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16. Abstract

Flexible bases (unbound aggregate bases) are an integral part of flexible pavements in Texas, and the state has many good quality aggregate sources. However, aggregates vary in mineralogy and in physical properties. These variations often have a profound effect on the engineering response of these materials. This is certainly true when one considers the effect of moisture on the strength, stiffness response and volumetric response of aggregate systems used as flexible bases.

Study 1432 focuses on the properties of flexible bases that lead to cracking and general loss of strength and stiffness of flexible aggregate bases. The problems associated with cracking and other forms of deterioration in aggregate bases in Texas are moisture- and thermal-related. Research in TxDOT study 2-8-73-18 described the mechanisms responsible for moisture- and thermal-related cracking and distress. With this study as a reference, study 1432 focuses on identifying efficient and reliable tests which can be used in design and construction specifications to limit cracking within the flexible base and hence the resulting distress.

Report 1432-2 describes the laboratory testing protocol used to evaluate aggregates for susceptibility to cracking and strength loss due to thermal effects. The report further defines a dielectric screening test and associated criteria which appears to have the potential to quickly and accurately differentiate among aggregates with high, moderate and low susceptibility to moisture and thermal (including freeze-thaw) distress. The credibility and reliability of this test is being verified by intense mineralogical, strength and mechanical testing in simulated freeze-thaw environments.

Report 1432-2 and its companion report 1432-1 discuss the criteria for selecting the aggregates and pavement case histories for this study.

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THERMAL CRACKING IN FLEXIBLE BASES IN TEXAS: AN INTERIM REPORT ON THE LABORATORY AND FIELD TESTING PROTOCOL

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Research Report 1432-2
Research Study Number 0-1432
Research Study Title: Guidelines for the Design and Construction of Flexible Bases to Control Thermal Cracking in Pavements

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IMPLEMENTATION STATEMENT

This is an interim report for the continuing study 0-1432 dealing with thermal cracking in road bases in Texas. This report presents the methodology and protocol for lab and field testing of selected aggregate sources and pavement case histories. The report also identifies the aggregate sources and pavement case histories selected for lab and field evaluation and how these sources and projects were selected.

This report is a companion to report 1432-1 which describes in detail the process for selection of pavement case histories and initial condition surveys conducted on these pavement sections.

The testing and evaluation protocol discussed in this report will be completed in year two. Based on the results of year two, the researchers will recommend design and construction specification changes to minimize moisture damage and thermal cracking in aggregate bases.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation (TxDOT), or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. The engineer in charge of the project is Dallas N. Little, P.E. #40392.

TABLE OF CONTENTS

Page

LIST OF FIGURES

LIST OF TABLES

Table Page

SUMMARY

Flexible bases (unbound aggregate bases) are an integral part of flexible pavements in Texas, and the state has many good quality aggregate sources. However, aggregates vary in mineralogy and in physical properties. These variations often have a profound effect on the engineering response of these materials. This is certainly true when one considers the effect of moisture on the strength, stiffness response and volumetric response of aggregate systems used as flexible bases.

Study 1432 focuses on the properties of flexible bases that lead to cracking and general loss of strength and stiffness of flexible aggregate bases. The problems associated with cracking and other forms of deterioration in aggregate bases in Texas are moisture- and thermal-related. Research in TxDOT study 2-8-73-18 described the mechanisms responsible for moisture- and thermal-related cracking and distress. With this study as a reference, study 1432 focuses on identifying efficient and reliable tests which can be used in design and construction specifications to limit cracking within the flexible base and hence the resulting distress.

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Report 1432-2 and its companion report 1432-1 discuss the criteria for selecting the aggregates and pavement case histories for this study.

CHAPTER 1: INTRODUCTION

BACKGROUND

Many miles of pavements in Texas are constructed with aggregate bases. These pavements are typically designed and constructed with a thin asphalt concrete surface over a flexible base layer. The base layer is therefore often the major structural component in such pavements. Thermal-and moisture-related effects can sometimes cause cracks in base layers. These cracks reflect through the asphalt surface and the load carrying capability of the entire pavement structure is thereby reduced. Thermal-and/or moisture-related cracking within the base layer is a problem that must be addressed to improve our ability to construct and manage structurally adequate pavements.

