were made where construction techniques and field testing were unable to be performed: Dubuque, Waterloo, and Council Bluffs. On June 4, 2004, the city of Dubuque was visited for documentation of utility restorations. There were no utility cut restorations occurring during the visit, however a new subdivision had a water main placed about 5 to 6 feet deep (1.5 m to 1.8 m) earlier in the month. The city of Dubuque uses the nuclear gauge, as a quality control device, to determine proper compaction. This subdivision was constructed by a new construction group and because of the use of the nuclear gauge, the city was able to determine that correct compaction levels were not being met during construction and therefore 80 feet (24.4 m) had to be reconstructed. With continual use of the nuclear gauge the construction workers were able to reach the 95% Proctor compaction level needed for compliance with the city of Dubuque. Noticeable settlement occurred on utility cuts where no inspector was on site, therefore since May 2004 the city of Dubuque now monitors private contractors.

Waterloo was visited on June 15, 2004, and again a testing site was difficult to locate and document. A representative from Waterloo reported that there were few complaints of failed trenches. The city uses complaints from the public to determine if construction techniques are providing adequate results. The research team was brought to several sites that had been constructed in the fall of 2003 that were to be surfaced with a permanent patch.

Council Bluffs was visited on November 4, 2004 with intentions of testing a site on Indian Hills Road; however, due to safety reasons, this site was no longer available for documentation and testing. The same contractor was working on a new subdivision, but they were at early stages in the construction. The research team did complete a preliminary testing evaluation of the site. During the first stage of compaction, a hand tamper, vibrating plate, and sheepsfoot were all used as compaction devices. After completion of testing, we were notified that the excavation would be compacted again in the future, therefore these testing results are not valid. The contractor used the nuclear gauge to determine proper compaction levels.

Summary of Observations from City Visits

- Backfill material used in the trenches varies from one city to another.
- The thicknesses observed in backfilled lifts often exceed the maximum depth of 12 inches as recommended by many cities.
- No moisture control of backfill material was observed to be used in the field.
- Backfill materials were compacted using large compaction equipment, which was observed as getting very close to the edge of the cut. Damage to pavement surfaces along the perimeter of the excavation occurred in these situations.
- Using large compaction equipment also resulted in achieving better compaction at the center of the utility cut compared to the edges of the cut and will be discussed later.
- During the excavation, material sloughing off extended into the zone of influence.
- Saturated excavated materials were observed to be cleaned into the trench during the backfilling process.
- Field and laboratory tests were performed on backfill material samples and are documented in the next two sections.

FIELD INVESTIGATION

Field testing was conducted to determine properties such as dry density, moisture content, stiffness, and deflection. Measurements of dry density and moisture content are important for the determination of compactive properties of backfill materials in the field. Stiffness is an equally important parameter, when compared to dry density and moisture content, which defines an engineering property of the soil. Furthermore, deflections were determined to assess the amount of distress occurring in and around the utility cut.

Testing Methods

The tests conducted in the field on utility restoration sites are the Nuclear Density Gauge, Dynamic Cone Penetrometer (DCP), GeoGauge, Clegg Hammer, and Falling Weight Deflectometer. These tests were used for correlationing and directly obtaining soil properties during construction. Statistical analyses were conducted, including mean, standard deviation and coefficient of variation to evaluate the consistency of the field values.

Nuclear Density Gauge

The nuclear density gauge is an in situ device that measures both in-place density (lb/ft ^{3}) and moisture content (percent). This test is typically conducted according to ASTM D2922. This test requires certification since it emits radiation, therefore limiting operator use of the device. The two types of emitted radiation that generate data include gamma ray and neutron radiation. The gamma ray generates the density values and the neutron radiation generates the moisture reading. The source can be inserted up to 12 inches (30.5 cm) into the testing surface and measures a volume of 0.22 ft^3 (6229.7 cm^3). As a result of the radiation, many governmental agencies are eliminating the use of the Nuclear Density Gauge. The Nuclear Density Gauge used in the field was manufactured by Humboldt Manufacturing and conducted according to the manufacture specifications.

Dynamic Cone Penetrometer (DCP)

The DCP is an in situ device where measurements of penetration per blow (mm/blow) are obtained. In 2003, ASTM published a standard for use of the Dynamic Cone Penetrometer (DCP) (ASTM D 6951), Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications. The device works by using a standard 17.6 pound (8 kg) hammer, which is lifted to the handle and dropped to the anvil, forcing the rod to penetrate the compacted soil area. The greater the number of blows needed to penetrate the rod into the soil, the stiffer the material. The rate of penetration or penetration index (DCPI) is determined by calculating the weighted average using the following equation (Sawangsuriya and Edil 2004):

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

where:
H=total penetration depth

z=layer thickness
DCPI=penetration index for z

The rate of penetration (DCPI) has been correlated to the California Bearing Ratio, an in situ strength parameter (ASTM 2003). The CBR correlation for soils other than CL below CBR 10% and CH soils is as follows:

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

GeoGauge

The GeoGauge is a relatively quick and easy in situ test that directly generates stiffness (MN/m) and modulus (MPa) values of soils. Stiffness is equivalent to a force per displacement. These values are obtained by a 10 to 17 N force transmitted to the base of the instrument reading 25 frequencies between 100 and 196 Hz (Sawangsuriya and Edil 2004). As a result, the stiffness readings are generated as an average of the force per frequency transmitted (Sawangsuriya and Edil 2004). The test is limited to readings reaching about a 12 inches (300 mm) depth below the testing surface.

Clegg Hammer

The Clegg Hammer is a quick and easy in situ test that generates a Clegg Impact Value for further correlations with CBR, a determination of soil strength. ASTM Standard D5874, Standard Test Method for Determination of the Impact Value (IV) of a Soil, has been written for use of the Clegg Hammer. It is performed by dropping a 9.9 pound (4.5 kg) hammer from a height of 18.0 inches (45.7 cm). The hammer is dropped four times from the marking on the hammer body, where the highest IV (drop four) is read, indicating the deceleration of the hammer. Four blows are used since consistent results have been obtained through experiments, indicating that it produced adequate results and a greater number of blows were insignificant or had little effect on the IV (ASTM 1995). The relationship used for the determination of CBR is (Clegg 1986) as follows:

$$CBR = \left(0.24(IV) + 1\right)^2$$

Falling Weight Deflectometer (FWD)

The FWD is a device used to determine pavement structural properties. In this research, it is used to compare the vertical displacement (i.e., deflection) responses in and around the excavation. The decrease in deflection is an indication of a stiffer material and therefore increasing pavement life. This is done where a weight is dropped in a step loading sequence of approximately 6,000 pounds, 9,000 pounds, and 12,000 pounds (2722, 4082, and 5443 kg) which was chosen for comparison of subgrade reactions. This loading sequence is chosen based on loads applied as a result of different traffic levels and experience provided by the

Iowa DOT. The deflection basins (maximum point of deflection) are used to generate profiles of deflection under the loads stated above. Figure 51 shows the FWD that used in determining the profiled deflections.

Figure 51. Falling weight deflectometer

Results from Field Testing

Field testing was performed in Ames, Cedar Rapids, Davenport, and Des Moines. The testing results in Ames, Cedar Rapids, and Des Moines reflect data obtained from the surface of the trench before pavement surfacing. In Davenport, testing took place approximately 2 feet (0.61 m) from the surface since no further construction was to be completed the day of the visit. Testing at each lift would have been ideal at all sites, but due to safety reasons this was not feasible.

Ames: Hayes Avenue & 20th Street

The site in Ames is in a high traffic area, with both a high school nearby and heavy loading from the bus system. The site is shown in Figure 22 of the Construction Observations section, where its location is on the east bound lane next to the gutter pan. This trench was tested in three different locations to determine the uniformity of the construction process.

As a result of time constraints, only one nuclear gauge reading was obtained in the imported material. The nuclear density gauge generated a moisture content of 6.3%. The dry density values indicated a value of 115.6 pcf (18.4 kN/m^3). Comparing this dry density value to a calculated relative density, according to Table 7 in the literature review, this material was compacted to a medium dense state. According to Table 6 in the literature review, typical values for maximum dry unit weight and optimum moisture content of this compacted soil is 110.0 pcf to 125.0 pcf (17.3 kN/m^3 to 19.6 kN/m^3) and 11% to 16%, respectively. The dry density was in this range, however, the moisture content was significantly lower than optimum.

The impact values from the Clegg Hammer indicate a high value of 7.3 and low value of 5.9. The mean impact value obtained from the Clegg Hammer was 6.6, with a coefficient of variance of 0.99%. A high CBR value of 7.6% and low of 5.8% was calculated for the surface. The average CBR value was 6.7% with a standard deviation of 1.2 and coefficient of variance of 18.3%. According to Table 12 in the literature review, CBR values are below typical values for a SM classified soil of 20% to 40%.

The Dynamic Cone Penetrometer indicated an average mean Penetration Index (DCPI) of 26.7, with a coefficient of variance of 46.5%. A high DCPI value of 1.6 inches/blow (41.0 mm/blow) and low of 0.7 inches/blow (18.3 mm/blow) was obtained. Based on the mean DCPI values obtained, a mean CBR value of 11.3%, with a coefficient of variance equal to 41.2% was determined. Again, the CBR values resulted in values below the typical range of 20% to 40%.

The CBR values of all testing locations in imported material using the DCP indicate high and low values of 17.4% and 3.7% (see Figure 52). According to Table 17 in the literature review, typical CBR values for an SM classified material ranges from 20% to 40%, indicating the values obtained from the field are lower than these typical values. Based on Figure 52, the CBR results appear to be relatively consistent throughout the trench. The native material in the cut back region indicates a stiffer response near the surface, but with depth, these results had a similar stiffness response with the imported material. This may be an indication of the loss in lateral support during the excavation.

Cedar Rapids: Miami Drive & Sherman Avenue

The site in Cedar Rapids is in a low traffic area, but heavy loading from the bus system exists. The site is shown in Figure 22 in the Construction Observations section, where its location is on the south bound lane near the intersection of Miami Drive and Sherman Avenue. This trench was tested in nine different locations to determine the uniformity of the construction process.

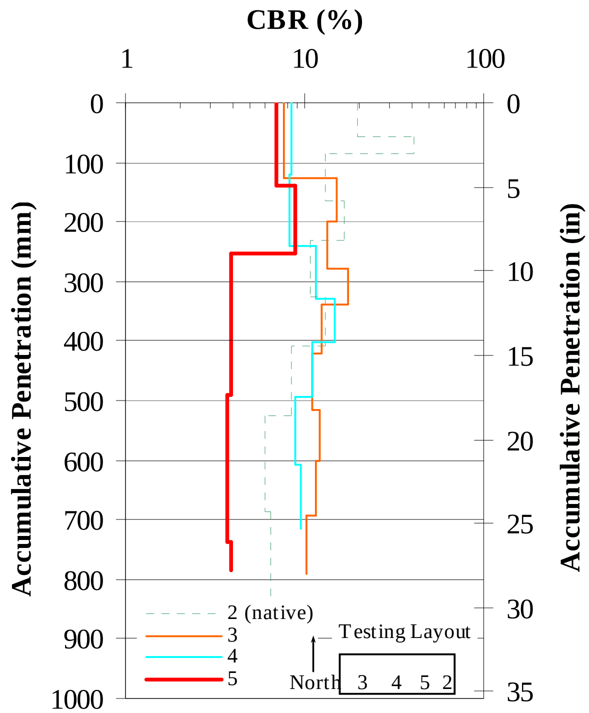


Figure 52. CBR profile for Ames

The nuclear density gauge generated moisture content results ranging from a high of 7.0% to a low of 5.0%, with a mean value of 5.7% and coefficient of variance equal to 13.3%. The dry density results ranged from a high of 126.6 pcf and low of 118.5 pcf (19.9 kN/m^3 and 18.6 kN/m^3). The mean was 122.9 pcf (19.3 kN/m^3), with a coefficient of variance of 2.1%. Using the mean dry density value to calculate relative density and Table 7 in the literature review, this material has an average classification of being in a dense state. Table 6 in the literature review indicates typical maximum dry unit weights and optimum moisture contents range from 105 pcf to 125 pcf (16.5 kN/m^3 and 19.6 kN/m^3) and 19% to 11%, respectively. The dry density values obtained in the field were in the upper range of typical values, however, the moisture contents were well below this typical range.

The GeoGauge test resulted in a high modulus value of 87.8 MPa and low of 65.6 MPa. The mean was 73.5 MPa, with a coefficient of variance of 9.0. The material stiffness values ranged from 10.1 MN/m to 7.6 MN/m. The mean stiffness value was 8.5 MN/m with a coefficient of variance equal to 8.2%.

The Clegg Hammer test resulted in a Clegg Impact Value (IV) ranging from 16.8 to 7.8. The mean IV value was 10.8, with a coefficient of variance of 25.2%. A range of CBR values calculated using the Clegg Hammer were a high of 25.3% to a low of 8.2%. The mean was 12.9%, with a coefficient of variance of 49.6%. These values ranged from just above to just below typical values of 10% to 20% stated in Table 12.

The Dynamic Cone Penetrometer resulted in a mean DCPI value of 0.72 inches/blow (18.3 mm/blow), with a coefficient of variance of 46%. Using the mean DCPI values for each location, CBR values ranged from a high of 25% to a low of 4.9%. A mean value of 13.3%

was obtained with a coefficient of variance of 41.6%. Again, these values ranged from just above to just below typical values of 10% to 20%, as stated in Table 12.

DCP results directly obtained from the field (i.e., no DCPI weighted average value) indicate a high CBR value of 40.5% and low of 2.6% (see Figure 53). When comparing these values to typical values stated in Table 12 of the literature review, this SC material had values above and below these the typical 10% to 20% CBR values. The results from Figure 53 indicate a higher CBR value near the center of the trench, to a low CBR value near the edge. Again, the CBR values are relatively consistent though the trench.

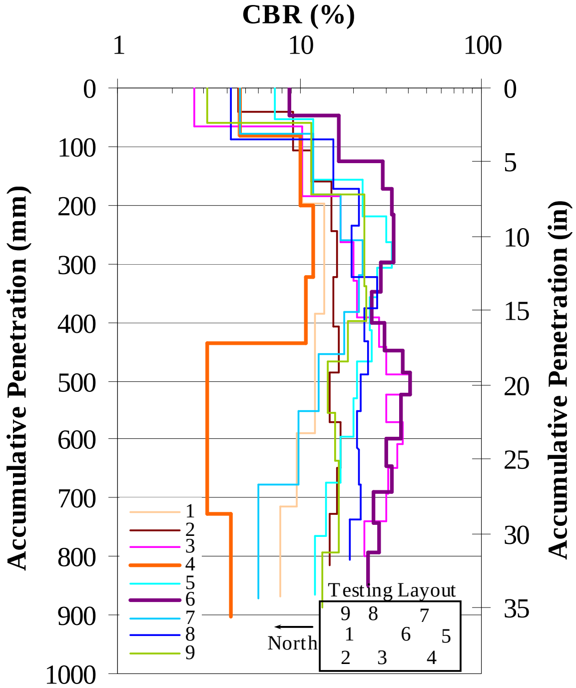


Figure 53. CBR profile for Cedar Rapids

Davenport: Iowa Street & 5th Street

The site in Davenport is in an alley on Iowa Street and 4th Street. The site is shown in Figure 22 of the Construction Observations section. This trench was tested in four different locations to determine the uniformity of the construction process.

The nuclear density gauge generated moisture content results ranging from a high of 7.8% to a low of 6.3%. The mean value was 7.1%, with a coefficient of variation of 9.3%. The nuclear density gauge also produced results for dry density with a high of 129.1 pcf (20.3 kN/m^3) and low of 122 pcf (19.2 kN/m^3). The mean was 127 pcf (19.9 kM/m^3), with a coefficient of variance of 2.7%. According to Table 7 in the literature review, this material has been compacted to dense state according to relative density standards. Table 6 in the literature review indicates a typical maximum dry density value of 115 pcf to 130 pcf (18.1 kN/m^3 to 20.4 kN/m^3) and OMC from 14% to 9%. Density values obtained were in the middle to upper range of these typical values. The moisture content was just below typical optimum moisture contents reported.

The GeoGauge resulted in a high modulus value of 80.5 MPa and low value of 58.7 MPa, with a mean value of 69.8 MPa and coefficient of variance of 17.2%. The material had a high stiffness of 9.3 MN/m and a low value of 6.8 MN/m, with a mean value of 8 MN/m, and coefficient of variance of 17.2%.

The Clegg Hammer resulted in a high IV of 12.8 and low value of 7.9. The mean IV achieved was 11.4% with a high coefficient of variance of 25.2%. A mean CBR value was 13.9%, indicating a low value compared to typical values of 20% to 40% in Table 12.

The Dynamic Cone Penetrometer resulted in an average mean DCPI value of 25 mm/blow, with a high coefficient of variance of 47.2%. A mean CBR value calculated from the mean DCPI was 9.2%, with a coefficient of variance of 36.2%. Again, this resulted in a low CBR value when compared to typical values of 20% to 40%.

DCP results directly obtained from the field indicate a high CBR value of 37.8% and low of 2% (see Figure 54). When comparing these values to typical GC classified materials values of 20% to 40% stated in Table 12 of the literature review, values resulted at or below this typical range. Figure 54 indicates a stiffer response with depth and again the CBR values were fairly uniform with depth, except location four where stiffness decreased.

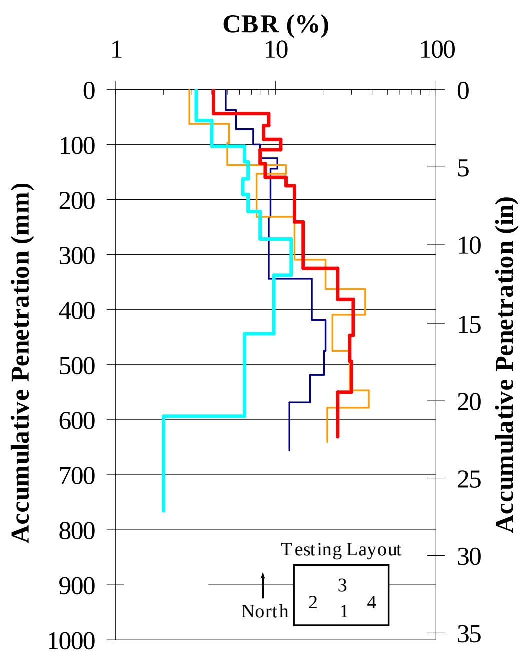


Figure 54. CBR profile for Davenport

Des Moines: E. 28th Street & E. Grand Avenue

The site in Des Moines is near the intersection of East 28th Street and East Grand Avenue. The street has bus traffic as well as frequent travel from vehicles. The site can be seen in Figure 22. This trench was tested in eight different locations to determine the consistency of the construction process.

The nuclear density gauge generated moisture content results ranging from a high of 11.7% to a low of 5.4%. The mean value was 7.6%, with a coefficient of variation of 20.8%. The highest dry density value obtained was 113.5 pcf (17.8 kN/m^3) and low value of 99.3 pcf (15.6 kN/m^3). The average was 105.9 pcf (16.6 kN/m^3), with a coefficient of variance of 2.9%. Comparing a mean calculated relative density values to Table 7 in the literature review, the material was compacted to a dense state. Table 6 in the literature review indicates a maximum dry unit weight of 110 pcf to 130 pcf (17.3 kN/m^3 and 20.4 kN/m^3) and optimum moisture content between 16% and 9%.

The GeoGauge resulted in a high modulus value of 51 MPa and low of 35.9 MPa, with a mean of 41 MPa and a coefficient of variance of 8.5%. The material had a high stiffness of 5.9 MN/m and a low value of 3.3 MN/m. The mean was 4.6 MN/m with a coefficient of variance of 11.8%.

The Clegg Hammer resulted in a high IV value of 12 and low of 4.8. The mean value was 8.1 with a coefficient of variance of 28.6%. The CBR values ranged from 15.1% to 4.6%, with a mean of 8.6% and therefore resulted in values below typical values of 20% to 50%.

The Dynamic Cone Penetrometer resulted in a mean DCPI value of 0.7 in/blow (17.9 mm/blow), with a coefficient of variance of 30%. A mean CBR value calculated from the mean DCPI was 12.5% with a coefficient of variance of 28.4%, again below typical values.

DCP results directly obtained from the field indicate a high CBR value of 34.9% and a low of 2.7% (see Figure 55). When comparing these values to typical SW classified materials values of 20% to 50% stated in Table 12 of the literature review, values resulted at and below this typical CBR range of this material. Figure 55, shows the material compacted near the center to have a stiffer response when compared to the material near the edge and again, the CBR values trend was fairly uniform.

A summary of the field results discussed above is shown in Tables 14 and 15, where data is organized according to each city. Values of high, low, mean, standard deviation, and coefficient of variation are also indicated for each test completed. When comparing high and low values, a trend should be observed where as the Clegg Impact Values increase, the DCPI values decrease, and the stiffness values increase. In other words, as the material becomes stiffer, the DCPI values decrease and the Clegg Impact Values increase. However, the use of granular material may be a result of these contradicting results and variability. It can also be observed from the results that moisture content is relatively consistent, with mean values ranging from 5% to 7%. This may be a result of the material being placed at ambient temperatures, with no additional moisture control.

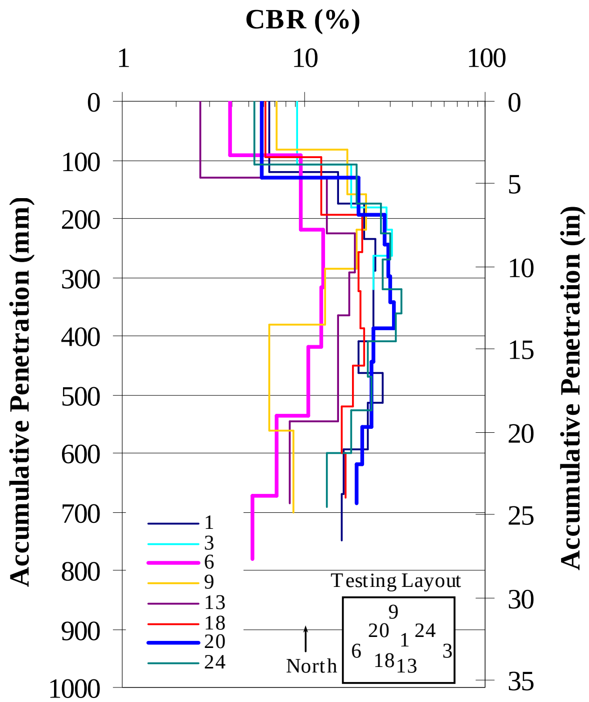


Figure 55. CBR profile for Des Moines

Dynamic Cone Penetration Analysis

As mentioned above, the Dynamic Cone Penetrometer (DCP) was conducted on four trench restoration sites in Ames, Cedar Rapids, Davenport, and Des Moines. Further analysis was conducted with the data to illustrate the stiffness of a material based on the number of blows per 10 cm, a method of evaluation mentioned in the literature review. Since the maximum penetration depth of the DCP used in the field was up to 1000 mm (3.33 feet), profiles reflect the top 3 feet of the testing area. Stiffness measurements were made entirely either in native material or imported material, since the trenches range from 8 to 10 deep feet as mentioned in the Construction Observations sections.

Readings in the field were obtained with the number of blows ranging from 1 to 10 for a given penetration depth, therefore an average blow count was calculated to determine the number of blows per 3.9-inch (10 cm) depth. Essentially, the data was broken into 3.9-inch (10 cm) depth profiles, to determine how many blows it would take to penetrate each layer. This was determined a feasible assumption since CBR data using the DCP was also plotted as an average with depth. As Figure 56(a), (b), and (d) illustrates, the number of blows needed to penetrate a 3.9-inch (10 cm) depth, tends to increase, level off, and then decrease with greater depth; however, Figure 56(c) has an increasing pattern, with a slight decrease with depth at 24 inches (600 mm). This decrease in the number of blows with depth may be a result of the large lift thicknesses used in the field. The larger the lift thickness, the more difficult it is to get proper compaction in the lower portion of the lift. The plots indicate a reduction in the number of blows to penetrate a 3.9-inch (10.0 cm) depth at approximately

1.5 feet (500 mm) below the backfilled surface. This would indicate that lifts should not exceed 1.5 feet (500 mm), as a result of a trend of decreasing values below this depth.

The greater the number of blow counts a material needs to penetrate this 10 cm depth, the stiffer the material is in this range. Therefore, the maximum blow count and depth was determined for each city. Ames DCP profile indicates a maximum blow count of seven at a 100 mm or 10 cm (3.9 inches) depth between about 7.9 to 19.7 inches (200 to 500 mm). In other words, seven blows were needed to penetrate the material from a depth of 7.9 inches to 11.8 inches (200 mm to 300 mm) and 19.7 inches to 15.7 inches (300 mm to 400 mm) for location three in Figure 56. Cedar Rapids indicated a maximum blow count of 18 to penetrate a 3.9-inch (100 mm) depth between 23.6 to 27.6 inches (600 to 700 mm) at location three. Testing conducted in Davenport indicated a maximum blow count of 13 to penetrate at a depth between 15.7 inches and 19.7 inches (400 and 500 mm) at location three, as well as 13 blows for location two to penetrate at a depth from 19.7 to 23.6 inches (500 to 600 mm). The site in Des Moines indicates a maximum of 15 blows between 3.9-inch (100 mm) depth of 11.8 inches to 15.7 inches (300 mm to 400 mm). These values indicate regions where stiffness is greatest, as well as the greatest number of blows obtained per 3.9 inches (100 mm) for a specific material, according DCP field data. Further testing should be conducted on each material, for potential direct correlations to be used in the field. The lift thicknesses in cities were estimated based on the observations. Ames used about a 2-foot (0.61 m) lift, Cedar Rapids about 3 feet (0.91 m) and Davenport about 4 feet (1.2 m). The construction of the Des Moines site was conducted before the research team arrived.

