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Investigation of improved utility cut repair techniques to reduce settlement in repaired areas

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Investigation of improved utility cut repair techniques to reduce settlement in repaired areas

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Signatures have been redacted for privacy

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ABSTRACT

Pavement settlement occurring in and around utility cuts is a common problem that draws significant resources for maintenance. Recently, a survey as well as field and laboratory investigations were carried out to identify the factors that contribute to the settlement of utility cut restorations in pavement sections throughout Iowa. Survey responses were received from seven cities in Iowa. Responses indicate: imported granular material generally is used as backfill material, utility cut restorations often last less than two (2) years, and during construction limited quality control is used.

To evaluate the performance of existing utility cuts, and document and evaluate the construction practices across Iowa, seven cities were visited where utility cut restorations were observed. From these observations it was concluded that: backfill material varies from one city to another, backfill lift thickness often exceeds 12 in. (30.5 cm), and the backfill is often placed at or near the bulking moisture content.

Backfill materials were subsequently characterized in the laboratory using gradation, specific gravity, relative density, and collapse tests. The collapse tests show that the backfill materials are placed in the field at moisture contents that have collapse potential ranging from approximately five to 25 percent. At three utility cut trenches the construction practices, backfill materials, and compaction process were documented and monitored for almost a year, using Falling Weight Deflectometer (FWD) and by measuring elevations. From the deflection data of the FWD it was observed that the area around the perimeter of the utility cut restoration showed the maximum deflection. This area, called the zone of influence, is a result of the native material surrounding the utility cut trench losing lateral support as excavation begins. Furthermore, this native material generally does not get re-compacted during utility cut restorations. The zone of influence was found to extend about two to three feet beyond the trench perimeter.

Simple design and construction procedures using two different backfill materials and geosynthetic reinforcement are suggested to reduce the settlement problems around utility trenches. Future monitoring and evaluation of constructed trial trenches will further indicate the performance of utility cuts using these construction techniques.

INTRODUCTION

Utility cuts are made in completed pavement sections to install electric, water and wastewater utilities, as well as drainage pipes under roadways. 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, it was 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 therefore 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 14.0 ft (4.3 m) long and 25.8 ft (7.9 m) wide, with elevation shots taken on the centerline 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 in (2.8 cm) elevation drop between the assumed trenching excavation limits and existing pavement, indicating significant settlement on the patched or reconstructed site.

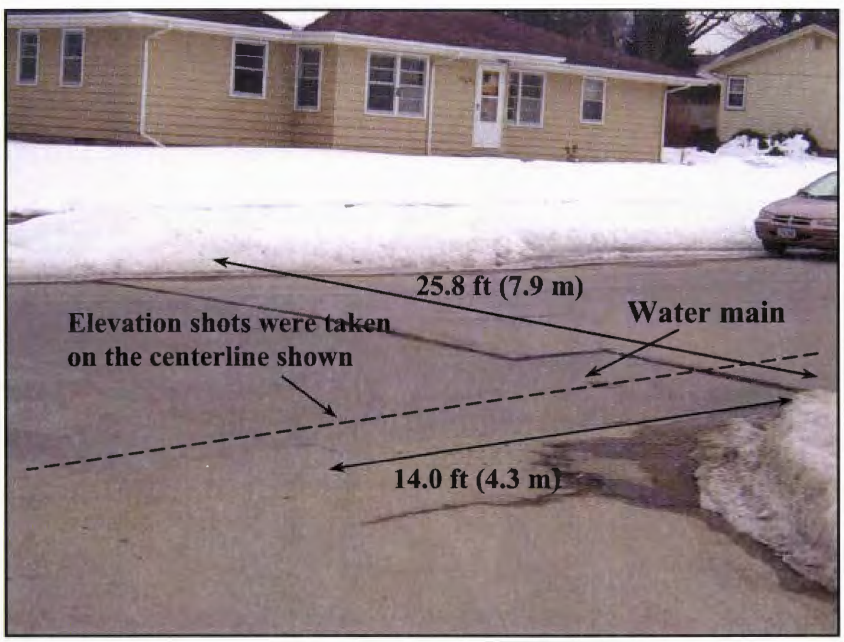


Figure 1. Poorly performing utility cut in asphalt pavement.

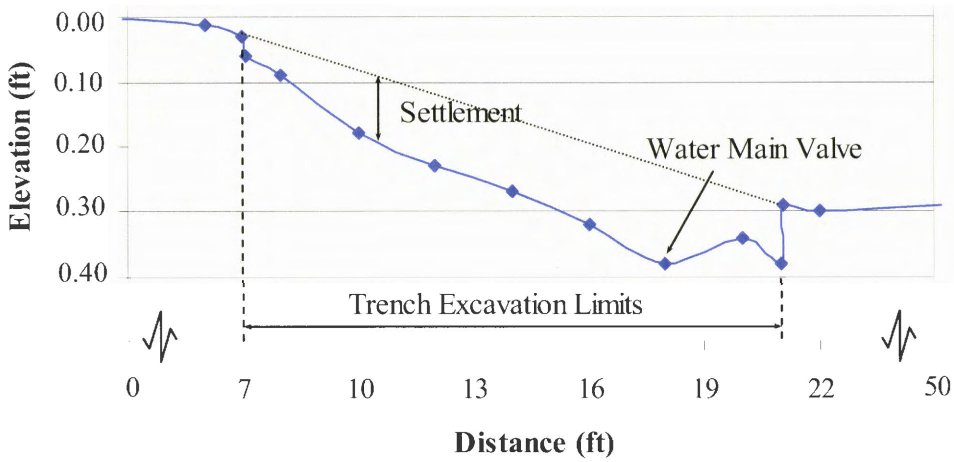


Figure 2. Settlement profile of poorly performing utility cut in asphalt pavement.

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 ft (1.1 m) long and 8.3 ft (2.5 m) wide, with elevation shots taken along the centerline of the trench as shown in Figure 3. Elevation differences of 0.12 in (0.30 cm) and 0.48 in (1.22 cm) were measured along the edge of the assumed excavation limits of the utility cut (see Figure 4). With nearly 0.5 in (1.3 cm) of difference in elevation, this amount of settlement was noticeable in a moving vehicle.

Utility cuts, specifically water main breaks, are made throughout the year. Breaks that occur in the winter months generally result in 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 seen in Figure 5.

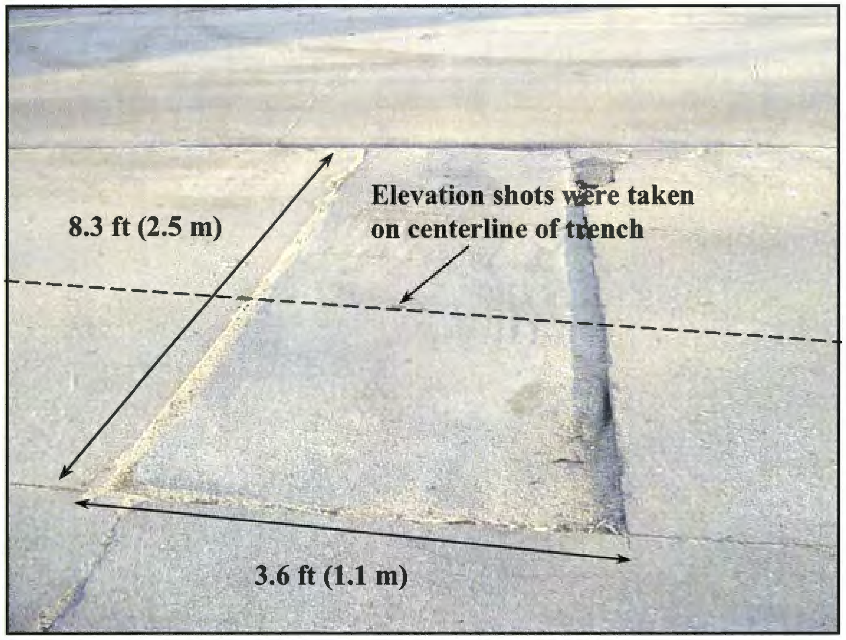


Figure 3. Poorly performing utility cut in concrete pavement.

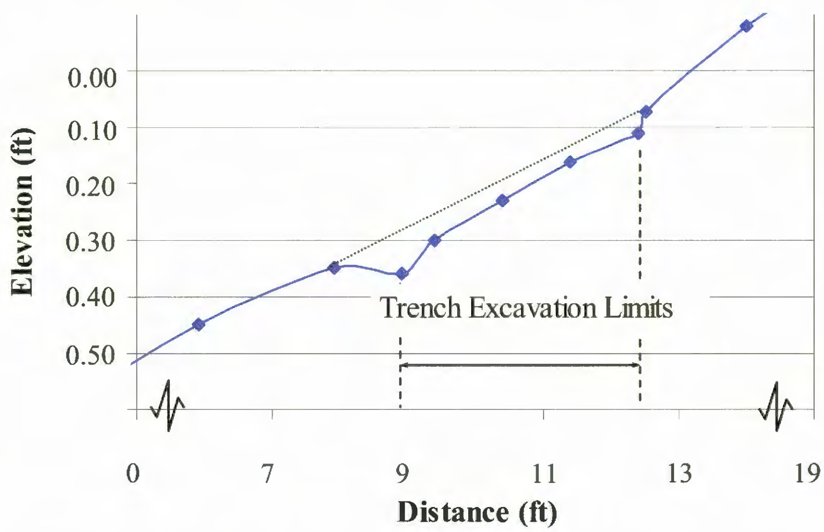


Figure 4. Settlement profile of poorly performing utility cut in concrete pavement.

Figure 5. Temporary cold patch in Cedar Rapids.

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).

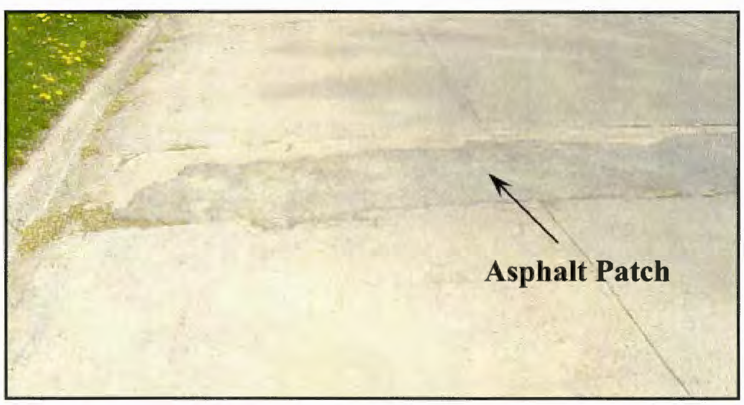


Figure 6. Asphalt patch on top of concrete patch to "repair" the settlement problem.

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 pavements, 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, etc.). The cost of repairing pavements as a result of poorly performing 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 thesis 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 fail 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 eight years. Furthermore, Tiewater (1997) indicates that several cuts in a roadway can lower the road life by 50 percent. 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 10 or more for pavement less than five years old.

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

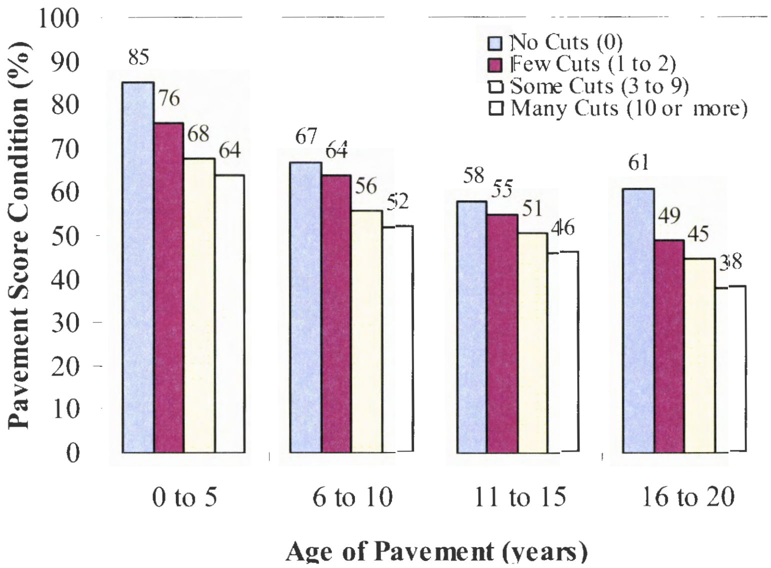


Figure 8. Utility Cut Effects on Pavement Condition (from the Department of Public Works City and County of San Francisco 1998).

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.

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 two 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 three feet (one meter) 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.

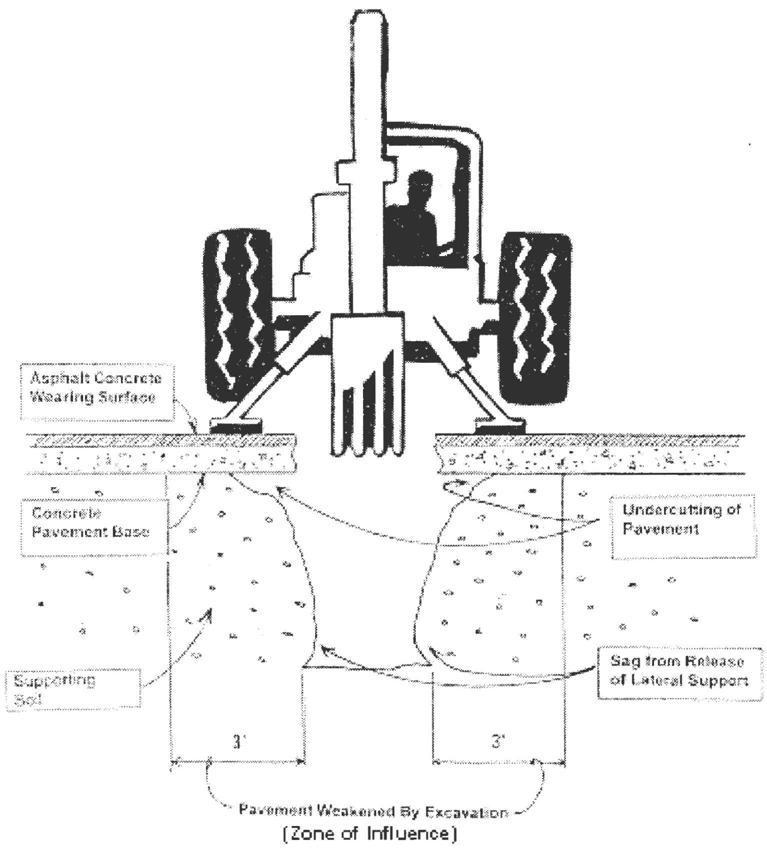


Figure 9. Overstressing of the pavement and natural materials adjacent to the trench (modified from the Department of Public Works City and County of San Francisco 1998).

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 2.0 ft (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 1.0 ft (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 5.0 ft (1.5 m) long by 4.0 ft (1.2 m) wide, had a zone of influence area extending 3.0 ft (0.91 m) on all sides of the trench for asphalt and macadam pavements. APWA (1997) stated very little damage occurring in 9.0 in (22.9 cm) thick 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.

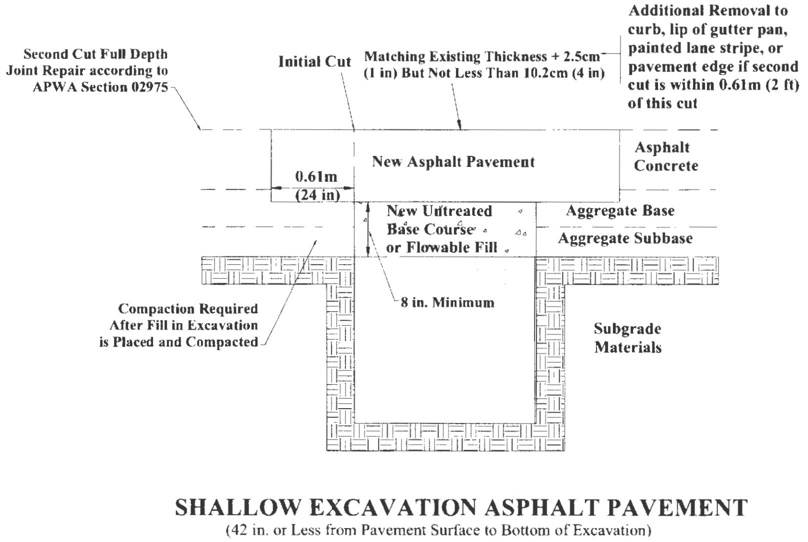


Figure 10. Salt Lake City T-Section Cross Section for a Shallow Excavation (from Peters 2002).

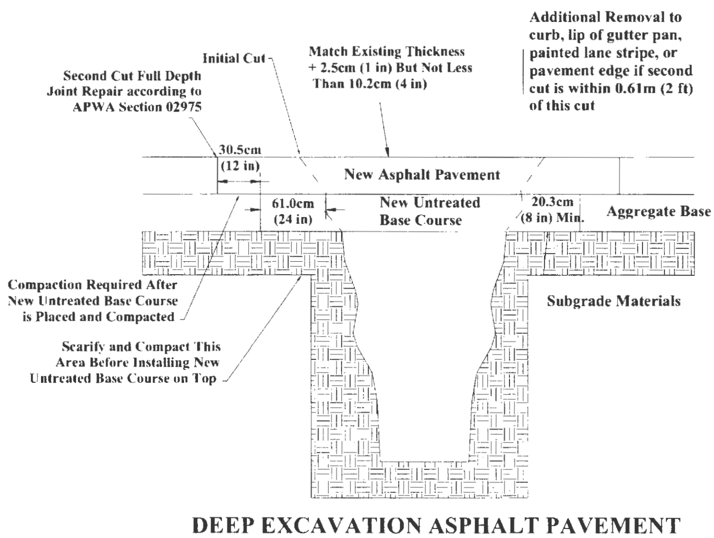


Figure 11. Salt Lake City T-Section Cross Section for a Deep Excavation (from Peters 2002).

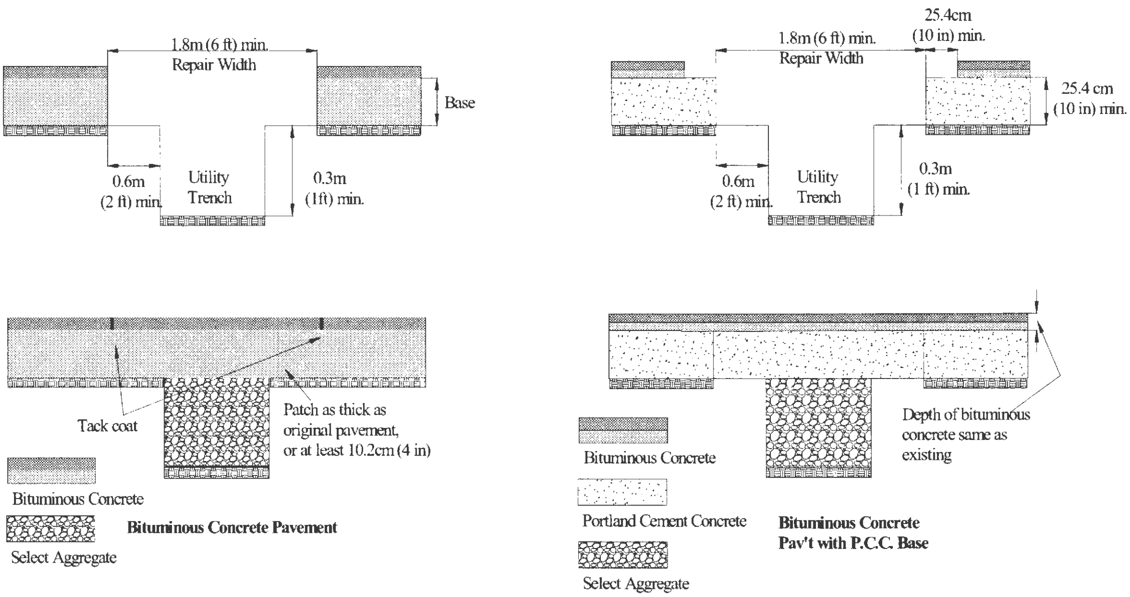


Figure 12. T-section cross-sections (from APWA 1997).

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 (0.61 to 0.91)
Washington State (granular)	2 (0.61)
Washington State (flowable)	1 (0.30)
Ohio	3 (0.91)

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 ft (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 ft (4.57 m) section, 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 ft (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 ft 6 in (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) chosen for a restoration can impact future settlement. Cohesive clay type backfill material 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.

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

Jayawickrama *et al.* (2000) states that many State Department of Transportation (DOTs) require granular material that classifies as a A-1 or A-3 according to AASHTO M145 (see Table 3). Iowa DOT suggests 100 percent passing the 75 mm (3 in) sieve, 20 to 100 passing the 2.36 mm (#8), and 0 to 10 percent 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 percent passing sieve 1 ½ in (37.5 mm), 95 to 100 percent passing the 1 in (25 mm) sieve, 25 to 60 percent for the ½ in (12.5 mm) sieve, and 0 to 10 percent for #4 (4.75 mm) sieve as opposed to the old gradation where 100 percent passing sieve 1 ½ in (37.5 mm), 95 to 100 percent passing the 1 in (25 mm) sieve, 35 to 70 percent for the 3/4 in (19.0 mm) sieve, 25 to 50 percent for the 1/2 in (12.5 mm) sieve, 10 to 30 percent for the 3/8 in (9.5 mm) sieve and 0 to 5 percent 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 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, percent 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 (from Jayawickrama *et al.* 2000).

Soil Class	Soil Class	Soil Group Symbol D2487	Description	Percent Passing Sieve Sizes 1 1/2 in. (40mm)	Percent Passing Sieve Sizes No. 4 (4.75mm)	Percent Passing Sieve Sizes No. 200 (0.075mm)	Atterberg Limits LL	Atterberg Limits PI	Coefficients Uniformity Cu	Coefficients 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 or no fines		>50% of "Coarse Fraction"				>6	1 to 3
		SP	Poorly-graded sands and gravelly sands; little or 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 Gradation-Percent Passing		**SUDAS Specification**	
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 in to 6 in (8 cm to 15 cm) thick lifts. APWA (1997) indicates that the thickness of backfill lifts generally range from 4 in (10 cm) to 12 in (31 cm), with 6 in (15 cm) being the most common depth and 12 in (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 in (20 cm). Washington DOT specifies placing material in 6 in (15 cm) lifts. Iowa, Florida, and Illinois use compacted lifts of 6 in (15 cm). However, Florida states that in the top zone (area near the surface), 12 in (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 in

(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 in (31 cm).

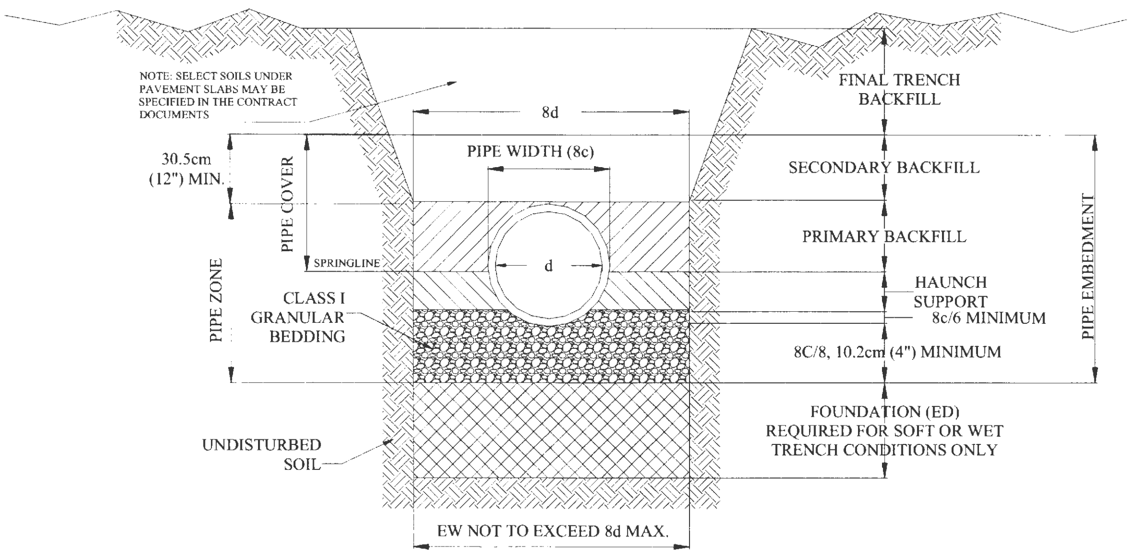


Figure 13. Typical trench cross-secion (from SUDAS 2004).

Hancor Inc. (2000) suggests that backfill around thermoplastic pipes be placed in layers of 4 in (10 cm) to 6 in (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 in (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 6 in (15 cm) for pipe diameters of 4 in (10 cm) to 48 in (122 cm) and for pipe with diameters between 54 in (137 cm) and 60 in (152 cm), a minimum final backfill depth of 12 in (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 the different backfill layers mentioned above.

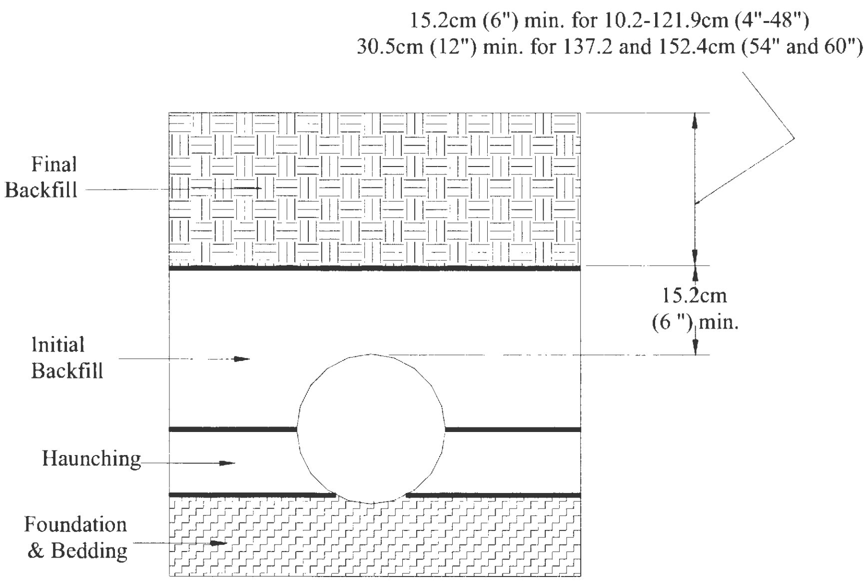


Figure 14. Typical backfill cross-section for thermoplastic pipes (from Hancor Inc 2000).