Texas has a diverse environment including regions with and without freezing temperatures and varying amounts of rainfall. Frost depth ranges from 12 to 50 cm in freezing regions in Texas (Jumikis 1977); therefore thermal activity is typically limited in surface and base layers of pavements in Texas. A low temperature cracking mechanism of asphalt concrete however cannot explain the large amount of transverse cracking observed in the no-freeze regions of Texas. Where it occurs, the alternate freezing and thawing brings about severe thermal stresses in pavement materials that can contribute to cracking. Carpenter and Lytton (1977) report that thermal fatigue caused by freeze-thaw cycling is a major cause of transverse cracking in flexible bases in West Texas.

In Texas the problem of cracking in bases is observed in different climatic regions. It is likely that different distress mechanisms, which may or may not be thermal-related, are active in Texas. Some cracking may be solely related to a loss of shear strength within the base due to the ingress of water.

There have always been concerns regarding the quality of base materials in Texas. Sometimes for economic reasons, relatively low quality materials are used to take advantage of local materials for pavement construction. These materials may be susceptible to thermal-and/or moisture-related cracking. Moreover, different coarse aggregate types (e.g., limestone, silicone and iron ore gravel and caliche) are used in construction in Texas. It is important to investigate the effect of coarse aggregate type on the cracking potential of bases.

Premature cracking in road bases in some newly constructed projects has been observed in the San Angelo and Yoakum districts. It is possible that these base layers were compacted at wet of optimum moisture contents and experienced significant shrinkage cracking on drying. Both transverse and random cracking are manifested on such prematurely cracked projects. At this stage no conclusive reasons can be given for premature cracking in flexible bases.

It is essential to understand how the environment, physical and mineralogical properties of aggregates and construction practices, influences the cracking in road bases. This will help in minimizing the problems of thermal and premature cracking in road bases which are not considered in TxDOT design and construction specifications at this time.

STUDY OBJECTIVES AND SCOPE

Study 1432, entitled "Developing Guidelines for the Design and Construction of Flexible Bases to Control Thermal Cracking in Pavements", was commissioned to investigate the problem of thermal-and/or moisture-related cracking in flexible bases which is prevalent in many regions in Texas. The following is the summary of objectives set forth for this study:

- Determine the extent of thermal cracking in flexible bases in Texas,
- Develop comprehensive design specifications for flexible bases that will minimize thermal cracking yet fulfill other design criteria,
- Develop improved construction specifications for flexible bases, which account for such factors as moisture content, compaction density and other construction and design parameters,
- Investigate methods of improving poor performing aggregates (i.e., use of stabilizers), and
- Develop guidelines to structurally evaluate cracked pavements and to propose optimum repair strategies for cracked pavements based on technical and economic considerations.

RESEARCH APPROACH

The research approach for study 1432 is primarily based on a comprehensive field and laboratory testing program to investigate base material from selected case study projects. Figure 1.1 shows a flowchart for the research approach used in this study.

A literature search was conducted for the study. Relevant references were reviewed and important conclusions were documented.

A broad level assessment was set up by the researchers to determine the extent of the thermal cracking problem in road bases in Texas. The PMIS database, which contains data for the observed transverse cracking on the pavement surface, was used to conduct an analysis to determine the extent of transverse cracking in Texas.

An experimental design was set up for the study which included three important factors: temperature, moisture and soil type. A screening of districts was conducted to identify candidate districts for study projects. Several TxDOT district offices were contacted, field visits were carried out and case study projects were selected.