Table 14. Field testing results for Nuclear Gauge and GeoGauge

City / Sample	Number of testing locations	Nuclear Gauge		GeoGauge	
		Moisture Content	Dry Density	Modulus	Stiffness
Units		(%)	(lb/ft^3)	MPa	MN/m
Ames / 3/8 minus	1(Nuclear Gauge)				
High		6.3	115.6	-	-
Low		-	-	-	-
Mean		-	-	-	-
Standard Deviation		-	-	-	-
Coefficient of variance		-	-	-	-
Cedar Rapids / Crushed Concrete	9				
High		7	126.6	87.8	10.1
Low		5	118.5	65.6	7.6
Mean		5.2	122.9	73.5	8.5
Standard Deviation		0.7	2.5	6.6	0.7
Coefficient of variance		13.3	2.1	9	8.2
Davenport / ¾ minus	4				
High		7.8	129.1	80.5	9.3
Low		6.3	122	58.7	6.8
Mean		7.1	127	69.8	8
Standard Deviation		0.7	3.4	12	1.4
Coefficient of variance		9.3	2.7	17.2	17.2
Des Moines / Manufactured Sand	16				
High		11.7	113.5	51	5.9
Low		5.4	99.3	35.9	3.3
Mean		7.6	105.9	41	4.6
Standard Deviation		1.6	3.1	3.5	0.5
Coefficient of variance		20.8	2.9	8.5	11.8

Table 15. Field test results for DCP and Clegg Hammer

City / Sample	Number of testing locations	DCP		Clegg Hammer	
		Penetration Index	CBR	CIV	CBR
Units		(mm/blow)	(%)	Clegg Reading	$=(0.24(CIV)+1)^2$
		wt.avg	$292/(PI^{1.12})$		
Ames / 3/8 minus	3(DCP), 2(Clegg Hammer)				
High		41.0	11.3	7.3	7.6
Low		18.3	4.6	5.9	5.8
Mean		26.7	8.5	6.6	6.7
Standard Deviation		12.4	3.5	0.99	1.2
Coefficient of variance		46.5	41.2	15	18.3
Cedar Rapids / Crushed Concrete	9				
High		38.3	25.0	16.8	25.3
Low		9.0	4.9	7.8	8.2
Mean		18.3	13.3	10.8	12.9
Standard Deviation		8.4	5.5	2.7	2.7
Coefficient of variance		46.0	41.6	25.2	49.6
Davenport / ¾ minus	4				
High		42.6	12.0	12.8	16.6
Low		17.3	4.4	7.9	8.4
Mean		25.0	9.2	11.4	13.9
Standard Deviation		11.8	3.3	2.3	2.4
Coefficient of variance		47.2	36.2	20.4	34.7
Des Moines / Manufactured Sand	8				
High		25.9	15.6	12.0	15.1
Low		13.7	7.6	4.8	4.6
Mean		17.9	12.5	8.1	8.6
Standard Deviation		5.4	3.6	2.3	2.4
Coefficient of variance		30.0	28.4	28.6	61.9

When comparing the DCP blow count profiles in Figure 56 to the CBR plots in Figures 52, 53, 54, and 55, a trend was observed where the greater the number of blows needed to penetrate a 3.9-inch (100 mm) depth, the higher the CBR value obtained. When comparing the Ames data, the maximum number of blows for location three was 7 per 3.9 inches (100 mm), with a CBR value of approximately 15% and Cedar Rapids with maximum of 18 blows per 3.9 inches (100 mm), with approximately 43%. Davenport had a maximum of thirteen blows per 3.9 inches (100 mm) indicating a CBR value of 30% and Des Moines, with a maximum blow count of fifteen blows per 3.9 inches (100 mm), with a CBR value of 35%. Typical CBR values according to Table 13, indicate Ames material to have CBR values between 20% and 40%, Cedar Rapids between 10% and 20%, Davenport between 20% and 40%, and Des Moines between 20% and 50%. These typical CBR values obtained were then compared to the data obtained from each material (i.e., the number of blows per 3.9 inches (100 mm)). Material in Ames indicated CBR values on the lower range of typical values, Cedar Rapids resulted in CBR values significantly higher than typical values, and both Davenport and Des Moines indicated CBR values in the middle of typical CBR values.

Case Study

The city of Ames leaves constructed trenches unpaved for about one to two weeks, to let settlement occur under traffic before surfacing the trench. Therefore testing was done at the completion of the trench construction and 20th Street and then again two weeks later when surfacing preparations began. The testing conducted includes the nuclear gauge, DCP, and the Clegg Hammer. These tests were done to obtain dry density, moisture content, and stiffness values. These tests were conducted to determine if there are significant advantages to leaving a trench open for several weeks. Figure 57 shows the rough edges that are formed in the pavement during the trench excavation site before the removal of pavement, and Figure 58 shows the site during pavement removal where the trench edges are reshaped with an approximate 2-foot (0.61 m) cutback. Note in Figure 58 the amount of material disturbance resulting on the site due to the pavement removal.

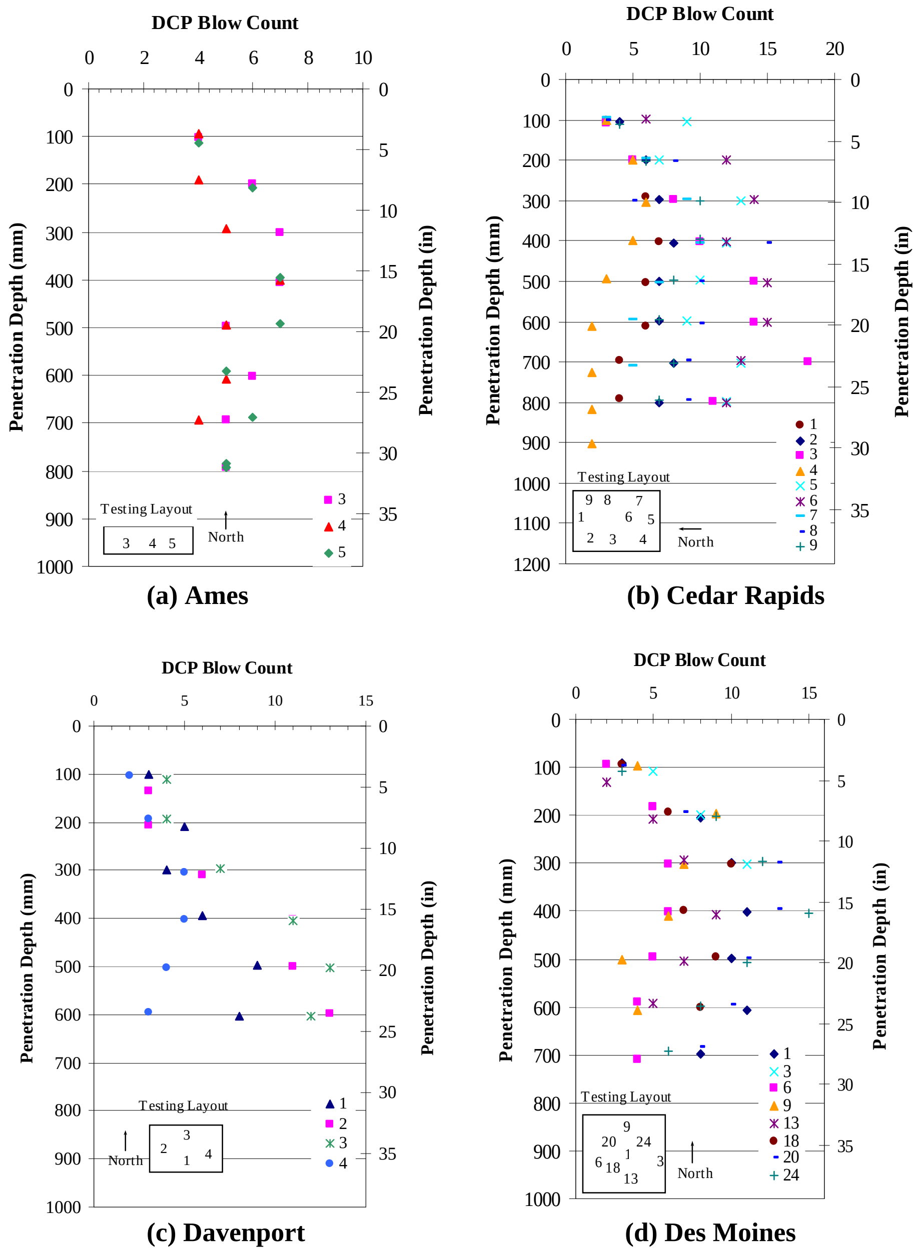


Figure 56. DCP blow count profiles

Figure 57. Site in Ames two weeks after construction

Figure 58. Pavement removal

Once pavement in the cutback region was removed, testing was completed on undisturbed, disturbed and native material throughout the trench. Figure 59 shows the placement of the five testing locations in the site layout. Locations one and two were tested in the native material after pavement removal and location four and five were tested on disturbed backfill material that occurred during the removal of pavement. Location three was tested on undisturbed material, before it was affected by the backhoe during pavement removal, and then again after it had been disturbed.

The results from the field two weeks after testing have been compared to average values obtained from testing after construction of the trench. The moisture content increased from an average of 4.7% after construction to 6.1% in an undisturbed state (location three). The dry density in the undisturbed state after two weeks was slightly higher with a value of 119.5 pcf (18.8 kN/m^3) compared to mean dry density originally at 118.8 pcf (18.7 kN/m^3).

Figure 59. Testing layout of trench

When comparing the DCP data, the undisturbed state two weeks after construction was slightly lower with a mean DCPI of 0.52 inches/blow (13.3 mm/blow) compared to and average of 0.59 inches/blow (15.1 mm/blow) two weeks prior, however once disturbed (location four and five) the mean DCPI increased to values ranging from 0.72 inches/blow (18.3 mm/blow) to 1.61 inches/blow (41.0 mm/blow). The calculated CBR value using the mean DCPI, was slightly stronger after two weeks in the undisturbed state with a CBR value of 16.1% compared to the average value of 14.1% after construction, however, once the site was disturbed for pavement removal, the CBR decreased to values ranging from 11.3% to 4.6%. Typical CBR values for this material ranged from 20% to 40%, indicating the field data to be lower than typical values.

The mean CIV obtained from the Clegg Hammer was 14.9 originally, compared to 13.2 obtained two weeks later. The disturbed locations had a lower CIV value of 7.3 and 5.9. Using these CIV values to calculate CBR, results showed that the material tested after construction had a higher CBR value of 20.9% compared to 17.4%, when the trench was left unpaved for several weeks. The disturbed state before surfacing began had significantly lower CBR values of 7.6% and 5.8%. A summary of these results are listed in Tables 16 and Table 17.

Table 16. Ames: Nuclear Gauge data comparison

City / Sample	Number of testing locations	Nuclear Gauge	
		Moisture Content	Dry Density
Units		*(%)*	*(lb/ft^3)*
Ames /3/8 minus	3		
High		5.4	119.4
Low		4.3	117.9
Mean		4.7	118.8
Standard Deviation		0.6	0.8
Coefficient of variance		13.6	0.7
Ames /3/8 minus (after 1 week open)	5		
1 (native material)		11.3	130.0
2 (native material)		10.9	128.6
3 (undisturbed)		6.1	119.5
3		-	-
4		-	-
5		6.3	115.6

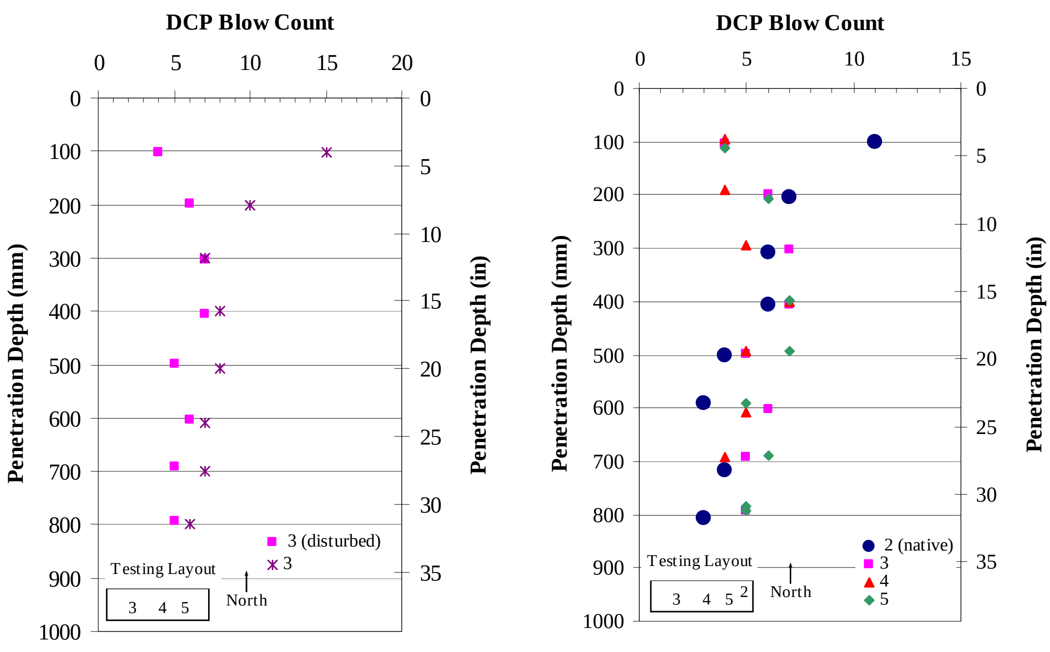
Table 17. Ames: DCP and Clegg Hammer data comparison

City / Sample	Number of testing locations	DCP		Clegg Hammer	
		Penetration Index	CBR	CIV	CBR
Units		*(mm/blow)*	*(%)*		*(%)*
		wt.avg	$292/(PI^{1.12})$		$=(0.24(CIV)+1)^2$
Ames /3/8$^-$	3				
High		16.9	15.0	15.0	21.2
Low		14.1	12.3	14.7	20.5
Mean		15.1	14.1	14.9	20.9
Standard Deviation		1.6	1.5	0.2	1.1
Coefficient of variance		10.4	10.9	1.0	1.6
Ames /3/8$^-$ (after 1 week open)	5				
1 (native material)		-	-	12.1	15.2
2 (native material)		22.2	9.1	15.5	22.3
3 (undisturbed)		13.3	16.1	13.2	17.4
3		18.3	11.3	-	-
4		20.9	9.7	7.3	7.6
5		41.0	4.6	5.9	5.8

The native material in the cutback region was tested using the Nuclear Gauge, Clegg Hammer, and DCP. The native material had a high dry density of 128.6 pcf and 130.0 pcf (20.2 kN/m^3 and 20.5 kN/m^3). When comparing the native material to the dry density of the imported material, the dry density difference was approximately 10.0 pcf (1.6 kN/m^3). The native material had a mean DCPI value of 0.87 inches/blow (22.2 mm/blow), with a calculated CBR value of 9.1% and had a CIV value of 12.1 and 15.5, with CBR values of 15.2% and 22.3%.

The data shows that after testing the trench near the surface, there was no significant advantage in leaving trenches open for several weeks. The material was loosened by the disturbance when pavement was removed. If further compaction with a vibratory source were to be used after the pavement cutback, the strength of the material may have increased.

The DCP blow counts were again compared with respect to a 3.9-inch (10 cm) penetration depth. Figure 60(a) indicates the disturbed material had a lower number of blows need to penetrate 3.9 inches (100 mm), near the top 7.9 inches (200 mm) of the trench. The disturbed material indicated a maximum blow count of seven to penetrate between 11.7 inches to 15.7 inches (300 mm and 400 mm). The undisturbed material indicated a maximum blow count of eight to penetrate a depth of 11.7 inches to 15.7 inches (300 mm to 400 mm). Figure 60(b) shows the DCP profile of the imported material and native material. The native material was stiffer at the top 7.9 inches (200 mm) of the trench. From 11.7 inches (300 mm) and deeper, the imported material showed a greater number of blow counts per 3.9 inches (10 cm), indicating a slightly stiffer material. The decrease in stiffness of the native material may be an indication of the loss in lateral support during the excavation.



(a) Disturbed and undisturbed material **(b) Native and disturbed material**

Figure 60. Ames DCP profile

Falling Weight Deflectometer Results

To monitor trench settlement and weakened areas, the Falling Weight Deflectometer (FWD) has been used following the pavement surfacing of the trench. The sites that have been monitored include (1) 20th Street in Ames & Hayes Avenue, (2) Miami Drive & Sherman Avenue in Cedar Rapids, and (3) East Grand Avenue & East 28th Street in Des Moines. Appendix B contains raw data from each FWD testing location.

Ames: 20th Street & Hayes Avenue

The FWD was tested on a pavement surface consisting of 8 inches of asphalt. The construction of this trench involved a pavement cutback before pavement could be placed on the excavated area. The trench was originally tested on November 22, 2004 and again on April 11, 2005 with the FWD. The dimensions of the original utility cut and pavement cut are shown in Figure 61. FWD responses were tested at 17 feet and 2 feet (5.2 m and 0.6 m) from the east and west edge of the cutback, the center of the cutback, and the east and west edge and center of the trench, to determine the effect of the influence zone on the trench. The 17-foot (5.2 m) deflection in the far field of the utility cut area was measured assuming this point represents the response of undisturbed pavement (i.e., utility cut has negligible influence on the pavement system). Figure 61 shows these locations and Figure 62 shows the response profiles of the maximum point on FWD deflection basins. Figure 62 shows profiles for test #1 (November 22, 2004) and test #2 (April 11, 2005), therefore indicating deflection results with time. It is evident from the profile that within this cutback region material is weakened, resulting in a noticeable deflection. This cutback region is located in the zone of influence (2 feet to 3 feet (0.6 m to 0.9 m) around the perimeter) as discussed earlier. The deflection in this influence zone was significant compared to deflections at other points in the trench as a result of a decrease in lateral support during the excavation. Compaction in this region before surfacing may have strengthened this area and lowered the deflection.

As the literature review stated, an increased deflection in this zone of influence is an indication of premature patch deterioration resulting from a strength reduction of material in this zone. Figure 62 also indicates a minimum deflection near the center of the trench and is comparable to the deflection existing in the far field. When comparing the FWD results with time, the profiles indicate an increase deflection within this approximate five-month period. The deflection difference (i.e., from test #1 to test #2), ranged from a maximum and minimum value of 11 mils and 2 mils at a 12,000 pound (5443 kg) load. The 9,000 pound (4082 kg) load had a maximum and minimum deflection difference of 8 mils and 2 mils. The lighter loading was run at two different loadings and therefore cannot be compared. Note that the first test was conducted in the November and the second test in April, therefore a seasonal effect is visible in the deflections. The figure also shows that lighter loads (e.g., 3000 pound (1361 kg) loads induced by cars) result in a lower deflection when compared to greater loads simulating loads, such as 9,000 pound (4082 kg), induced by trucks.

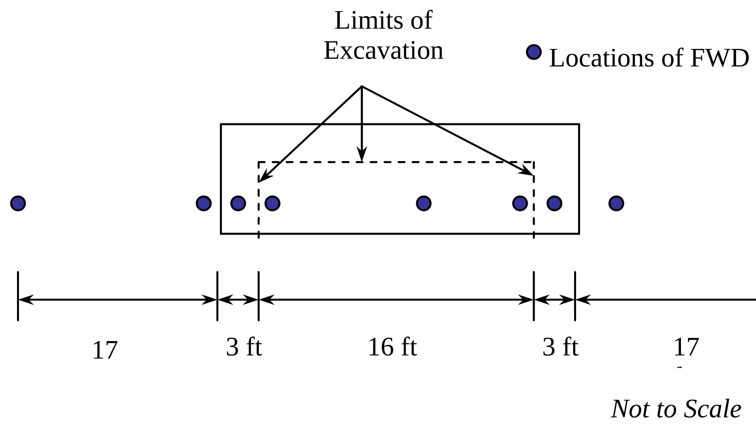


Figure 61. Ames FWD layout

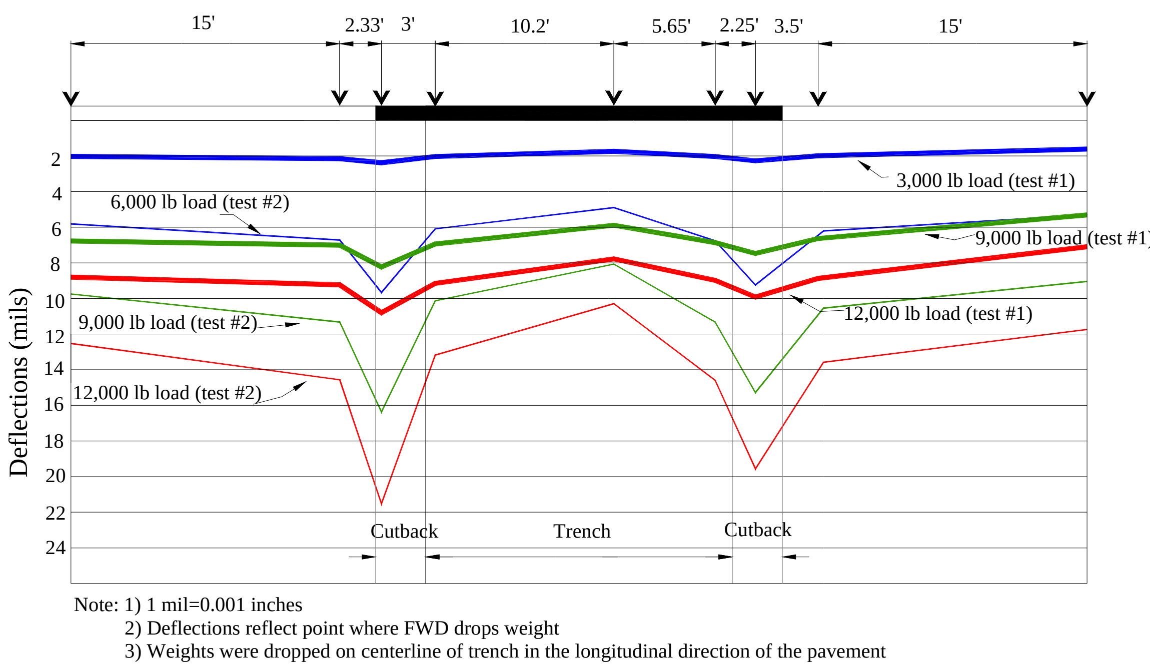


Figure 62. Ames FWD response profile

Cedar Rapids: Miami Drive & Sherman Avenue

The FWD was tested in a composite material with 6 inches (15.2 cm) of concrete and a 2-inch (5.1 cm) asphalt overlay. During the construction of this trench in Cedar Rapids, the edge was weakened by the backhoe rolling over the open edge while moving out of the way for a dump truck. This represents a situation where a cutback and further compaction in this region may have been advantageous. The site was visited about three months after construction and raveling was observed on the pavement. Figure 63 illustrates the pavement distress surrounding the trench.

This site was tested October 25, 2004 (test #1) and then again April 20, 2005 (test #2) to see the effect of deflections with time. When designing the FWD testing layout, this damaged region was of great importance to determine what effect additional stress has on the edge of the open excavated area. Figure 64 shows the FWD drop locations and Figure 65 shows the influence zone again causing the greatest deflection, specifically near the damaged edge of the trench. The distressed point of the trench, was missed on the second visit. A difference in deflections with time ranged from 0.5 mils to 12 mils at a load of 9,000 pounds (4082 kg). Again the seasonal effect of the ground thawing increased the deflections observed in the data.