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

Table 6. Typical Properties of Compacted Soils (modified from NAFAC 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 in.
GW	Well graded clean gravels, gravel- sand mixtures	125-135	11-8	5×10^{-2}	40-80	300-500
GP	Poorly graded clean gravels, gravel sand mixtures	115-125	14-11	10^{-1}	30-60	250-400
GM	Silty gravels, poorly graded gravel- sand-silt	120-135	12-8	$>10^{-6}$	20-60	100-400
GC	Clayey gravels, poorly graded gravel-sand-clay	115-130	14-9	$>10^{-7}$	20-40	100-300
SW	Well graded clean sands, gravelly sands	110-130	16-9	$>10^{-3}$	20-40	200-300
SP	Poorly graded clean sands, sand-gravel mix	100-120	21-12	$>10^{-3}$	10-40	200-300
SM	Silty sands, poorly graded sand-silt mix	110-125	16-11	5×10^{-5}	10-40	100-300
SM-SC	Sand-silt clay mix with slightly plastic fines	110-125	15-11	2×10^{-6}	5-30	100-300
SC	Clayey sands, poorly graded sand- clay-mix	105-125	19-11	$5 \times >10^{-7}$	5-20	100-300
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	$5 \times >10^{-7}$	……	
CL	Inorganic clays of low to medium plasticity	95-120	24-12	$>10^{-7}$	15 or less	50-200
OL	Organic silts and silt-clays, low plasticity	80-100	33-21	……	5 or less	50-100
MH	Inorganic clayey silts, elastic silts	70-95	40-24	$5 \times >10^{-7}$	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.

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.

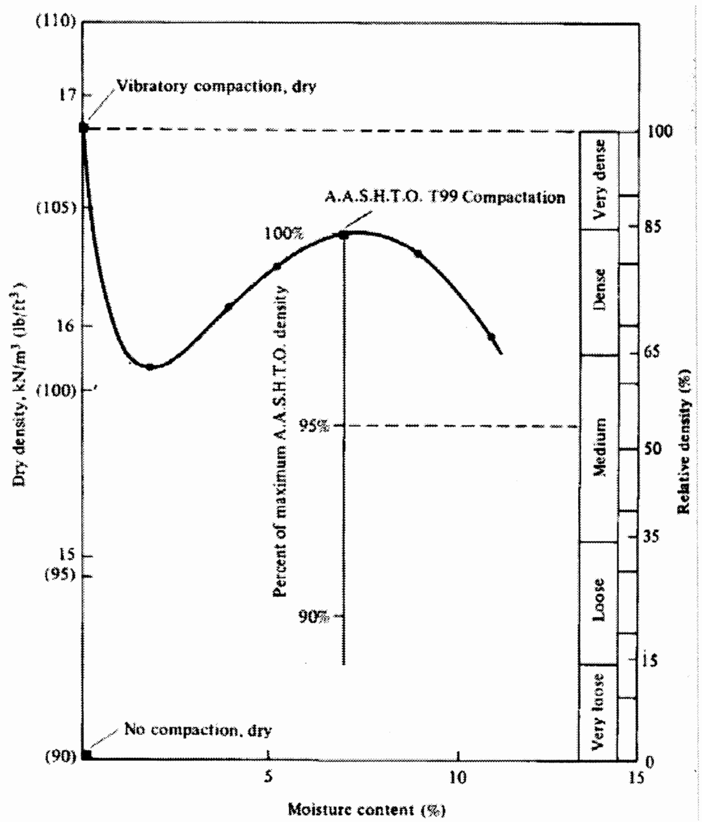


Figure 15. Relative density vs. AASHTO T99 Compaction.

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 this bulking phenomenon which occurs in granular materials. Spangler and Handy (1982) explain that the addition of a small amount of water, about six to eight percent, to sand 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 percent (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) indicate also 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 85 to 90 percent compared to a compaction requirement of 95 percent 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 material after construction of the trench as a result of factors such as infiltration or seasonal variations in the groundwater table and therefore decreasing stability of the internal tension. The bulking moisture content region is a critical factor resulting in 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 percent maximum dry density using standard Proctor, is required for backfill materials (APWA 1997). NAVFAC (1986) requires achieving 90 percent of maximum density using modified proctor and a maximum layer thickness of 8 in (20 cm) (see Table 8). As Sowers (1979) indicates in Table 9, based on experience, materials have a variety of representative percent 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 ft (1 m) below the subgrade, compaction ranging from 97 to 100 percent standard Proctor is required and material exceeding one meter 3 ft (1 m), should have a compaction of 94 to 97 percent required standard Proctor to achieve good compaction.

Table 8. Compaction Requirements (modified from NAVFAC 1986).

Fill Utilized for:	Required Density, Percent of Modified Proctor	Tolerable Range of Moisture About Optimum, (Percent)	Maximum Permissible Lift Thickness, Compacted (in.)	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 vibratory roller	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 ft) of subgrade under pavements***

Class 2 ***Deeper parts (to 10 m (30 ft)) of fills under pavements***

Class 3 ***All other fills requiring some degree of strength or compressibility***

In Iowa, SUDAS requires the final trench backfill area to achieve compaction of 95 percent of maximum standard Proctor and the bedding region 90 percent 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 in (37.5 mm) sieve) should have compaction of 90 percent 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 percent standard Proctor is required, (see Figure 13).

State DOTs compaction specifications for backfills are as follows. Ohio's structural backfill should be compacted to 96 percent maximum dry density (www.dot.state.oh.us). Iowa DOT requires 95 percent 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 percent maximum standard density should be obtained (www.dot.state.fl.us). However, they state that for metal and plastic pipes, the cover zone (area around the pipe) to be at least 95 percent maximum density (www.dot.state.fl.us). California DOT requires a relative compaction of at least 95 percent (www.dot.ca.gov). Washington DOT suggests that material which is placed above the pipe zone, be compacted to 95 percent maximum density (www.wsdot.wa.gov). The pipe zone should be compacted to 90 percent maximum density (www.wsdot.wa.gov). Table 10 compares these various state compaction requirements.

Table 10. State Compaction Comparison (from www.dot.state.oh.us,. www.erl.dot.state.ia.us, www.dot.state.fl.us, www.dot.ca.gov, www.wsdot.wa.gov)

State	Required Compaction
Florida	Min. 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 percent modified proctor, had settlement ranging from zero to 1/8 inch, whereas material compacted below 90 percent modified proctor, showed settlement up to and exceeding ½ inch (APWA 1997). Therefore the study concluded that backfill material compacted at 90 percent 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 4.0 in (10.2 cm) thick lift, in order to obtain a 90 percent 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 2) 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 lbs. and a tamper base with a maximum of 6 in x 6 in (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.

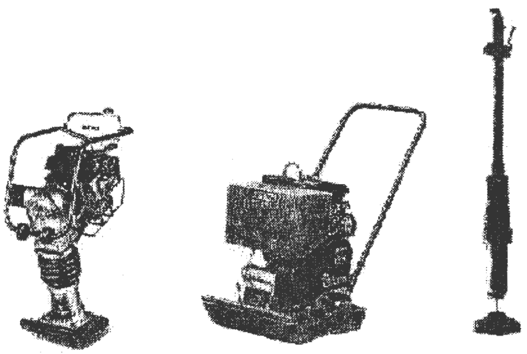


Figure 16. Compaction equipment from left to right: impact rammer, vibratory plate, and compressed-air tamper (from Jayawickrama *et al.* 2000).

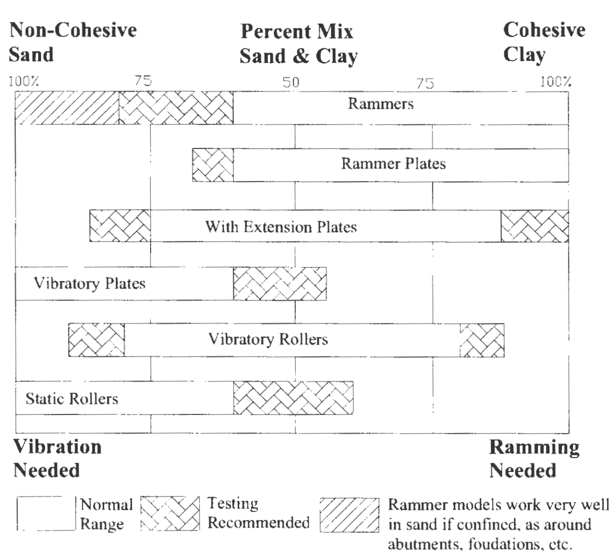


Figure 17. Guide to compaction equipment (from Hancor Inc. 2000).

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, flyash slurry, flowable flyash, 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 (ACI 1994). The cement acts as a binder and impacts cohesion and strength, whereas 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, 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 30 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) its strength and durability, 2) ability to be excavated in the future, assuming the mix design was designed correctly, 3) little field inspection is required, 4) traffic delay is minimal, 5) elimination of settlement once the mix has cured, 6) excavation costs are lower as a result of the self compacting properties of CLSM (i.e. no compaction equipment needed and therefore construction of narrow trenches), and 7) it can be used all year round (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 in using CLSM as a backfill material. One disadvantage with CLSM includes potential long-term delays in construction procedures due to setting time needed as a result of mixing (Gassman *et al*. 2001). Another disadvantage with CLSM is that since it is a flowable material, pipes have the potential to float (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). A CLSM material also initially costs more than a granular material to fill a trench (Jayawickrama *et al.* 2000). Finally, if a compressive strength is too high, future excavation of the trench can be difficult and time consuming (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 3 ft (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.

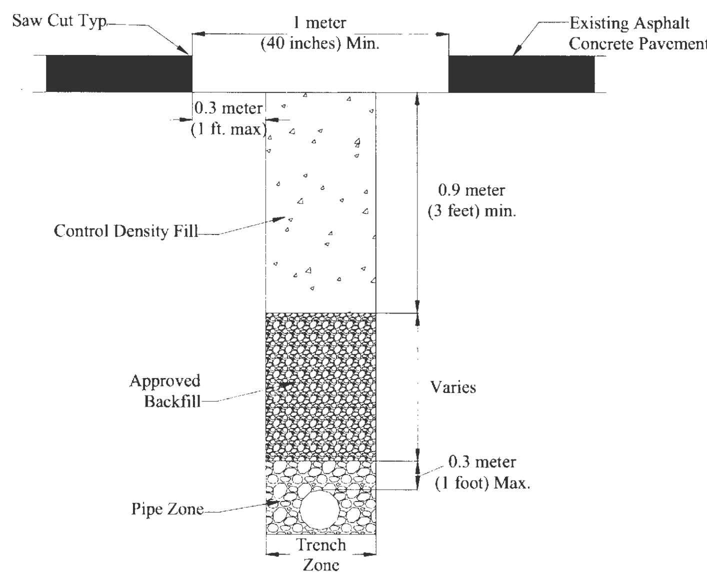


Figure 18. Typical trench from WSDOT cross-section using cdf as backfill material (from WSDOT).

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 10 to 30 minutes (see Table 11). The mixture of pulverized fuel ash (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 (from 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 a compaction procedure as follows: 1) For each lift the moisture content should be tested for compliance with the optimum moisture content, 2) the amount of time for a lift to be compacted and 3) compaction performed from the outer region of the trench towards the center to eliminate excess soil on the edges and form a connection with the trench walls and soil (APWA 1997). When using native material, compaction density should be tested on the excavated soil for compliance (APWA 1997). SoCalGas also recommends that the backfill be compacted to 90 percent 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", "to wet", and "too dry"(APWA 1997). Another advantage to the moisture meter is the ability to determine moisture contents with the use of devises 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 percent 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 8 in (20 cm) thick base course should be used, along with 1 in (3 cm) of asphalt plus any additional asphalt, minimum of 10 cm (4 in), to reach the existing pavement. Asphalt should be placed in 3 in (8 cm) lifts and compacted to 96 percent 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 are 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. etc., 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 5.1 cm (2 in) 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 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 in (20 mm) a new testing area is to be located about 11.8 in (300 mm) away. If more than 0.80 in (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 in (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). The site passes the test if this value is 0.75 in (19 mm) or lower (Burnham 1997). Burnham (1997) suggests from testing a silty/clay material DPI (DCP's penetration index) should be less than or equal to 1 in/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 ¼ in (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 18 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 Jr. 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: organic 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 clays	3 to 5
Pt: peat and highly organic soils	<1

With a background of 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 result in effects that may result in 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 an overlay of 0.75 to 1.5 in. (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 assuring 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 Observations from the Literature Review

- Millions of dollars in some cities have been spent on the maintenance and repair of utility cuts.
- A new trend in permit fees is the requirement of a structural integrity fee, which is a fee assessed to cover the costs of future repairs
- Utility cuts in a roadways result in an estimated decreased pavement life up to 50%.
- A majority of the settlement occurring in utility cuts occurs in the top two feet of an excavation.
- The use of a trenchless method for excavation can eliminate the impact a cut has on a roadway and lower traffic interruptions.
- The subgrade softening due to the zone of influence has been found to lower the structural capacity along the perimeter of a trench 50 to 65% in two years.
- Correction for the zone of influence requires a pavement cutback of two to three feet to be removed.
- A cutback was found to perform best when conducted after backfill has been compacted into the trench.
- A majority of United States Department of Transportation groups use a granular backfill material with an AASHTO classification of A-1 and A-3.
- A well-graded, gravel sand mixture with very few to no fines, is a good material for fill in roadways with or without frost heave potential.
- Lift thicknesses should be between four and twelve inches, with six inches most commonly used.
- Three major components of compaction are gradation, moisture, and compaction equipment.
- Most jurisdictions state granular compaction requirements according to standard Proctor.
- CLSM eliminates future settlement that may occur when using a granular material.
- CLSM does not require the use of compaction equipment.

- Flowable Fills are advantageous in confined areas, with strengths ranging from about 50 to 100 psi needed for potential future excavations.
- The DCP is an inexpensive devise that has been found to be a useful in some cities in determining proper compaction.
- A minimum Clegg hammer value of 18 is recommended for proper compaction for pavement surfaces.

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.

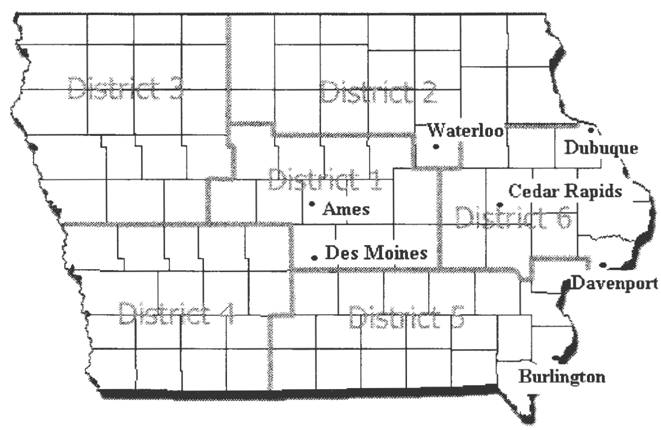


Figure 19. Survey responses from various Iowa cities (modified from www.dot.state.ia.us/tranreg.htm).

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 predominate 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 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 an overestimated number of breaks. Overall, the months of January and December have a majority of the natural breaks occurring, whereas breaks occurring in May and throughout the summer is thought to be a result of the beginning of the construction season.

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. As a result, including all data except the year 2003, an average of 24 water main breaks occur in Ames annually.

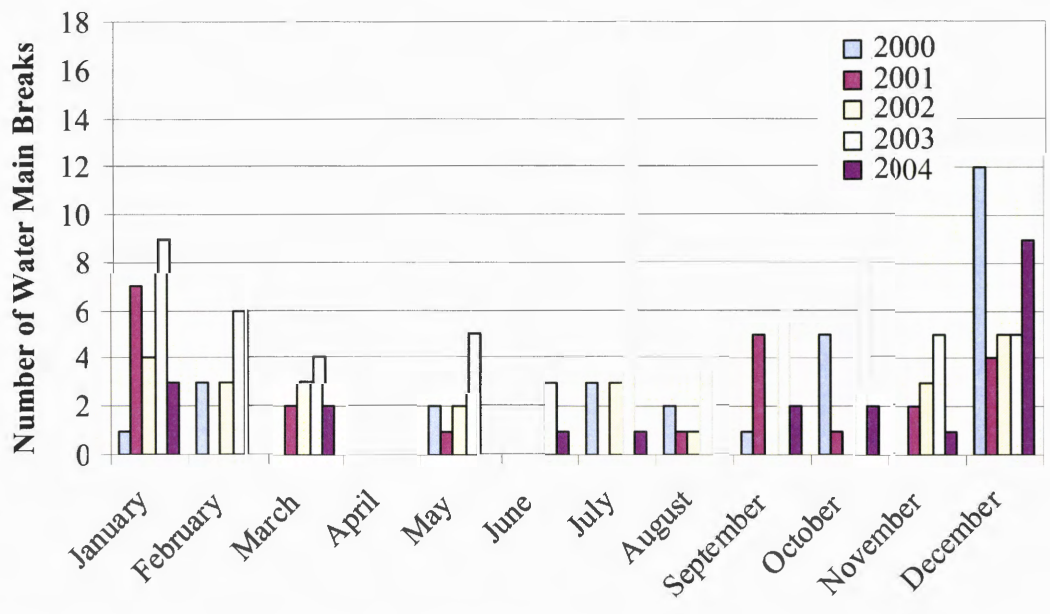


Figure 20. Monthly distribution of water main breaks in Ames, IA (Ames Street Department database).

As stated in the literature review, if a trench is constructed properly, pavements should last for 15 to 20 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.

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

The personal 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 a trenches 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 do not document accruing expenses for breaks and repairs, however Davenport estimated that these breaks and repairs cost the city about $150,000 a year for a five-man crew and Ames estimated annually $70,000. This is a significant amount of money that could be saved if proper construction practices were conducted during the original trench construction.

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: full depth saw cuts, use of manufactured sand, and ability for 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 state 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 ¾ 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, 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 percent standard Proctor density, however Davenport states 90 percent standard Proctor should be used to 18 inches below finished subgrade and 95 percent 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 cases like these, 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 12 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 a budget cut. These in-house crews do not necessarily complete the excavation and compaction process, but rather complete the surfacing of the excavation.

Quality control and quality assurance is of great importance in the proper construction of trenches. Five cities, 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 obtain, however there is no fee assessed for receiving this permit. Des Moines requires a permit fee and 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 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 titled EXCAVATE Version 2001, where fees are calculated. Waterloo has a \$10 permit fee and \$50 mobilization fee plus additional fees depending on the amount and type of surfacing material used for the excavation repair.

Summary of Observations from the Utility Cut Survey

- In most cities, little documentation on construction practices was available. Therefore many of the values obtained were primarily estimated values from the city personnel's experience.
- Using statistical data from Ames, January and December are the prominent months for water main breaks.
- All of the cities surveyed believe that their standard of practice provides satisfactory results, however there was no documentation.
- Almost all state that utility cut restorations were reported to last two years or less.
- Each city has adopted their own method of repair practice in which they follow.
- A variety of materials are being used as backfill material and chosen according to regional availability. These materials include: native material, select material containing no organics, Class A crushed stone, material passing ¾ inch, limestone crusher dust, flowable fills, 3/8 inch limestone chips, and manufactured sand.
- Burlington is the only city using a flowable fill and indicates its use is providing good results.
- Dubuque, Waterloo, and Des Moines require 95 percent standard Proctor compaction. Davenport requires 90 percent standard Proctor to 18 inches below finish grade and 95 percent above that region.
- Inspection in most cases is visual and not by the use of instrumentation, such as the nuclear gauge.
- Permit fees from responding cities indicated that fees varied from no charges to as much as $1000 across Iowa.

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 include: Ames, Cedar Rapids, Council Bluffs, Davenport, Des Moines, Dubuque, and Waterloo (see Figure 21).

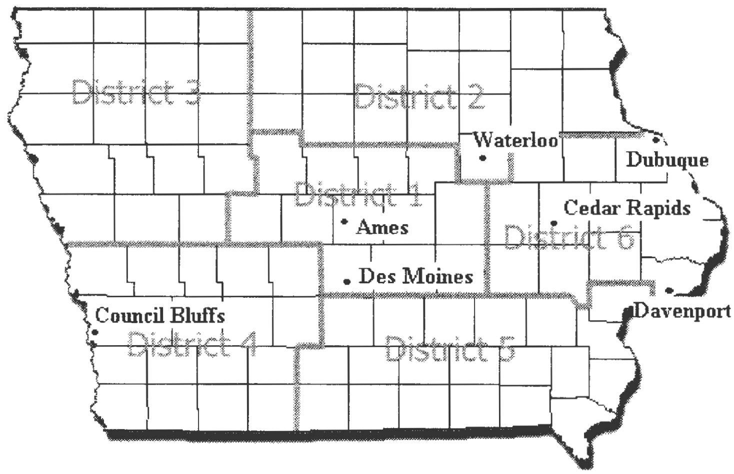


Figure 21. District Map of Iowa (modified from www.dot.state.ia.us/tranreg.htm).

Field Observations of Iowa Practices

A variety of construction practices and material use has been observed throughout 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 (one meter) 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.

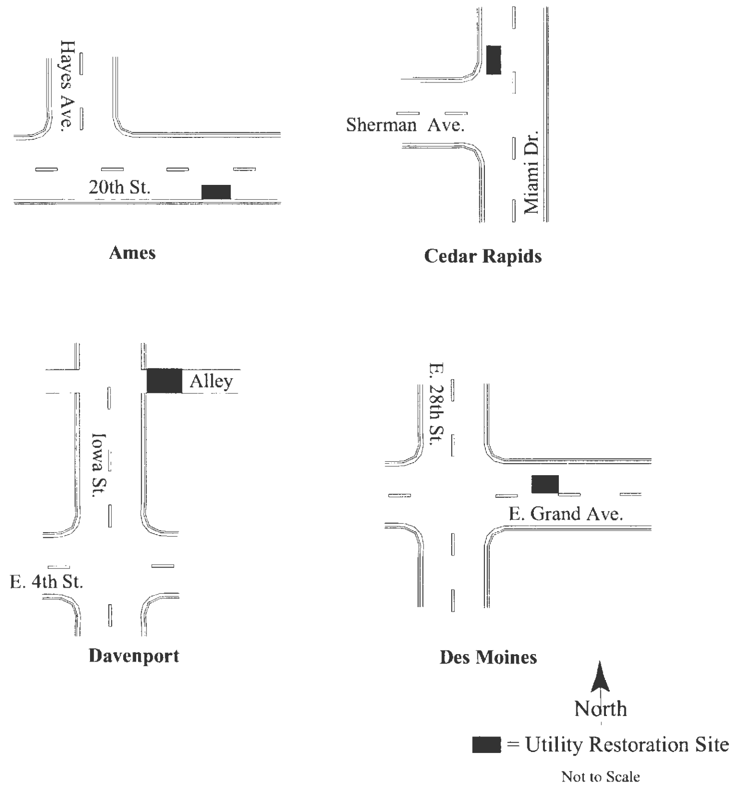


Figure 22. Iowa utility restoration site locations.

Due to the unstructured occurrences of utility breaks and 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 without extensive testing 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 16 ft (5 m) long, 6 ft (2 m) wide and about 10 ft (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 10 ft (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 4.0 ft (1.2 m) of 1-inch limestone was placed as a bedding and backfill material up to 2.0 ft (0.6 m) above the pipe crown. This bedding material was compacted in 2.0 ft (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 2.0 ft (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 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 ft (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 again. 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 ft (2.3 m) long, and 8.5 ft (2.6 m) wide, with a depth of 6 ft (1.8 m). This utility cut was constructed by the city of Ames, in pavement consisting of 10 in (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 ½-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 ft (2.4 m) wide, 12 ft (3.7 m) long and about 10 ft (3.0 m) deep. 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 two inch by four inch 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 ¾-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 3.0 ft to 4.0 ft (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 six inches of concrete and two 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 damage (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 15.0 ft (4.6 m) wide, 13.0 ft (4.0 m) long and 10.0 ft (3.0 m) deep.

Imported backfill material consisted of 1 ½-inch limestone as a bedding material and a backfill material with a maximum of ¾-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 6.0 in (15.2 cm) lifts. However, the material was placed in approximately 4.0 ft (1.2 m) lifts, 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 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.

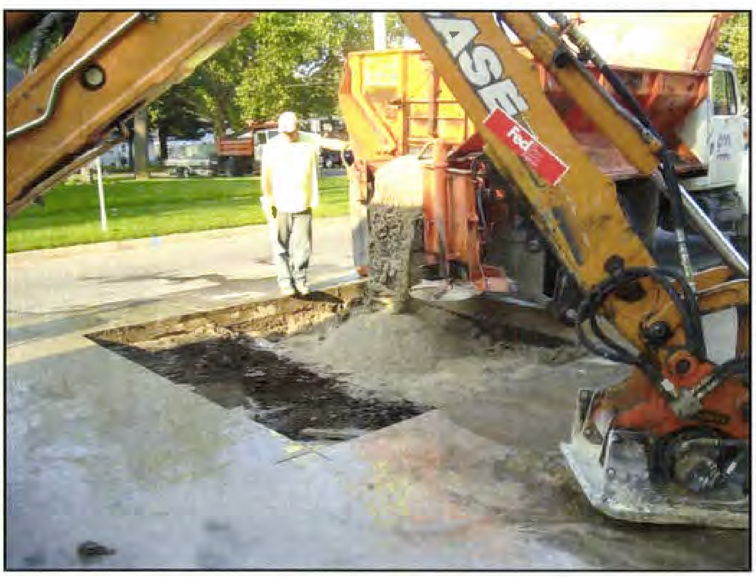


Figure 47. Adding additional manmade sand to the trench.