A comprehensive field evaluation program is underway to investigate the case study projects. The program consists of visual condition surveys, Falling Weight Deflectometer (FWD) testing, Dynamic Cone Penetrometer (DCP) testing and Ground Penetrating Radar (GPR) surveys. The purpose of the field evaluation program is to obtain information about the pavement's condition and to estimate in situ properties of the base materials used in the case study projects. Initial condition surveys of selected pavement case study projects have been completed. In future condition surveys, results will be examined in conjunction with the results obtained from other field and laboratory tests. This will help in understanding the relationship among the observed distresses, in situ pavement properties and laboratory test results.

Base material samples will be obtained from each case study project for laboratory evaluation. Material samples will also be taken from selected sources (pits and quarries) in some districts. The laboratory testing program is aimed at investigating thermal susceptibility and strength properties of base materials. Several tests will be carried out in the laboratory such as dielectric tube probe tests, Texas triaxial tests, pedological, mineralogical and fabric analysis of

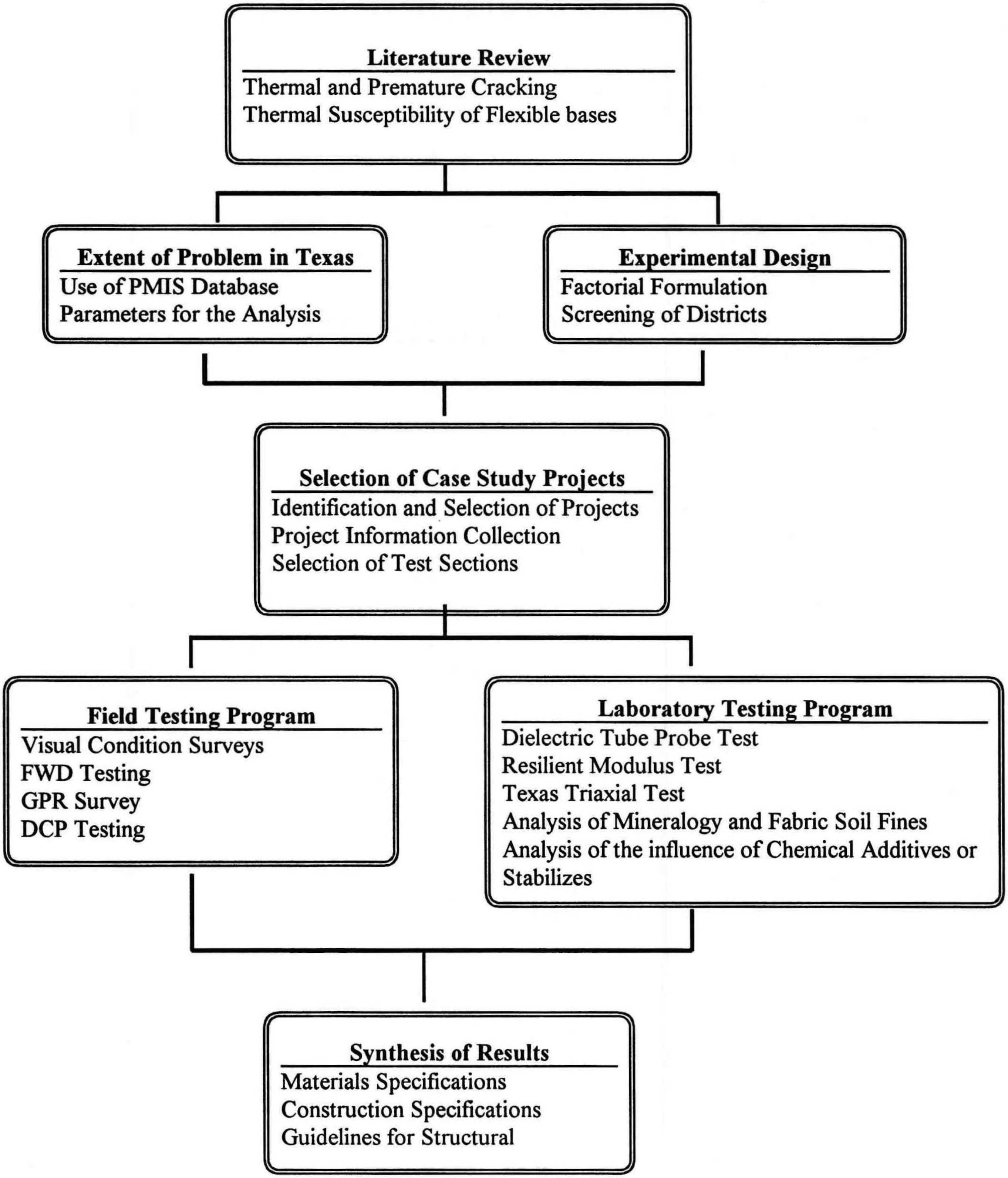


Figure 1.1 Research Approach Used for the Study 1432.

the soil fines fraction and analysis of the effect of chemical additives on the soil fines fabric and general stability of its aggregate bases.

Results from the field and laboratory evaluation will be synthesized and the cause of thermal cracking in flexible bases will be established. Specifications will then be developed for base materials to control thermal cracking. Construction procedures will also be investigated for their role in causing thermal and/or premature cracking in bases. Finally, guidelines will be developed for structural evaluation of cracked pavements, and optimum repair strategies will be proposed to maintain and rehabilitate cracked pavements.

REPORT ORGANIZATION

This report supplements report 1432-1. Report 1432-1 is divided into five chapters: Introduction, Extent of Cracking in Texas, Experiment Design and the Selection of Case Study Projects, Conditions Surveys and Conclusions and Recommendations. Report 1432-1 focuses on the selection of projects for field monitoring and study and the primary considerations in the selection of these projects. This report more definitively addresses the laboratory and field testing program for the entire study and presents some of the data and findings from the first year of the laboratory testing program.