Figure 63. Cedar Rapids pavement distress

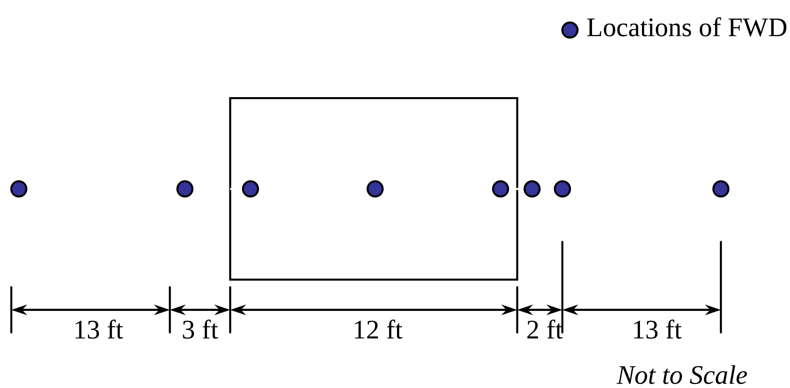
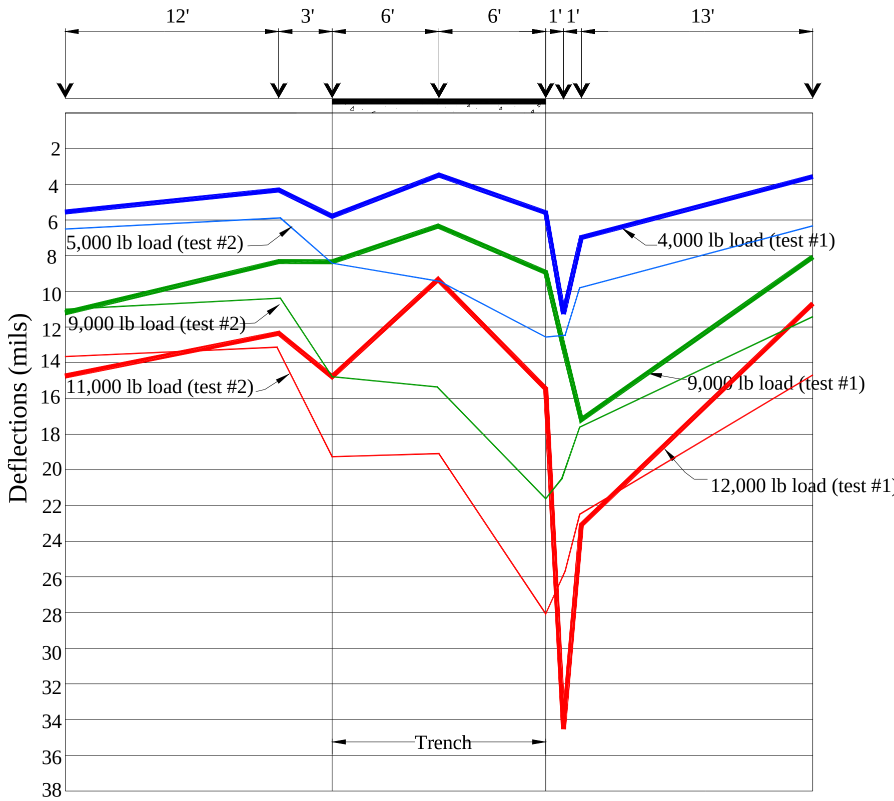


Figure 64. Cedar Rapids FWD layout



Note: 1) 1 mil=0.001 inches
2) Deflections reflect point where FWD drops weight
3) Weights were dropped on centerline of trench in the longitudinal direction of the pavement

Figure 65. Cedar Rapids FWD response profile

Des Moines: E. 28th Street & E. Grand Avenue

The FWD was tested on 8 inches of concrete pavement. The Des Moines site was constructed with a cutback, but again no compaction was performed in the cutback region. The utility restoration was tested with time on October 25, 2004 (test #1) and then again April 13, 2004 (test #2). Figure 66 shows the FWD drop locations and Figure 67 shows the FWD profile. Again the influence zone around the trench shows the deflection to be greater in this region. The figures show that a concrete patch provides lower deflection values in the zone of influence. This trench was also tested with time on October 25, 2004 and then again April 13, 2004. During the second visit, however points in the cutback region were missed on the left hand side of the trench. Figure 67 shows the deflections significantly less in concrete pavements as opposed to asphalt or composite pavements.

In general, each FWD plot indicates a significant lower vertical deflection in the region just outside the excavated area, leading to an indication of decreased pavement life.

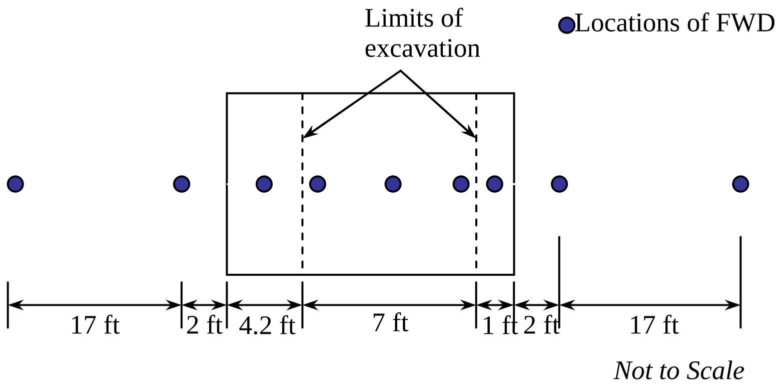


Figure 66. Des Moines FWD layout

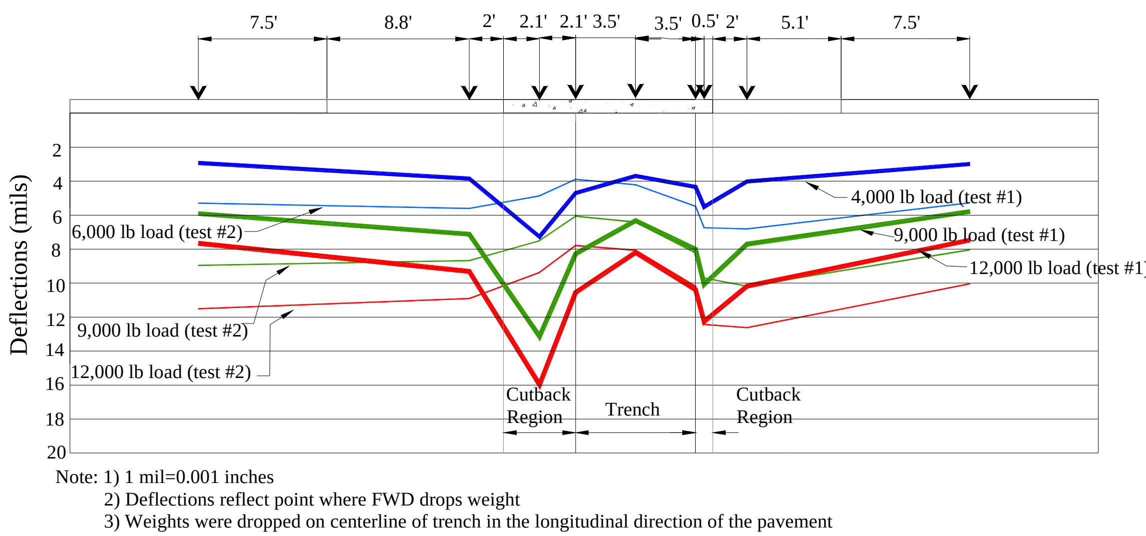


Figure 67. Des Moines FWD response profile

Summary of Findings from Field Testing

- Using the nuclear gauge, the average measured dry density values for the compacted backfill material in Ames was 115.6 lb/ft^3 at a moisture content of 6.3%, in Cedar Rapids was 122.9 lb/ft^3 at a moisture content of 5.2%, in Davenport was 127.0 lb/ft^3 at a moisture content of 7.1%, and in Des Moines was 105.9 lb/ft^3 at a moisture content of 7.6%.
- Backfill materials used in Cedar Rapids and Ames (classified as SM) and Davenport (classified as GC) provided higher density values compared to the manmade sand (classified as SP) used in Des Moines, although the moisture contents wer similar.
- Mean CBR values using the DCP correlation is summarized in the following table:

Table 18. Mean CBR values/DCP correlation

City	Mean CBR (%)
Ames	8.5
Cedar Rapids	13.3
Davenport	9.2
Des Moines	12.5

- Mean CBR values calculated using the Clegg Impact Values is summarized in the following table:

Table 19. Mean CBR values/Clegg impact

City	Mean CBR (%)
Ames	6.7
Cedar Rapids	12.9
Davenport	13.9
Des Moines	8.6

- DCP results using 10 cm/blow results indicate a stiffness reduction at approximately 1.5 feet below the surface, which is approximately the surface of the previous layer.
- It was observed that waiting two weeks after construction of a trench to affect the surface patch made little difference in the strength of the backfill near the surface since the material was disturbed and no further compaction was used.
- When comparing CBR profiles, in most cases it was observed that higher CBR were obtained in the center of the trench and lower values along the utility cut edge. The response of FWD tests shows the same trend. This could be a result of using large compaction equipment.
- Visible distress was seen near the utility cut edge in Cedar Rapids by visual observations and deflection data using the FWD.
- The “zone of influence” in the cutback region is apparent from the profiles constructed using FWD data.
- Recommendations regarding design values for the use of the DCP in compaction monitoring cannot be made at this time because of a need to continue monitoring restoration performance.

LABORATORY INVESTIGATION

Laboratory tests were conducted on various types of backfill material. These tests include particle size distribution curves with sieve and hydrometer analysis, Atterberg limits, specific gravity, water content, standard Proctor, and minimum and maximum relative density according to the corresponding American Society for Testing and Materials (ASTM) Standards. A granular collapse test was also performed; however, no standard exists for this test. These laboratory tests were performed to determine material properties and classify the materials used in the field, as well as compliment field data obtained.

Testing Methods

Particle size distribution & Hydrometer

This test was conducted according to ASTM D422, Standard Test Method for Particle-Size Analysis of Soils. A 50 gram sample was used in the Hydrometer Analysis for determining the amount of fine-grained particles passing the #200 sieve.

Atterberg Limits

This test was performed according to the ASTM D 4318-95a, the Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The results assist in the classification of the materials

Specific Gravity

Specific Gravity was first completed according to ASTM D854-92, Standard Test Method for Specific Gravity of Soils. Since obtaining results was relatively time consuming, the test was completed again using the Helium Pychometer. The test was conducted according to the standards outlined by Quantachrome Instruments, the manufacture of this devise. Results were found to be more accurate and time efficient when using the Helium Pycnometer and therefore was used for specific gravity determination of the remaining samples.

Minimum and Maximum Density using the Vibrating Table

A majority of state DOTs use ASTM and AASHTO Proctor test for granular materials, however it is difficult to achieve well defined optimum moisture content and maximum dry density for these materials using the proctor test (Jayawickrama et al. 2000). Therefore ASTM D4253 and ASTM D4254 represent standards for the determination of maximum and minimum index density and unit weight of soils using a vibrating table for granular materials. Hence, materials using ASTM D4253 and ASTM D4254 are more applicable since granular material used in the field is generally compacted using a vibrating plate. Using results from these tests, a relative density value can be determined. ASTM D4253 defines maximum index density/unit weight as “the reference dry density/unit weight of a soil in the densest state of compactness that can be attained using standard laboratory compaction procedures that minimizes particle segregation and breakdown” and minimum index density/unit weight as “the reference dry density/unit weight of a soil in the loosest state of compactness at which it can be placed using standard laboratory procedure which prevents bulking and minimizes particle segregation”. During the testing of material samples, the materials were reused as a result of the limited amount of material available.

Standard Proctor

The Standard Proctor test was conducted according to ASTM D698-91, Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort. Material for the Cedar Rapids sample was not reused. Material used in Ames and Des Moines was reused due to the amount of material available.

Granular Collapse Test

A granular collapse test was conducted to determine the collapse potential of a granular material. This test was completed using a clear plexi-glass 8-inch (20.3 cm) diameter cylinder with an open top and bottom. Geofabric was used on the bottom of the cylinder to minimize the amount of fines lost during the collapse simulation. The cylinder was placed on a five gallon bucket allowing height measurements to be made easily. The material was placed by dumping it from a height of 3 feet (0.91 m). The material height in the cylinder ranged from 6 inches to 12 inches (15.2 cm to 30.5 cm) deep and the initial height was measured in three locations. This apparatus is illustrated in Figure 68. Water was added by spraying the side of the cylinder to prevent an induced collapse due to water pressure. Height measurements were taken until collapse was complete, generally two flooding cycles. Since the material was placed loose (i.e., no mechanical compaction), this simulation represented a worse-case scenario. The collapse index (CI) was calculated as shown below:

$$CI = \left(\frac{\Delta H}{H_i}\right)*100$$

where:
ΔH=initial height-final height
H_i=initial height

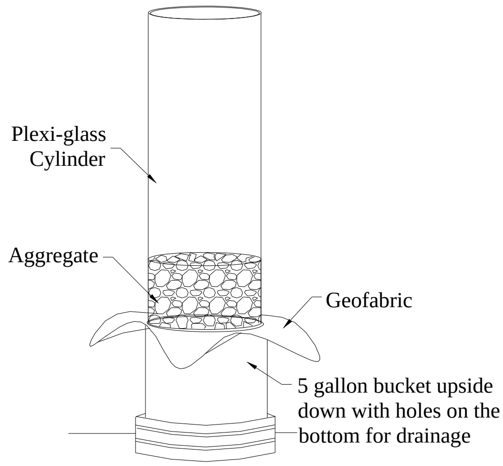


Figure 68. Granular material collapse potential apparatus

Results from Laboratory Testing

Classification

A sieve analysis was conducted on all imported samples obtained from the field visits. The gradations of each sample obtained, as well as the gradation specified by the Iowa Department of Transportation (Iowa DOT) for use as a backfill material, are shown in Table 20. Table 21 shows the gradation of limestone screenings and the SUDAS specification for granular material that is suitable as bedding and backfill material (Class I).

The Iowa DOT granular backfill gradation limits for trench backfill materials (Gradation No. 32), was compared with the gradations of backfill materials used by different cities throughout Iowa. The granular backfill specification in Iowa is relatively broad, thereby allowing a variety of qualified backfill materials for use. The results obtained from the sieve analyses are plotted in Figure 69, along with the IDOT Specification. It can be seen that the results remain in the specified range, except for material passing the No. 200 sieve. The material obtained from Des Moines was found to be on the upper end of the required gradation provided by Iowa DOT. Backfill materials of Ames 3/8 minus, Cedar Rapids 3/4 minus, and Davenport samples have a percentage passing sieve No. 200 greater than the percentage allowed by Iowa DOT Gradation No. 32, indicating a high fine content. Coefficient of uniformities were calculated for each material and shown in Table 22. The results indicate that all of the materials are well graded, with an exception of the SUDAS specification.

Table 20. City gradations

		Ames/ 3/8 minus	Des Moines/ Manufactured sand	Cedar Rapids/ ¾ minus	Davenport/ ¾ minus	Iowa DOT-No.32
Sieve Size	**Diameter**	**% Passing**	**% Passing**	**% Passing**	**% Passing**	**% Passing**
3 in	76.2	100	100	100	100	100
1 in	25.4	100	100	100	100	-
3/4 in	19.05	100	100	96.2	90.1	-
3/8 in	9.525	98.9	99.1	78.9	56.2	-
No.4	4.75	74.4	98.1	60.8	36.5	-
No.8	2.3876	-	-	-	-	20 to 100
No.10	2	46.5	80.2	45.4	24.8	-
No.20	0.85	37	47.8	34	20.4	-
No.40	0.425	28.9	27.5	30.8	17.9	-
No.60	0.25	22.4	15.8	29.6	16.6	-
No.100	0.15	17.9	11.5	28.3	15.5	-
No.200	0.075	14.4	10	26.8	14.2	0 to 10

Table 21. Limestone screenings and SUDAS material gradation specification

		Limestone screenings	SUDAS Specification
Sieve Size	**Diameter**	***% Passing***	***% Passing***
1½ in	38.1	100	100
1 in	25.4	100	95 to 100
3/4 in	19.05	100	-
1/2 in	12.7	100	25 to 60
3/8 in	9.525	100	-
No.4	4.75	97.7	0 to 10
No.8	2.3876	-	-
No.10	2	71	-
No.20	0.85	55.1	-
No.40	0.425	39.8	-
No.60	0.25	29.4	-
No.100	0.15	22.3	-
No.200	0.075	17	-

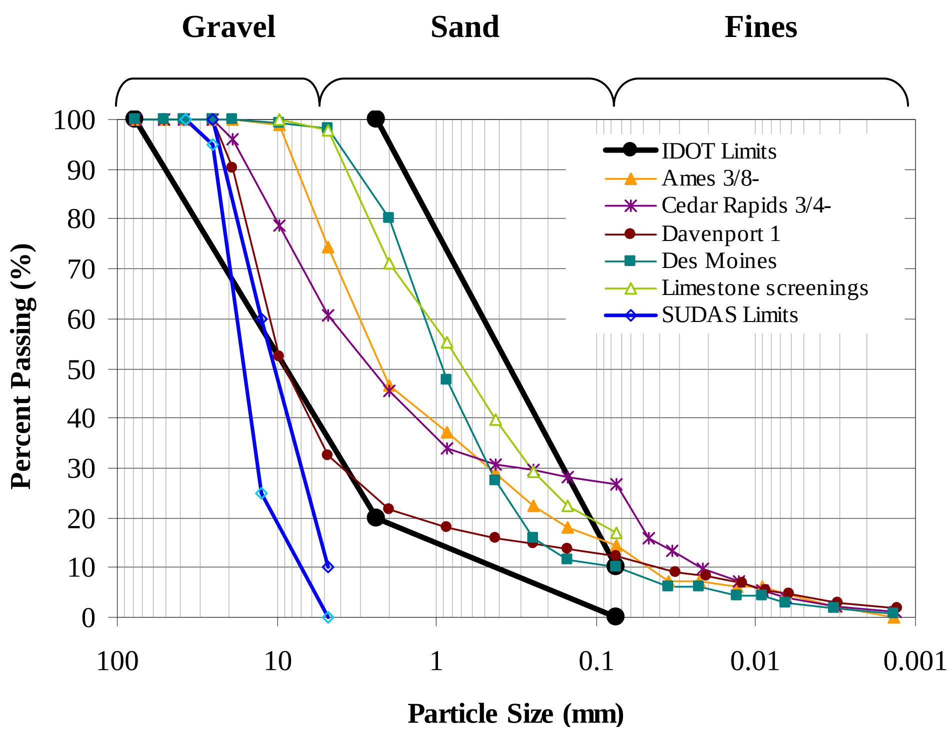


Figure 69. City gradation plot

Table 22. Coefficient of uniformity comparison

Material	Coefficient of Uniformity
Ames/ 3/8 minus	75
Des Moines/ Manufactured sand	16.7
Cedar Rapids/ ¾ minus	225
Davenport/ ¾ minus	275
Iowa DOT-No.32 (Upper Limit)	6.7
Iowa DOT-No.32 (Lower Limit)	33.3
Limestone screenings	83
SUDAS Specification (Upper Limit)	2.8
SUDAS Specification (Lower Limit)	2.6

Table 23 shows a summary of the results obtained from gradation analysis, specific gravity, and relative density laboratory tests for granular backfill materials used in the field at Ames, Cedar Rapids, Davenport, and Des Moines. According to the Unified Soil Classification System (USCS), Ames 3/8 minus classifies as a silty sand (SM), Cedar Rapids 3/4 minus classifies as a clayey sand (SC), Davenport backfill material as a clayey gravel (GC), and Des Moines backfill material as SW-SM.

According to Table 2 in the literature review provided by NAVFAC (1986), these soils range from 4 to 10 in desirability as a fill in a roadway, with 1 being the most desirable and 14 being the least desirable. Des Moines manmade sand (SW) was ranked a 2 where frost heave is possible. Ames 3/8 minus was ranked 10 and Cedar Rapids 3/4 minus (SC) was ranked a 6 for fills in roadways with possible frost heave.

The literature review also indicates that a majority of backfills used in various states fall into the AASHTO classification of A-1 and A-2 which is stated to be an excellent to good subgrade material. The backfill materials used by different cities in Iowa are all classified in one of those categories.

Table 23. Laboratory results of imported material

City / Sample	**Soil Classification**				**Specific Gravity**	**Maximum Density**	
Units	**AASHTO**		**USCS**			**γ_{Max}** *(lb/ft^3)*	**Bulking Water Content** *(%)*
Ames / 3/8 minus	A-1-a	Stone fragments, gravel and sand	SM	sand/silt	2.67	140	6 to 8
Cedar Rapids / ¾ minus	A-2-4	Silty or clayey gravel & sand	SC	sand/clay	2.76	130	7 to 10
Davenport / ¾ minus	A-1-a	Stone fragments, gravel and sand	GC	gravel/contains clay	2.74	140	4.5 to 7
Des Moines / manufactured sand	A-1-b	Stone fragments, gravel and sand	SW-SM	Well graded sand/silt	2.7	138	7.5 to 11

Bulking Moisture Phenomena

The bulking moisture phenomena discussed in the compaction methods section of the Literature review is a critical aspect occurring in granular materials at a certain moisture contents. A microscopic view of the capillary tension or suction occurring on the surface of the granular particles was obtained using a light microscope. The granular material was obtained from Des Moines and was wetted to a moisture content of 9% and magnified to 200 μm (see Figure 70). To further explain this bulking moisture phenomenon, a schematic and description of the bulking moisture affect on granular particles is shown in Figure 71. In this figure, a plot in the upper portion indicates the bulking moisture content range and furthermore the increase in collapse potential of the material in this region. An illustration of the bulking effect is shown in the bottom of this Figure. From left to right, the granular particles are initially dry, then water is added to the material, with the addition if more water, a suction forms between particles forming tension and an air void. With the addition of more water the tension is released and collapse occurs, leading to a more dense material.

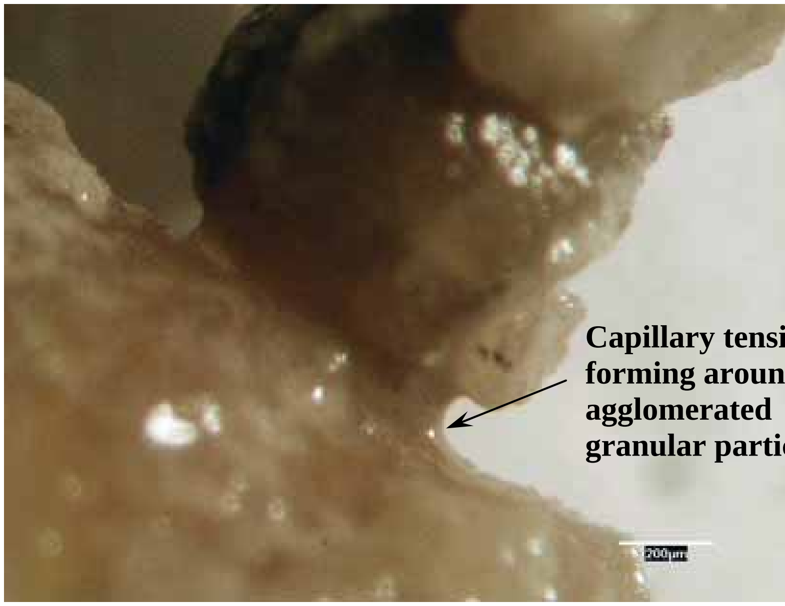


Figure 70. Microscopic view of capillary tension

Relative Density or Minimum and Maximum Density

The minimum and maximum density tests were used to determine the bulking moisture content and relative density of granular materials used throughout Iowa. The minimum and maximum density tests only require testing at an oven dry state; however this test was conducted further by increasing moisture contents for determination of the bulking moisture content. Ideally, materials in the field should be placed at a moisture content exceeding the bulking moisture content to prevent the collapse (i.e., settlement) of granular particles.

A backfill material known as 3/8 minus limestone is generally used in Ames for utility cut restorations. This material has a bulking moisture content of 7% (see Figure 72). The nuclear gauge was used in the field to determine moisture content in several locations throughout the trenches top layer, which was found to range from 4.3% to 5.4%. This material at the surface was placed just under the critical bulking moisture content which increases the potential of collapse due to seasonal changes of moisture contents and could be watered to overcome this collapse potential. Figure 72 also shows a maximum and minimum compacted dry density of 140 pcf and 90 pcf (22.0 kN/m^3 and 14.1 kN/m^3) respectively, with a density difference of approximately 50 pcf (7.9 kN/m^3) for this material.

The Cedar Rapids 3/4 minus material has a bulking moisture content at 8.5% (see Figure 73). In the field, the trench top layer was tested in several areas with a maximum moisture content of 7% and a minimum moisture content of 5%. This material was placed just below the bulking moisture content. Therefore this material should have been watered in the field to exceed the critical region and lower collapse potential. Figure 73 shows a maximum and minimum dry density of about 130 pcf and 85 pcf (20.4 kN/m^3 and 13.3 kN/m^3) respectively, with a density difference of approximately 45 pcf (7.1 kN/m^3) for this material.

The material used in Davenport has a bulking moisture content of 5.5% as shown in Figure 74. The moisture contents of this material used in the field ranges from 6.3% to 7.8%. This material was placed above the bulking moisture content. Figure 74 shows a maximum and minimum compacted dry density 140 pcf and 85 pcf (22 kN/m^3 and 13.3 kN/m^3) respectively, with a density difference of approximately 55.0 pcf (8.6 kN/m^3).

The manufactured sand obtained from Des Moines had a bulking moisture content of 9% (see Figure 75). After testing the site in the field, a maximum water content obtained was 11.7% and a minimum of 5.4%. Therefore, backfill material was placed at and around the bulking moisture content. Figure 75 shows a maximum and minimum compacted dry density of about 135 pcf and 80 pcf (21.2 kN/m^3 and 12.6 kN/m^3) respectively, with a density difference of approximately 55.0 pcf (8.6 kN/m^3).