The pavement consisted of 8.0 in (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. These cities include: 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 5.0 ft to 6.0 ft (1.5 m to 1.8 m) deep 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 80.0 ft (24.4 m) had to be reconstructed. With continual use of the nuclear gauge the construction workers were able to reach the 95 percent 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. After speaking with a representative from Waterloo, it was stated 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 which 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

- During the excavation, material sloughing off extends into the zone of influence mention in the Literature review.
- Damage to pavement surfaces in many cases occurred along the perimeter of the excavation as a result of tough working conditions involved and in maneuvering the backhoe.
- Backfill material used in the trenches varied from each city.
- Moisture Control of backfill material was not observed in the field.
- The thicknesses in backfilled lifts exceed the recommended maximum depth of 12.0 in (30.5 cm) recommended by many cities according to the Literature review.
- No small compaction equipment was used to compact confined areas, such as the cavities noted on the Davenport site.
- In several cases, refuse was incorporated into the backfill material.
- Saturated excavated materials were cleaned into the trench during the backfilling procedures.
- Field and laboratory tests have been 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 contents are important for the determination of compactive properties of backfill materials in the field. Stiffness is a parameter, equally as important 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 correlations and directly obtaining soil properties during construction. Statistical analysis including mean, standard deviation and coefficient of variation were calculated 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 (%). This test is typically tested 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 in (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 lb (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:

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

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 in (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 lb (4.5 kg) hammer from a height of 18.0 in (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):

$$CBR = (0.24(IV) + 1)^2$$

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 lb, 9,000 lb, and 12,000 lb (2722, 4082, and 5443 m) 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 ft (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 option 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 in/blow (41.0 mm/blow) and low of 0.7 in/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 51, 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.

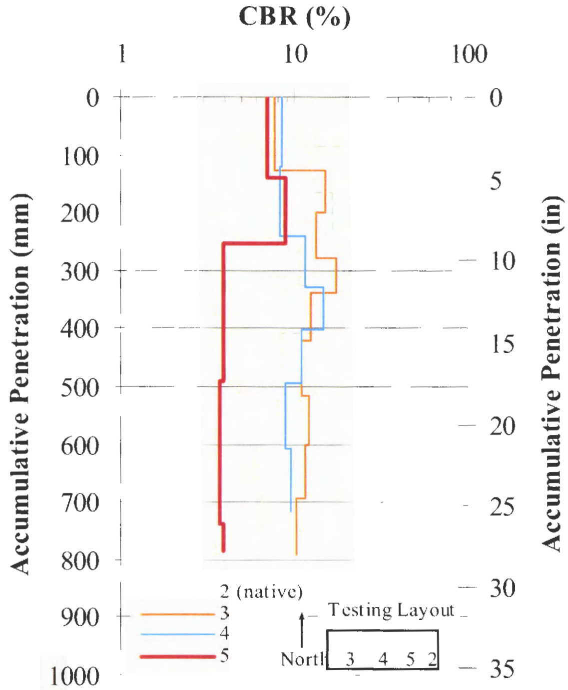


Figure 52. CBR Profile for Ames.

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 blow typical values of 10 to 20% stated in Table 12.

The Dynamic Cone Penetrometer resulted in a mean DCPI value of 0.72 in/blow (18.3 mm/blow), with a coefficient of variance of 46.0%. Using the mean DCPI values for each location, CBR values ranged from a high of 25.0% 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 blow typical values of 10 to 20% 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.

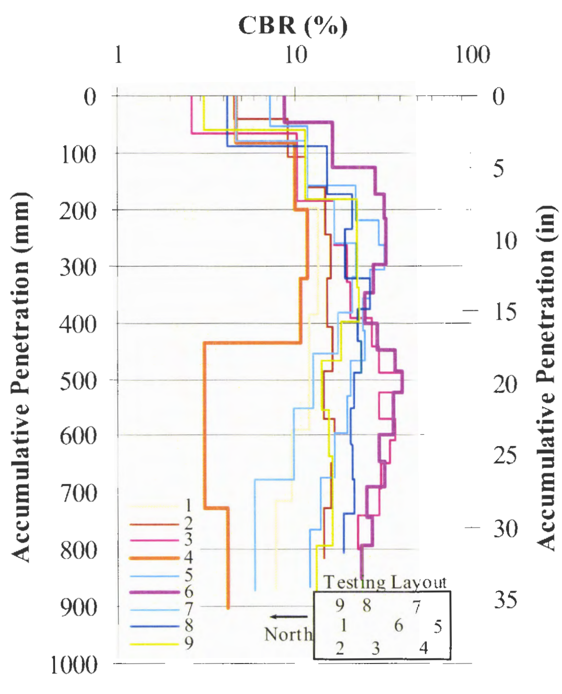


Figure 53. CBR Profile for Cedar Rapids.

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 122.0 pcf (19.2 kN/m^3). The mean was 127.0 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 115.0 pcf to 130.0 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 8.0 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 25.0 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 2.0% (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.

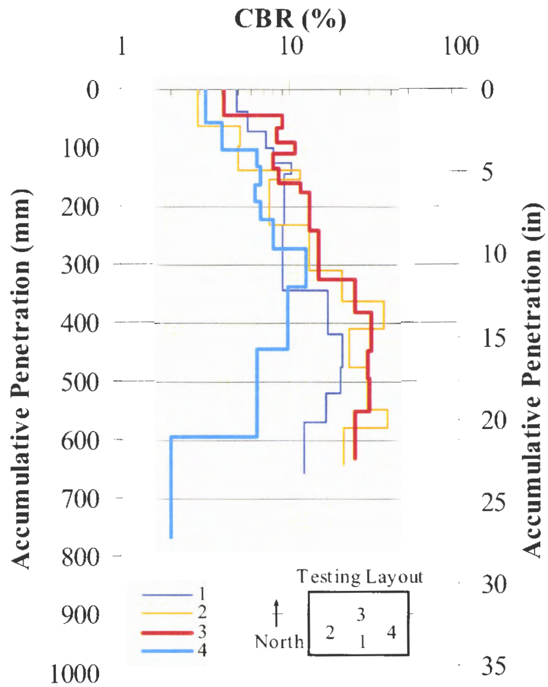


Figure 54. CBR Profile for Davenport.

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 51.0 MPa and low of 35.9 MPa, with a mean of 41.0 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 12.0 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 30.0%. 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 are shown in Tables 14 and 15, where data is organized according to each cities. 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.

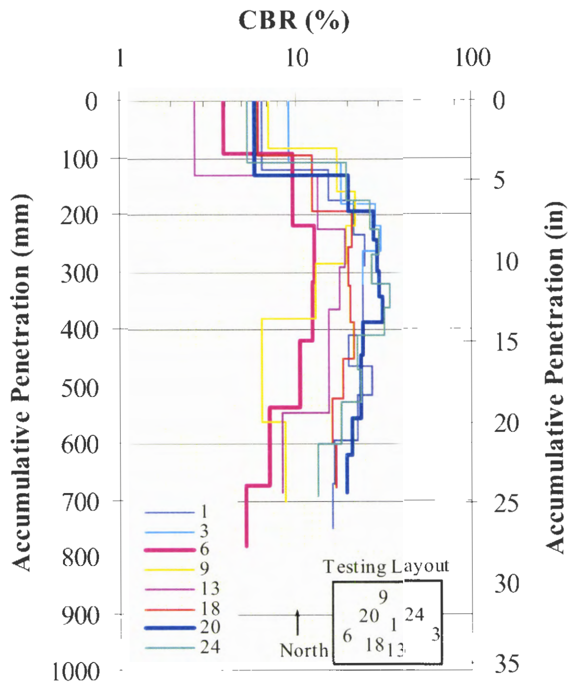


Figure 55. CBR Profile for Des Moines.

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 ft), profiles reflect the top three feet of the testing area. Stiffness measurements were made entirely either in native material or imported material, since the trenches range from eight to ten feet deep 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 in (10 cm) depth. Essentially, the data was broken into 3.9 in (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 56A, B and D illustrate, the number of blows needed to penetrate a 3.9 in (10 cm) depth, tends to increase, level off, and then decrease with greater depth, however Figure 56C has an increasing pattern, with a slight decrease with depth at 24 in (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 in (10.0 cm) depth at approximately 1.5 ft (500 mm) below the backfilled surface. This would indicate that lifts should not exceed 1.5 ft (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 in) depth between about 7.9 in to 19.7 in (200 to 500 mm). In other words, seven blows were needed to penetrate the material from a depth of 7.9 in to 11.8 in (200 mm to 300 mm) and 19.7 in to 15.7 in (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 in (100 mm) depth between 23.6 in to 27.6 in (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 in and 19.7 in (400 and 500 mm) at location three, as well as 13 blows for location two to penetrate at a depth from 19.7 in to 23.6 in (500 to 600 mm). The site in Des Moines, indicates a maximum of 15 blows between 3.9 in (100 mm) depth of 11.8 in to 15.7in (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 in (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 2.0 ft (0.61 m) lift, Cedar Rapids about 3 ft (0.91 m) and Davenport about 4 ft (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.

		Nuclear Gauge		GeoGauge	
City / Sample	**Number of testing locations**	**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.

		DCP		Clegg Hammer	
City / Sample	**Number of testing locations**	**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 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 in (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 in (100 mm), with a CBR value of approximately 15% and Cedar Rapids with maximum of 18 blows per 3.9 in (100 mm), with approximately 43%. Davenport had a maximum of 13 blows per 3.9 in (100 mm) indicating a CBR value of 30% and Des Moines, with a maximum blow count of 15 blows per 3.9 in (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 in (100 mm)). Material in Ames, indicated a 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 2.0 ft (0.61 m) cutback. Note in Figure 62, the amount of material disturbance resulting on the site, due to the pavement removal.

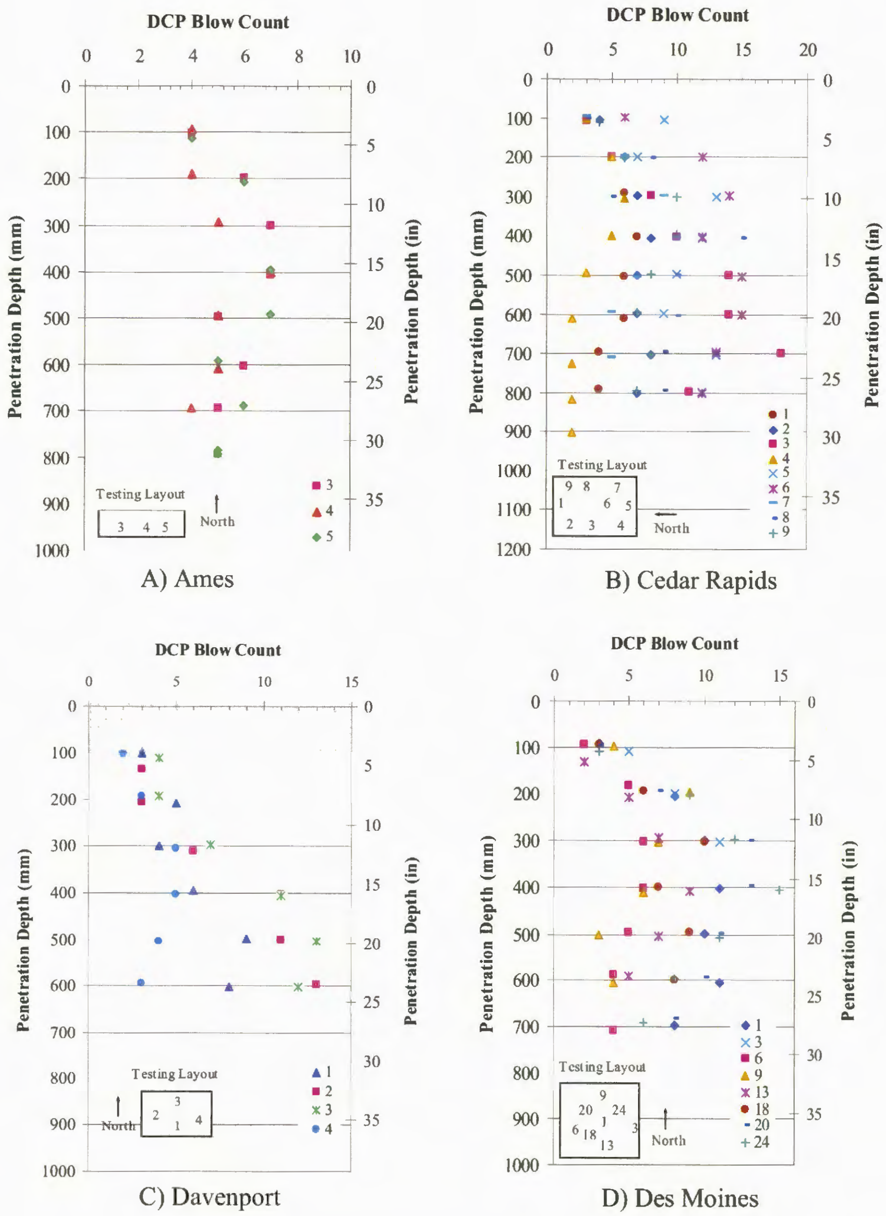


Figure 56. DCP Blow Count Profiles.

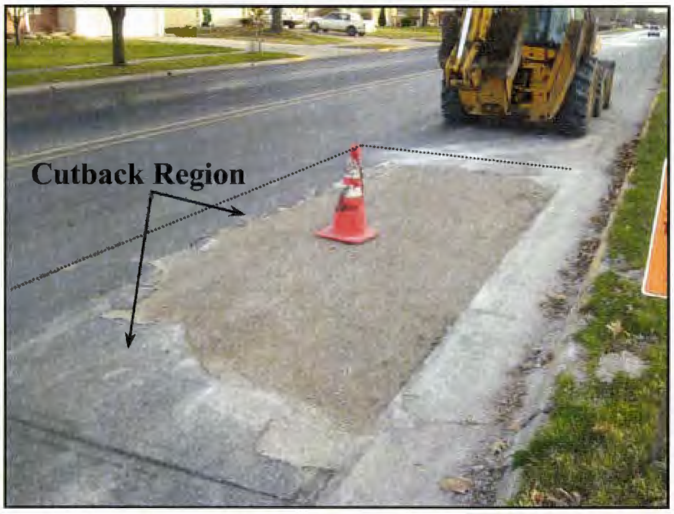


Figure 57. Site in Ames two weeks after construction.

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. Location 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 in/blow (13.3 mm/blow) compared to and average of 0.59 in/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 in/blow (18.3 mm/blow) to 1.61 in/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 above are listed in Tables 16 and Table 17.

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 in/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.

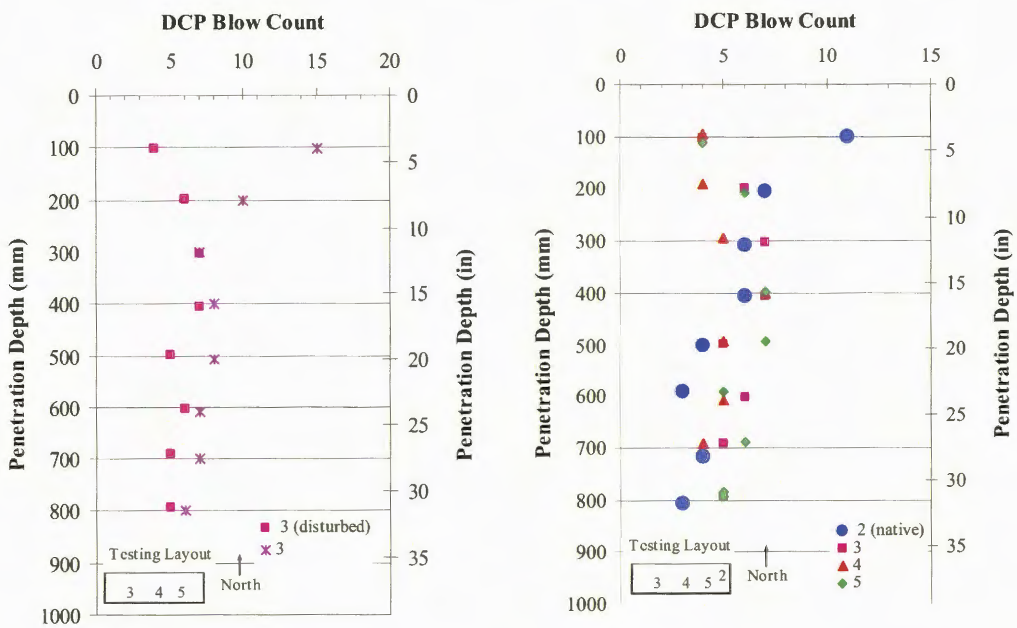
The DCP blow counts were again compared with respect to a 3.9 in (10 cm) penetration depth. Figure 60A indicates the disturbed material had a lower number of blows need to penetrate 3.9 in (100 mm), near the top 7.9 in (200 mm) of the trench. The disturbed material indicated a maximum blow count of seven to penetrate between 11.7 in to 15.7 in (300 mm and 400 mm). The undisturbed material indicated a maximum blow count of eight to penetrate a depth of 11.7 in to 15.7 in (300 mm to 400 mm). Figure 60B shows the DCP profile of the imported material and native material. The native material was stiffer at the top 7.9 in (200 mm) of the trench. From 11.7 in (300 mm) and deeper, the imported material showed a greater number of blow counts per 3.9 in (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.

Table 16. Ames: Nuclear Gauge data comparison.

City / Sample	Number of testing locations	Nuclear Gauge Moisture Content	Nuclear Gauge 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 Penetration Index	DCP CBR	Clegg Hammer CIV	Clegg Hammer 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



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 eight 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 17.0 ft and 2.0 ft (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 17.0 feet (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 (2.0 ft to 3.0 ft (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 lb (5443 kg) load. The 9,000 lb (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 lb (1361 kg) loads induced by cars) result in a lower deflection when compared to greater loads simulating loads, such as 9,000 lb (4082 kg), induced by trucks.

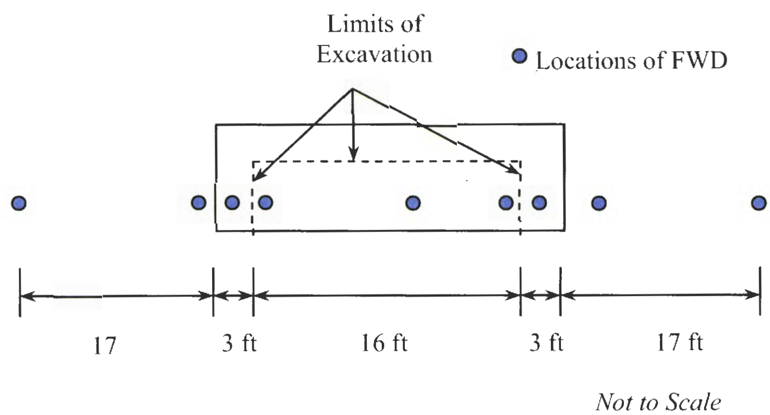


Figure 61. Ames FWD Layout.

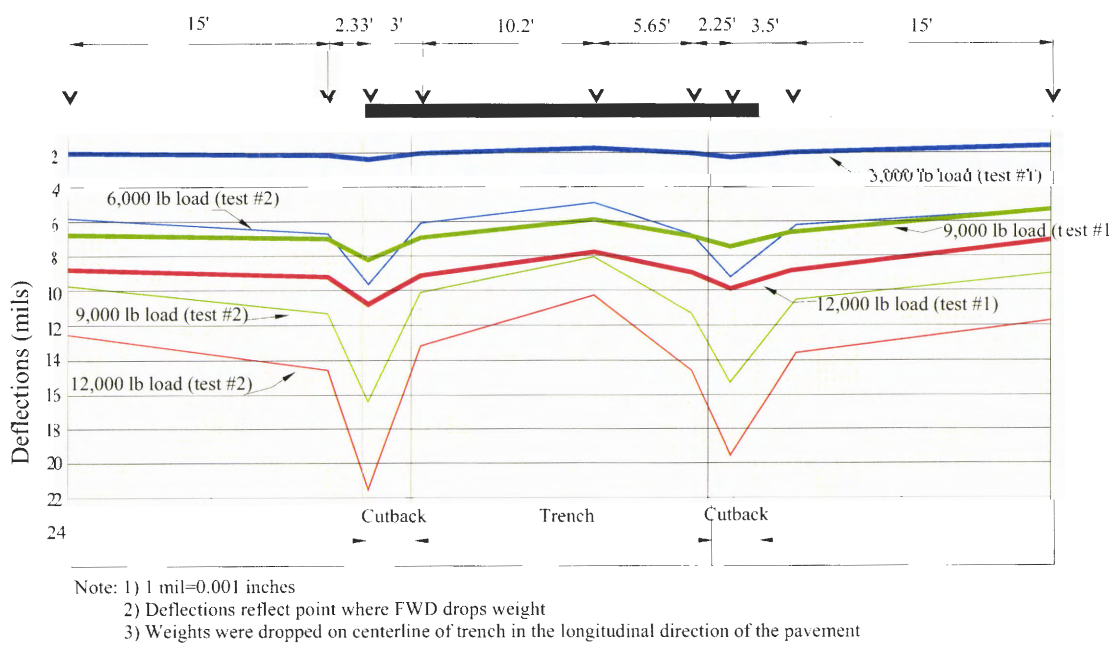


Figure 62. Ames FWD response profile.

Cedar Rapids: Miami Drive & Sherman Avenue

The FWD was tested in a composite material with 6.0 in (15.2 cm) of concrete and a 2.0 in (5.1 cm) of 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 and then again April 20, 2005 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 lb (4082 kg). Again the seasonal effect of the ground thawing increased the deflections observed in the data.

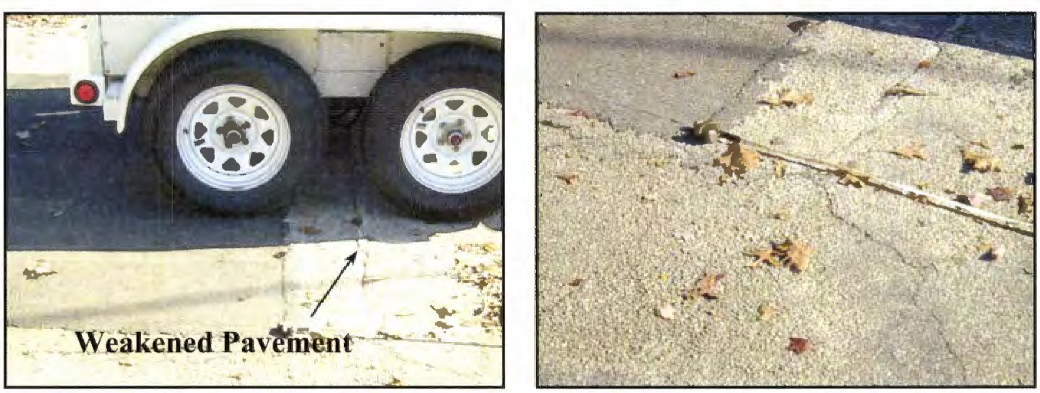


Figure 63. Cedar Rapids pavement distress.

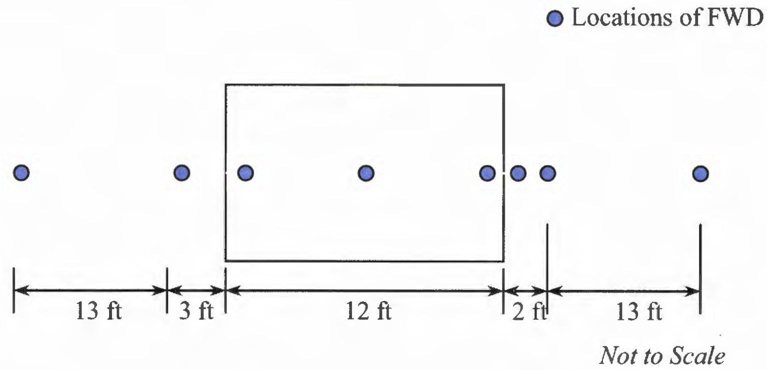


Figure 64. Cedar Rapids FWD Layout.

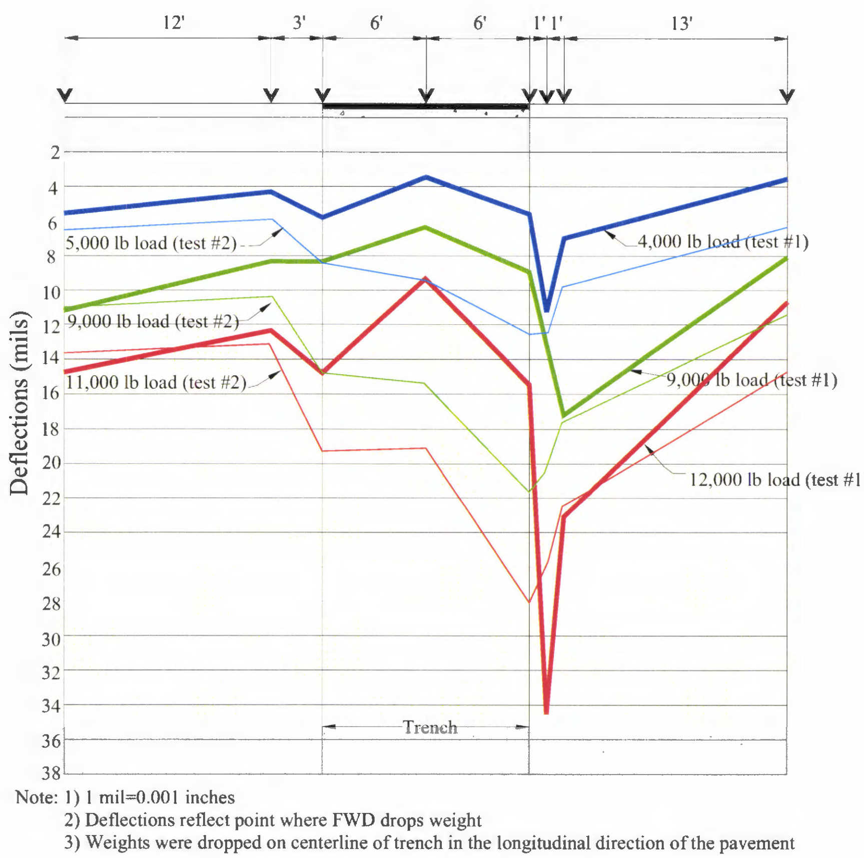


Figure 65. Cedar Rapids FWD response profile.