This report is divided into six chapters. The first is the introduction. The second is a summary of pertinent literature which is being used in this study and is the basis for the laboratory and field testing protocols adopted for this study.

Chapters 3 and 4 describe the laboratory and field testing protocols, respectively, and explain the tests adopted for the testing protocol and the information which we expect to get from these tests as well as the practical benefits of this information. Chapters 5 and 6 summarize results of the laboratory and field testing program to date and present pertinent findings from these results and discuss directions for research in year two. Finally, chapter 6 presents conclusions and recommendations from the first year of this study.

CHAPTER 2: SUMMARY OF PERTINENT LITERATURE

GENERAL

The base layer is often the primary structural layer in an asphalt pavement unless the asphalt concrete layer is substantially thick (greater that about 200 mm). Very few Texas pavements were originally designed and constructed with such thick asphalt surface layers; therefore, the base layer is often the major structural component of the pavement. Consequently, it is imperative to treat this layer as a critically important structural contributor. Since cracking within the base layer can certainly diminish its load-carrying capability, the problems associated with thermal and traffic load induced cracking within the base layer greatly impact the performance of the entire pavement structure. Thermal-or moisture-related cracking within the base is a problem that must be addressed as this cracking reflects through the asphalt surface layer.

Concerns have been raised regarding the quality of base materials used in Texas and that these lower quality materials are susceptible to freeze-thaw damage. Economical sources of type 1 bases do not exist in many parts of Texas, particularly along the Gulf Coast, south Texas, the High Planes and east Texas. Since these aggregate sources may not meet the rather tight specification requirements of type 1 bases with regard to Texas triaxial strength, abrasion resistance, plasticity, etc., it is essential to understand how the highly variable physical properties and mineralogical properties of these different aggregate sources influence their behavior in the pavement environment. In other areas, stabilizers are used to bring marginal materials up to type 1. In some instances, these stabilizers may be the cause of some of the reported cracking.