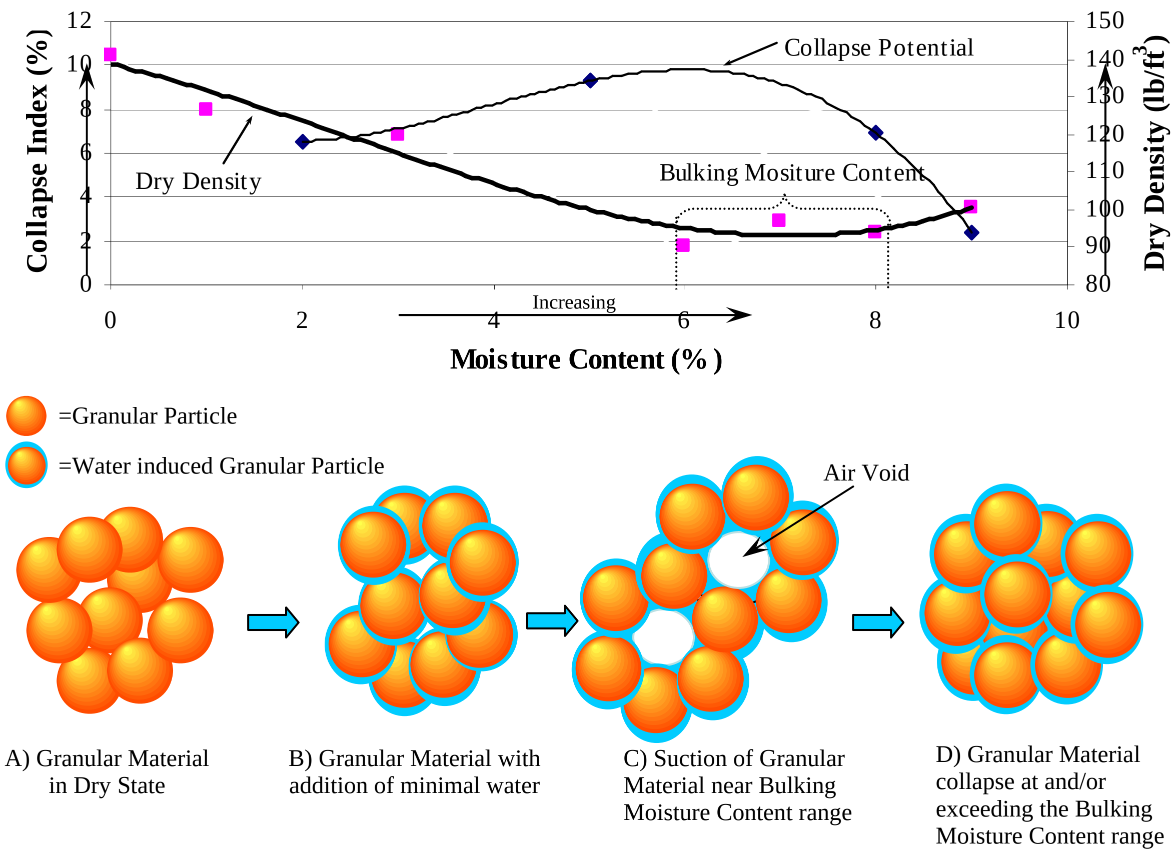


Figure 71. Bulking moisture schematic

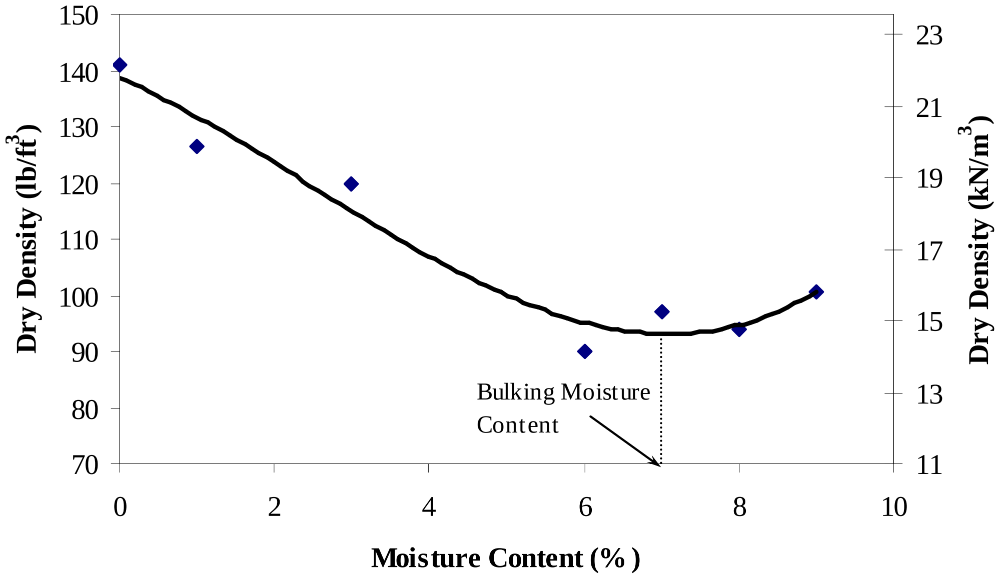


Figure 72. Ames 3/8 minus maximum density test results, SM

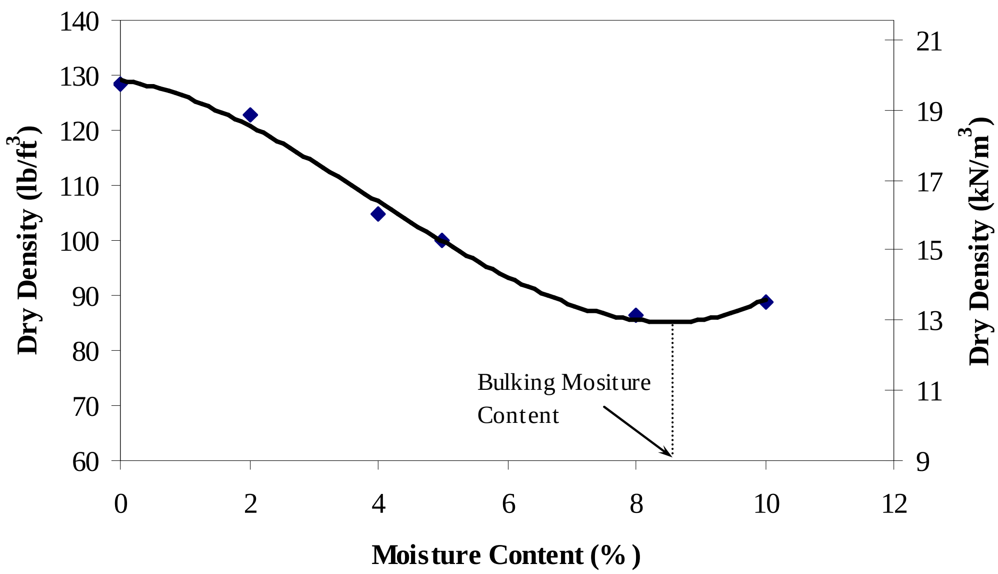


Figure 73. Cedar Rapids 3/4 minus maximum density test results, SC

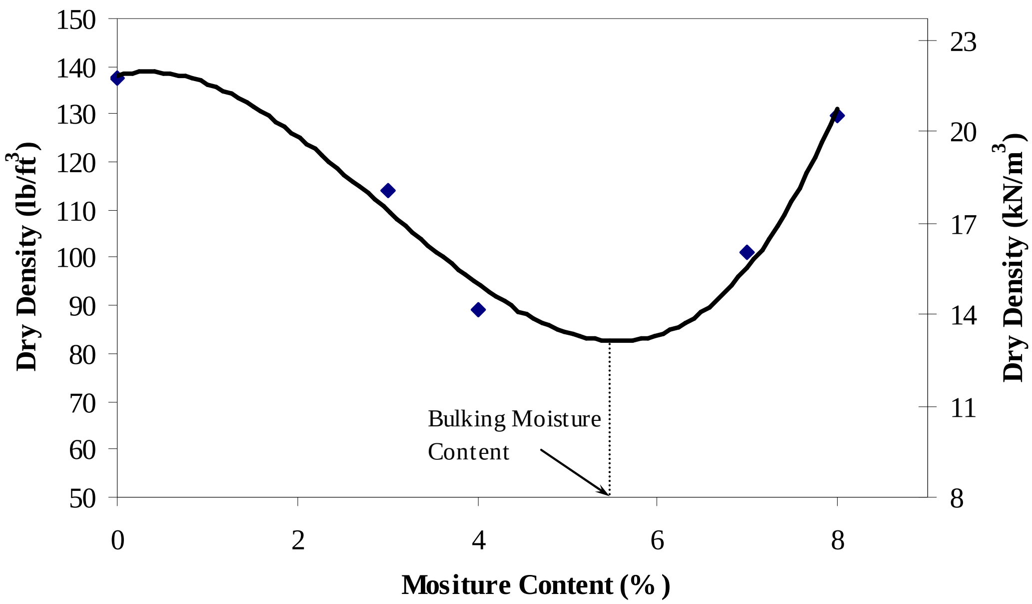


Figure 74 . Davenport maximum density test results, GC

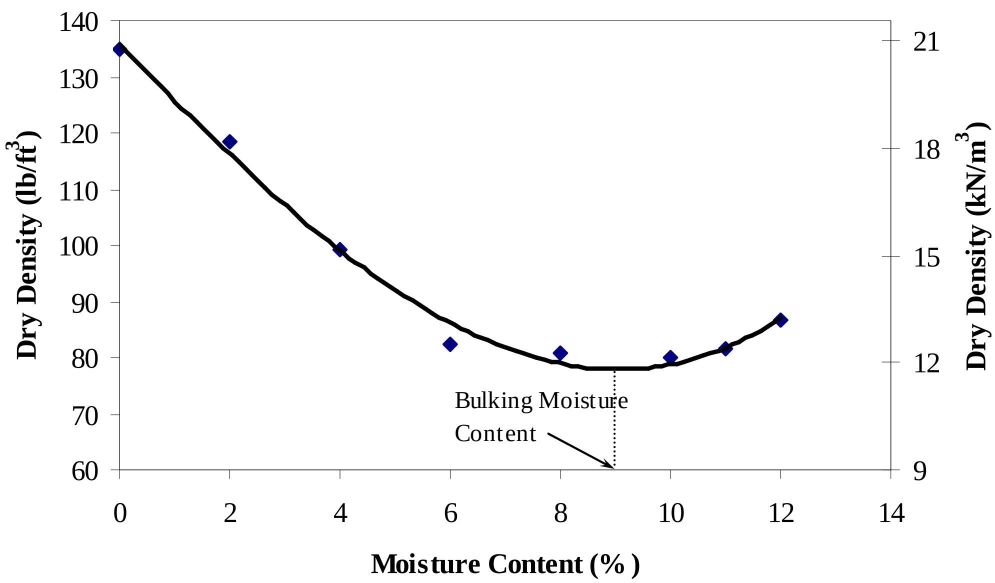


Figure 75. Des Moines maximum density test results, SW-SM

Table 24, shows a summary of moisture contents from the field, bulking moisture contents, and maximum densities obtained in the laboratory.

Table 24. Moisture content and maximum density summary

Sample	Classification	γMax (lb/ft^3)	*W*% (Bulking)	*W*% (Field)
Ames	SM	140	7	4.3 to 5.4
Cedar Rapids	SC	130	8.5	5 to 7
Davenport	GC	140	5.5	6.3 to 7.8
Des Moines	SW-SM	138	9	5.4 to 11.7

Since backfill materials used in utility cuts at several locations across Iowa had a moisture content within or just below the bulking moisture content, a granular collapse potential test was conducted on these materials to further investigate the collapse mechanism. The collapse index is shown in Figures 76, 77, and 78. Ames 3/8 minus indicates a collapse of approximately 9%, Cedar Rapids, 8.5%, and Des Moines, 24%. The SUDAS Class I specification was tested in addition to the samples currently used in the field (see Figure 79). The SUDAS Class I specification indicated a very low collapse potential of approximately 0.4%. Limestone screening had the highest collapse potential of approximately 35% (see Figure 80). Therefore, the collapse potential obtained from the granular collapse test varied from about 35% to less than 0.5%, depending on the material tested. Table 25 shows a summary of the engineering properties of each material.

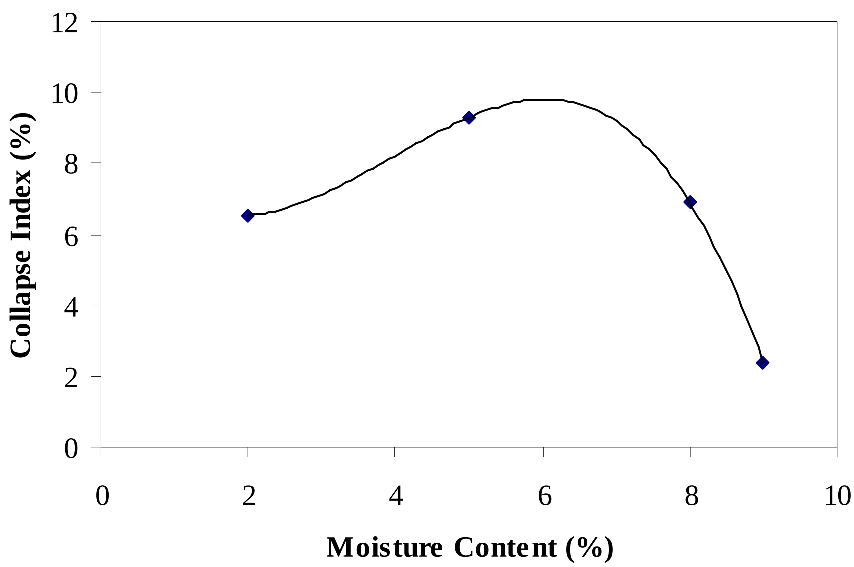


Figure 76 . Ames 3/8 minus collapse index profile, SM

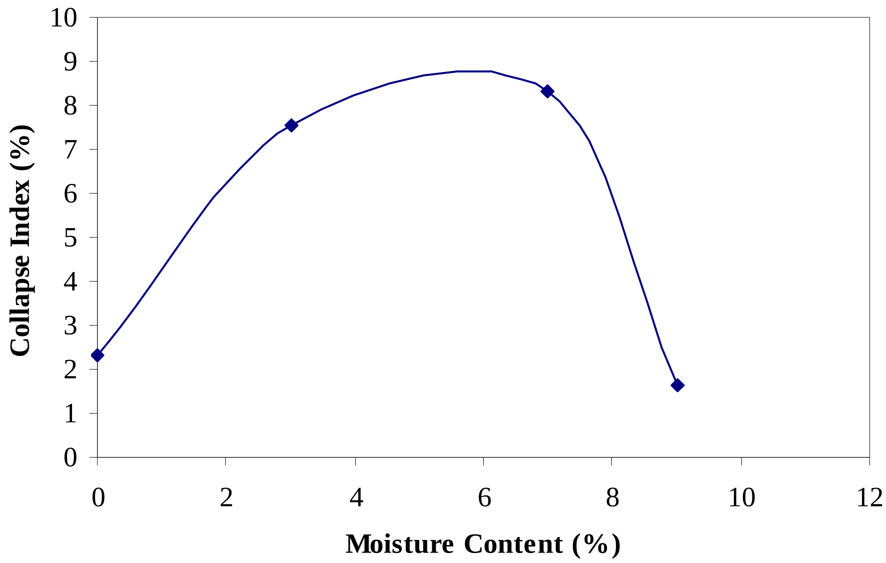


Figure 77. Cedar Rapids ¾ minus collapse index profile, SC

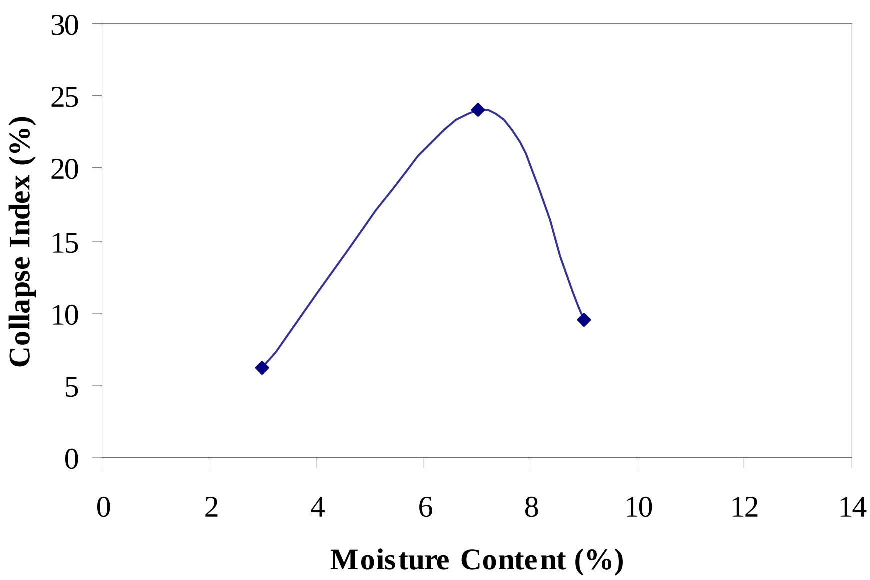


Figure 78. Des Moines manufactured sand collapse index profile, GC

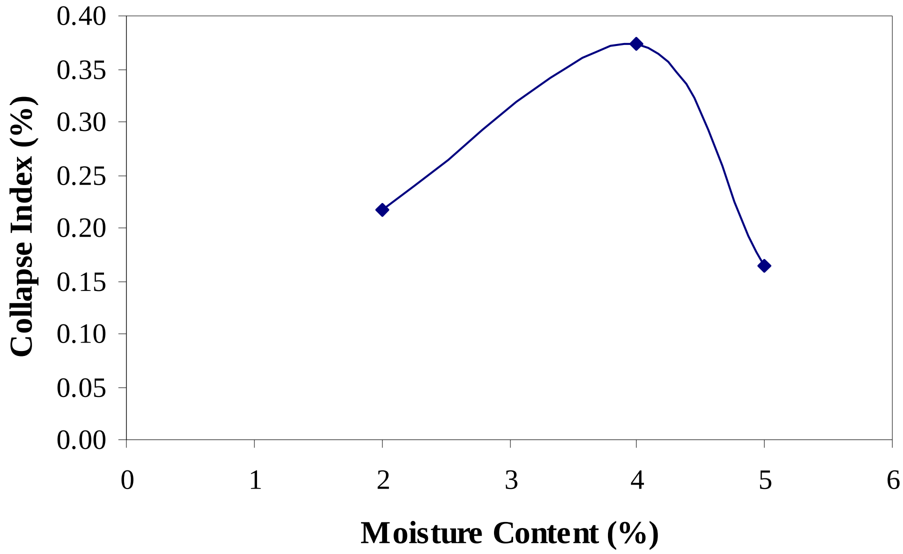


Figure 79. SUDAS collapse index profile, SW-SM

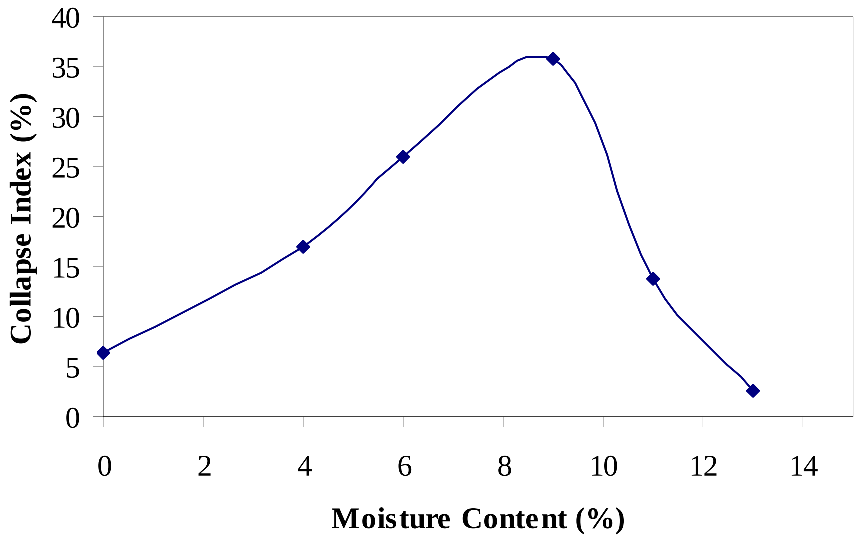


Figure 80. Limestone screenings collapse test, SW-SM

Table 25. Engineering properties of imported material

Material	Classification AASHTO (USCS)	% Sand	% Fines	Cu	Cz	% Collapse
Ames 3/8"	A-1-a (SM)	60.0	14.4	75.0	1.3	9.0
Cedar Rapids 3/4"	A-2-4 (SC)	34.0	26.8	225.0	1.0	8.5
Davenport	A-1-a (GC)	20.1	12.4	275.0	36.4	-
Des Moines	A-1-b (SW-SM)	88.1	10.0	16.7	1.9	24.0

Following the collapse index test, the bulking moisture content of several materials was compared with percent saturation for all backfill materials used. The degree of saturation is defined as the percentage of water a material has with respect to the maximum amount of moisture that a material can obtain for saturation (Spangler and Handy 1982). The degree of saturation was calculated to determine what amount of saturation needed to exceed the bulking moisture content region. Figures 81, 82, and 83 indicate that the bulking moisture range may be exceeded if the material is at about 40% saturation.

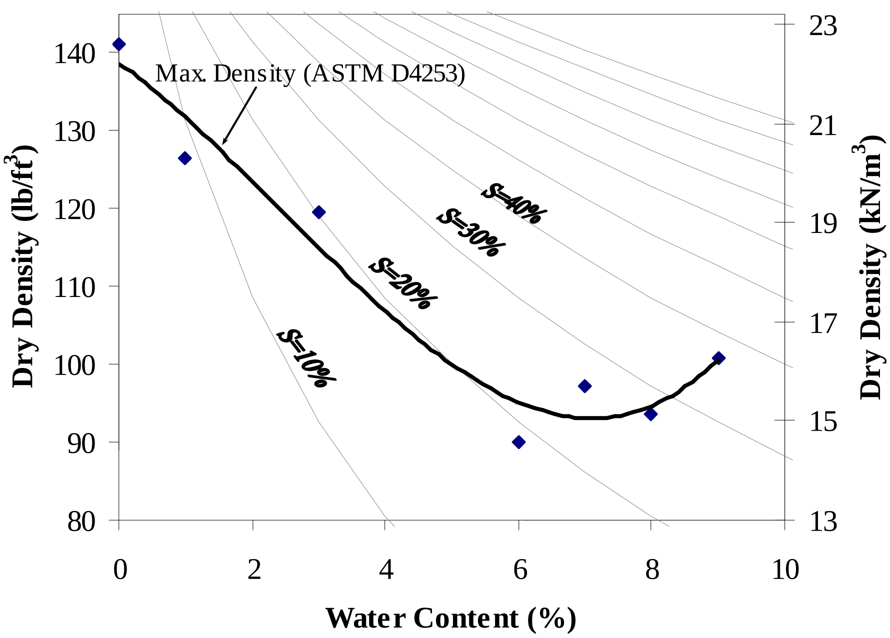


Figure 81. Degree of saturation, Ames, IA

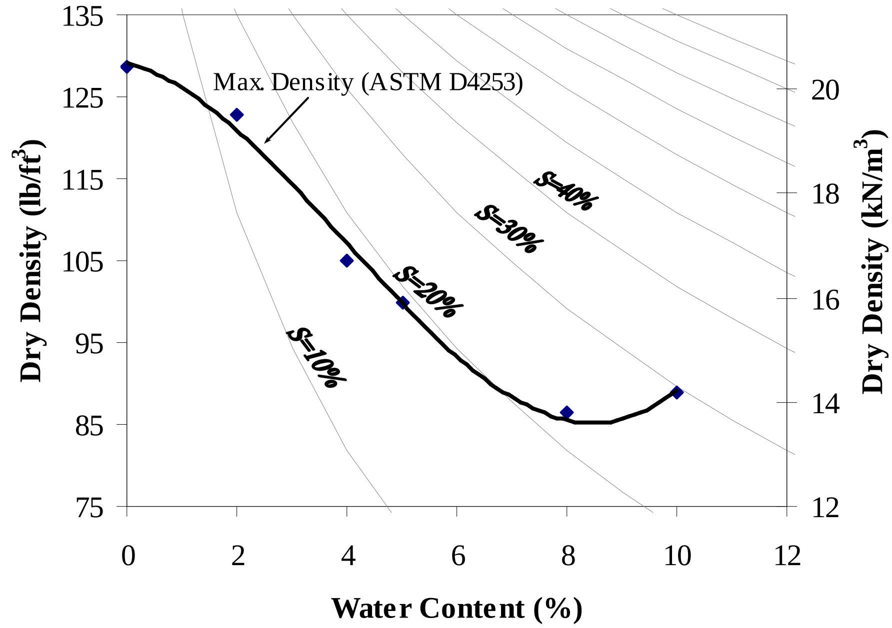


Figure 82. Degree of saturation, Cedar Rapids, IA

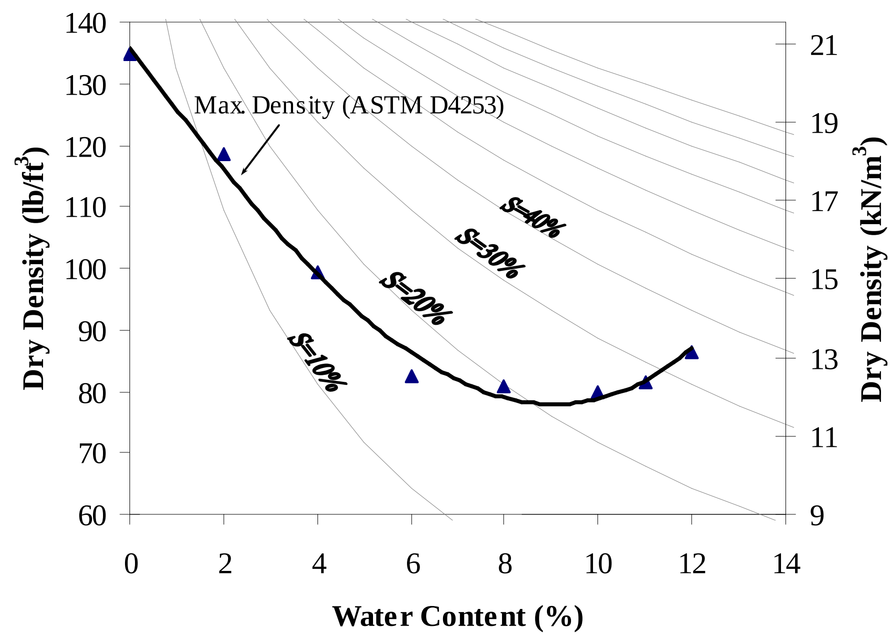


Figure 83. Degree of saturation, Des Moines, IA

Standard Proctor

From the survey results, standard Proctor is generally used as the method of defining compaction requirements in the field. The standard Proctor is conducted in the laboratory to indicate a maximum density and optimum moisture content. Figure 84 shows a typical standard Proctor curve for cohesive soils. To illustrate the difficulty in determining the relationship of density and moisture in a granular material, mentioned in the Literature review, standard Proctor tests were conducted. Figures 85, 86, and 87 are plotted comparing test results from a standard Proctor and minimum and maximum density test. The standard Proctor test is conducted using an impact, whereas the minimum and maximum density test uses a vibrating table, similar to the type of compaction produced in the field. The maximum density tests show a more distinct curve, in comparison to standard Proctor results.