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

The FWD was tested on eight 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.

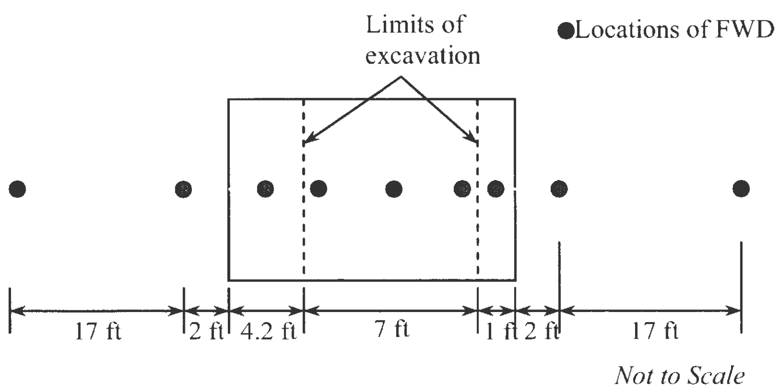
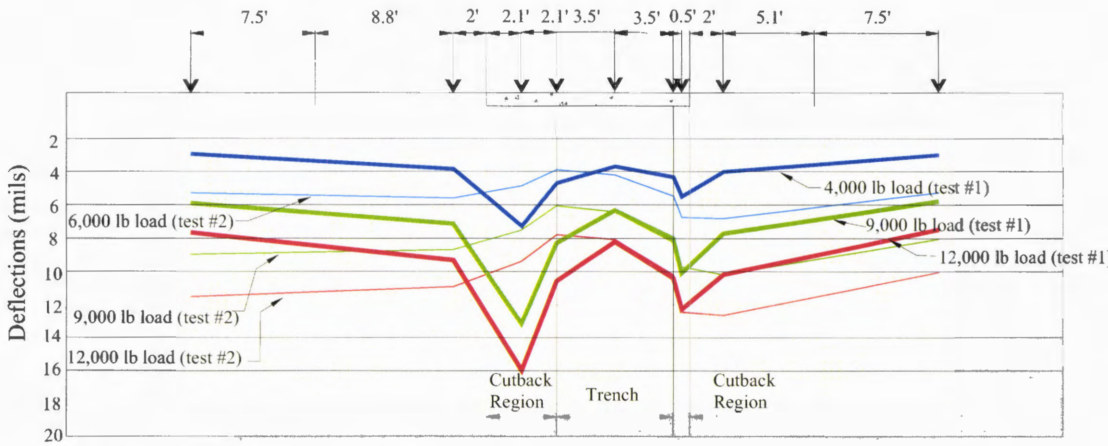


Figure 66. Des Moines FWD Layout.

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.



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 67. Des Moines FWD response profile.

Summary of Observations from Field Testing

- An average dry density value for Ames was 115.6 lb/ft^3, Cedar Rapids was 122.9 lb/ft^3, Davenport was 127.0 lb/ft^3, and Des Moines was 105.9 lb/ft^3.
- Aggregate materials such as that used on Cedar Rapids, provided higher relative density values, compared to the manmade sand in Des Moines.
- Mean CBR values using the DCP correlation were: Ames 8.5%, Cedar Rapids 13.3%, Davenport 9.2%, and Des Moines 12.5%.
- Mean CBR values using the Clegg Impact Values were: Ames 6.7%, Cedar Rapids 12.9%, Davenport 13.9%, and Des Moines 8.6%.
- 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.
- DCP results using 10 cm/blow results indicate a stiffness reduction at approximately 1.5 ft below the surface.
- Waiting two weeks after construction of a trench, made little difference in the strength of the trench near the surface since material was disturbed and no further compaction was used.
- 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 cannot be made at this time because of a need to continue monitoring restoration performance for an additional two years.

LABORATORY INVESTIGATION

Laboratory tests were conducted on various types of backfill material. These tests include: particle size distribution curves with a 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 minimal amount of material available.

Granular Collapse Test

The granular collapse test was conducted to determine the collapse potential of a granular material. This test was completed using a clear plexi-glass 8.0 in (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 3.0 ft (0.91 m). The material height in the cylinder ranged from 6.0 in to 12.0 in (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 by:

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

where: ΔH=initial height-final height

H_i=initial height

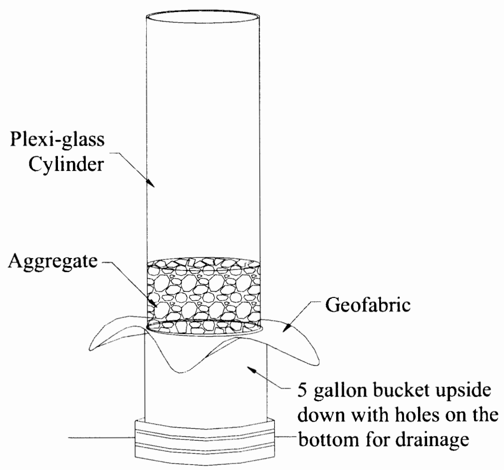


Figure 68. Granular Material Collapse Potential Apparatus.

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 18. Table 19 shows the gradation of limestone screenings and the SUDAS specification for granular material that is suitable as bedding and backfill material.

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 ¾ minus, and Davenport samples have percent passing sieve No. 200 greater than the percent allowed by Iowa DOT Gradation No.32, indicating a high fine content. Coefficient of uniformities were calculated for each material and shown in Table 20. The results indicate that all of the materials are well graded, with an exception of the SUDAS specification.

Table 18. 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 19. 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	-

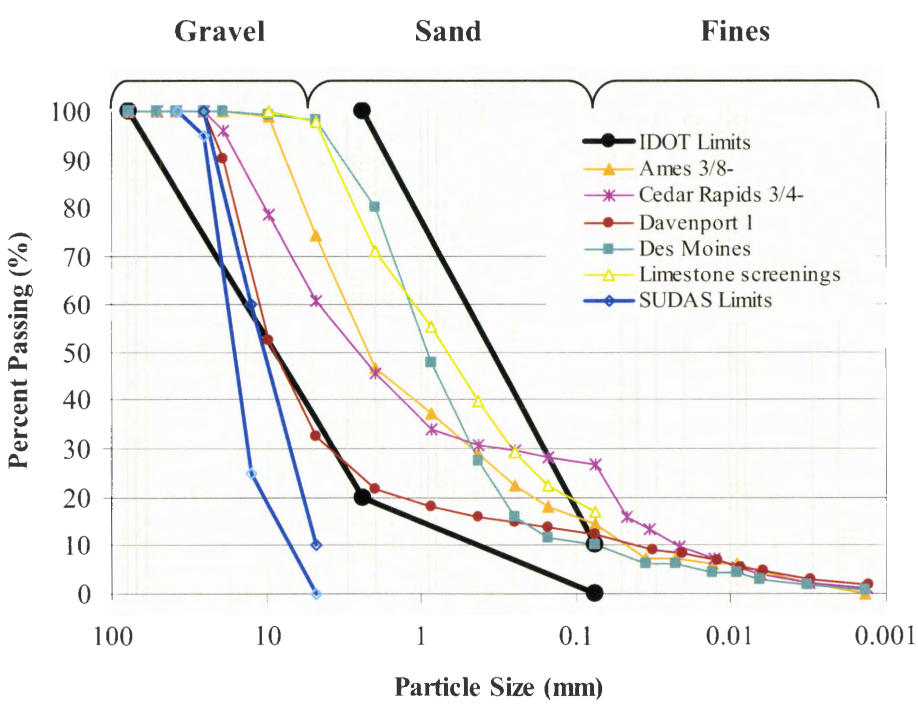


Figure 69. City Gradation Plot.

Table 20. 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 21 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 ¾ 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 one being the most desirable and 14 being the least desirable. Des Moines manmade sand (SW) was ranked a two where frost heave is possible. Ames 3/8 minus was ranked ten and Cedar Rapids ¾ minus (SC) was ranked a six 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 21. Laboratory Results of Imported Material.

City / Sample	Soil Classification				Specific Gravity	Minimum & Maximum Density	
Units	AASHTO		USCS			γ_{Max}	Bulking Water Content
						(lb/ft³)	*(%)*
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 clay 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 / manmade	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 mentioned 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 can be seen 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.

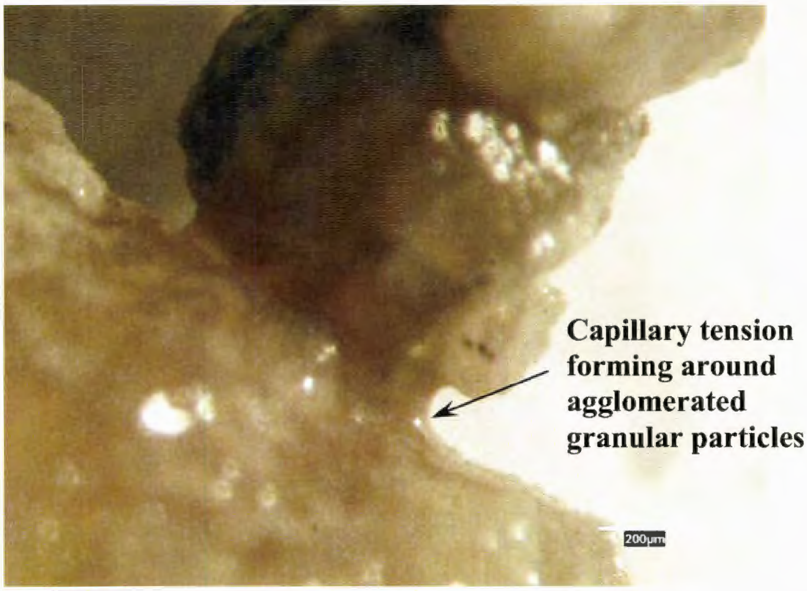


Figure 70. Microscopic view of capillary tension.

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

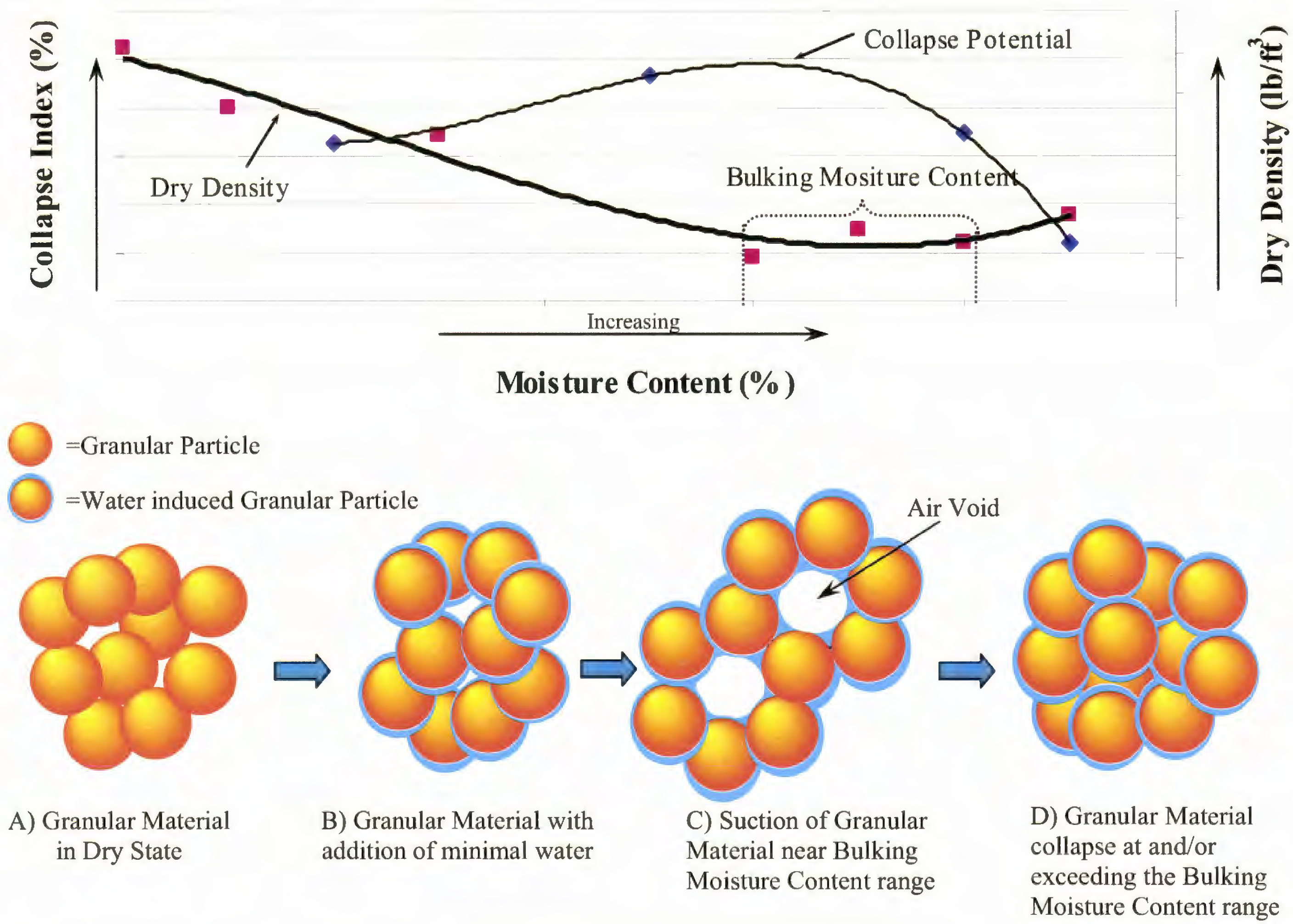


Figure 71. Bulking Moisture Schematic.

compacted dry density of 140.0 pcf and 90.0 pcf (22.0 kN/m^3 and 14.1 kN/m^3) respectively, with a density difference of approximately 50.0 pcf (7.9 kN/m^3) for this material.

The Cedar Rapids ¾ 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 130.0 pcf and 85.0 pcf (20.4 kN/m^3 and 13.3 kN/m^3) respectively, with a density difference of approximately 45.0 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 140.0 pcf and 85.0 pcf (22.0 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 9.0% (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 135.0 pcf and 80.0 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).

Table 22, shows a summary of moisture contents from the field, bulking moisture contents and maximum densities obtained in the laboratory.

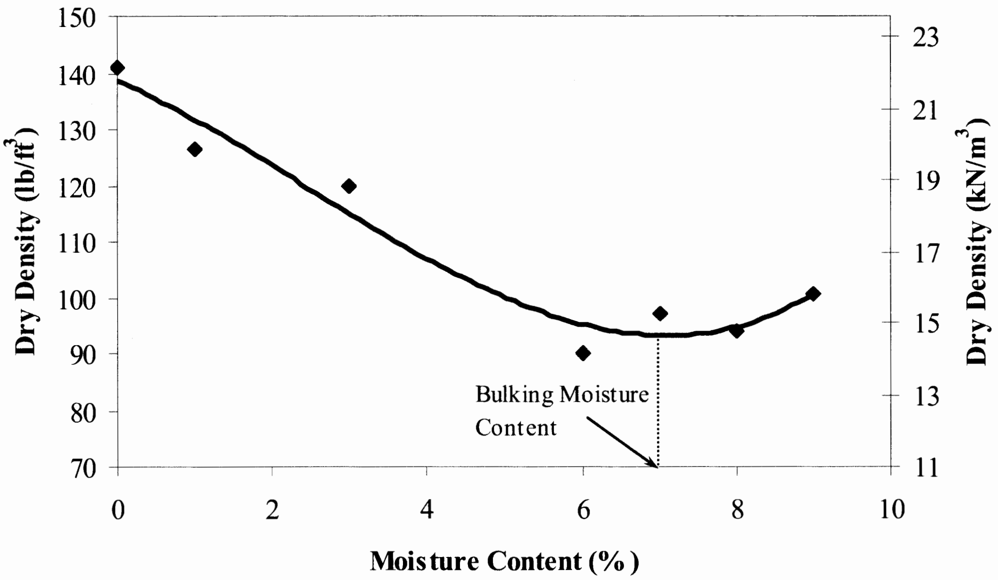


Figure 72. Ames 3/8 minus Maximum Density Test Results.

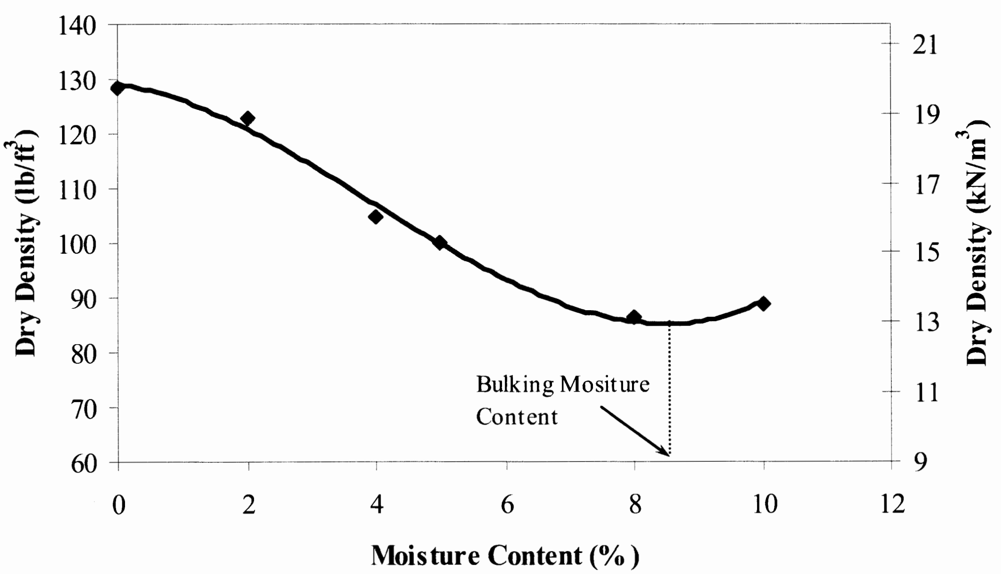


Figure 73. Cedar Rapids ¾ minus Maximum Density Test Results.

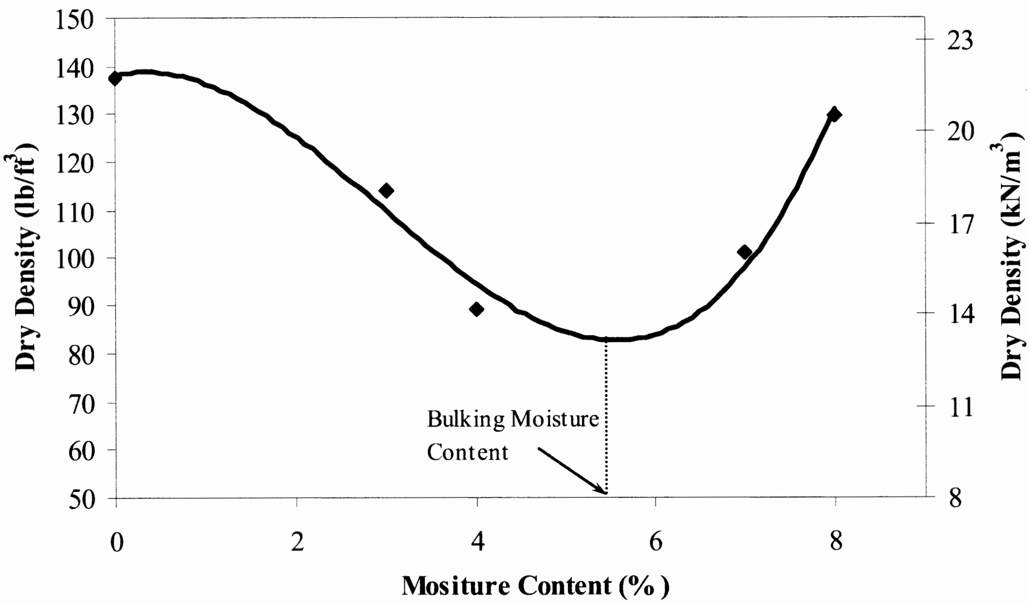


Figure 74. Davenport Maximum Density Test Results.

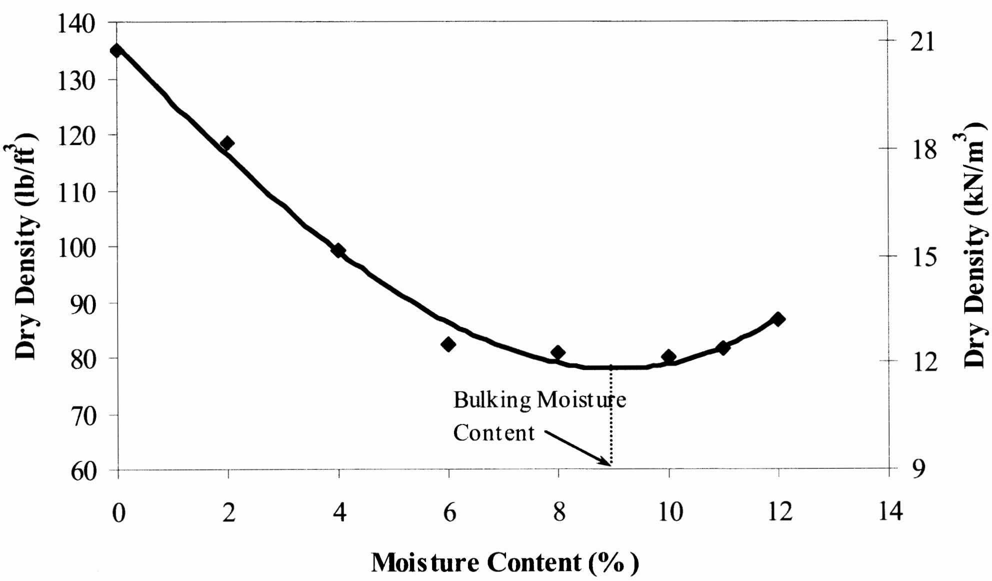


Figure 75. Des Moines Maximum Density Test Results.

Table 22. 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 9.0%, Cedar Rapids 8.5%, and Des Moines 24.0%. The SUDAS specification was tested in addition to the samples currently used in the field (see Figures 79). The SUDAS specification indicated a very low collapse potential of approximately 0.4%. Limestone screening had the highest collapse potential of approximately 35.0% (see Figure 80). Therefore, the collapse potential obtained from the granular collapse test varied from about 35.0% to less than 0.5%, depending on the material tested. Table 25 shows a summary of the engineering properties of each material.

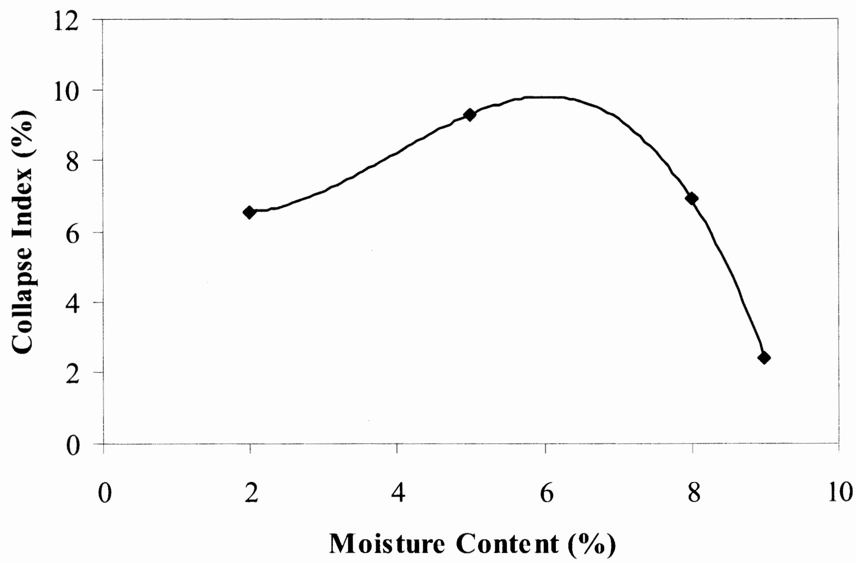


Figure 76. Ames 3/8 minus collapse index profile.

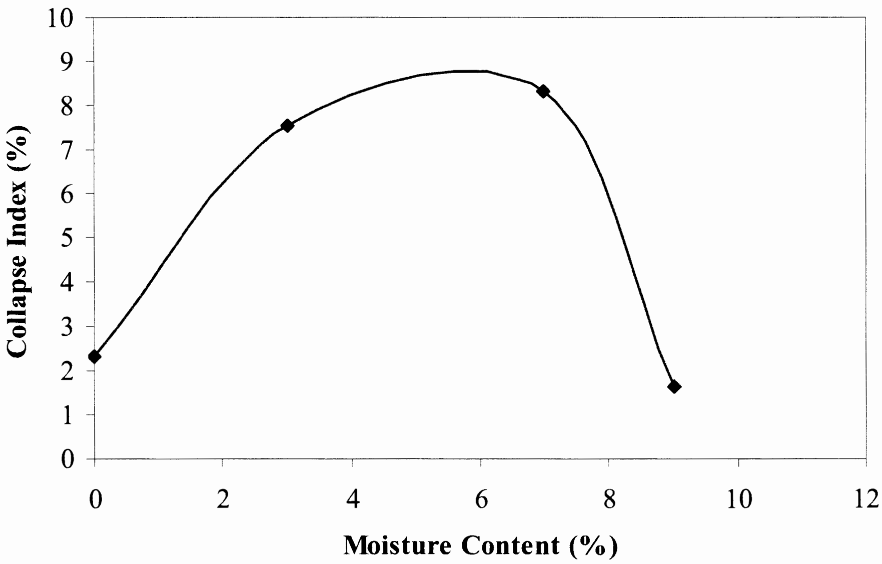


Figure 77. Cedar Rapids ¾ minus collapse index profile.

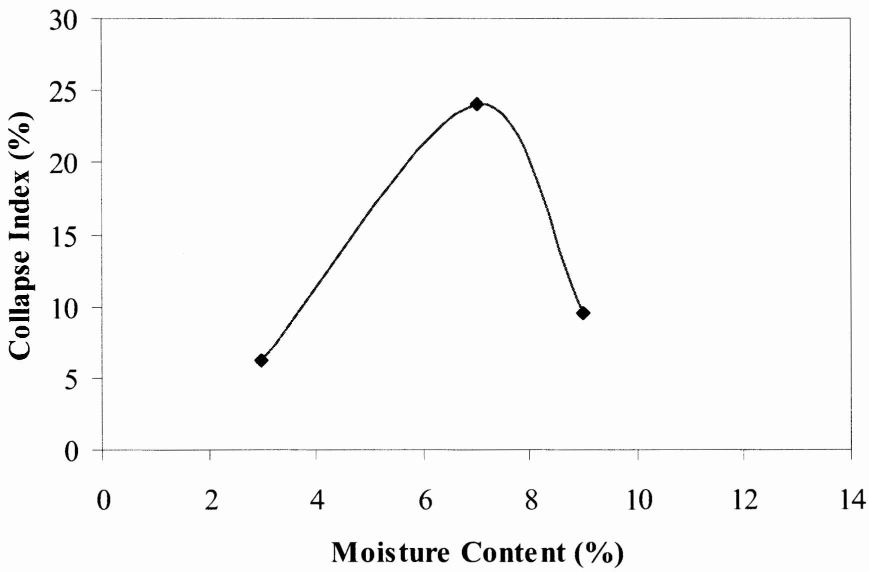


Figure 78. Des Moines manufactured sand collapse index profile.

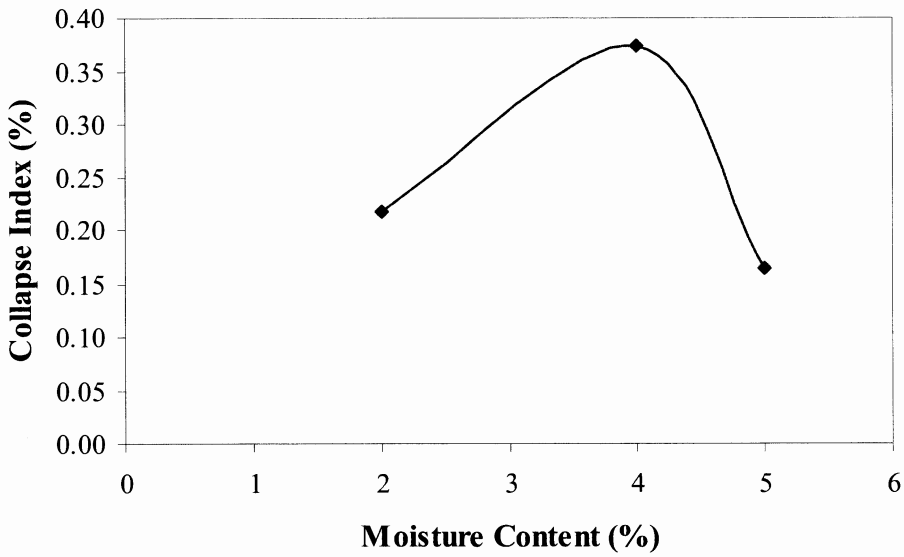


Figure 79. SUDAS collapse index profile.

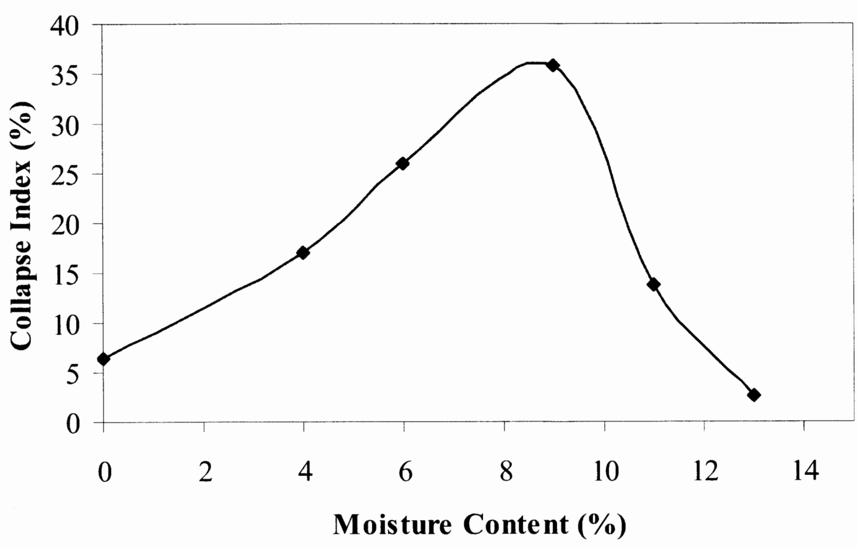


Figure 80. Limestone screenings collapse test.

Table 23. 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 were 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. Figure 81, 82 and 83 indicate that the bulking moisture range may be exceeded if the material is at about 40% saturation.

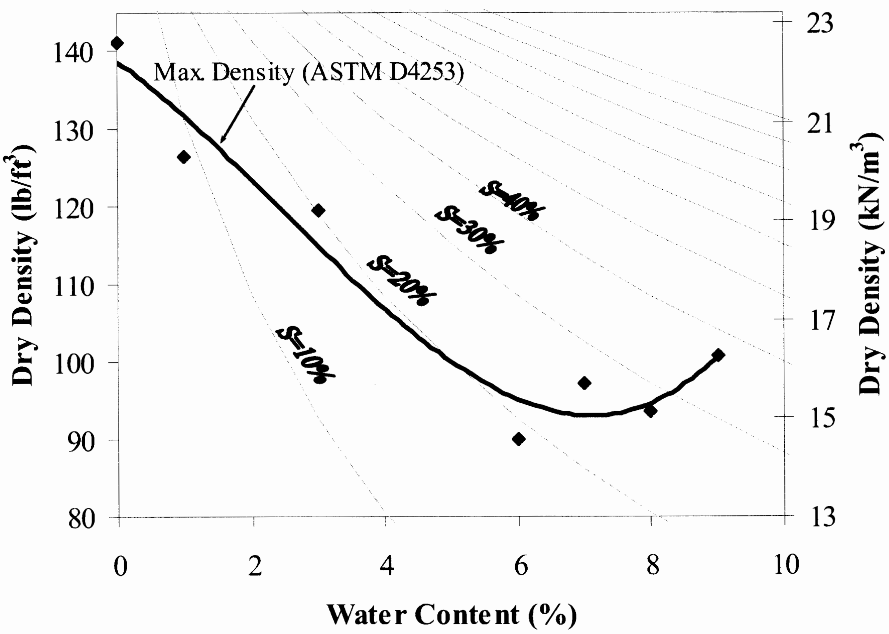


Figure 81. Degree of Saturation, Ames, IA.

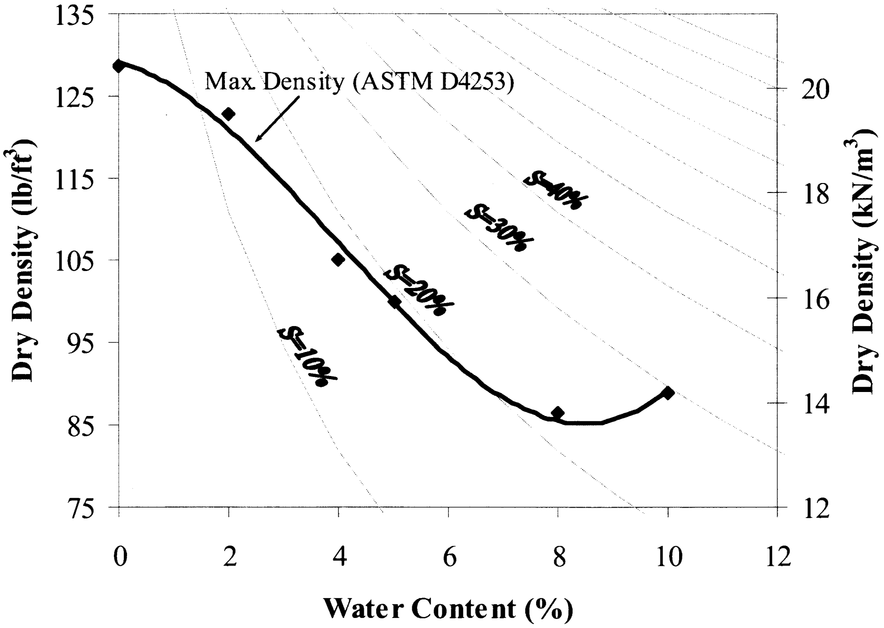


Figure 82. Degree of Saturation, Cedar Rapids, IA.

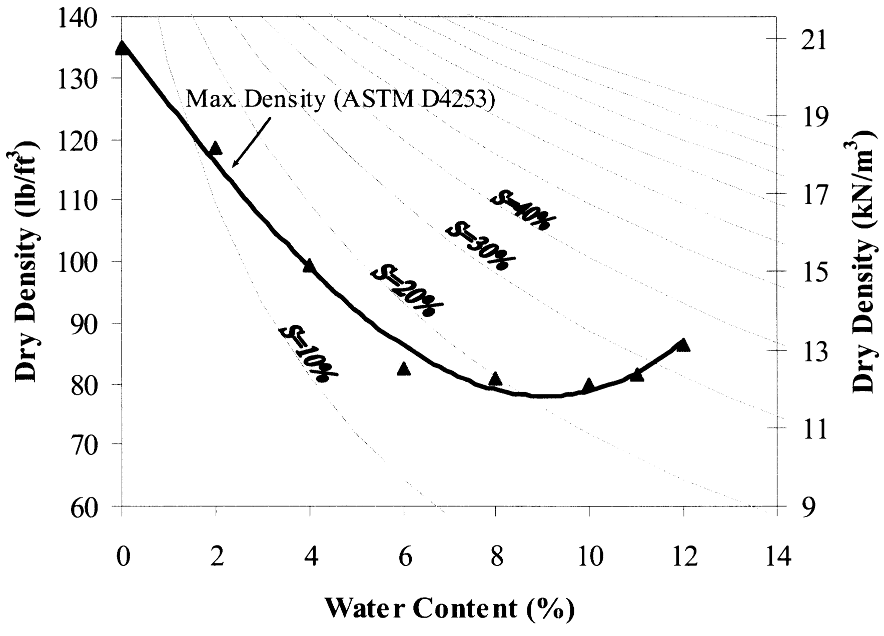


Figure 83. Degree of Saturation, Des Moines, IA.

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 6.0%. Cedar Rapids ¾ minus material indicates a majority of the values higher than density values achieved with the standard Proctor between 5.0% and 7.0%, 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 5.0% and 11.0%.

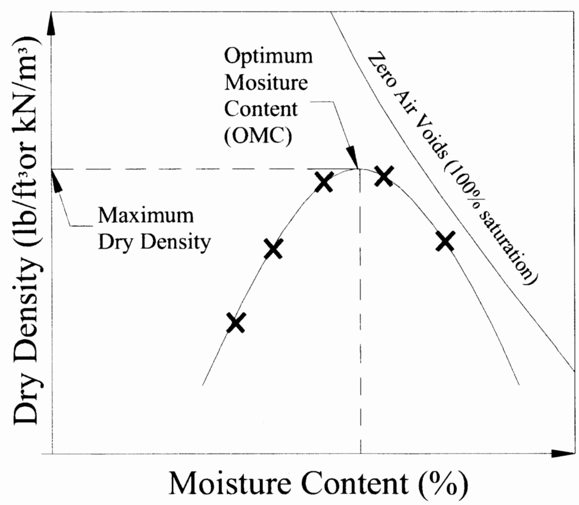


Figure 84. Typical Standard Proctor Curve (Modified from Departments of the Army and the Air Force).

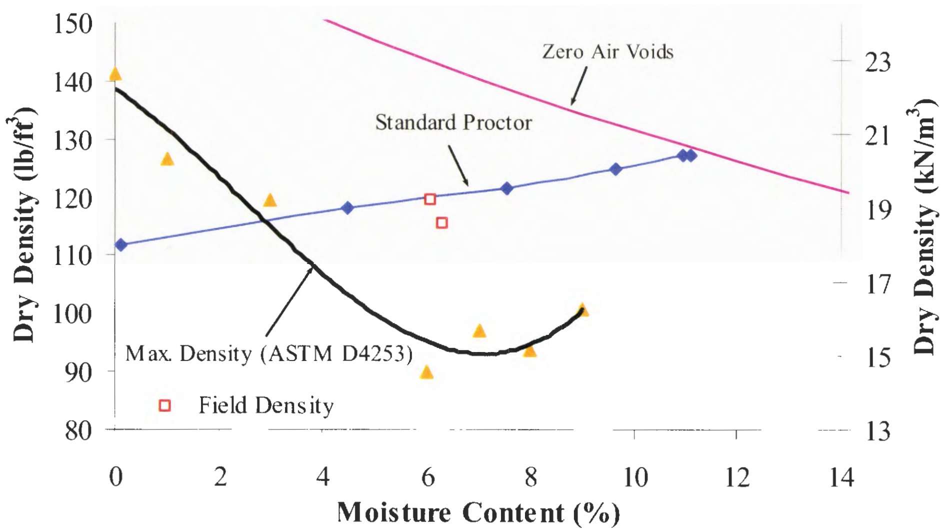


Figure 85. Ames: Standard Proctor vs. Maximum Density.

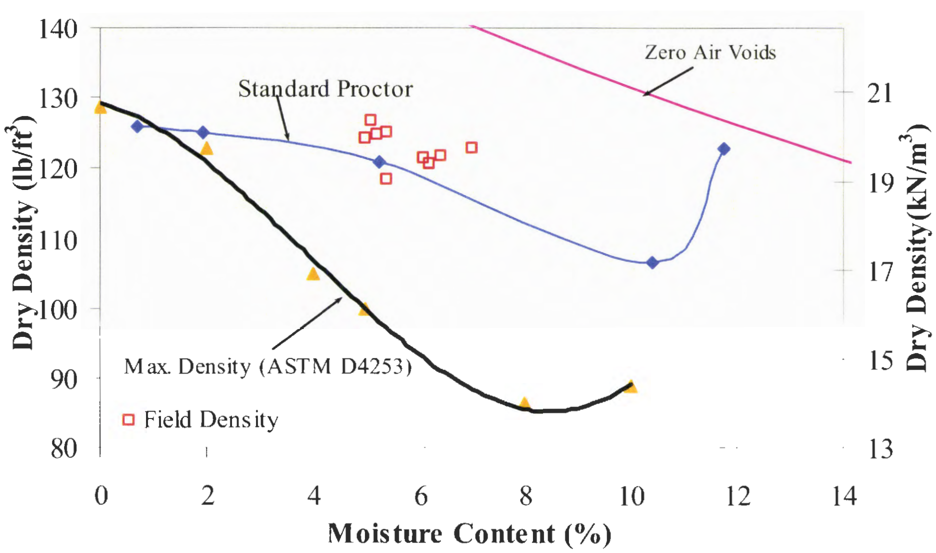


Figure 86. Cedar Rapids: Standard Proctor vs. Maximum Density

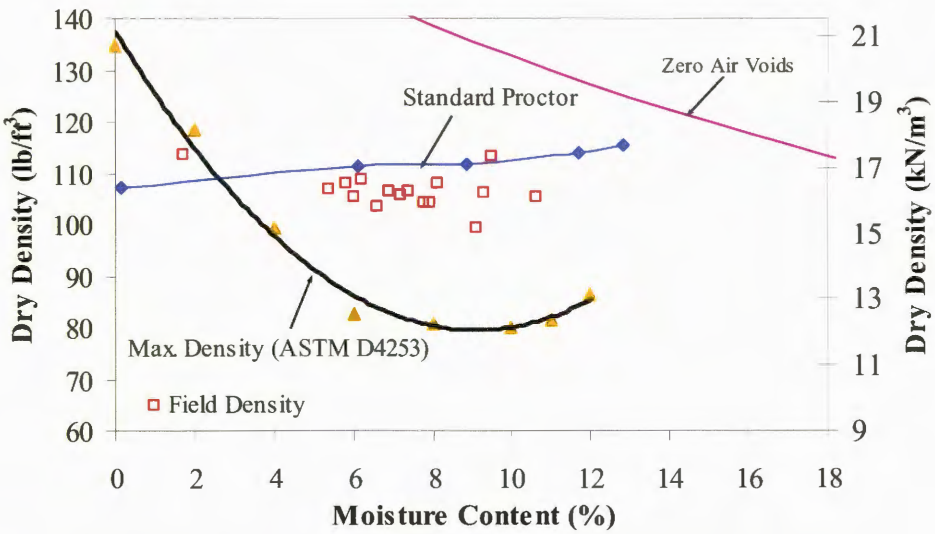


Figure 87. Des Moines: Standard Proctor vs. Maximum Density.

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 a oven dry state (i.e. zero percent moisture content) according to ASTM D 4253 and D 4254. Relative density (R.D.) is defined as:

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

Where:

γ_{field}= Dry density in the field (pcf or kN/m^3)

γ_{max}= Maximum dry density in the laboratory (pcf or kN/m^3)

γ_{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 percent 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 Figures 88, 89, 90, and 91 show, 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.

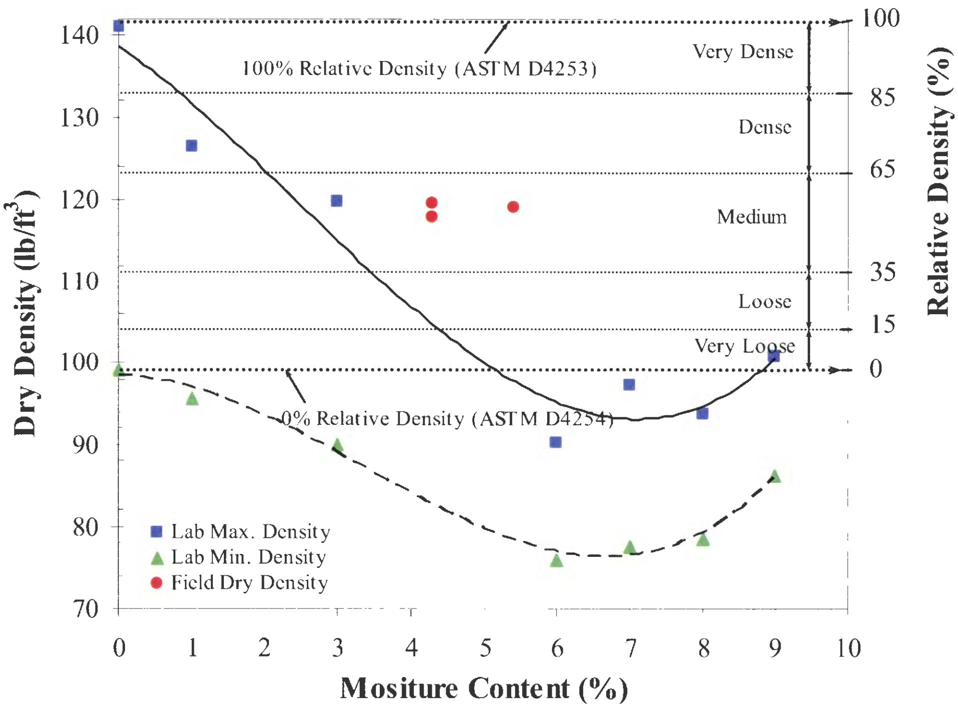


Figure 88. Ames 3/8 minus Relative Density Plot.

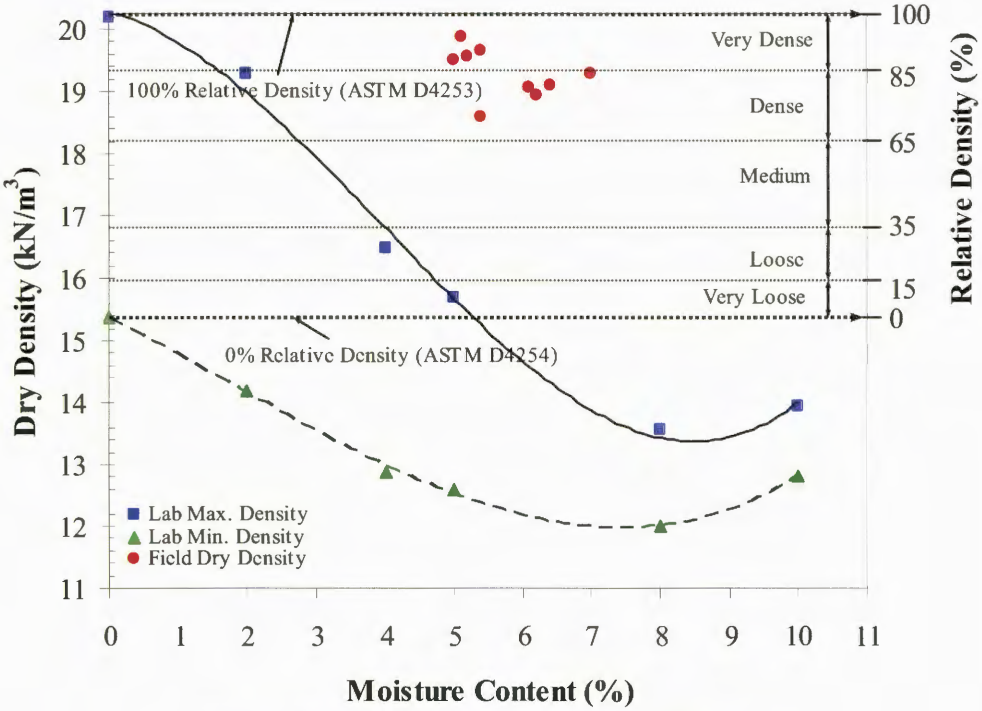


Figure 89. Cedar Rapids Relative Density Plot.

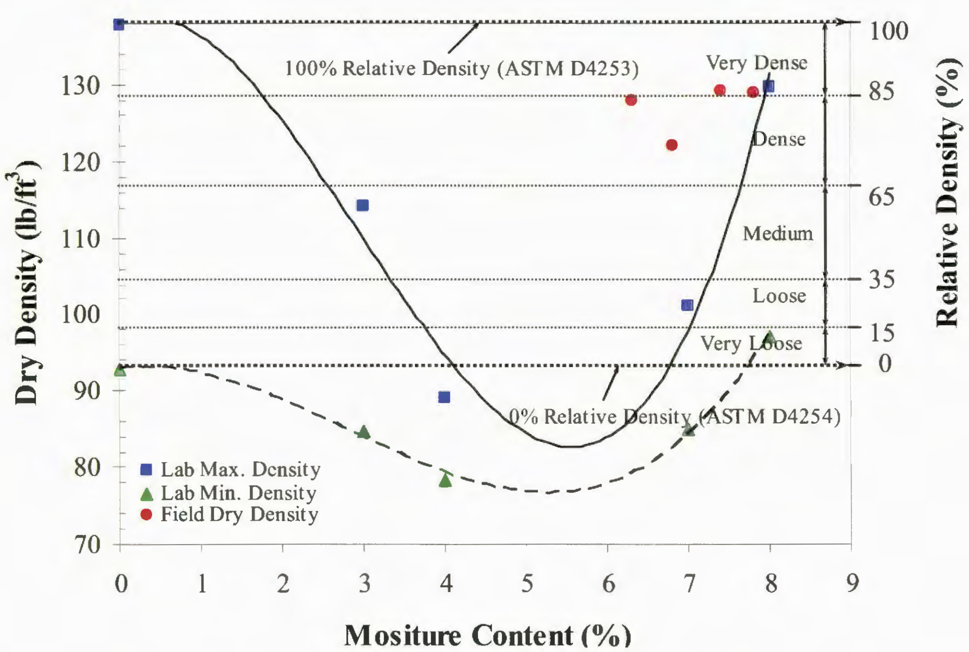


Figure 90. Davenport Relative Density Plot.

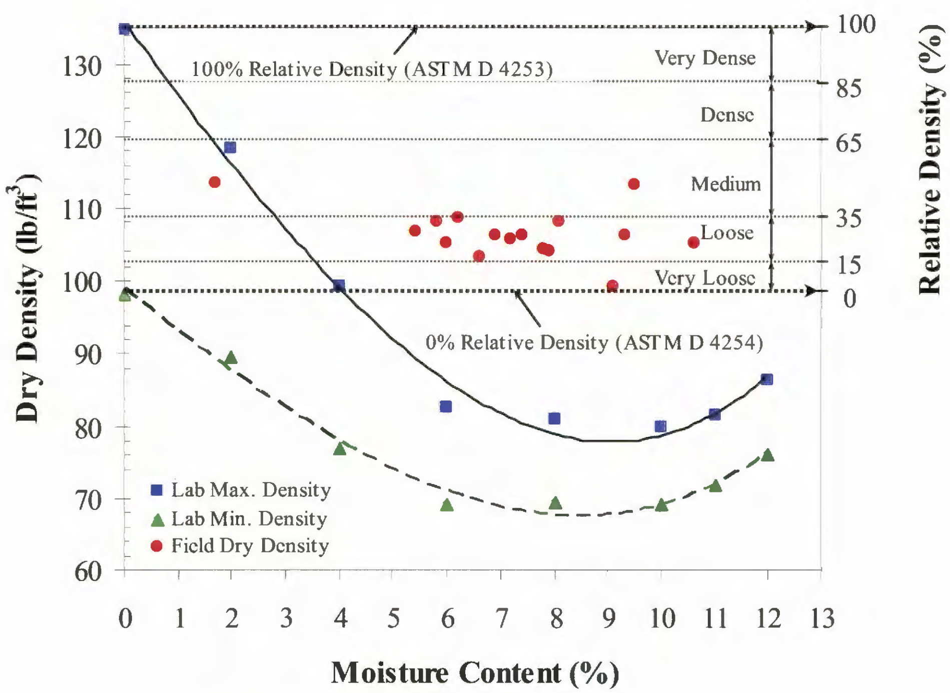


Figure 91. Des Moines Relative Density Plot.