Figures 85, 86, and 87 were also plotted for comparison of field data to the standard Proctor results obtained in the laboratory. Dry density values of Ames 3/8 minus material obtained in the field indicates values lower than the standard Proctor energy at 6%. Cedar Rapids 3/4 minus material indicates a majority of the values higher than density values achieved with the standard Proctor between 5% and 7%, with one value below the standard Proctor energy at 5.5% moisture. Des Moines has a majority of readings below standard Proctor energy at moisture contents between 5% and 11%.

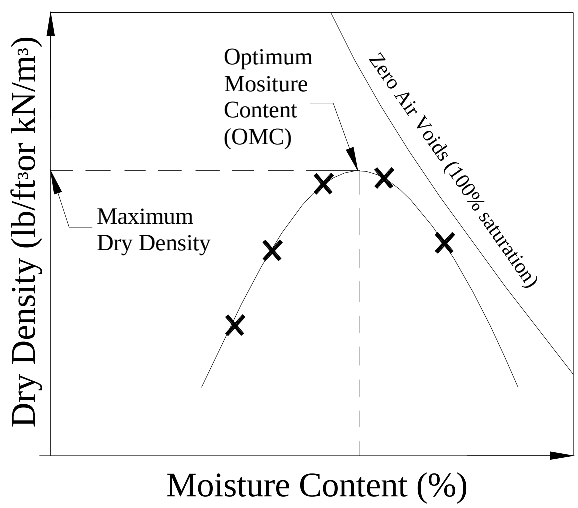


Figure 84. Typical standard Proctor curve

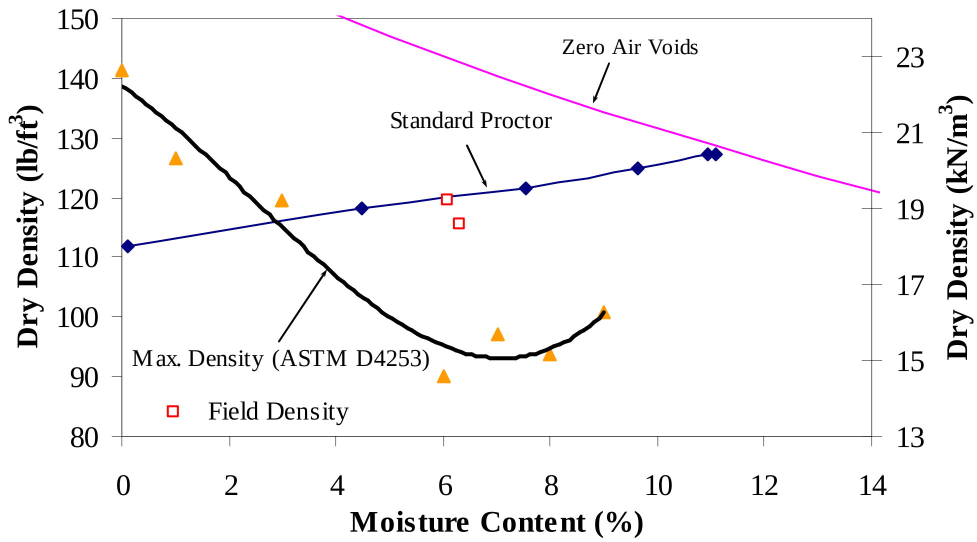


Figure 85. Ames: standard Proctor vs. maximum density

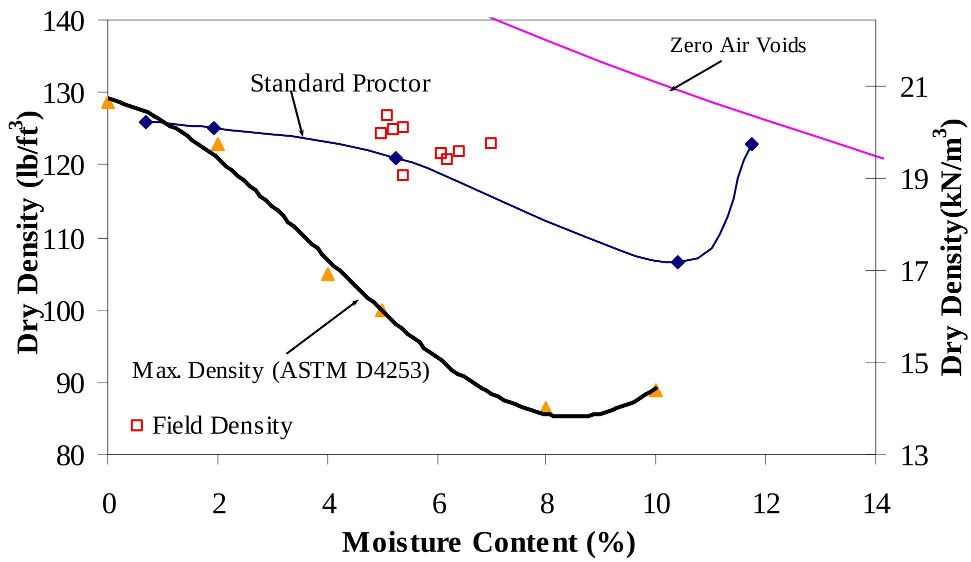


Figure 86. Cedar Rapids: standard Proctor vs. maximum density

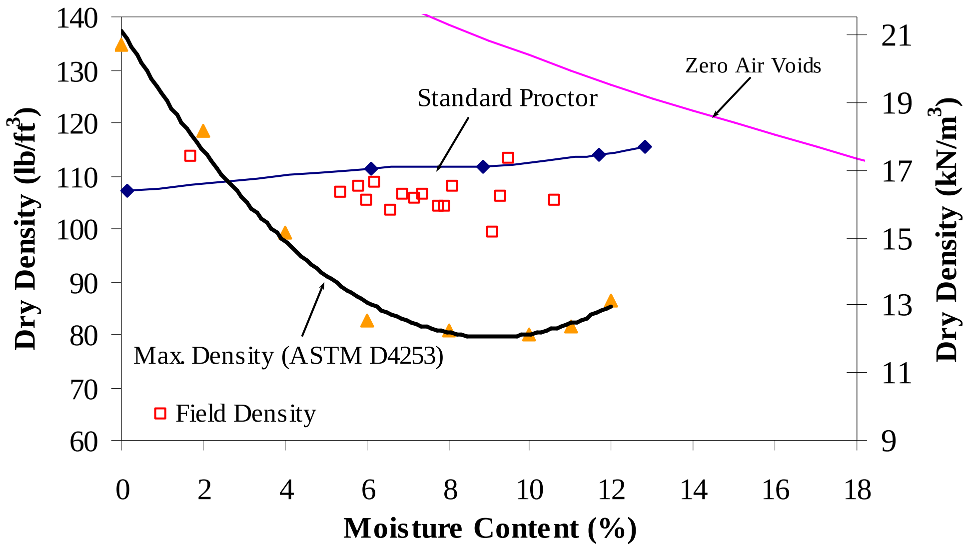


Figure 87. Des Moines: standard Proctor vs. maximum density

Design Charts

Since the compaction of granular materials provides a more distinct moisture-density curve when based on relative density (minimum and maximum density), design charts were generated for use in determining the relative density of materials in Ames, Cedar Rapids, Davenport, and Des Moines. These charts were devised based on the minimum and maximum density tests completed in the laboratory and field dry density values obtained from the nuclear density gauge. The relative density values are based on the minimum and maximum dry density values of a material obtained in an oven dry state (i.e., zero percent moisture content) according to ASTM D 4253 and D 4254. Relative density (R.D.) is defined as follows:

$$R.D. = \frac{\gamma_{\max}(\gamma_{field} - \gamma_{\min})}{\gamma_{field}(\gamma_{\max} - \gamma_{\min})} * 100$$

where:
γ_{field}= Dry density in the field (pcf or kN/m^3)
$\gamma_{\max}$= Maximum dry density in the laboratory (pcf or kN/m^3)
$\gamma_{\min}$= Minimum dry density in the laboratory (pcf or kN/m^3)

Figures 88, 89, 90, and 91, show the plot with relative density on the secondary y-axis. The percentages indicated on this axis are based on relative density classifications of very loose, loose, medium dense, dense, and very dense (see Table 7 in the Literature review). Relative density is depicted on these charts based on its nonlinear relationship with dry density (see Figure 92). A material compacted at 65% relative density is considered a dense material according to Table 7 in the literature review, so achieving this density in a trench would result in a densely compacted material.

As shown in Figures 88, 89, 90, and 91, field density exceeds the maximum density achieved in the laboratory with an increase in moisture content. This is the result of the material in the field compacted at a greater compaction energy, compared to the energy in the laboratory. The relative density results for Ames 3/8 minus indicate a medium dense material, Cedar Rapids, a dense to very dense, Davenport, a dense to very dense, and Des Moines, a very loose to medium dense material.

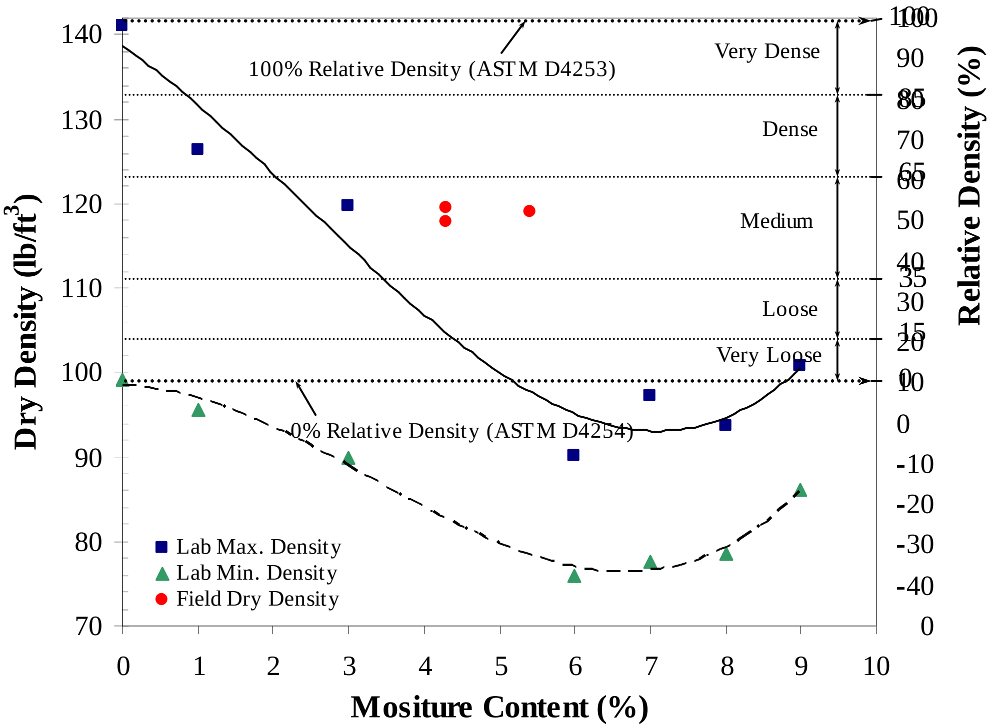


Figure 88. Ames 3/8 minus relative density plot

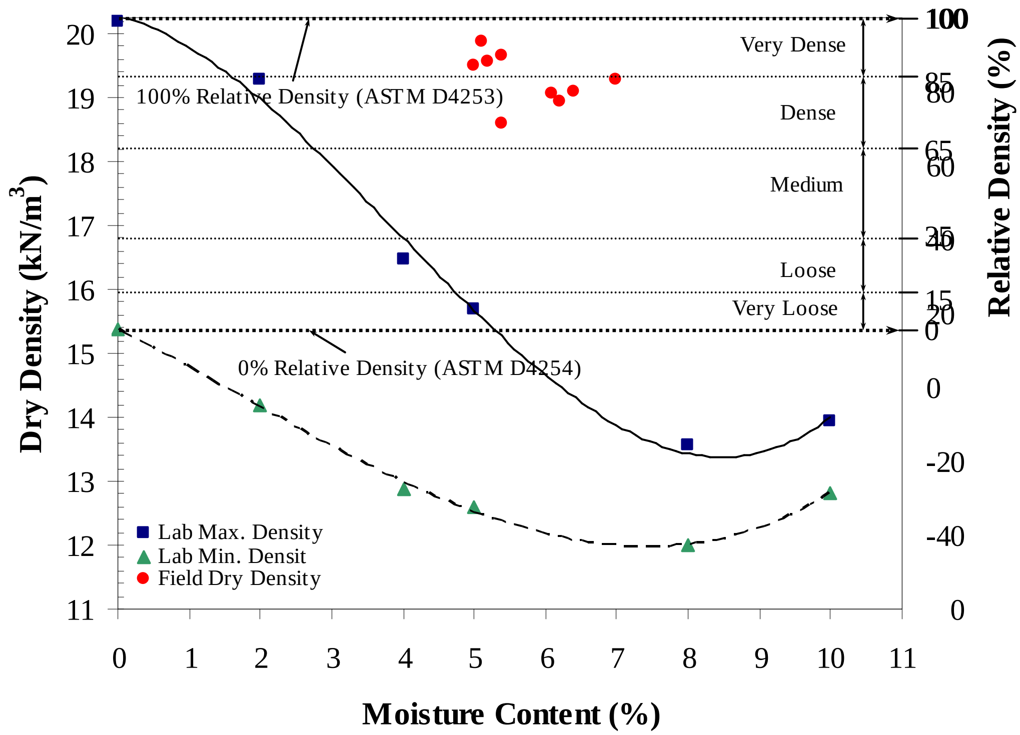


Figure 89. Cedar Rapids relative density plot

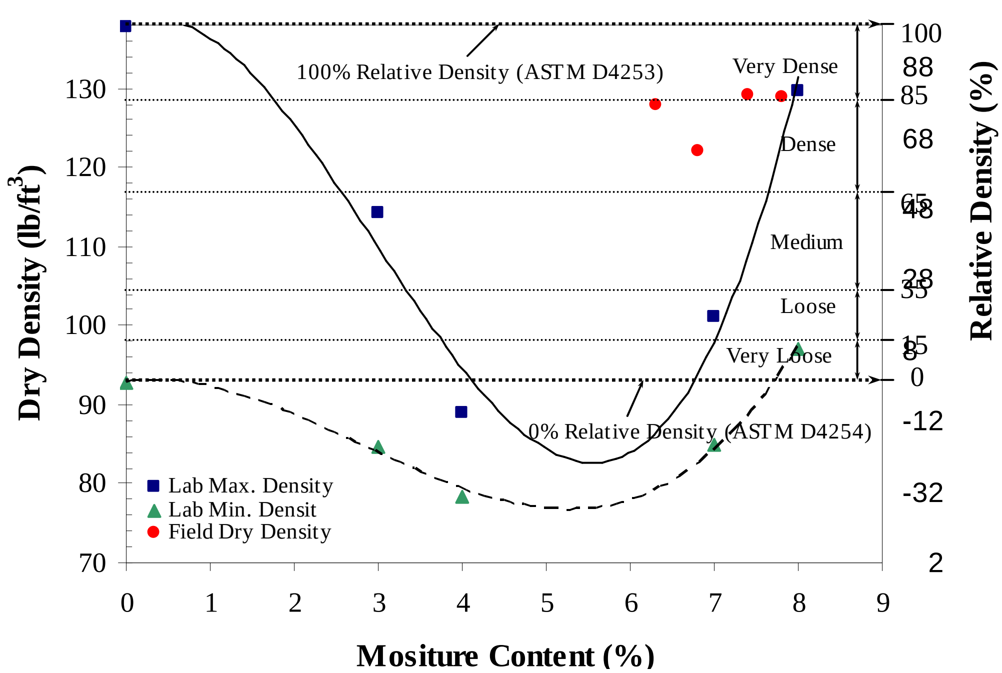


Figure 90. Davenport relative density plot

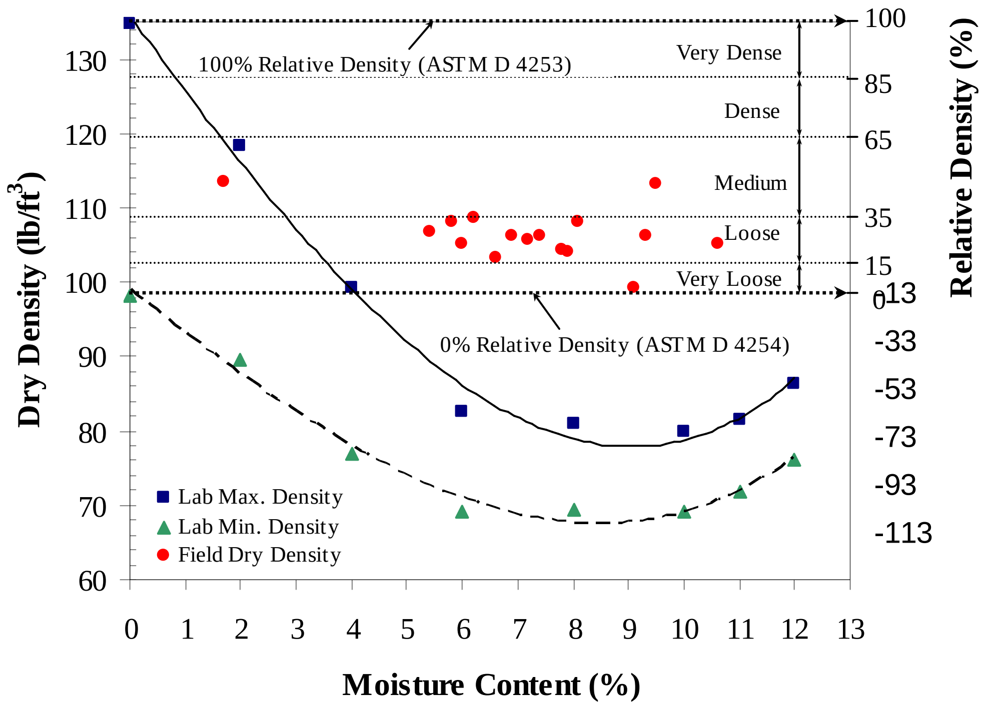


Figure 91. Des Moines relative density plot

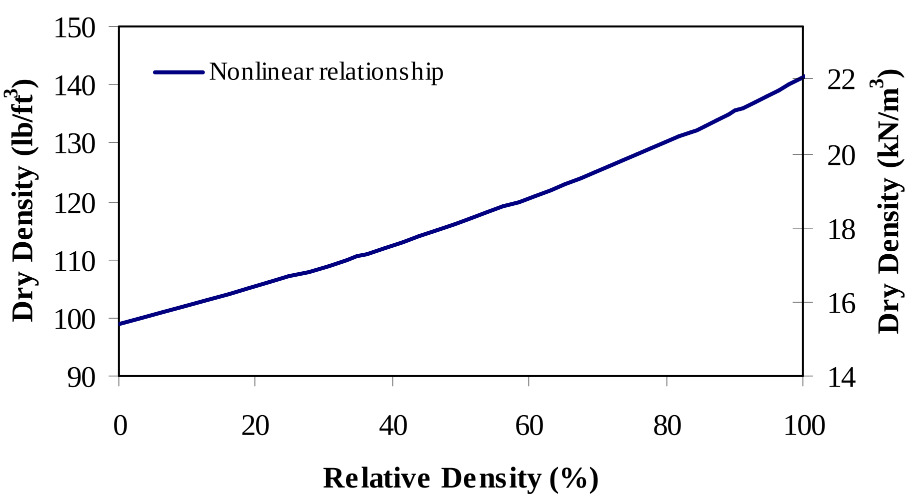


Figure 92. Relative density–dry density nonlinear relationship

Summary of Findings from Laboratory Testing

- Backfill materials used in Cedar Rapids and Davenport, which are classified as SM and GC, respectively, with% of sand not exceeding 35%, achieved relative densities of dense to very dense without a significant amount of compaction.
- Based on the relative density data, the backfill material used in Des Moines, which is classified as SP-SM with 88% sand, was difficult to achieve the required relative density. The material placed in the field was characterized as loose with relative density less than 35%.
- All backfill material used in the visited cities except Des Moines has fines content (Percent passing sieve No. 200) greater than the maximum limit allowed by Iowa DOT of 10% for backfill material gradation.
- Materials obtained from all cities are classified as excellent to good in use as a subgrade material according to AASHTO.
- Materials obtained from the field in Ames, Cedar Rapids, Davenport, and Des Moines were placed at or near the bulking moisture content which increase the settlement (collapse) potential.
- Collapse tests indicate a large collapse potential of 36% for loosely placed limestone screenings, 9% for 3/8-inch, 8.5% for 3/4-inch, and 24% for manufactured sand. The material specified in SUDAS had a low collapse potential of 0.35%. It was also observed that the collapse potential increases as the percentage of sand particles increase.
- The use of granular backfill materials may require watering the material in the trench to reduce settlement potential induced by moisture change.
- Saturating a material to 40% exceeds the bulking moisture content for all materials used in the visited cities.
- Relative density tests, rather than standard proctor tests should be used in the specifications for compaction requirements of granular materials used by all visited cities. Relative density of 65% is suggested as a minimum requirement of compaction for granular materials.
- Using the relative density design charts as a guide, correct compaction requirements for a given material can be determined.

SUMMARY AND CONCLUSIONS

Utility cuts are made in completed pavement sections to install electric, water, and wastewater utilities, as well as drainage pipes under roadways. If the repair construction is not done properly, the repaired pavement will settle relative to the original pavement. Several cities in the United States and abroad spend millions of dollars each year on maintenance and repairs of utility cuts made in pavements (APWA 1997). This research study was undertaken to improve utility cut construction practices in Iowa to increase the pavement life and reduce maintenance. This section summarizes the findings and conclusions of this research.

Relevant Literature

- Utility cuts, made in completed pavement sections to install several utilities under roadways, not only disturb the original pavement, but also the base course and subgrade soils below the cut. Utility cuts in a roadway affect the performance of the existing pavement as settlement and/or heave occurs in the backfill materials of the restoration. The Canada National Research Council indicates that excavations in pavements by utility companies reduce the pavement life by up to 50%.
- When a utility cut is made the native material surrounding the perimeter of the trench is subject to loss of lateral support. This leads to loss of material under the pavement and bulging of the soil on the trench sidewalls into the excavation. Subsequent refilling of the excavation does not necessarily restore the original strength of the soils in this weakened zone.
- Backfill materials and compaction requirements should include gradation, moisture control, lift thicknesses, and compaction equipment. The majority of Departments of Transportation in the United States use a granular backfill material with an AASHTO classification of A-1 and A-3. Most state DOTs use the standard Proctor test to specify optimum moisture content and required density and specify lift thicknesses less than or equal to twelve inches. Granular backfill requirements should be based on relative density with moisture control and no on standard Proctor.
- Correction for the zone of influence can be obtained with a pavement cutback of 2 to 3 feet removed and filled with compacted native soil or backfill materials. T-sections, and other similar engineered cross sections, have been used successfully to mitigate the zone of influence effects.
- QC/CA includes using the nuclear gauge, DCP, and Clegg Hammer. State DOTs specify 90% to 95% of standard Proctor, DCP penetration of not more than 3-¼ inch (129 mm) at 11 drops, or a minimum Clegg hammer value of 18 for proper compaction for materials under the pavement surface. All these values were used for general compaction requirements and not necessarily in utility cut regions.

Survey Results

- Surveys were sent to several cities across Iowa, and received back from Ames, Cedar Rapids, Davenport, Des Moines, Dubuque, Waterloo, and Burlington. Surveys include questions about permit fees, extent of the problem, construction requirements, and C/QA.
- All surveyed cities indicated that the current method of utility cut construction resulted in

satisfactory results and they all indicated that there was virtually no problem. However, the responses also indicated that utility cut pataches often last less than two years, a relatively short period. This discrepancy may be a result of minimal documentation kept on utility maintenance and repairs, as well as a personal opinion of the definition of a poorly performing utility cut.

- Using the statistical data provided by the city of Ames, January and December are the prominent months for water main breaks. This trend may be a result of frost loading which could substantially increase vertical loads (i.e., up to twice the original load) on buried pipes.
- Many cities throughout Iowa require permits before an excavation be made, however a fee is not assessed in all cases. A permit is a mechanism to track who conduted the work and when and fees generally attempt to recoup administrative costs.
- Construction requirements and materials used in the construction of a utility cut repair varied from one city to another. The material selection is based on regional availability, with each city using a different gradation and material.
- Although all surveyed cities use granular backfill materials, all used 90% to 95% standard Proctor requirements in their specifications which should be replaced by relative density requirements. Furthermore, quality control at the construction site is minimal, if at all. Dubuque and Waterloo do use the nuclear density gauge for regulating compaction requirements. In some cases, however an inspection program consists of only visual inspection.