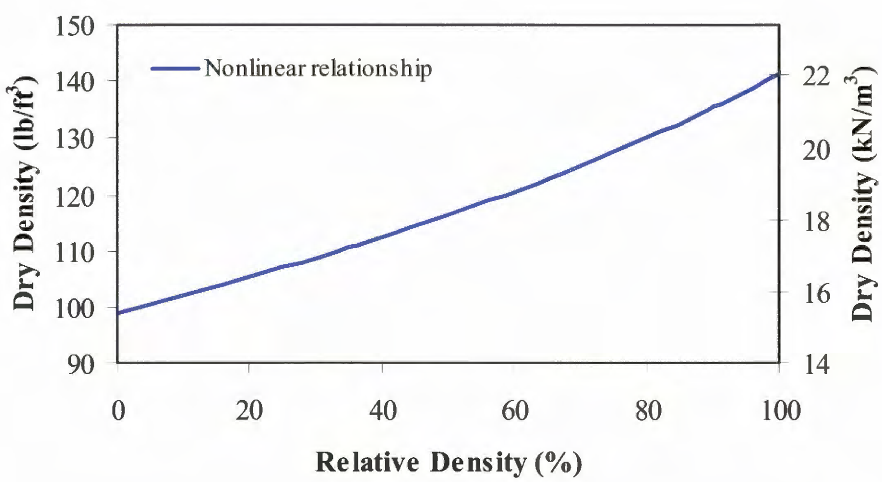


Figure 92. Relative Density-Dry Density nonlinear relationship.

Settlement Potential Evaluation

A derivation using the phase diagram provides an equation that can be used to predict potential future settlement based on the relative density a material achieves when placed in a trench. This derivation is located in Appendix C. Time was not a variable in the determination of Figure 93. This plot illustrates the potential settlement a material could encounter when compacted to a specific relative density. Material from Ames and Davenport were calculated based on a 10.0 ft (3.0 m) trench observed in the field and Cedar Rapids and Des Moines were calculated based on an 8.0 ft (2.4 m) trench. The utility cut constructed in Cedar Rapids, using ¾ minus imported backfill material, has the least potential for future settlement when considering factors such as field density, maximum density, and a given excavation height.

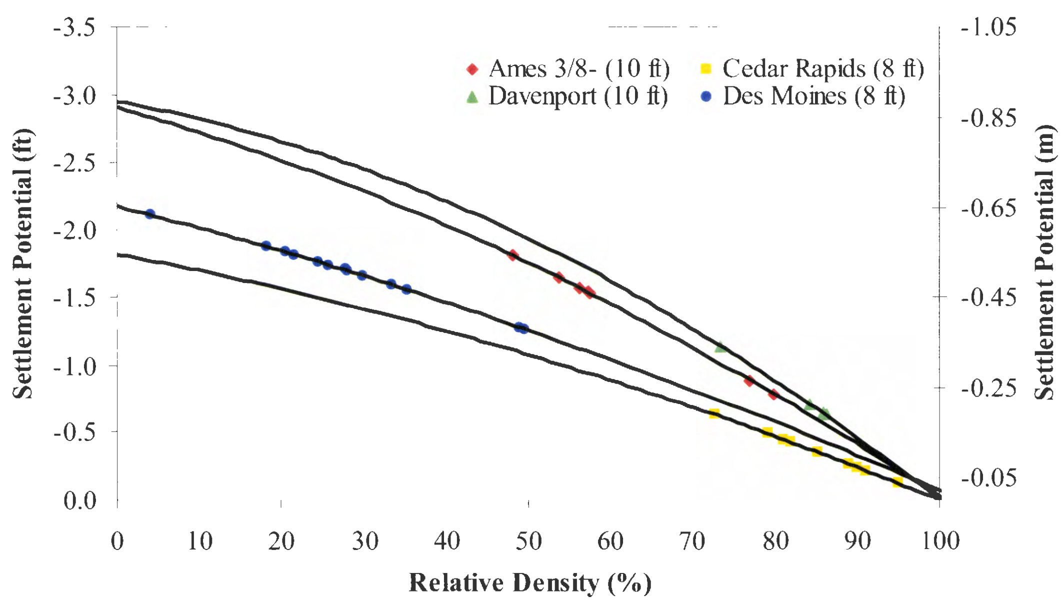


Figure 93. Estimated future settlement.

Summary of Observations from Laboratory Testing

- Materials used in the field are at or exceeding the IDOT gradation for material passing the No. 200 sieve.
- Materials obtained from all cities, according to AASHTO, are classified as excellent to good in use as a subgrade material.
- Material obtained from the field in Ames, Cedar Rapids, Davenport and Des Moines were at or near the bulking moisture content.
- Conclusions observed from the trial materials from the collapse tests indicate a large collapse potential of 36 percent in loosely placed limestone screenings, whereas the SUDAS specification had a low collapse potential of 0.4 percent as a result of a coarser material.
- The use of materials near the bulking moisture content, such as Ames and Des Moines, may require additional water once material is placed in the trench to reduce settlement potential induced by moisture collapse.
- Saturating a material to 40% exceeds the bulking moisture content range.
- Relative density should be stated as a level of compaction in jurisdictions, rather than standard Proctor.
- Using the relative density design charts as a guide, correct compaction requirements for a given material can be determined.
- Coarse material, like Cedar Rapids and Davenport imported material, obtained relative densities of dense to very dense without a significant amount of compaction.
- Based on the relative density data, material from Des Moines indicated that it was placed loose into the excavation.

DISCUSSION AND CONCLUSIONS

This section discusses practices and results mentioned above in the following sections: 1) Utility Cut Survey Results, 2) Utility Cut Construction Techniques, 3) Field Investigation, and 4) Laboratory Investigation.

Survey Results

The survey results discussed above indicate opinions based on city personnel from seven cities in Iowa. Discussions in this area include topics such as: permit fees, extent of the problem, construction requirements, and quality control.

Many cities throughout Iowa require the use of permits before an excavation be made, however a fee is not assessed in all cases. Ames indicated that no fee is required however a permit must be obtained and yet other cities are charging fees exceeding $200. This is just an example of the variance in permit requirements that jurisdiction's are implementing throughout Iowa. Ultimately, when fees are not assessed, maintenance and repairs of poorly performing utility cuts are paid for by tax payers. By implementing and updating permit fees with the growth of the economy, future restorations will be 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 considering the life of an undisturbed pavement can last approximately ten times this amount of time. This 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.

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 a sand backfill and now is the only city in Iowa that consistently uses a flowable fill for utility cuts.

Many cities require a compaction of 95% standard Proctor, with Davenport indicating 90% standard Proctor up to 18.0 in (45.7 cm) below finished grade and 95% above that area.

Quality control 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 excavation observed consisted of a cut and excavation. The utility was then repaired and backfilled with imported material. Lifts generally were 2.0 ft to 4.0 ft (0.6 m to 1.2 m) thick, 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 device. Pavement surfacing was placed anywhere from immediately after the utility cut was constructed, up to two weeks. Des Moines was the only city observed that plated the unpaved utility cut until surfacing was available.

The common practice of placing 2.0 ft to 4.0 ft (0.6 m to 1.2 m) 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.

It was often observed that saturated native materials were added to the excavation in an attempt to clean the utility cut area. This is an undesirable practice in two respects. First, a saturated material is very weak and has low compaction properties. Secondly, once a native material is disturbed, achieving its original density is extremely difficult, specifically in clay type native materials due to its disturbed structure. The use of native materials in an excavation also requires monitoring of the moisture content for optimum performance.

Another common practice was incorporating refuse, such as pop bottles, paper, and leaves into the trench. With the addition of these materials, the potential for the formation of voids increases, therefore leading to potential settlement in the future.

Ultimately, sites where construction was observed from the excavation to the backfilled trench, no quality control devices were used to assure compaction requirements were met. Furthermore, there was no moisture control of the imported backfill material placed into the trench.

Field Results

The field results were obtained from the following destructive and non-destructive devices: Nuclear Density Gauge, Dynamic Cone Penetrometer, Clegg Hammer, GeoGauge, and the Falling Weight Deflectometer.

The Nuclear Density Gauge generated dry density and moisture contents for each import backfill material. These values could then be used with laboratory data to calculate relative density values. This data indicated a dense to very dense compacted material in both Davenport and Cedar Rapids. The material obtained from Ames resulted in a medium dense material and Des Moines material was placed in a loose to very loose state. These results indicate an aggregate in the field can reach a high relative density with minimal effort. However backfill material consisting of sands, require more effort in achieving a dense relative compaction.

The DCP generated CBR data, indicated results fairly consistent with each other throughout the excavated area. With plotting the number of blows it took to reach a 3.9 in (10 cm), distance a trend showed that at approximately 1.5 ft (0.5 m) from the surface of the test, material stiffness decreased. This just reiterates that importance of lifts being placed in small lift thicknesses. Many specifications reported 12.0 in (30.5 in) or less. DCP data obtained from native material indicated a decreasing stiffness trend with the number of blows needed for a 3.9 in (10 cm) depth. This may have been a result of the loss in lateral support during the excavation. Another trend that was observed using the DCP was that a higher CBR value was obtained near the center of the excavated area in most cases and the lower CBR values were generally 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, as well as possibly decreasing the impact that heavy equipment, such as backhoes, on the zone of influence during compaction, as seen in Cedar Rapids.

CBR results from the DCP and the Clegg Impact Hammer, were not consistence with one anther. For the most part the Clegg Hammer had a higher range of calculated CBR values. However, in the case of utility trenches, it was observed in many cases that excess material was brushed or shoveled back into the trench without further compaction. Having loose material near the surface generates lower CBR values since the device measured impact values of the material from the surface. When comparing the results to values found in the literature review, a minimum Clegg hammer value of 18 is needed for proper compaction beneath a pavement surface, however when comparing all data obtained in the field, this was not reached at any site.

The FWD data indicated a decrease in deflections with time. This most likely is a result of the changes in seasons, since the first tests were conducted in the fall of 2004 and the second tests in the spring of 2005, therefore further monitoring should be conducted for one to two more years to seasonal deflections. The results also indicate the responses as a result of different loadings on the pavements due to a variety of vehicles. The FWD profiles show the large amount of deflection produced in the zone of influence. The Cedar Rapids data dramatically illustrates the damage that heavy equipment can have on the zone of influence on an open excavation. These profiles also indicate a minimal deflection near the center of the trench. The concrete pavement produced a lower amount of 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 then used with the field data to further classify the material properties.

The sieve analysis indicated a majority of the materials used in the field consisted of a well-graded material. The SUDAS specification was the only material defined as poorly graded.

The maximum density tests indicate a large density difference from a maximum density at zero percent to the bulking moisture content. Des Moines and Davenport have a density difference of 55.0 pcf (8.6 kN/m^3), Ames a difference of 50.0 pcf (7.9 kN/m^3) and Cedar Rapids with 45.0 pcf (7.1 kN/m^3). This is a significant change in density and shows the importance of having moisture control in the field.

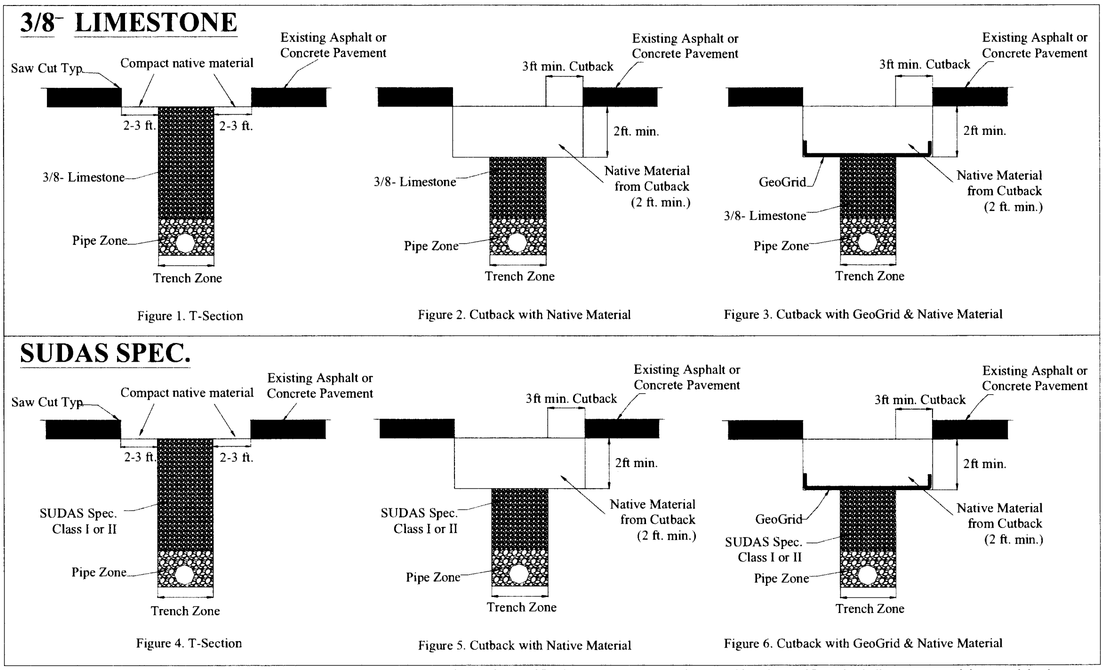
The collapse potential test indicated collapse potential ranging form 1% to 35%. The limestone screenings showed the largest collapse potential value with approximately 35%, indicating that this material would not be suitable for use as a backfill material. Fines may have an impact on settlement as well. Having too many fines in a gradation may actually promote settlement as a result of the fines migrating to the native material, therefore forming potential areas of settlement. The results of the collapse test indicated that fine sandy materials have a greater collapse potential, compared to aggregates. This is due to the bulking moisture content range that exists in granular material. Each material indicated a different bulking moisture content range, where this value should be exceeded in the field to minimize the effect of settlement due to moisture.

Standard Proctor and Relative Density tests were conducted on each imported material. Results for the Standard Proctor for Ames and Des Moines did not show a bulking region at low moisture contents, as did the relative density tests. This may be a result of the impact induced by the Standard Proctor for compaction, where the impact overcomes these capillary forces and therefore increasing the density of the material. Ultimately, compaction of granular materials should be specified according to relative density. Furthermore, the relative density test uses vibration to compact the material, similar to that used in the field.

The design charts generated, indicate a specified region of compaction for a material to obtain correct density. These charts can be used in the field if a quality control device, such as the nuclear density gauge were to be incorporated into the construction of the trench. With the nuclear gauge a dry density value can be obtained for a specific material and immediately determine the compaction state with respect to relative density.

Trial Trenches

After observations were made in the field and laboratory, six proposed trenches were designed with the goal of alleviating future settlement. The six trenches consisted of a T-section using material consisting of 3/8 minus and the material satisfying SUDAS specification. Three types of trenches were proposed with each material: 1) a T-section using a two to three feet excavation of pavement in the cutback region, 2) a 2.0 ft to 3.0 ft (0.61 m to 0.91 m) cutback and pavement removal, along with an excavation of 2.0 ft (0.61 m) deep into the native material, and 3) a trench constructed the same as (2) except a geosynthetic placed on the bottom of the excavated area adding strength and a bridge between the utility excavation. The cutback excavation incorporated into the last two trenches where placed in the cutback region two to three feet beneath the excavation for bridging purposes. A 2.0 ft to 3.0 ft (0.6 m to 0.9 m) cutback depth was excavated to compensate for the majority of settlement that was found to occur in backfill at 2.0 ft (0.6 m) beneath the pavement surface according to the Literature review. Cross-sections of these proposed trenches are illustrated in Figure 94.



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.

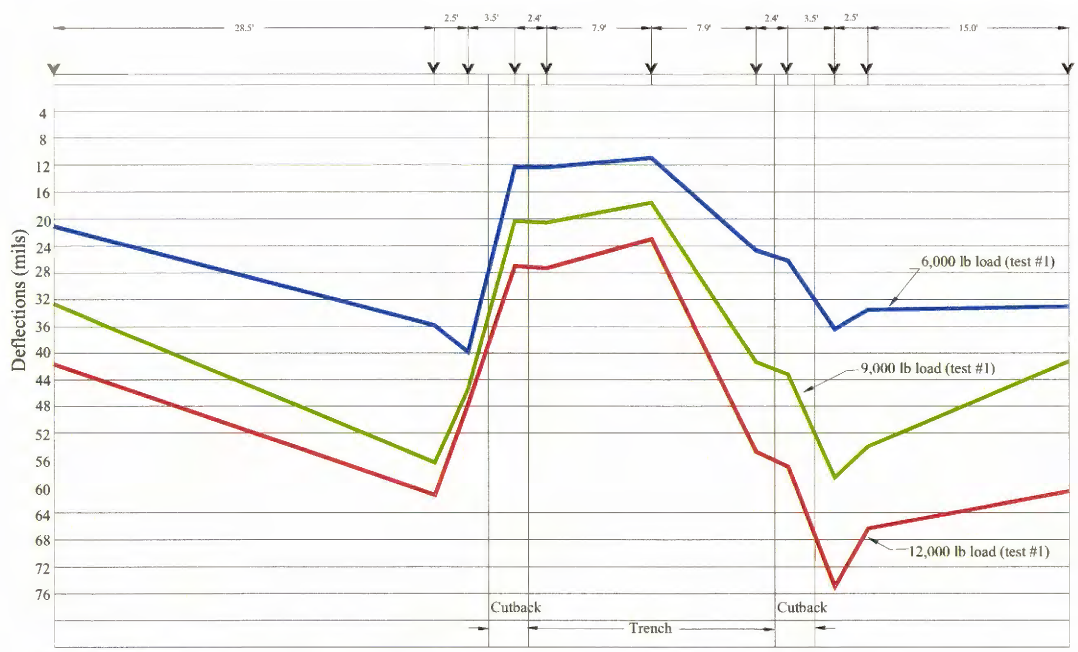
Figure 94. Proposed trenches in Ames, IA.

On June 16, 2005, a proposed utility cut restoration constructed in Ames was monitored with the FWD. The trench was 24.7 ft (7.3 m) long and 13.6 ft (4.1 m) wide. This utility cut consisted of a 3.0 ft (0.9 m) pavement cutback and a 2.0 ft (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 1.0 in (25.0 mm), a tensile strength at 2% strain of 280.0 lb/ft (4.1 kN/m), an ultimate tensile strength of 850.0 lb/ft (12.4 kN/m) and ultimate junction strength of 791.0 lb/ft (11.5 kN/m). The index properties in the cross-machine (transverse) direction include: aperture dimensions of 1.3 in (33.0 mm), tensile strength at 2% strain of 450.0 lb/ft (6.6 kN/m), ultimate tensile strength of 1300.0 lb/ft (19.0 kN/m), and ultimate junction strength of 1209.0 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 95 shows a picture of the geogrid placed inside the excavated area.

Figure 95. Geogrid being placed.

The FWD profiles can be seen in Figure 96, 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

2.0 ft (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 2.0 ft (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 96, illustrates this indistinctive zone of influence. Since the zone of influence is not distinct with this 2.0 ft (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 1.0 ft (0.3 m) vertical excavation of the native material in the cutback region, rather than a 2.0ft (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 96. 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.

From observations in the field it is recommended that material be placed in lifts less than 12.0 in (30.5 cm). Materials such as pop bottles, paper, or any objects other than the backfill material, should not be incorporated into the trench since they have the potential of creating voids and therefore increasing settlement potential. Saturated material should also be eliminated since the compaction properties of saturated materials are considerably weaker.

A majority of the imported backfill materials in the field were placed at or below the bulking moisture content range. It is recommended that watering trucks be on site to exceed this bulking region found for a given material, therefore alleviating settlement induced by moisture.

It is recommended that compaction be stated according to relative density, rather than standard Proctor, since material is compacted using vibration in the field. Material in the field should be compacted to 65% or above relative density to achieve a dense material. McCook (1996) has shown that relative density test values can be reasonably correlated to a 1 point field proctor test. Also, field compaction with a vibrating plate should be conducted after pavement removal, both in the excavated area and the cut back region.

Many cities require a cutback to be constructed in the field to compensate for the zone of influence, however, further monitoring needs to be conducted in Iowa to determine its performance. The proposed trench incorporating a 2.0 ft (0.6 m) excavation in the cutback region and geogrid showed a lower amount of deflection in the zone of influence and therefore should be monitored further to determine the performance of the trench with time.

Furthermore a seminar or informational session could 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 in utility construction should involve continued monitoring of the constructed trenches for approximately two years, further work with the DCP to determine its possible use as a quality control device in Iowa (i.e. the number of blows needed to reach a certain depth), and further studies with incorporating the use of geogrid and a 1.0 ft (0.3 m) excavation in the cutback region.

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APPENDIX A: CITY SURVEY RESULTS

Iowa Department of Transportation
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
482B Town Engr. Bldg.
CCEE Department
Iowa State University
Ames, IA 50011

OR

Mr. Dale Harrington, P.E.
CTRE
2901 South Loop Drive, Suite 3100
Ames, IA 50010

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 percent 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

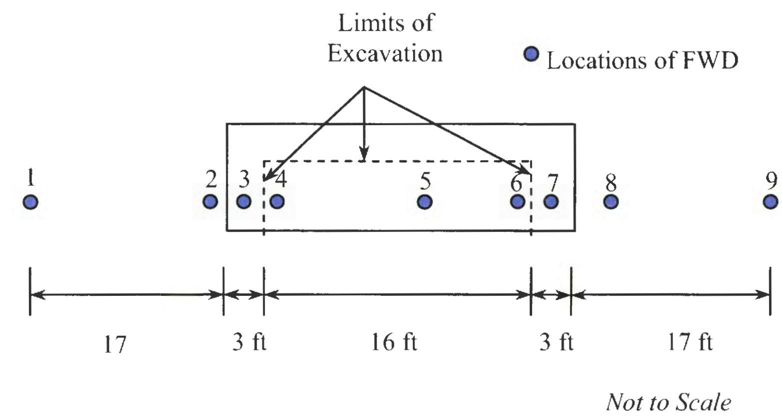


Figure 97. Ames 20th St. FWD Layout.

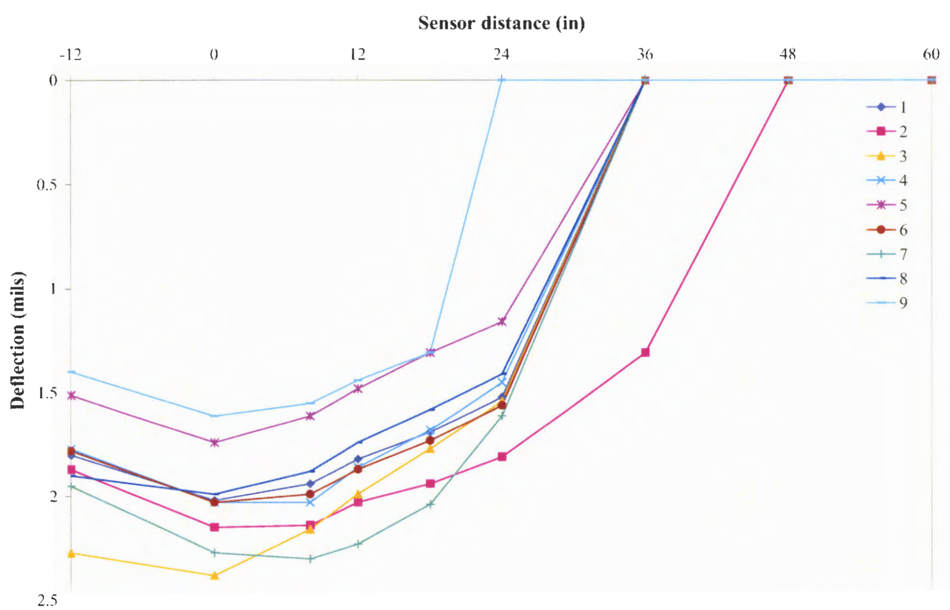


Figure 98. Ames test #1: 3000 lb. FWD raw data.

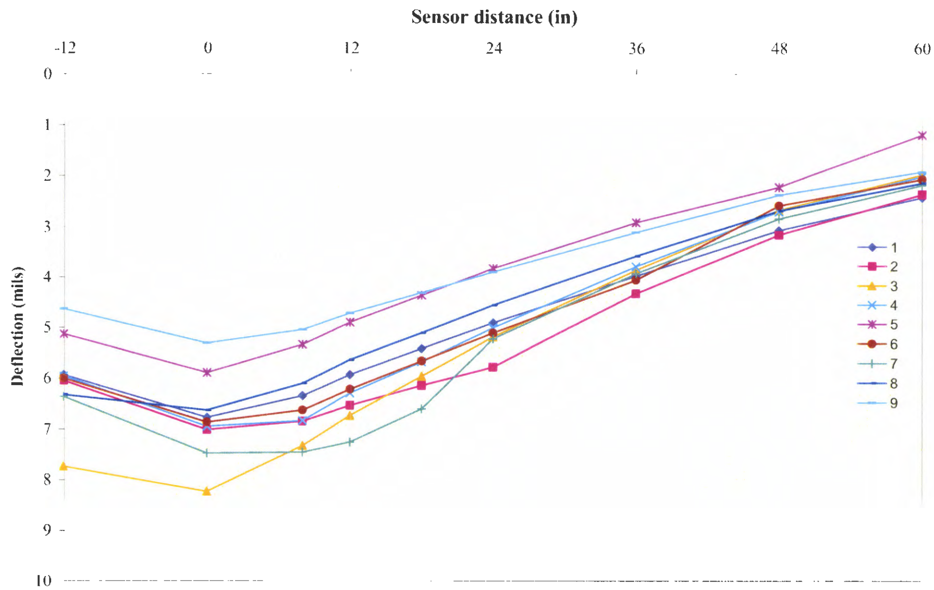


Figure 99. Ames test #1: 9000 lb. FWD raw data.

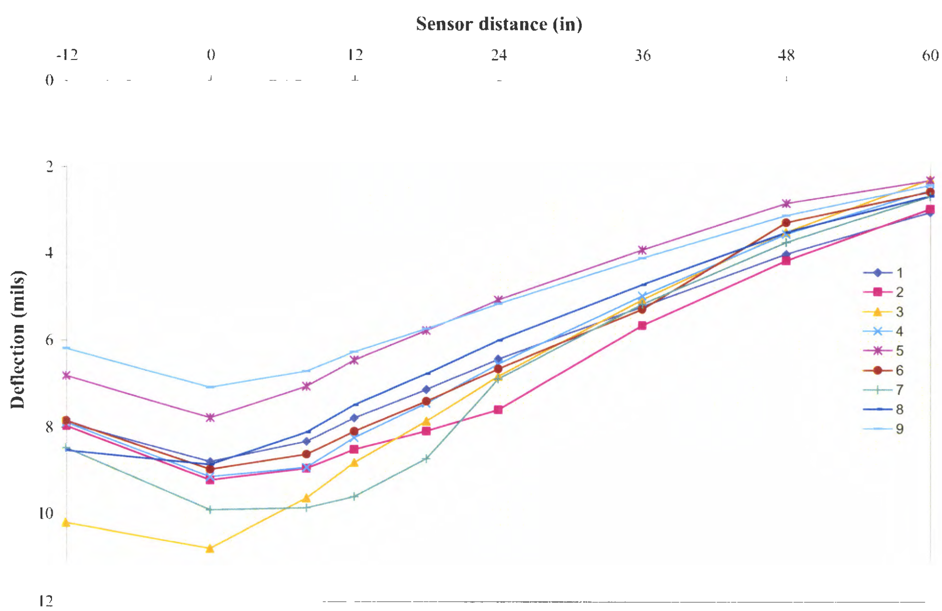


Figure 100. Ames test #1: 12000 lb. FWD raw data.

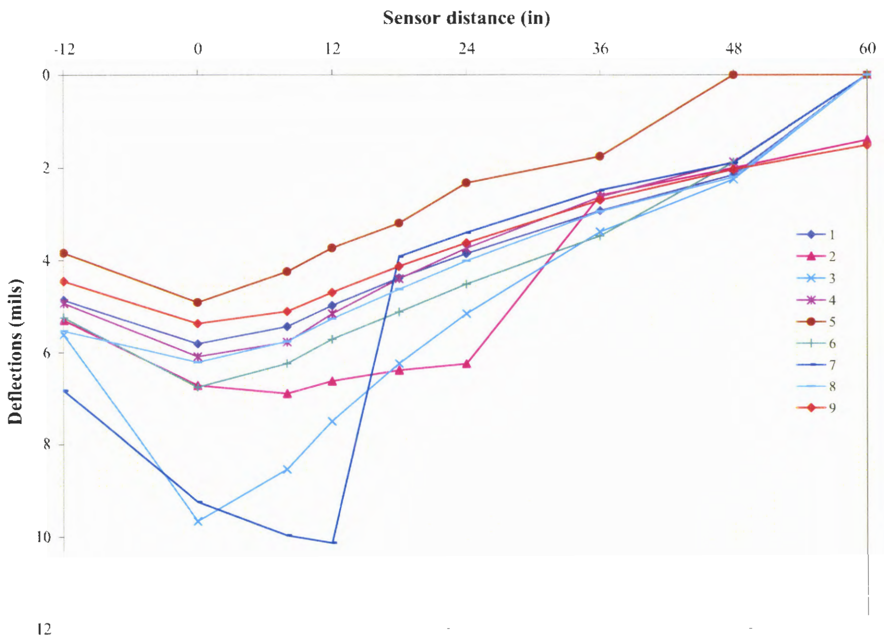


Figure 101. Ames test #2: 6000 lb. FWD raw data.

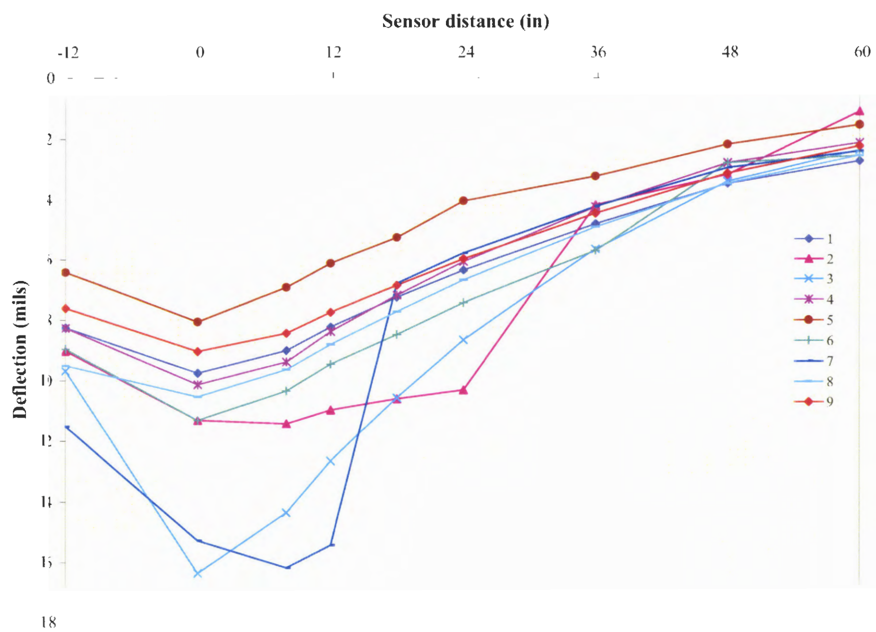


Figure 102. Ames test #2: 9000 lb. FWD raw data.

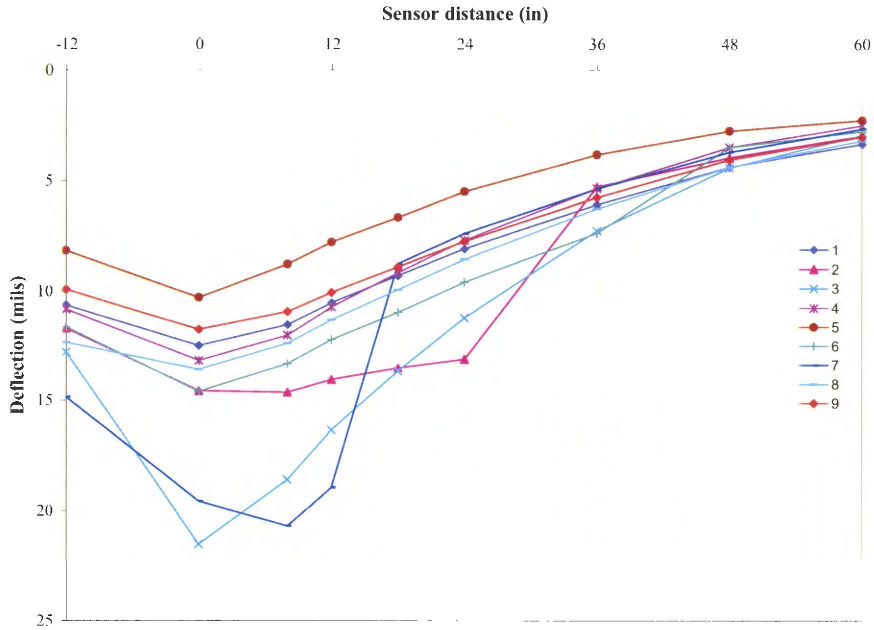


Figure 103. Ames test #2: 12000 lb. FWD raw data.

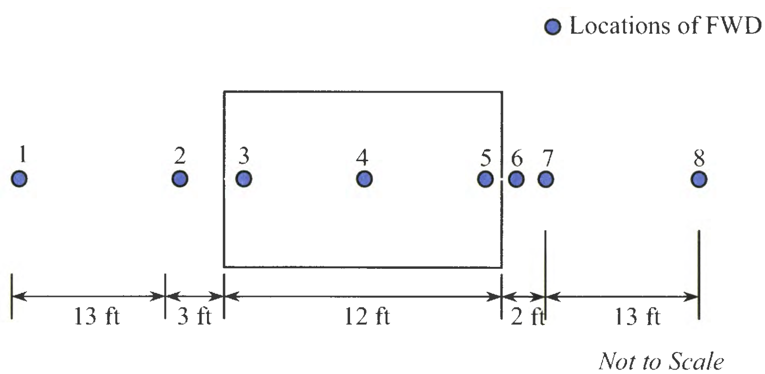


Figure 104. Cedar Rapids FWD layout.

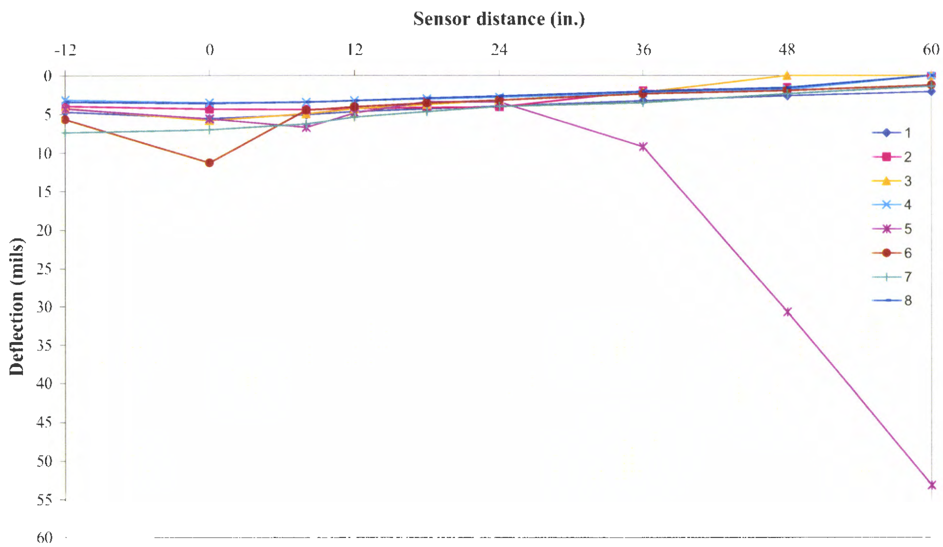


Figure 105. Cedar Rapids test #1: 4000 lb. FWD raw data.

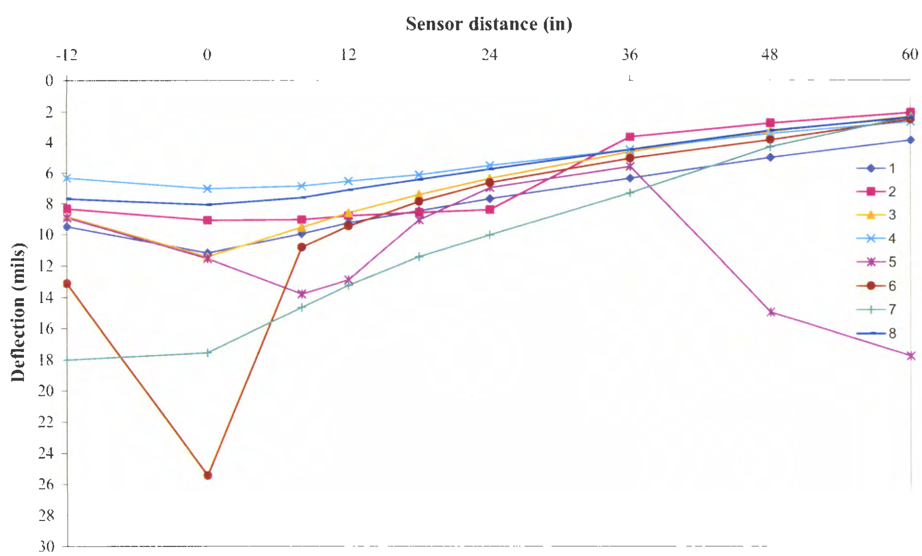


Figure 106. Cedar Rapids test #1: 9000 lb. FWD raw data.

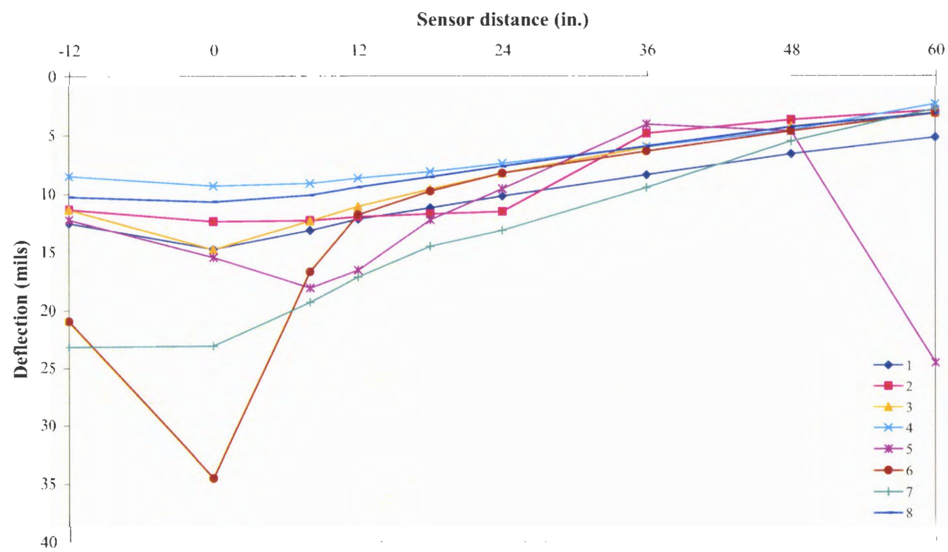


Figure 107. Cedar Rapids test #1: 12000 lb. FWD raw data.

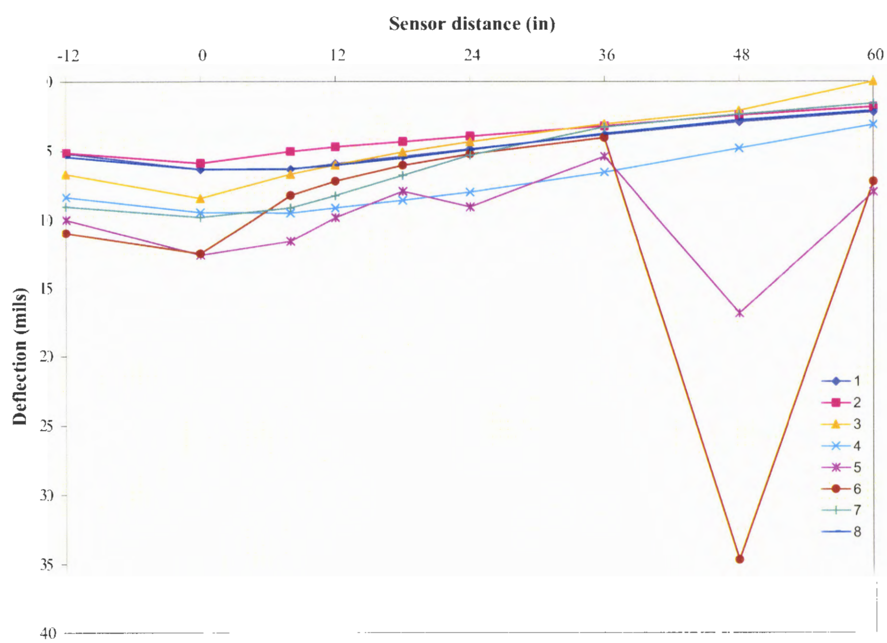


Figure 108. Cedar Rapids test #2: 5000 lb. FWD raw data.

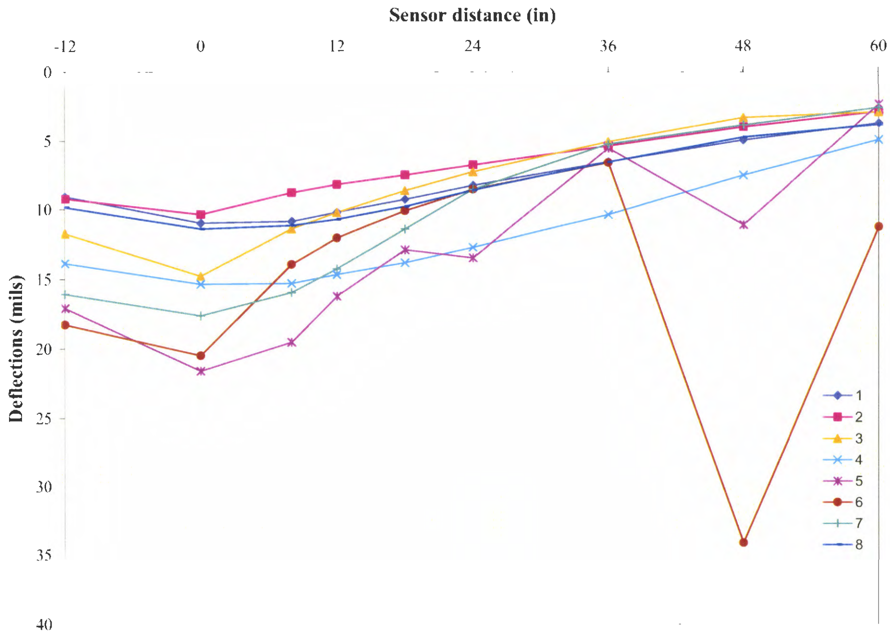


Figure 109. Cedar Rapids test #2: 9000 lb. FWD raw data.

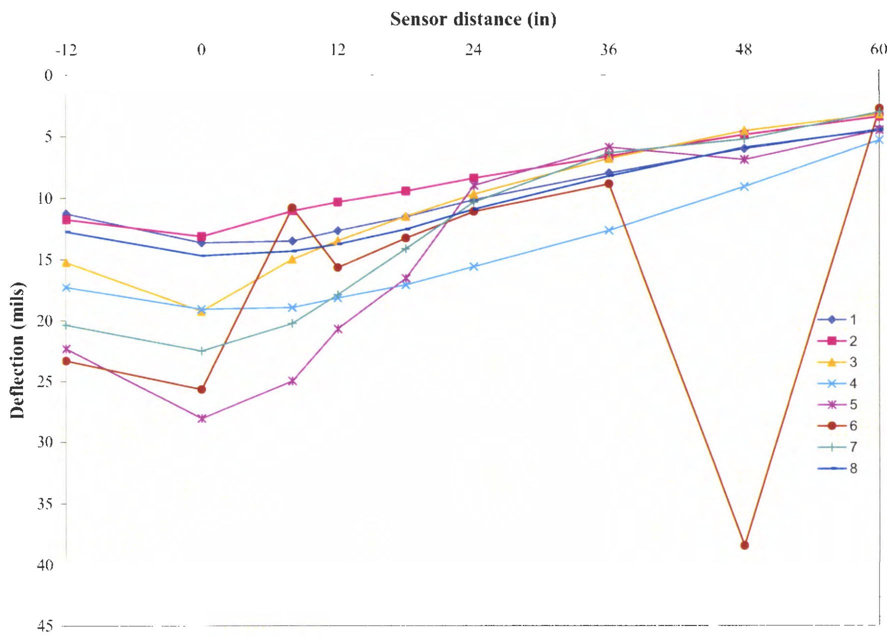


Figure 110. Cedar Rapids test #2: 11000 lb. FWD raw data.

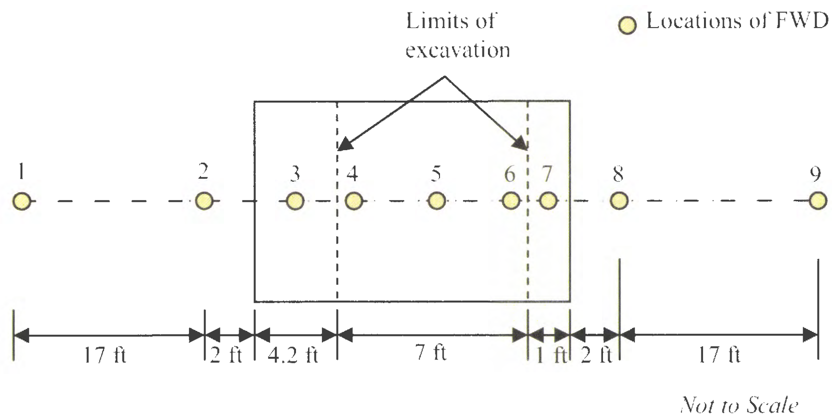


Figure 111. Des Moines FWD layout.

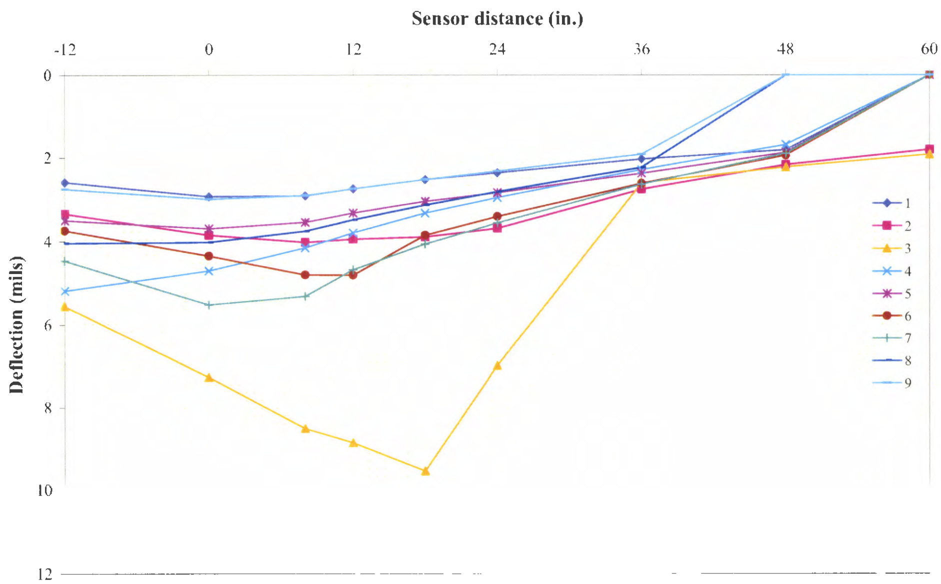


Figure 112. Des Moines test #1: 4000 lb FWD raw data.

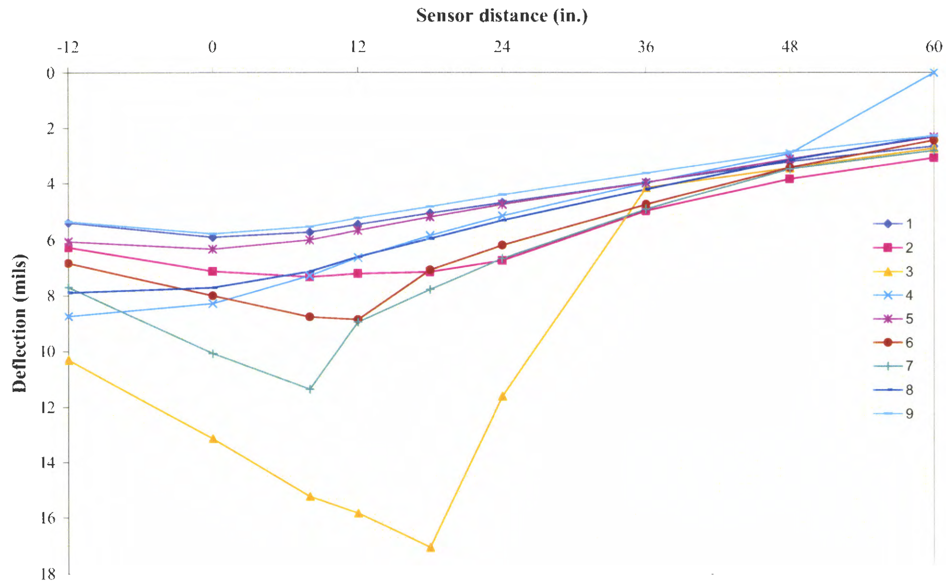


Figure 113. Des Moines test #1: 9000 lb FWD raw data.

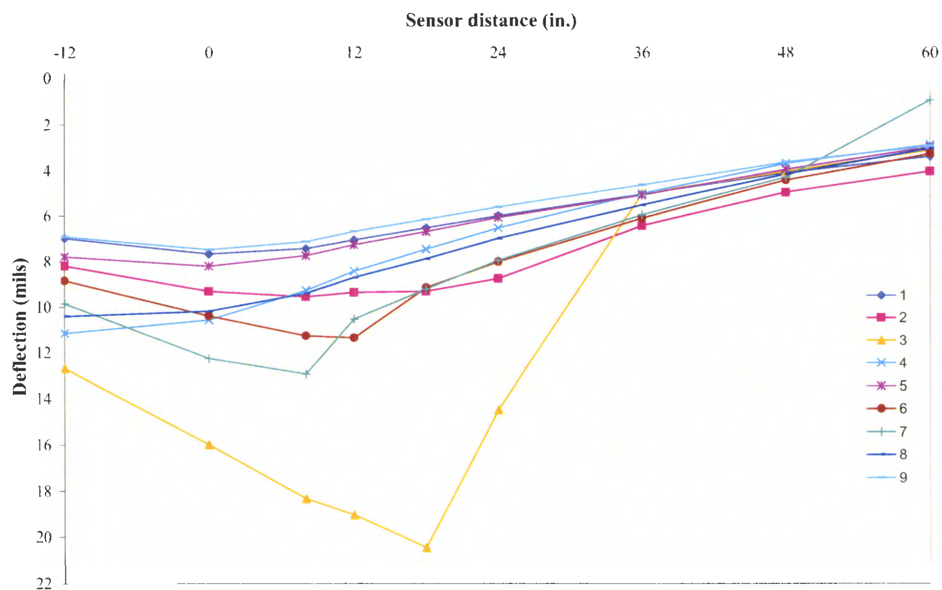


Figure 114. Des Moines test #1: 12000 lb FWD raw data.