Construction Techniques

- A typical utility cut repair consists of a pavement cut, excavation of soil materials, repair of the utility and backfilling of the trench, usually with imported materials. Lift thicknesses generally ranged from 2 feet to 4 feet, with compaction sporadically throughout the fill using a vibrating plate on the end of a backhoe. Pavement surfacing was placed anywhere from immediately after the utility cut was constructed to up to two weeks later. Des Moines was the only city observed that plated the unpaved utility cut until surfacing was available.
- Backfill materials were compacted using large compaction equipment, which was observed getting very close to the edge of the cut. This resulted in damage to pavement surfaces along the perimeter of the excavation.
- The common practice of placing 2-foot to 4-foot thick lifts lead to difficulty in obtaining adequate compaction. Essentially the material in the upper portion of the lift is compacted, however the vibration used to orient the soil particles into a more dense structure, tends to decrease with depth as shown from DCP profiles.
- Undesirable practices were observed as construction practices were observed. For example, it was often observed that saturated native materials were added to the excavation in an attempt to clean the utility cut area.
- Ultimately, sites where construction was observed from the excavation to the backfilled trench, no quality control devices were used to ensure compaction requirements were met. Furthermore, there was no moisture control of the imported backfill material placed into the trench. The method of obtaining the required compaction was based on experience, rather than a quality control program or device.

Field Results

- The backfill materials used in several utility cut sites were characterized using the Nuclear Density Gauge, Dynamic Cone Penetrometer, Clegg Hammer, GeoGauge, and the Falling Weight Deflectometer.
- The moisture content and dry density values measured in the field using the Nuclear Density Gauge were compared with the results of laboratory tests (relative density tests). Calculated relative density values indicate a dense to very dense compacted material in investigated utility cuts in both Davenport and Cedar Rapids. The backfill material used in Ames was placed at a medium density state; however, the backfill material used in Des Moines was placed in a loose to very loose state.
- The CBR values calculated using DCP test results were fairly consistent throughout the excavated area. CBR values were higher near the center of excavated areas when compared to CBR values near the edge of the trench. These profiles indicate that smaller compaction equipment may be needed to achieve uniform compaction throughout the trench.
- DCP data obtained from native material indicate a trend of decreasing the number of blows required for 3.9 inches (10 cm) penetration as a result of loss in lateral support during the excavation.
- When plotting the number of blows required to penetrate 3.9 inches (10 cm) into the ground, the DCP profile showed a trend of high CBR values at approximately 1.5 feet. Then the CBR values reduce with depth afterward as the effect of compaction decrease with depth for large lift thicknesses. This reiterates the importance of lift thicknesses being less than or equal to 12 inches.
- According to the available literature, a minimum Clegg Hammer Impact Value of 18 is needed for proper compaction beneath a pavement surface, however when comparing all data obtained in the field, this value was not reached at any site.
- The FWD results show larger deflection in the zone of influence which indicates the softening of this zone as a result of the cut. FWD results also show a trend of higher stiffness near the center of tested trenches as was also observed using DCP results.
- When subjected to FWD loading, concrete pavement at the edge of the utility cut produced a smaller deflection compared to the asphalt and composite pavement materials. The may be a result of the dowel bars located in the concrete aiding in the distribution of loads.

Laboratory Results

- The laboratory results were obtained from test methods, including sieve analysis, relative density, standard Proctor, and collapse tests. These results were used with the field data to further characterize the material properties.
- All backfill material used in the visited cities except Des Moines has fines content (Percent passing sieve No. 200) greater than the maximum limit allowed by Iowa DOT (i.e., 10%) for backfill material gradation. Furthermore, most of these materials were placed at or near the bulking moisture content which increase the settlement (collapse) potential.

- Collapse tests indicate a collapse potential of 9% for 3/8-inch used in Ames, 8.5% for 3/4-inch used in Cedar Rapids, and 24% for manufactured sand when loosely placed. Limestone screenings which were tested to characterize the potential use in utility cut applications had a 36% collapse when loosely placed. The material specified in SUDAS had a low collapse potential of 0.35%. It was noticed that the collapse potential increases as the percentage of sand particles increases.
- The use of granular backfill materials may require watering the material in the trench to reduce settlement potential induced by moisture change. Saturating a material to 40% exceeds the bulking moisture content for all materials used in the visited cities and could be used in the field during construction.
- Standard Proctor and Relative Density tests were conducted on each imported material. Results for the Standard Proctor for the materials used in Ames and Des Moines did not show the well-known bell-shape Proctor curve nor showed a bulking moisture content effect at low moisture contents, as was noticed in the relative density test.
- Compaction of granular materials should be specified according to relative density not according to Proctor tests. Relative density of 65% is suggested as a minimum requirement of compaction. Furthermore, the relative density test uses vibration to compact the material, which is similar to that used in the field.
- The generated design charts indicate a specified target region of compaction for a material to obtain the required density. These charts can be used in the field as a quality control check with the nuclear density gauge. Using the dry density value measured with the nuclear density gauge the state of compaction and relative density can be determined.
- Backfill materials used in Cedar Rapids and Davenport, which are classified as SM and GC, respectively, with percent of sand not exceeding 35%, achieved relative densities of dense to very dense without a significant amount of compaction.
- Based on the relative density data, the backfill material used in Des Moines, which is classified as SP-SM with 88% sand, was difficult to achieve the required relative density. The material placed in the field was characterized as loose with relative density less than 35%.

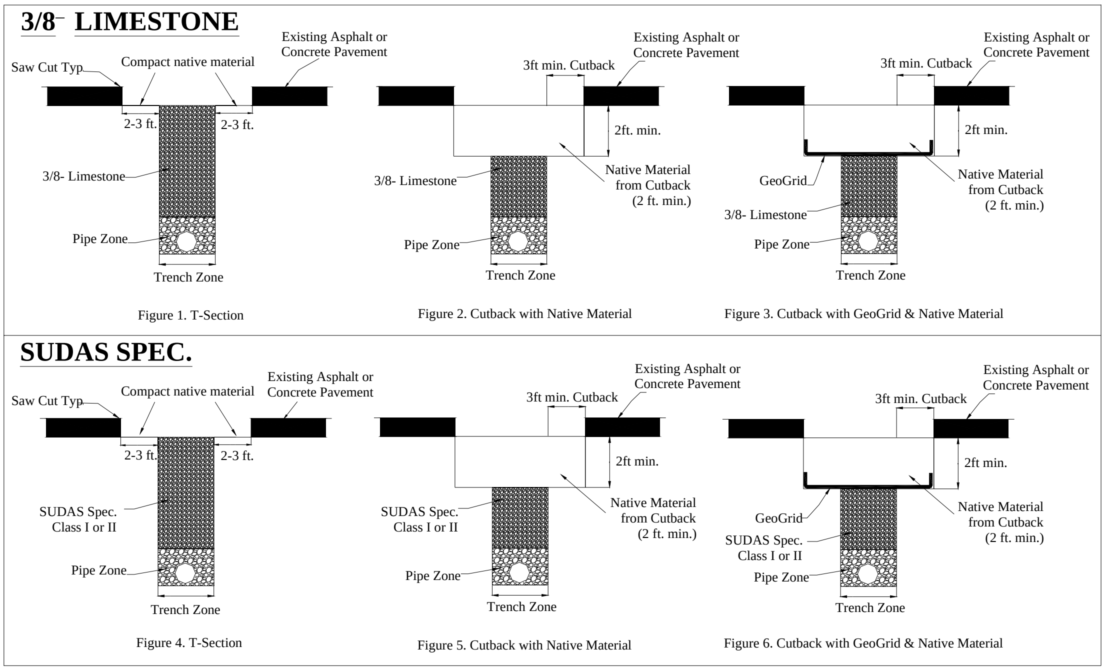
Trial Trenches

After observing the construction techniques and field and laboratory investigation, six trenches were designed and proposed to the city of Ames for construction with the goal of alleviating future settlement. Settlement expected to result from collapse and low compaction effort used in the field were avoided by using SUDAS Class I gradation backfill with 100% passing 1-1/2-inch sieve and with a maximum passing sieve No. 4 of 10%. The research team also tried to avoid settlement using a structural geogrid to bridge over the excavated area using 3/8-inch backfill material used in Ames with no moisture or compaction control. Three similar trenches were proposed using the two different backfill materials:

1. A T-section using up to 3 feet wide excavation around the perimeter of the cut and applying compaction to the surrounding native material in the cutback region.
2. A 2- to 3-foot cutback and pavement removal, along with an excavation of 2 deep into the native material. This material will be replaced with imported backfill material.

3. A trench constructed the same as (2) with a structural geogrid placed on the bottom of the excavated.

The cutback excavation incorporated into the last two trenches was placed in the cutback region 2 to 3 feet beneath the excavation for bridging purposes. A 2- to 3-foot (0.6 to 0.9 m) cutback depth was excavated to compensate for the majority of settlement that was found to occur in backfill at 2 feet (0.6 m) beneath the pavement surface according to the literature review. Cross-sections of these proposed trenches are illustrated in Figure 93.



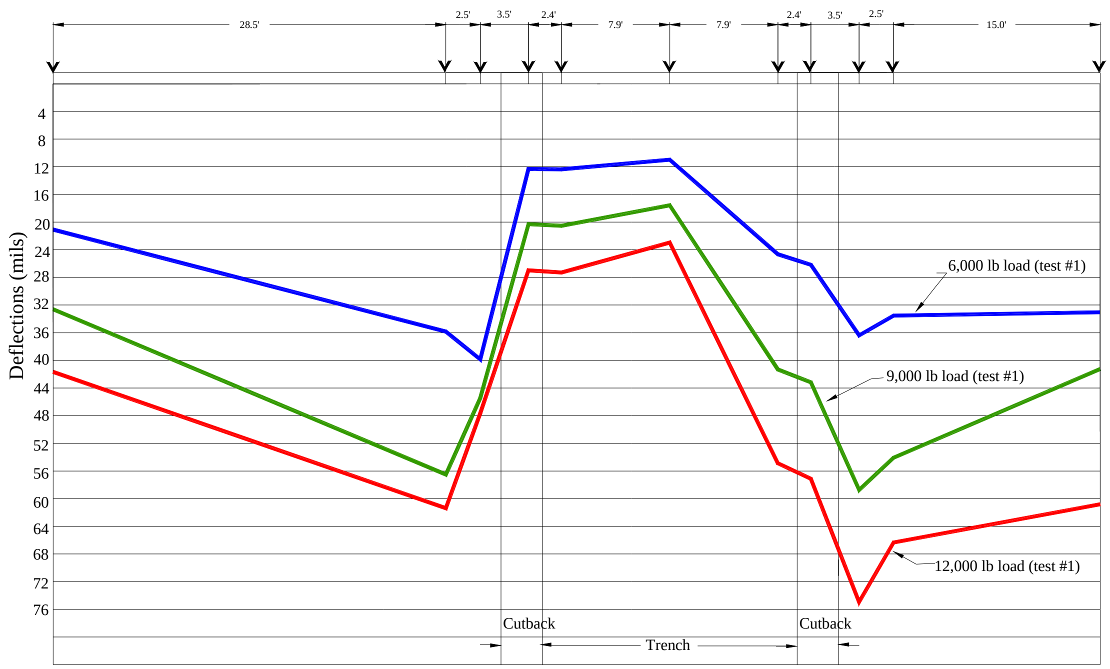
Note: 1: Trenches should be constructed to appropriate widths and depths according to site specifications. These cross-sections specify only specific cutback distances, material use, and depth of compacted native material.
2: If native material is not sufficient to fill trench, imported material (3/8- limestone or SUDAS Spec.) should be used to fill trench to the surface.

Figure 93. Proposed trenches in Ames, IA

On June 16, 2005, a proposed utility cut restoration constructed in Ames was monitored with the FWD. The trench was 24.7 feet (7.3 m) long and 13.6 feet (4.1 m) wide. This utility cut consisted of a 3-foot (0.9 m) pavement cutback and a 2-foot (0.6 m) vertical cut into the native material beneath this region, along with a geogrid placed to bridge the excavated cutback region and utility cut excavation. The geogrid used in the trench was a Tensar BX1100, formerly known as Tensar SS1. It is a polypropylene biaxial geogrid that has been approved by the Iowa DOT for subgrade stabilization. Its index properties in the machine (longitudinal) direction include aperture dimensions of 1 in (25 mm), a tensile strength at 2% strain of 280 lb/ft (4.1 kN/m), an ultimate tensile strength of 850 lb/ft (12.4 kN/m) and ultimate junction strength of 791 lb/ft (11.5 kN/m). The index properties in the cross-machine (transverse) direction include aperture dimensions of 1.3 in (33 mm), tensile strength at 2% strain of 450 lb/ft (6.6 kN/m), ultimate tensile strength of 1300 lb/ft (19 kN/m), and ultimate junction strength of 1209 lb/ft (17.7 kN/m). The purpose of incorporating the geogrid is to act as reinforcement for the backfill material, strengthening its properties. Figure 94 shows a picture of the geogrid placed inside the excavated area.

Figure 94. Geogrid being placed

The FWD profiles can be seen in Figure 95, where as in the previous FWD profiles shown, the center of the trench had a considerably low deflection compared to the surrounding regions of the utility cut restoration. The geogrid may have assisted in the lower deflections in this area. In the trenching limits, the right side of the excavation had greater deflections and an apparent zone of influence, when compared to the left side. This may be a result of the dump truck near the left side edge of the trench, over stressing the pavement as backfill material was dumped into the trench. This figure also indicates a shift in the zone of influence to regions outside the pavement cutback, since material was excavated to a depth of 2 feet (0.6 m) in the cutback region. This may be a result of the surrounding native material experiencing a loss in lateral support around the 2-foot (0.6 m) excavation. Ultimately, the construction sequence may have resulted in an influence zone around the cutback area however, when compared to previous FWD tests, the FWD did not show a clear zone of influence response at greater loadings. This may be an indication of a reduction of disturbance in the zone of influence using this construction technique. Figure 95 illustrates this indistinctive zone of influence. Since the zone of influence is not distinct with this 2-foot (0.6 m) vertical excavation in the cutback region, the research team is proposing the continuation of monitoring this trench and also a new utility cut restoration. This new restoration would consist of a 1-foot (0.3 m) vertical excavation of the native material in the cutback region, rather than a 2-foot (0.6 m) cutback and again the use of the geogrid.



Note: 1) 1 mil=0.001 inches
2) Deflections reflect point where FWD drops weight
3) Weights were dropped on centerline of trench in the longitudinal direction of the pavement

Figure 95. FWD profile for proposed utility cut with geogrid

SUGGESTED PRACTICES & RECOMMENDATIONS

With the conclusion of this research, practices and recommendations are proposed for future utility cut restorations. These recommendations are intended to improve the quality of the construction process, however further monitoring is recommended to determine the performance of the documented construction sites since settlement has been noted to occur in utility cuts within two years. Based on the field observations and measurements and laboratory testing; the following recommendations can be made.

- Proper compaction is generally determined according to Standard Proctor compaction in most cities. However, relative density should be used for the determination of granular compacted material. When determining compaction based on relative density, a value of 65% or greater should be obtained to achieve a densely compacted material.
- Moisture is one of the most important parameters in the evaluation of a material in Geotechnical Engineering. It has been shown throughout this research that moisture is an important factor in utility cut restorations. It has also been shown that much of the granular backfill material placed is at or near the bulking moisture content. It is recommended granular backfill for utility cut restorations be constructed at moisture contents exceeding the bulking moisture content region for the particular backfill used. The material as placed will then overcome the collapse potential that could be induced on the pavement patch as a result of infiltration or a rise in the groundwater table. Based on the results of the tests reported herein, granular backfill materials placed in this manner will achieve the recommended 65% relative density.
- It was observed in the field studies that instrumentation and quality control were rarely used to ensure standards and proper construction procedures were being met. Due to regulatory concerns, the use of the nuclear density gage for density control into the future is considered unlikely. The DCP provides an alternative density control method; however, correlations between the DCP and dry density would need to be established for specific backfill materials.
- The zone of influence has been shown to be a critical factor in the construction of these utility trenches. To compensate for the zone of influence effects on utility cut restorations, it is recommended that a pavement cutback of 2 to 3 feet laterally beyond the limit of the trench excavation be constructed. The pavement cutback and excavated area should be recompacted before the pavement surfacing is placed. To compensate for the zone of influence and to provide bridging over the trench backfill materials it is recommended that T-sections be used in repairing utility cuts. Although monitoring is continuing on the T-sections installed in Ames, at this time it is recommended that T-sections consist of a cutback laterially 3 feet from the edge of the trench excavation and that a particular attention be paid to the upper 3 feet of the recompacted material. This upper 3 foot zone can be constructed of either granular fill material or native cohesive materials, provided that proper moisture and density is achieved in the materials. Cohesive matierals placed in the upper 3 feet should be placed at a minimum of 95% of standard Proctor density and within two percentage points of optimum water content.
- The zone of influence has proven to be a critical factor in the construction of these utility trenches because of the loss of lateral support in the trenching limits. To compensate for the weakened material in this zone, it is recommended that a pavement cutback of two to

three feet be constructed. The pavement cutback and excavated area should be compacted again with a vibrating plate before the pavement surfacing is placed.

- SUDAS specifications and design requirements should be updated, in consultation with the districts, to reflect the recommendations herein.
- A seminar or informational session should be conducted with construction crew members to show the effects of poor construction and the factors that affect the performance result. This seminar would be useful in emphasizing the importance of good construction, since proper construction could minimize many of the existing problems. Along with good construction practices a good quality control and assurance program should be enforced. In many cases, just having someone on site promotes careful utility cut construction.

Future Research

A continuation of this research should be conducted to monitor the performance of the constructed trenches. According to survey results and previous studies, a restored trench will begin to show signs of settlement as early as two years, therefore to accurately determine the performance of the trenches, monitoring should continue for a minimum of two years.

It would be desirable to monitor the change in moisture content, the frost depth and the stresses around the pipe in the utility cut region as well as under the pavement in the cut region and the surrounding undisturbed pavement. This will help in understanding the mechanisms of pavement settlement, the difference in the response between backfill materials and native subgrade when subjected to freeze-thaw, and the changes of stresses on the pipe as a result of freezing.

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APPENDIX A: CITY SURVEY

Highway Division
Research Project TR-503

"Utility Cut Repair Techniques – Investigation of Improved Utility Cut Repair Techniques to Reduce Settlement in Repaired Areas"

Questionnaire Completed by: ____________________
Organization: ____________________
Address: ____________________
____
E-mail address: ____________________

Responses can either by E-mailed or faxed to Vern Schaefer (E-mail address: vern@iastate.edu; Fax number: 515-294-8216) or Dale Harrington (E-mail address: pcconc@iastate.edu; Fax number: 515-294-0467) or turned in as part of the discussion. Regarding the questions below, if you have a repair procedure, pictures or additional data that you are willing to share please mail them to:

Prof. Vern Schaefer	OR	Mr. Dale Harrington, P.E.
482B Town Engr. Bldg.		CTRE
CCEE Department		2901 South Loop Drive, Suite 3100
Iowa State University		Ames, IA 50010
Ames, IA 50011		

A. Does your agency have a standard method of repair of utility cuts?: Y ___ N ___

1. If yes to question A, does your method provide satisfactory results?: Y___ N___

B. If you answered yes to question A, please describe the standard method of repair. Please be as detailed as possible or attach written standards, if available. If you answered no, proceed to question C.

____
____

Specifically, with your standard method of repair, please answer the following questions:

1. What types of backfill materials do you allow? i.e., native materials, imported materials, special materials? ____________________

2. What type of compaction do you require of the backfill materials?
____

3. Are repairs surfaced with a temporary pavement? Y ___ N ___

a. If yes to question 3, please identify the temporary pavement material and how long the temporary patch is left in place.

____

____

b. If no to question 3, please indicate the type of permanent repair.

____

____

4. Do you have any quality control or quality assurance (QC/QA) requirements for utility cut repairs? Y ___ N ___

a. If yes to question 4, please identify (or attach) the QC/QA requirements.