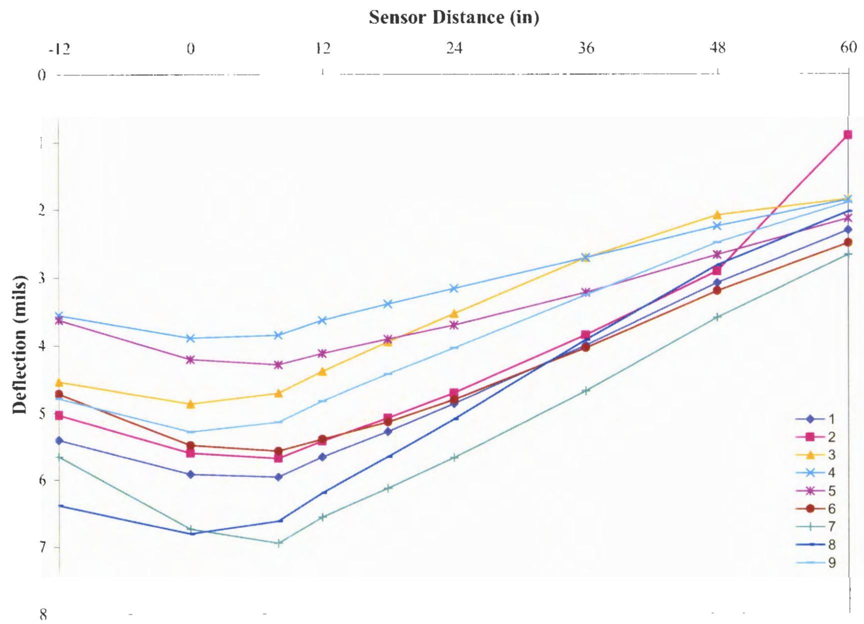


Figure 115. Des Moines test #2: 6000 lb. FWD raw data.

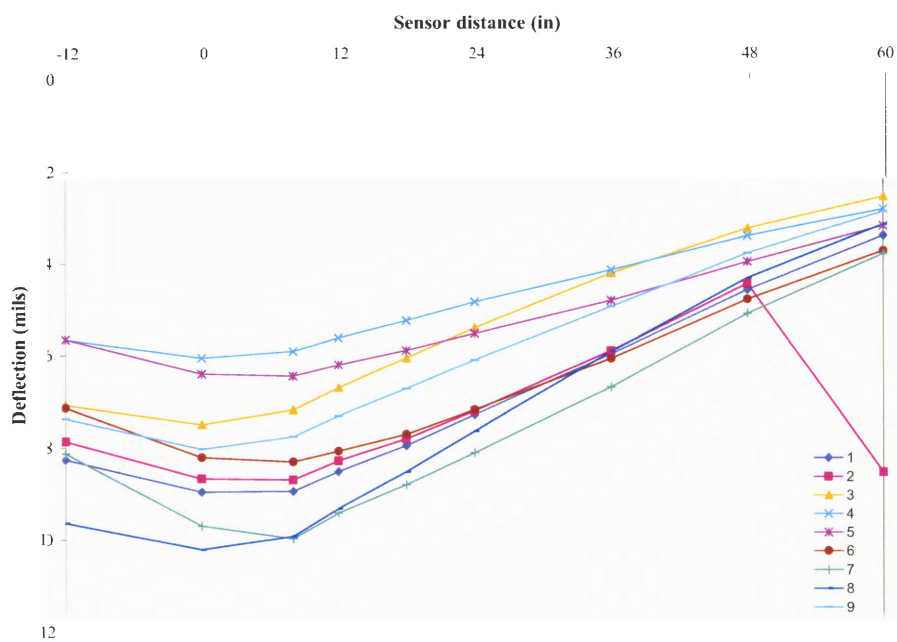


Figure 116. Des Moines test #2: 9000 lb. FWD raw data.

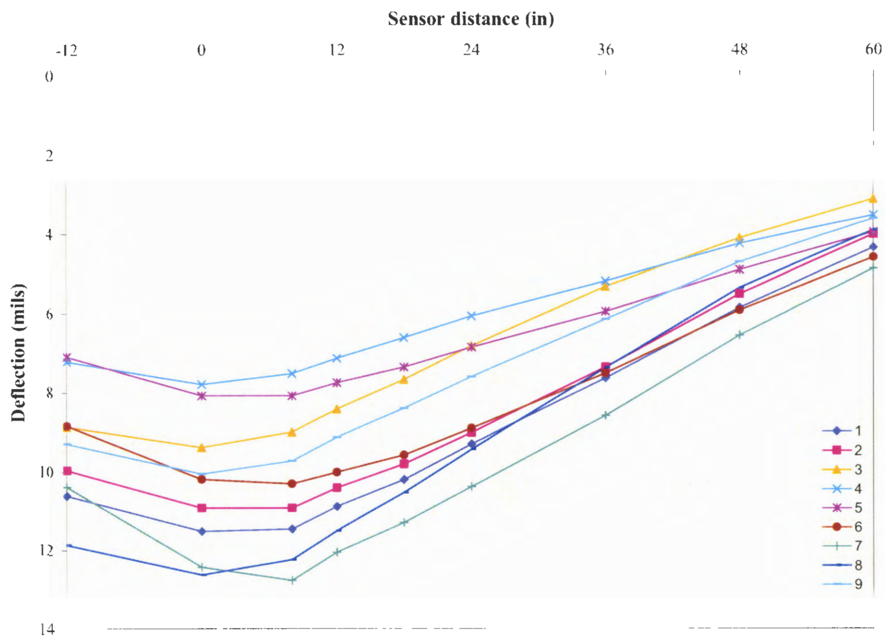


Figure 117. Des Moines test #2: 12000 lb. FWD raw data.

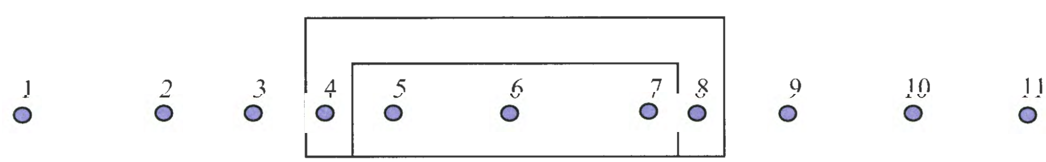


Figure 118. Ames: McKinley FWD layout.

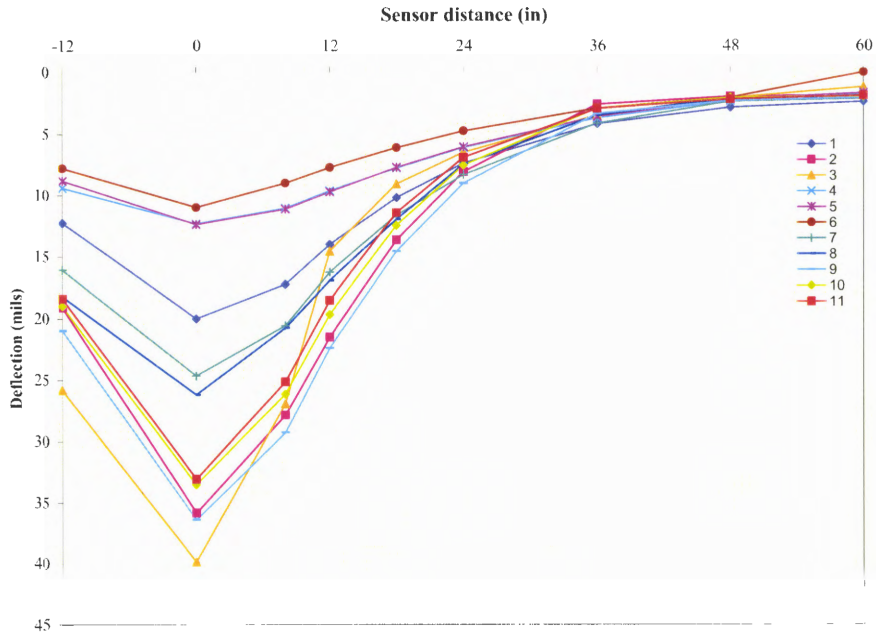


Figure 119. Ames McKinley St.: 6000 lb. FWD raw data.

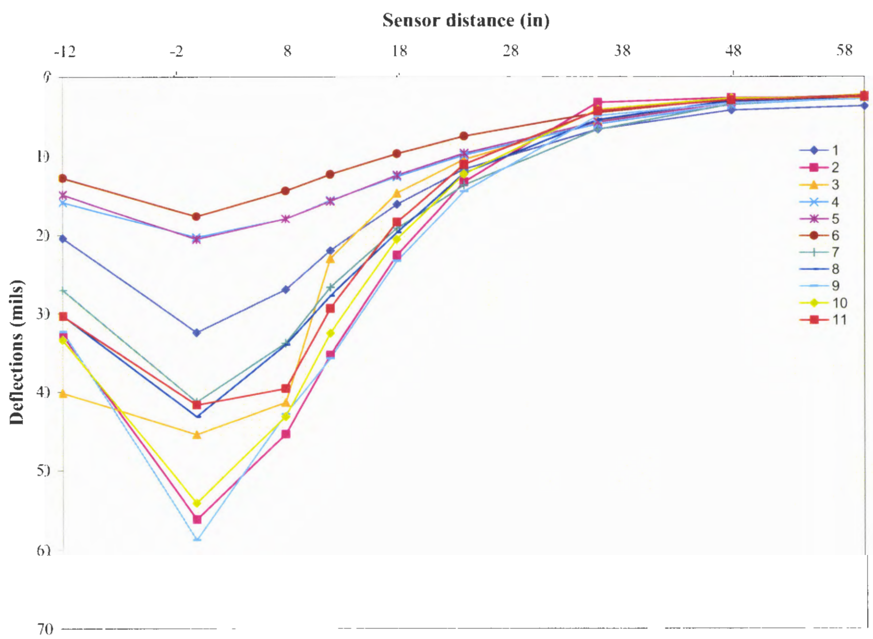


Figure 120. Ames McKinley St.: 9000 lb. FWD raw data.

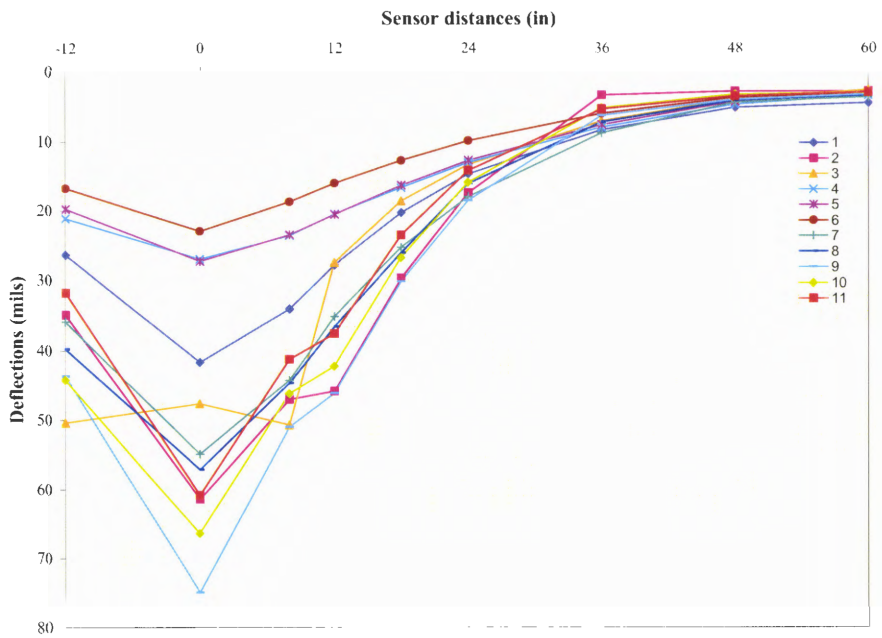
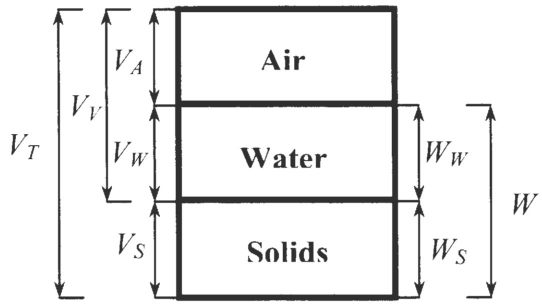


Figure 121. Ames McKinley St.: 12000 lb. FWD raw data.

APPENDIX C: SETTLEMENT DERIVATION

Derivation for Potential Trench Settlement:

Phase Diagram:



Settlement Equation:

$$S = \frac{\Delta e}{1+e_0} * H \longrightarrow \frac{(e_f - e_o)}{1+e_o} * H$$

Assume:

$$V_s = 1 \text{ and } V_t = V_v + 1$$

Therefore from phase relationships:

$$e = \frac{Vv}{Vs} \longrightarrow e = Vv$$

Also:

$$\gamma_d = \frac{W_s}{V_T} \qquad \gamma_d = \frac{W_s}{V_V + 1} = \frac{W_s}{e+1}$$

Then:

$$W_s = \gamma_d(e+1) \longrightarrow W_s = \gamma_d e + \gamma_d \longrightarrow e = \frac{W_s - \gamma_d}{\gamma_d} \Rightarrow e = \frac{Ws}{\gamma_d} - 1$$

Substitution into the Settlement Equation:

$$S = \frac{(e_f - e_o)}{1+e_o} * H \longrightarrow S = \frac{\left(\left(\frac{W_s}{\gamma_{d f}} - 1\right) - \left(\frac{W_s}{\gamma_{d o}} - 1\right)\right)}{1 + \left(\frac{W_s}{\gamma_{d o}} - 1\right)} * H$$

$$S = \frac{\frac{W_s}{\gamma_{d f}} - \frac{W_s}{\gamma_{d_o}}}{\left(\frac{W_s}{\gamma_{d o}}\right)} * H = \left(\frac{\frac{W_s}{\gamma_{d_f}}}{\frac{W_s}{\gamma_{d_o}}} - \frac{\frac{W_s}{\gamma_{d_o}}}{\frac{W_s}{\gamma_{d o}}}\right) * H = \left(\frac{\frac{W_s}{\gamma_{d_f}}}{\frac{W_s}{\gamma_{d_o}}} - 1\right) * H \boxed{= \left(\frac{\gamma_{d_o}}{\gamma_{d_f}} - 1\right) * H}$$

APPENDIX D: FIGURES IN METRIC UNITS

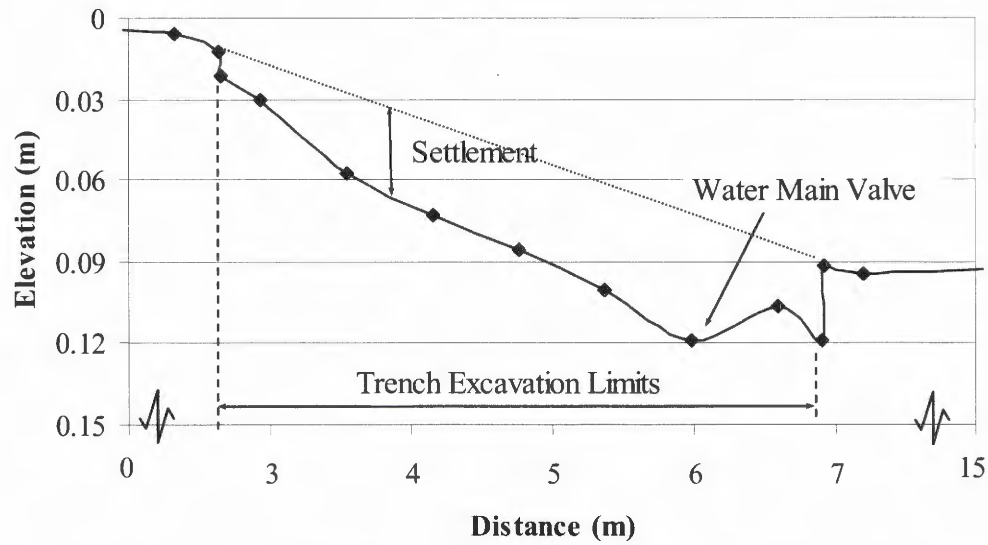


Figure 122. Settlement profile of poorly performing utility cut in asphalt pavement.

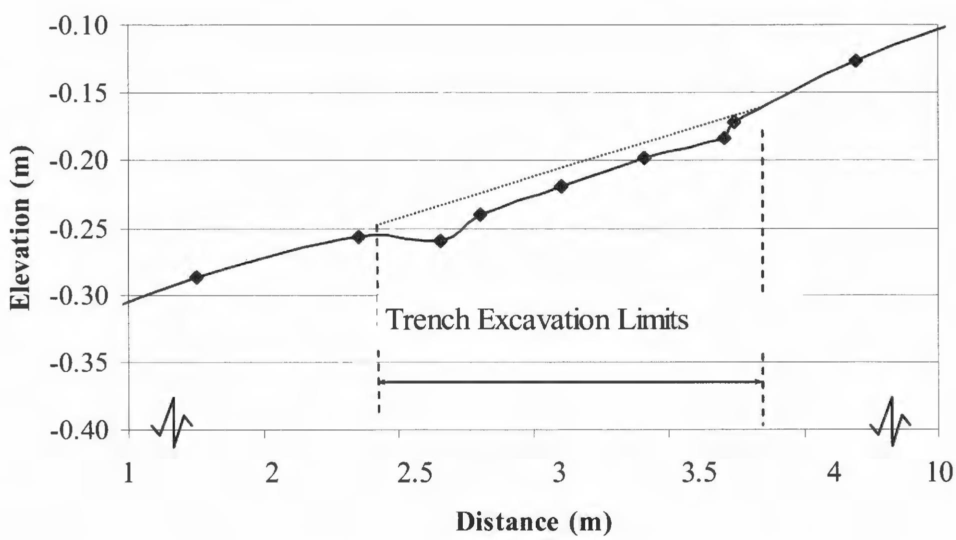


Figure 123. Settlement profile of poorly performing utility cut in concrete pavement.

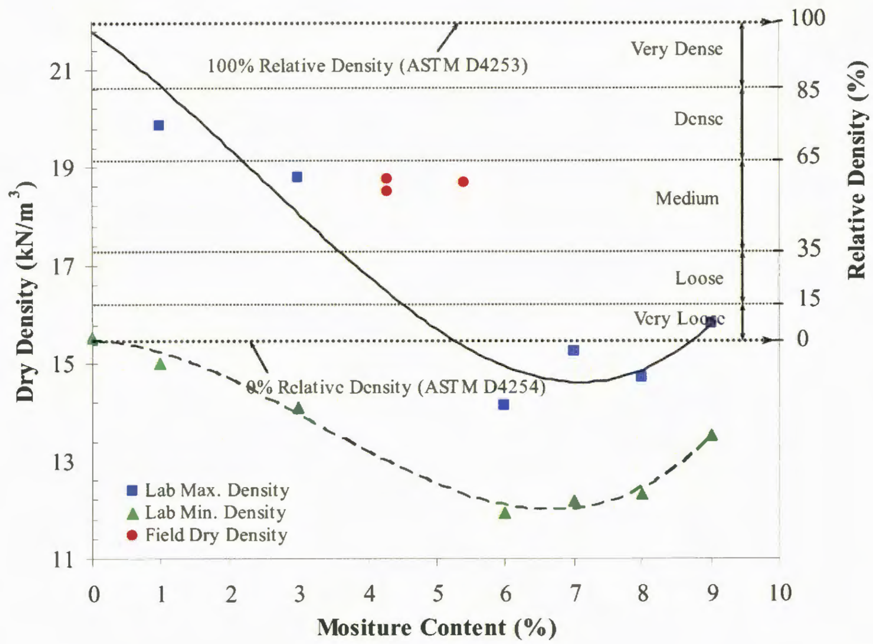


Figure 124. Ames 3/8 minus Relative Density Plot.

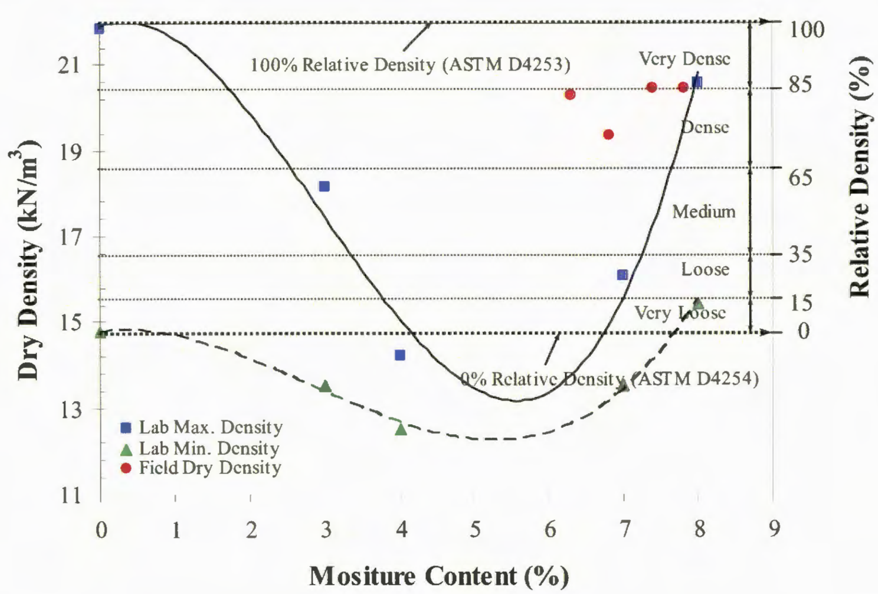


Figure 125. Cedar Rapids Relative Density Plot.

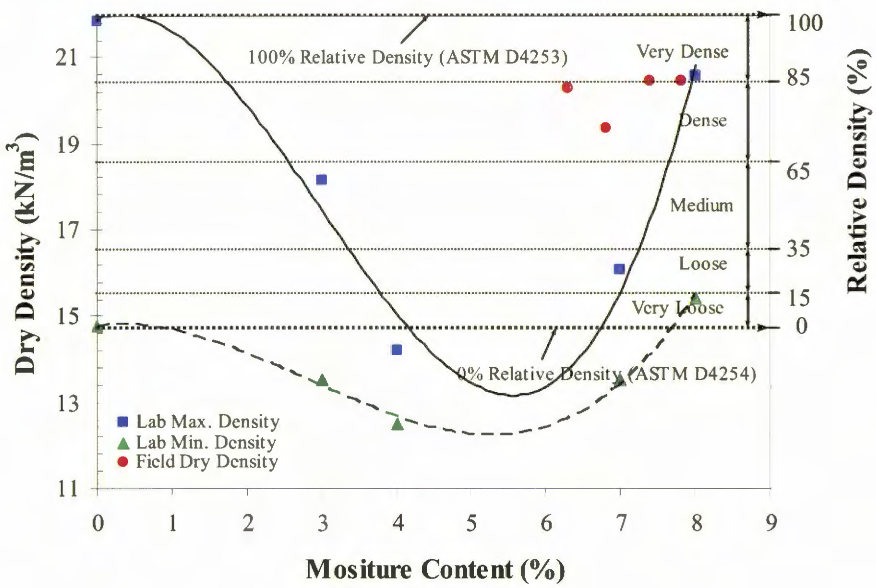


Figure 126. Davenport Relative Density Plot.

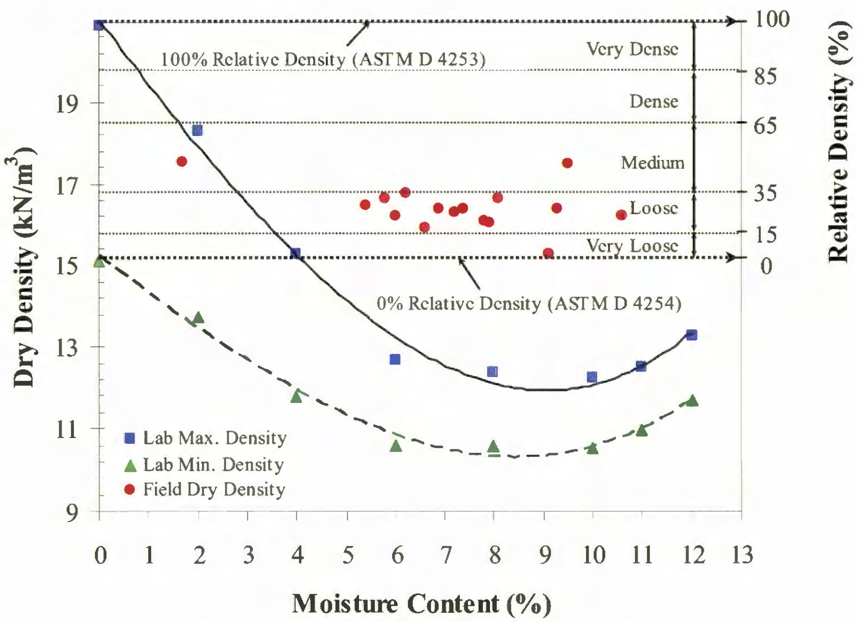


Figure 127. Des Moines Relative Density Plot.

Common unit conversions from English to Metric units of measurement include:

Table 24. English to Metric unit conversions.

Dimensions	English Units	Metric Units
Length	1 in	25.4 mm
Length	3.28 ft	1 m
Mass	1 lb_m	454 g
Unit Weight	62.4 lb/ft^3	9.81 kN/m^3