____

C. Does your agency use in-house crews to repair utility cuts?: Y ___ N ___

D. If known, what do the breaks and repairs cost your agency annually?

____

____

E. What is the predominate timing of breaks that require repair? i.e., winter, spring, summer, fall? ____

F. How many breaks do you have annually?

____

G. Have you changed repair practices recently? Y ___ N ___

1. If yes to question G, please identify the old practice and why you changed.

____

____

I. What percentage of repairs have experienced pavement performance problems? __________

J. How long do the typical repairs last before they have performance problems? __________

K. What, in your opinion, is causing the problems?

APPENDIX B: FALLING WEIGHT DEFLECTOMETER RAW DATA

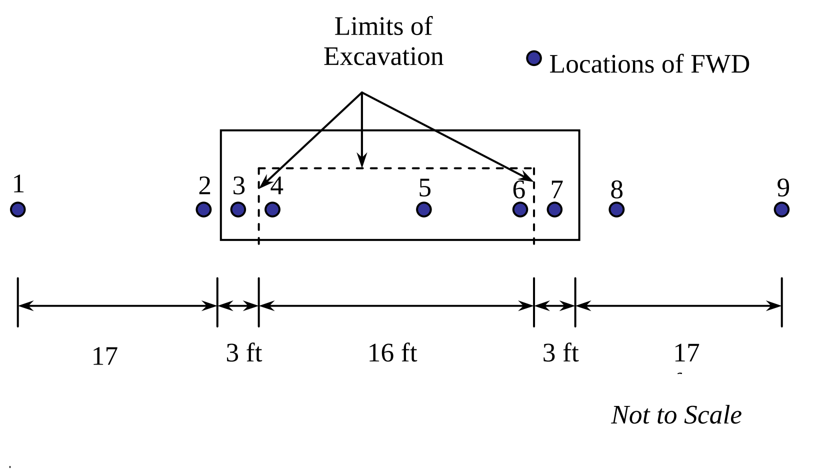

Figure B1. Ames 20th St. FWD layout

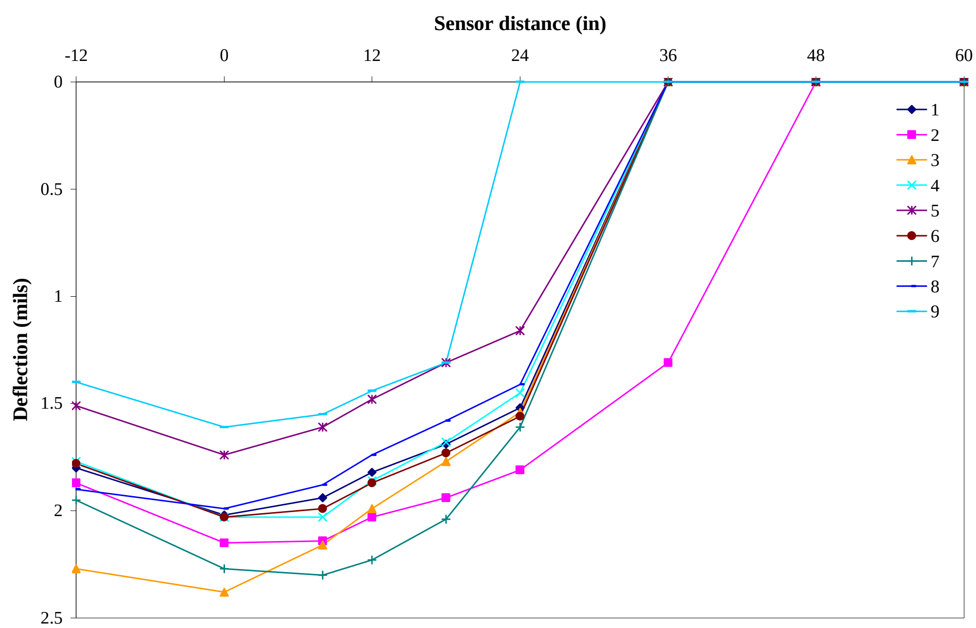

Figure B2. Ames test #1: 3000 lb. FWD raw data

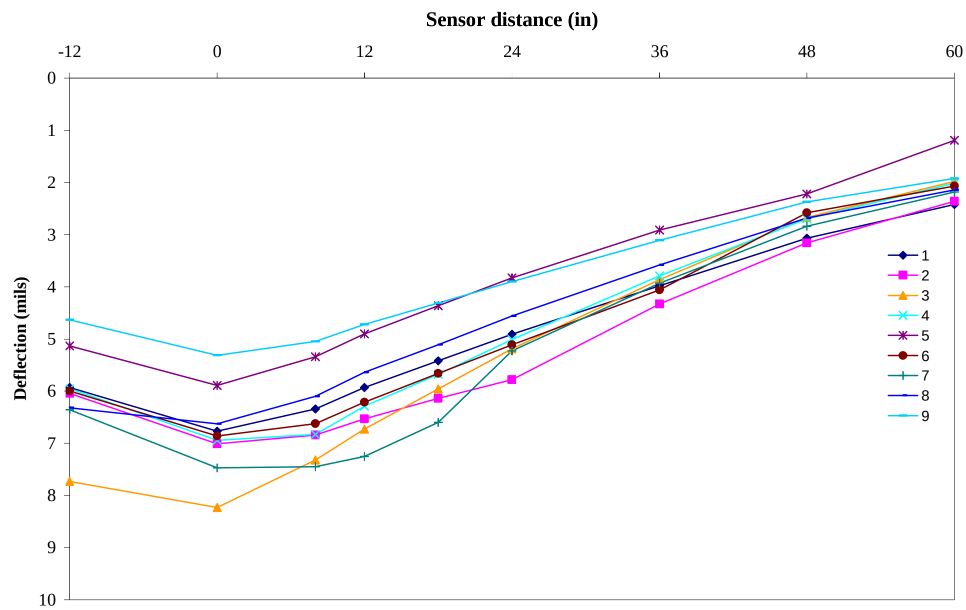

Figure B3. Ames test #1: 9000 lb. FWD raw data

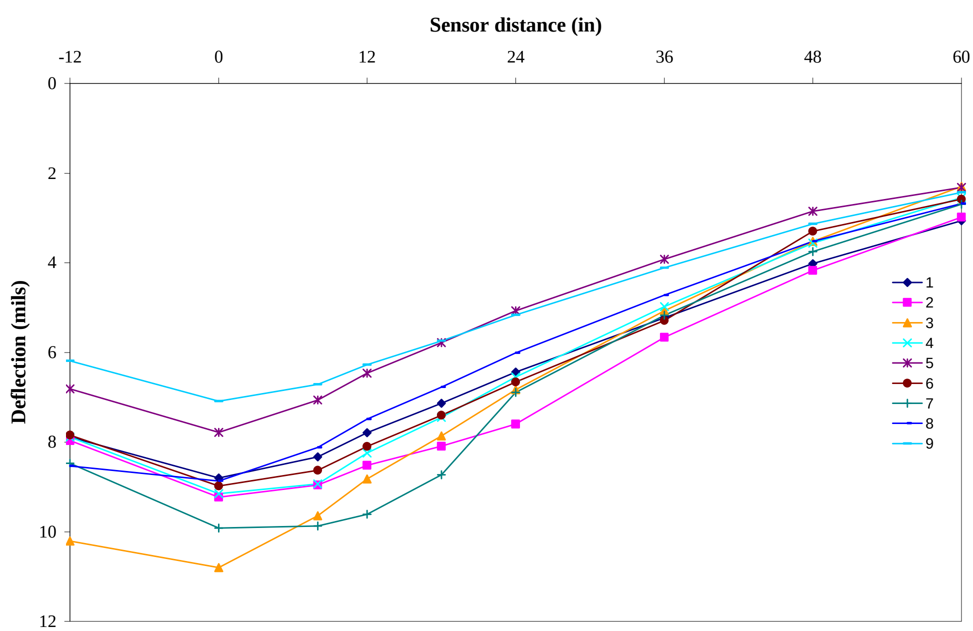

Figure B4. Ames test #1: 12000 lb. FWD raw data

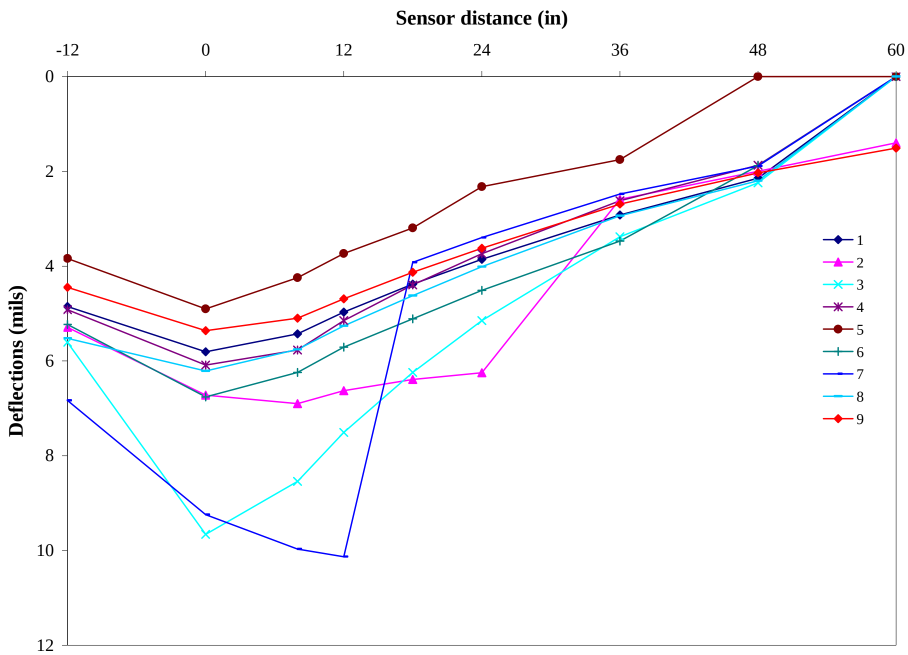

Figure B5. Ames test #2: 6000 lb. FWD raw data

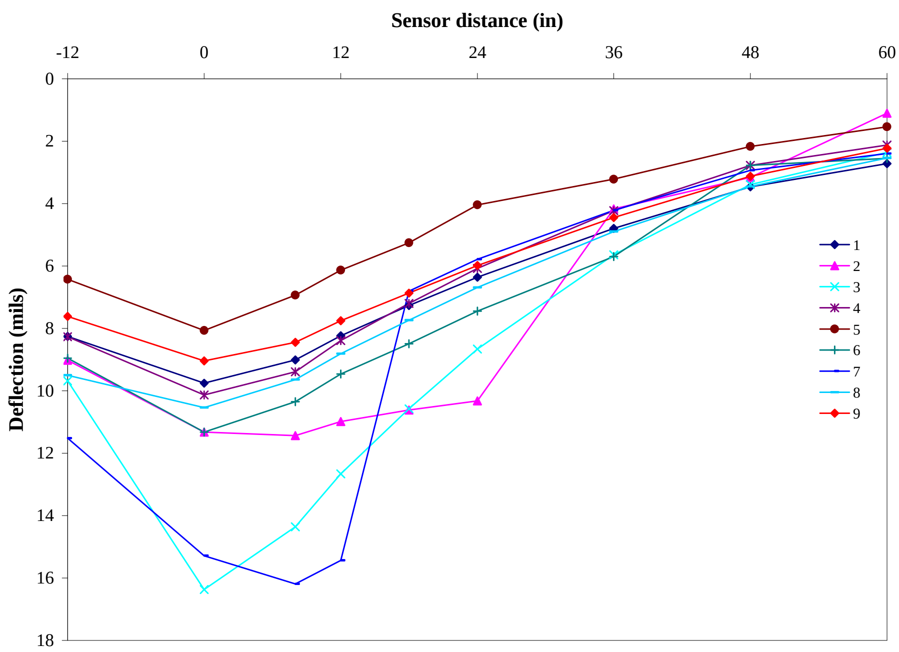

Figure B6. Ames test #2: 9000 lb. FWD raw data

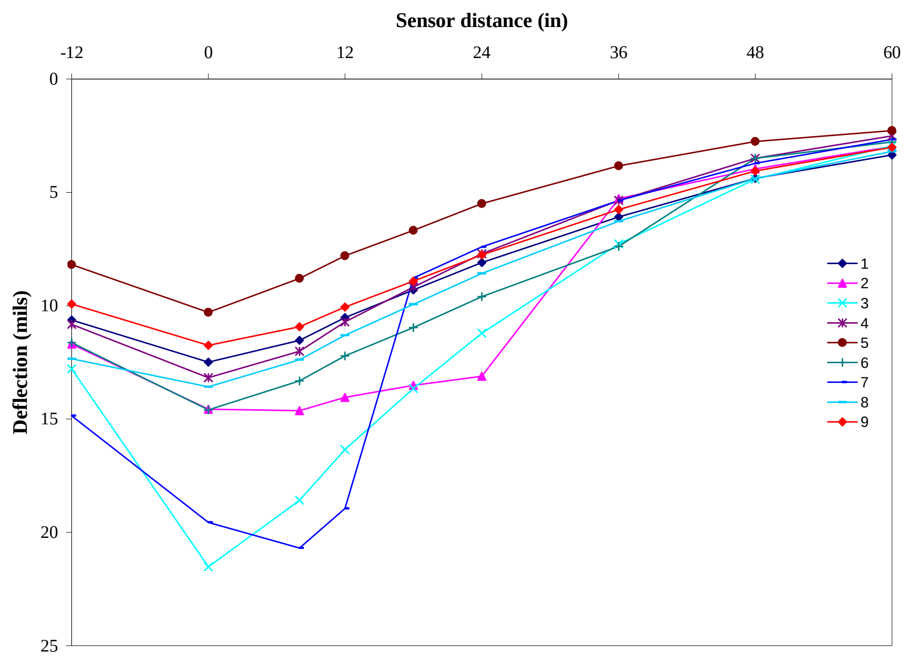

Figure B7. Ames test #2: 12000 lb. FWD raw data

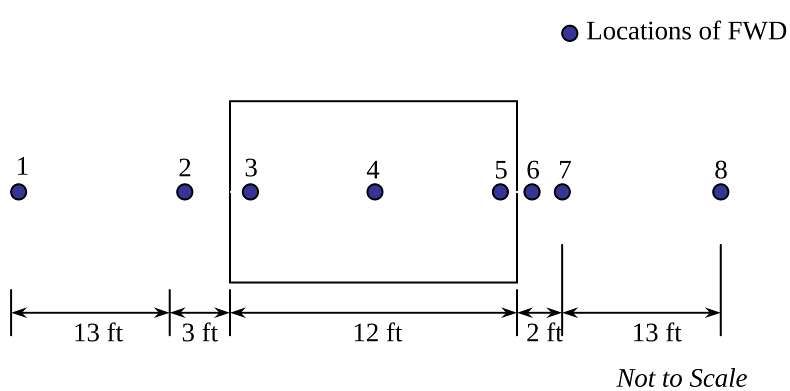

Figure B8. Cedar Rapids FWD layout

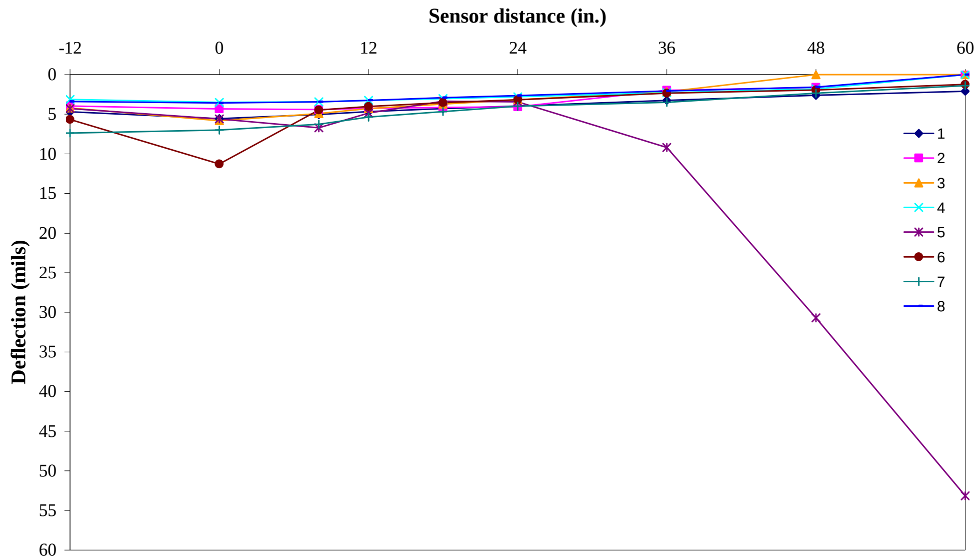

Figure B9. Cedar Rapids test #1: 4000 lb. FWD raw data

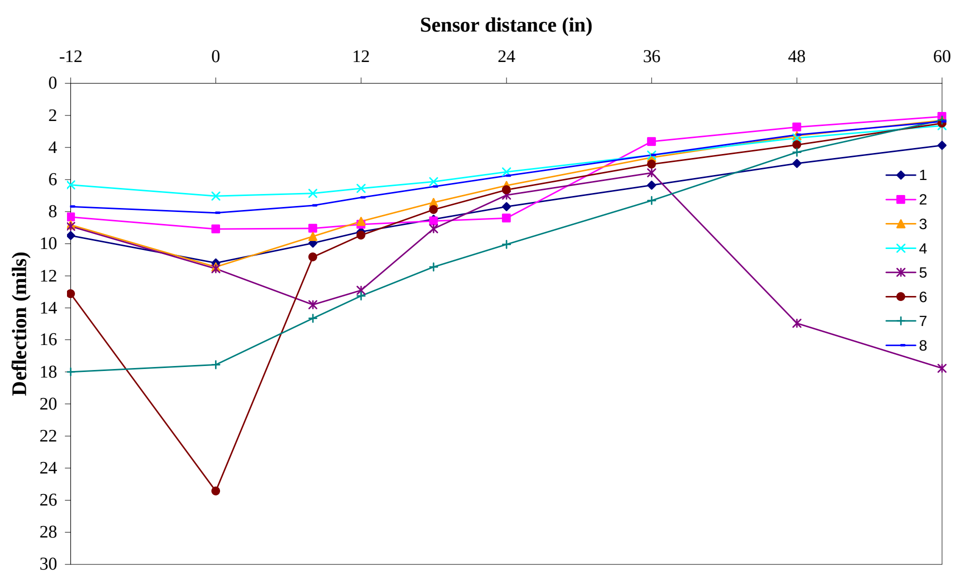

Figure B10. Cedar Rapids test #1: 9000 lb. FWD raw data

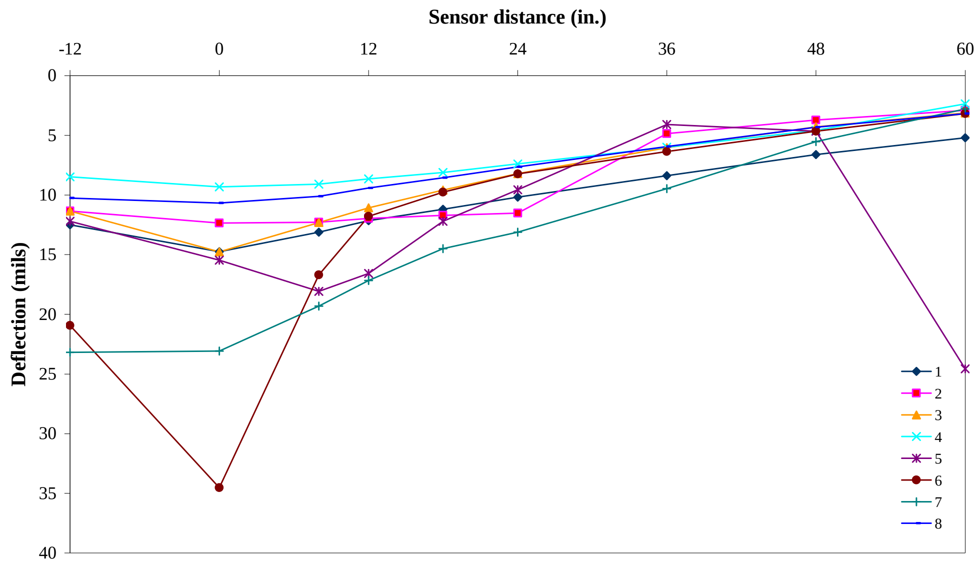

Figure B11. Cedar Rapids test #1: 12000 lb. FWD raw data

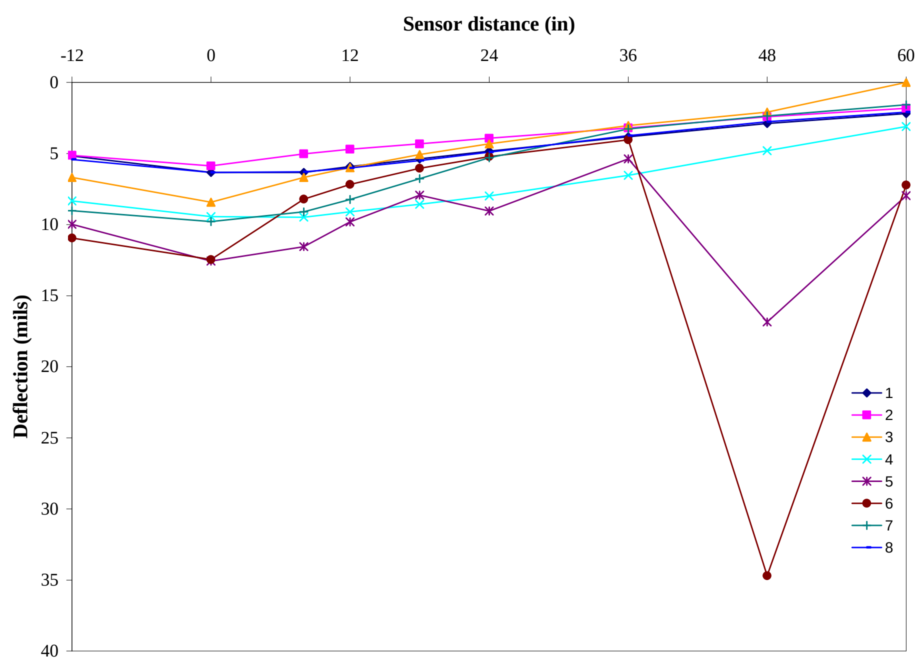

Figure B12. Cedar Rapids test #2: 5000 lb. FWD raw data

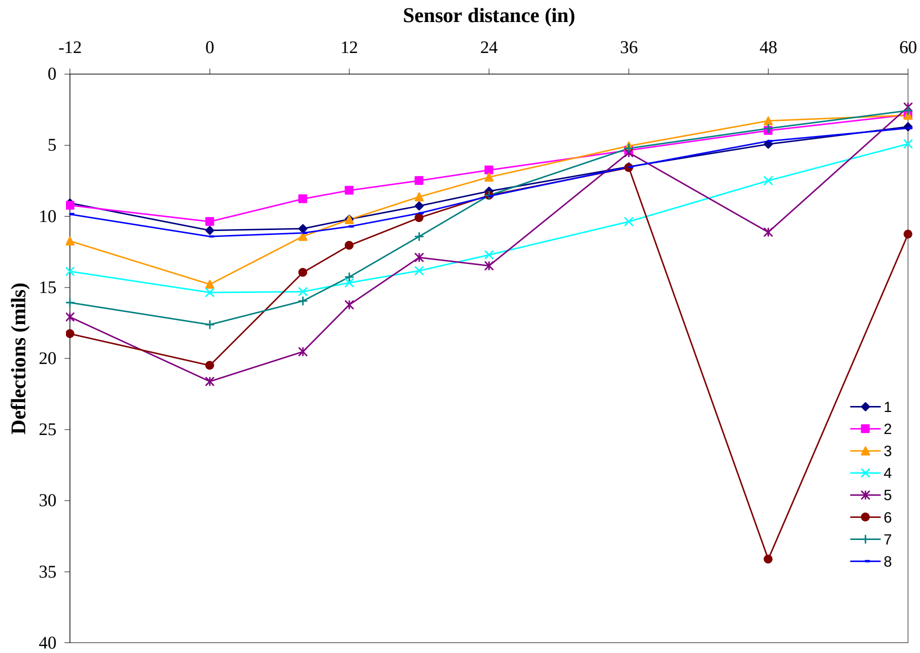

Figure B13. Cedar Rapids test #2: 9000 lb. FWD raw data

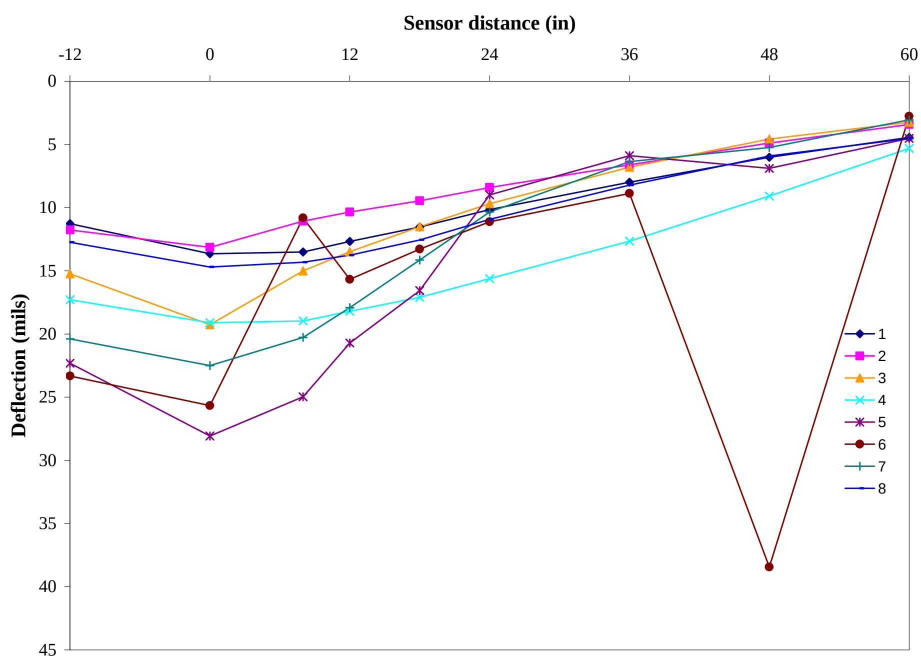

Figure B14. Cedar Rapids test #2: 11000 lb. FWD raw data

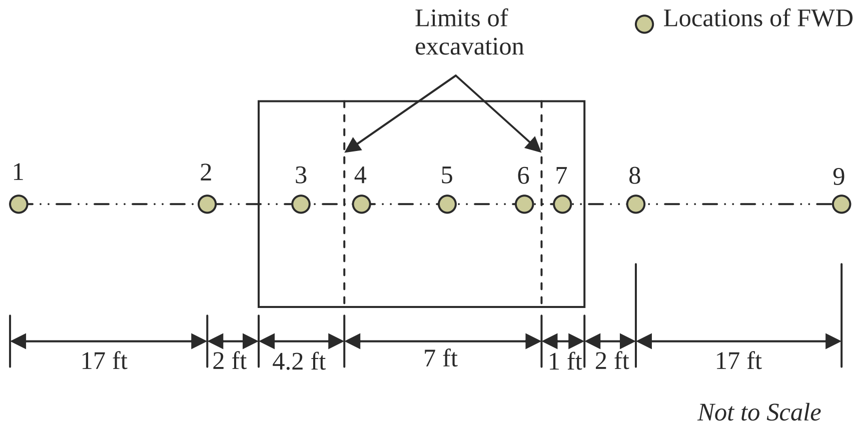

Figure B15. Des Moines FWD layout

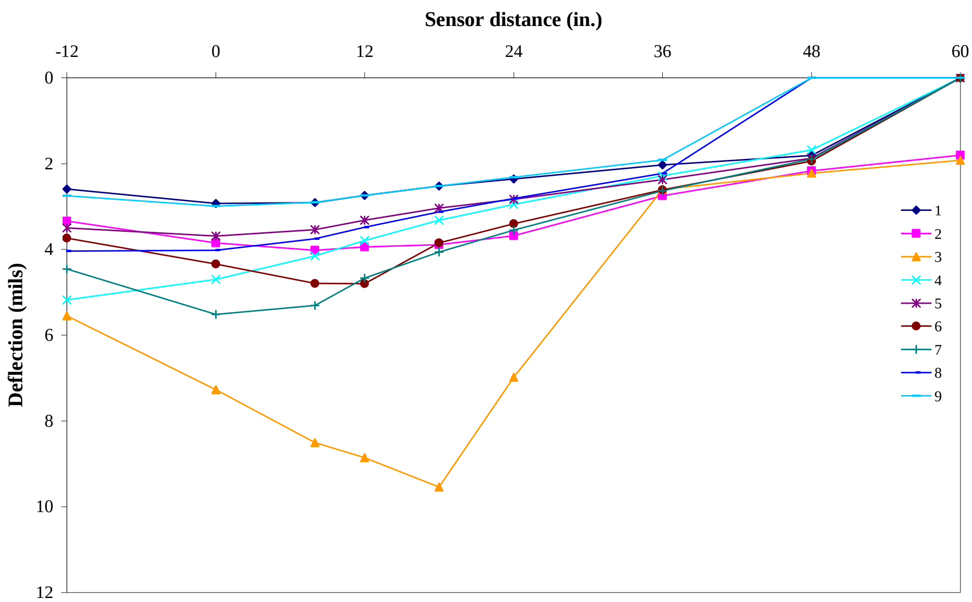

Figure B16. Des Moines test #1: 4000 lb FWD raw data

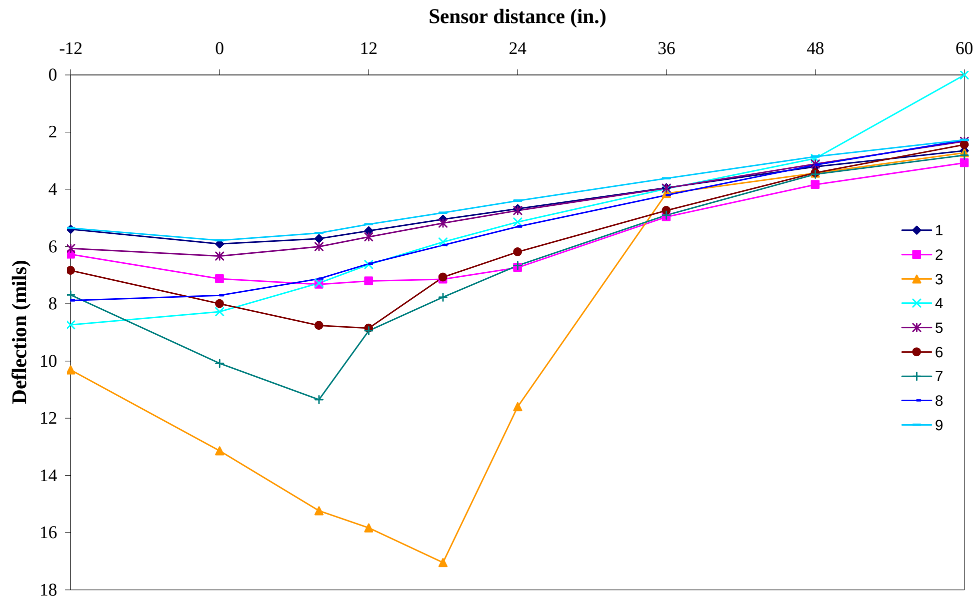

Figure B17. Des Moines test #1: 9000 lb FWD raw data

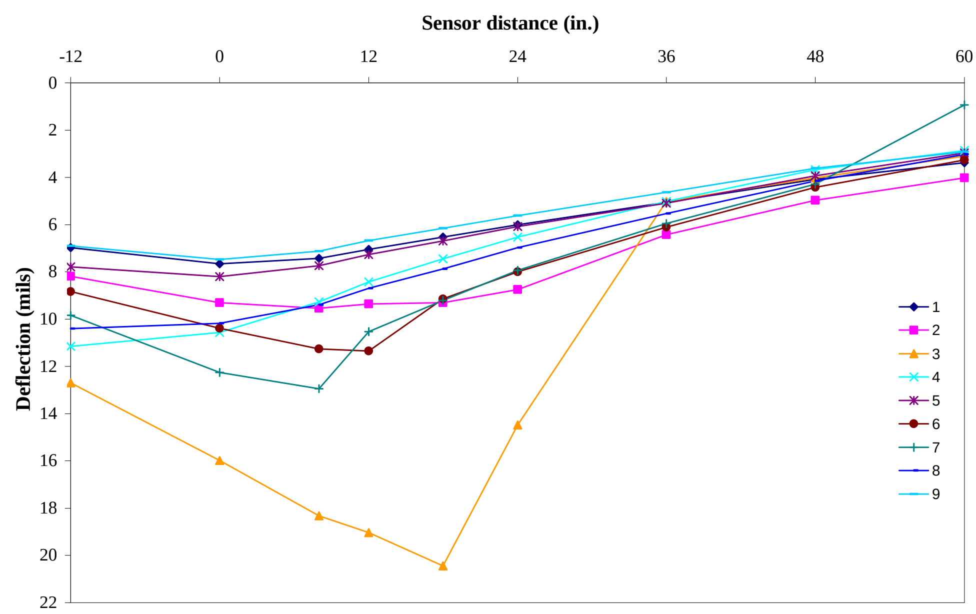

Figure B18. Des Moines test #1: 12000 lb FWD raw data

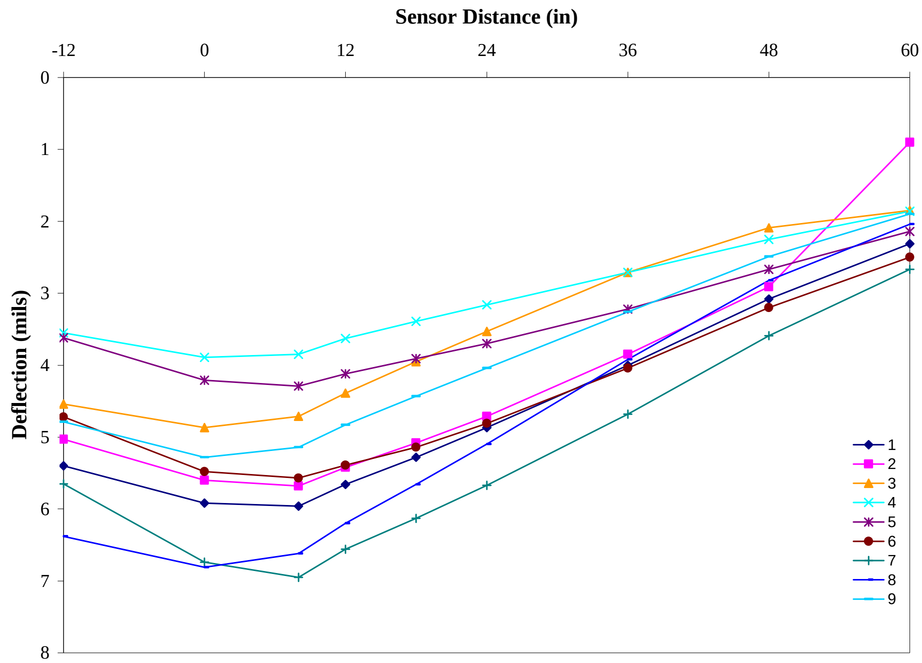

Figure B19. Des Moines test #2: 6000 lb. FWD raw data

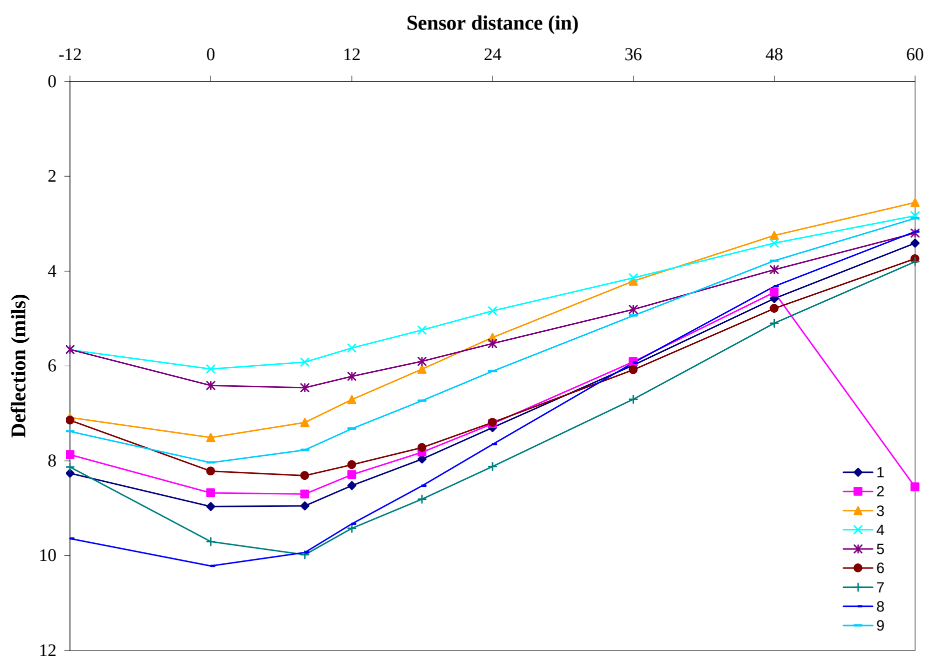

Figure B20. Des Moines test #2: 9000 lb. FWD raw data

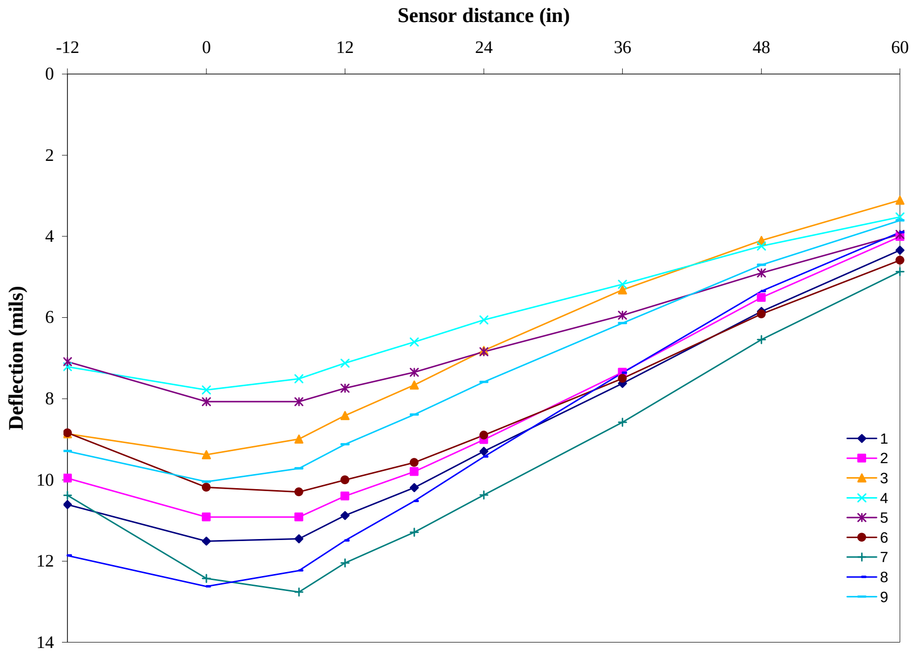

Figure B21 Des Moines test #2: 12000 lb. FWD raw data

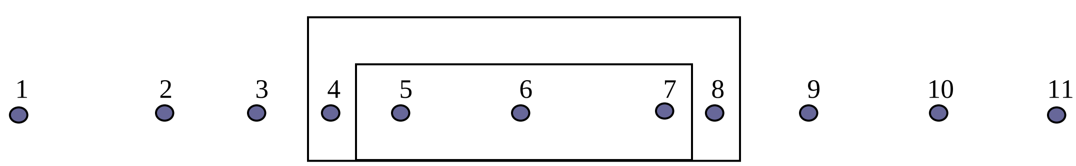

Figure B22. Ames: McKinley FWD layout

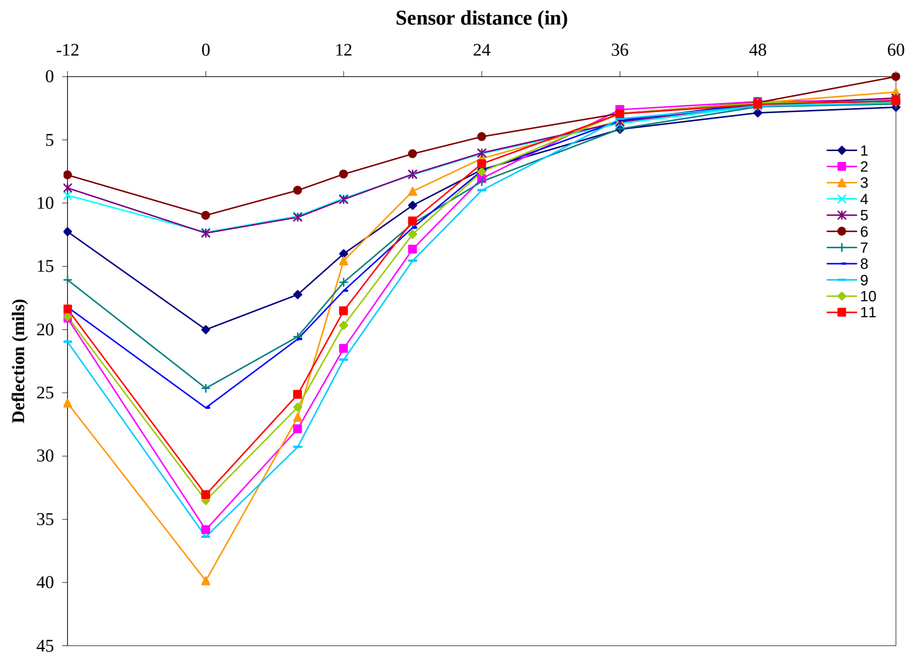

Figure B23. Ames McKinley St.: 6000 lb. FWD raw data

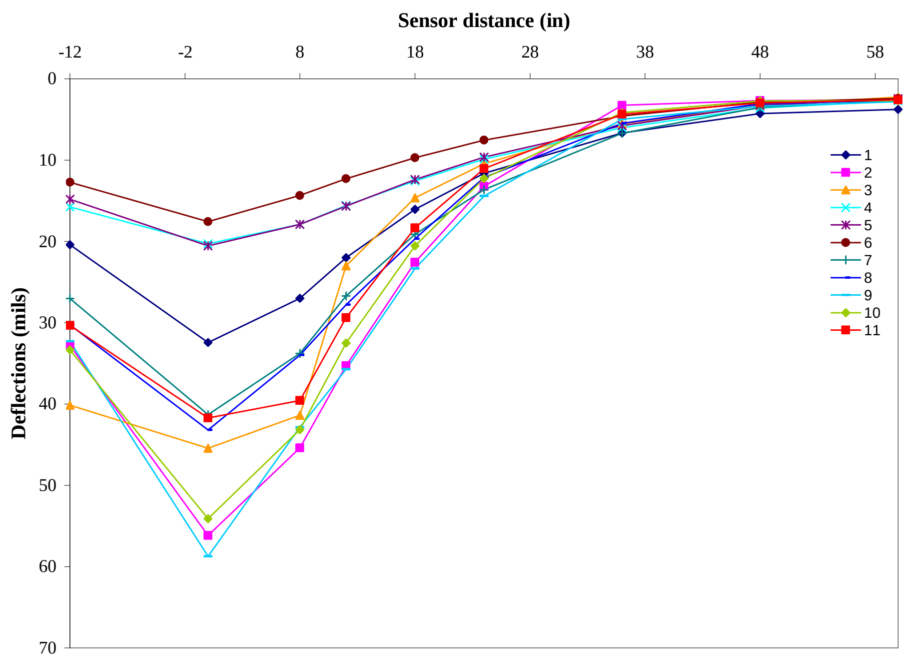

Figure B24. Ames McKinley St.: 9000 lb. FWD raw data

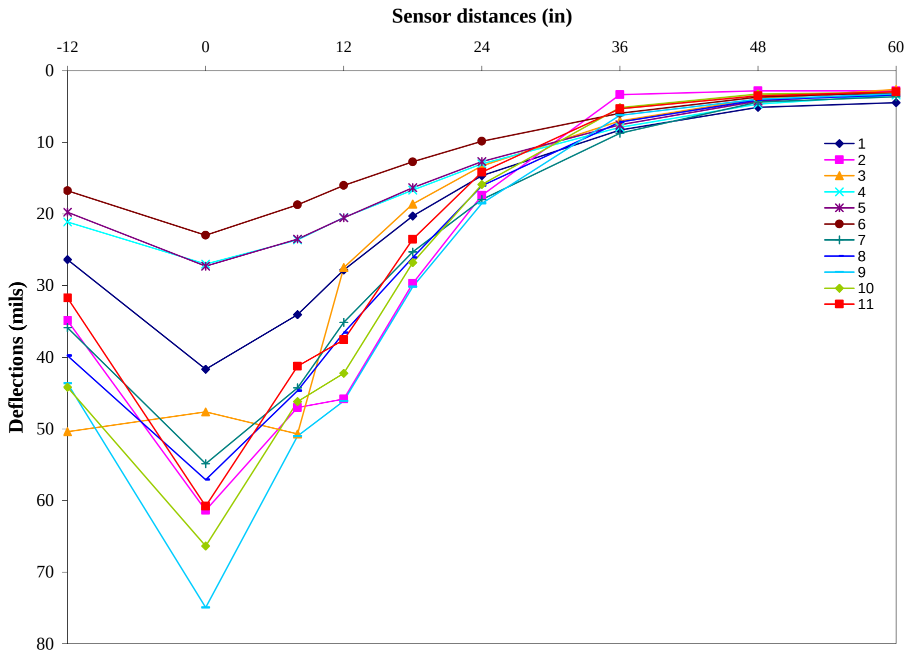

Figure B25. Ames McKinley St.: 12000 lb. raw data

APPENDIX C: FIGURES IN METRIC UNITS

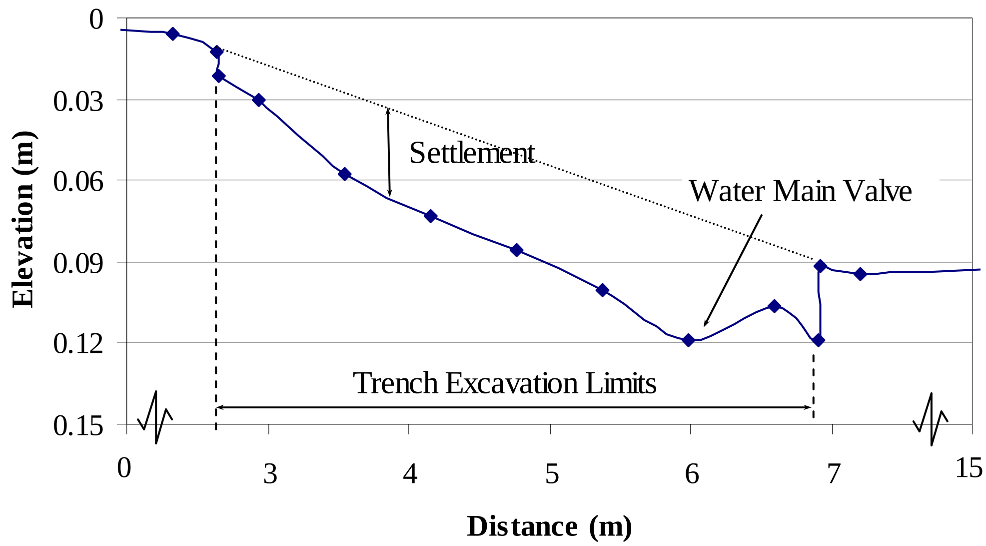

Figure C1. Settlement profile of poorly performing utility cut in asphalt pavement

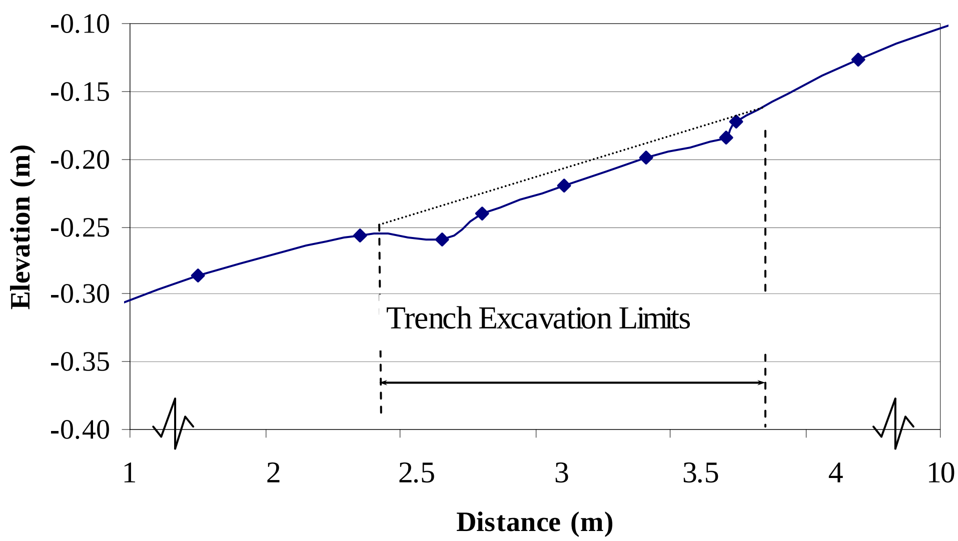

Figure C2. Settlement profile of poorly performing utility cut in concrete pavement

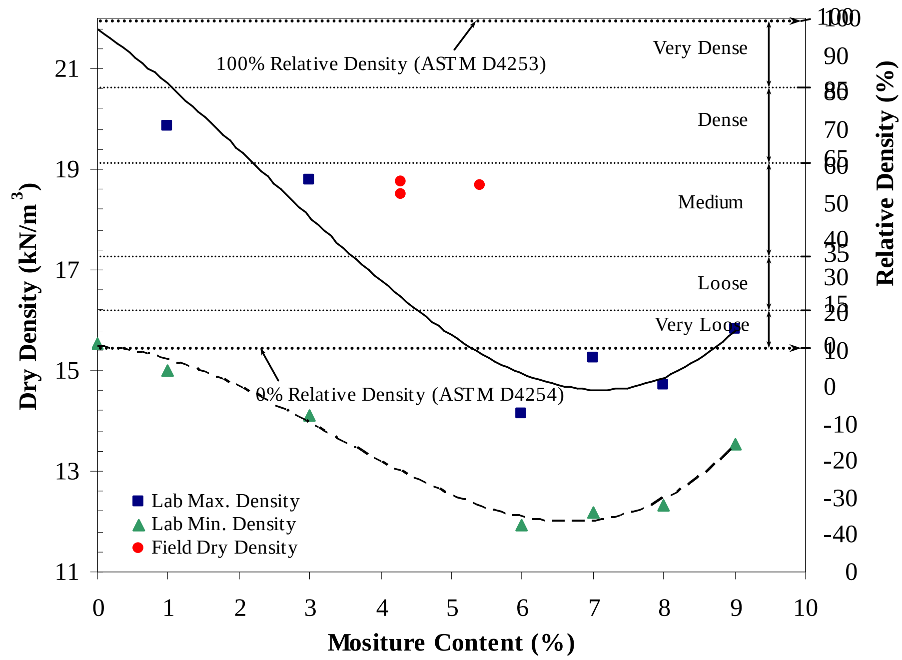

Figure C3. Ames 3/8 minus relative density plot

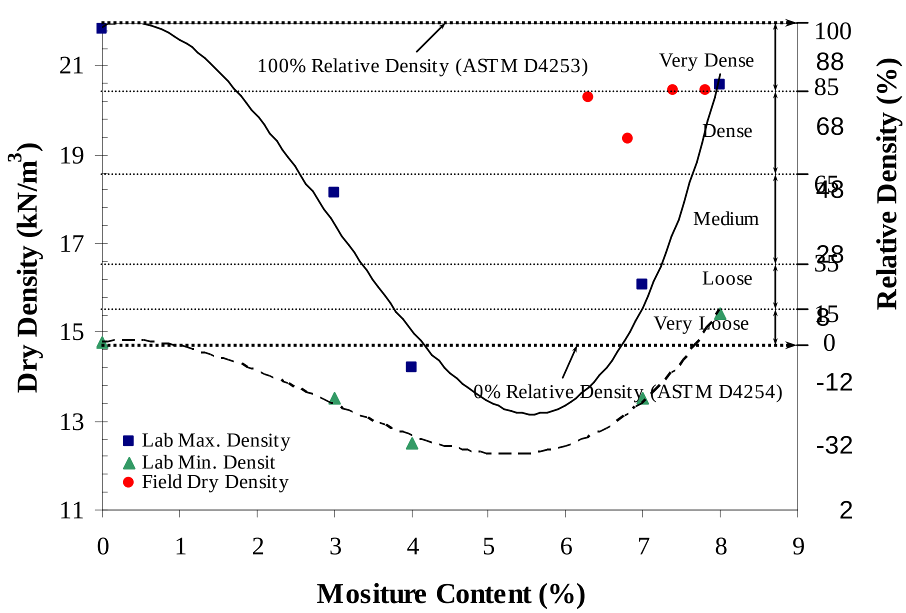

Figure C4. Cedar Rapids relative density plot

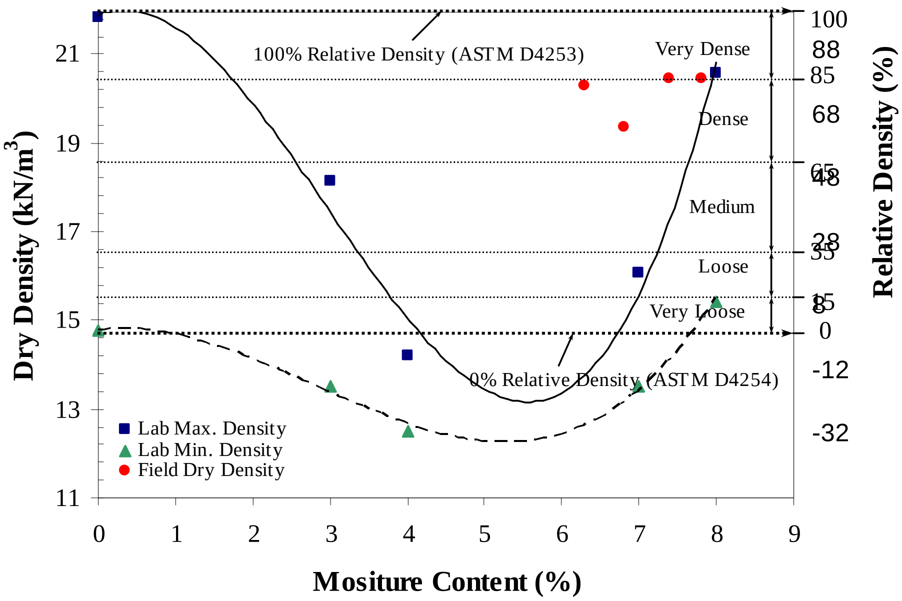

Figure C5. Davenport relative density plot

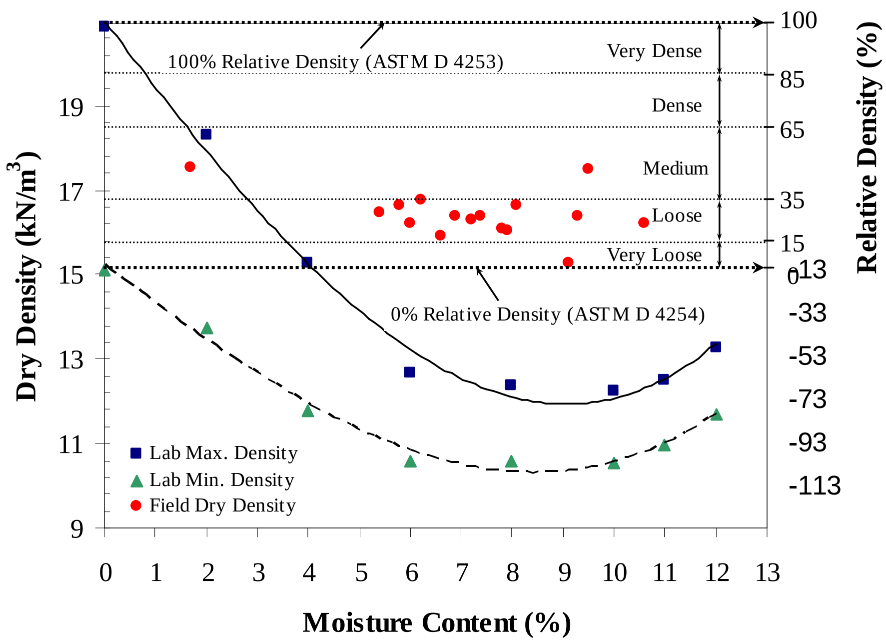

Figure C6. Des Moines relative density plot

Common unit conversions from English to Metric units of measurement are displayed in Table C1.

Table C1. English to metric conversions

Dimensions	English Units	Metric Units
Length	1 inch	25.4 millimeter
Length	3.28 feet	1 meter
Mass	1 pound	454 grams
Unit Weight	62.4 lb/ft ^{3}	9.81kN/m^3