PREVIOUS TXDOT RESEARCH (STUDY 2-8-73-18)

The most comprehensive study ever performed on the environmental deterioration of pavements in west Texas was summarized in Research Report 18-4F by the Texas Transportation Institute (Study 2-8-73-18). In this study Carpenter and Lytton (1977) reported that transverse cracking in west Texas is largely a product of freeze-thaw cycling which acts primarily in the base course. The base course undergoes volumetric contraction upon freezing that is an order of magnitude larger than that of the asphalt concrete surface. This contraction is related to the specific surface area of the clay mineral portion of the material. Carpenter and Lytton (1977) developed a mathematical model of the freeze-thaw contraction process to predict the occurrence and severity of freeze-thaw induced cracking.

Furthermore, Carpenter and Lytton (1977) proposed a theory of particle structure and reorientation based on field data and based on a theoretical interpretation of the Lennoard-Jones model for inter-particle forces. This theory was verified by Carpenter and Lytton (1977) while using scanning electron micrographs of base course materials with and without being subjected to freeze-thaw cycles. A computer model was developed which uses material properties to predict crack spacing caused by this contraction. The model uses actual climatic data to calculate the rate of crack growth and the change in crack spacing with time.

Carpenter and Lytton (1977) performed a series of stabilization studies aimed at identifying additives which may be effective in reducing susceptibility to thermally induced and freeze-thaw induced cracking. The study indicated that very low percentages of stabilizers may be effective, while higher percentages may actually induce more extensive cracking. In that study low percentages of gypsum (0.5 to 0.75%) were found to be effective in reducing volume changes induced by thermal fluctuations and freeze-thaw actions.

More specifically, the Carpenter and Lytton (1977) study concluded the following ideas.

1. Thermal susceptibility of the base course is a valid deterioration mechanism and the volumetric contraction activated by freezing and thawing is quite prevalent in base course materials in west Texas and probably in other parts of Texas.
2. Soil moisture suction, which is a measure of the energy state of moisture within the soil, is a parameter that directly relates the environment to the engineering behavior of the soil. The relationship between soil suction and thermal susceptibility accentuates the need to fully characterize a material by testing it in the environment in which it will be used.

3. Although clay contents of the base course are often relatively low, and within specifications, the mineralogy of the clay fines has a significant impact on the mechanism of cracking and deterioration.
4. The specific surface area and the ion concentration of the fines within the base materials impact interparticle forces and the manner in which these forces resist freeze and thaw action.
5. The mechanism of thermal susceptibility will produce a crack within the base course. Subsequent temperature cycles will propagate this crack through the asphalt surface course and produce more cracks within the base course. The rate and extent of reflection cracking within the asphalt concrete surface can be determined by using a compute model developed at TTI which utilizes the viscoelastic properties of the asphalt concrete surface course to predict reflection cracking within the surface layer.
6. The TTI computer model will allow the environmental damage caused by a thermally susceptible base course to be analyzed in a "stress and distress system" type of analysis. This allows one to assess the impact of changes in material properties of the pavement layers on frequency and severity of reflection cracking.
7. Stabilization of the fines in the base course may provide part of the answer to reduction of the susceptibility of bases to crack. However, the correct percentage of stabilizer additives to minimize cracking and thermally induce volume change probably falls within a constricted zone or region. Too little stabilizer may not be sufficient to react with the deleterious clay mineral fines, yet too much stabilizer may result in excessive pozzolanically induced or hydration cementation induced shrinkage cracking. This matter deserves more study.
8. It is imperative to fully study the properties of the base course in the frozen state. Little information is available at present concerning the frozen tensile strength and modulus values. It is the frozen properties that most severely affect the crack spacing and rate of propagation in the asphalt concrete.
9. The freeze coefficient is defined as the relative amount of volume change due to

thermal activity. The freeze coefficient is dependent on the soil suction which is in turn dependent on soil fabric, mineralogy and surface area as well as on level of compaction of the material. The freeze coefficient is normally small or slightly positive in the environment of east Texas where soil suctions are lower and where the moisture content of the base layer is maintained at nearly optimum moisture content (based on moisture-density relationships) conditions. However, the freeze coefficient can be considerably negative indicating contraction in west Texas. Thus freeze contraction can be very active in west Texas but not in east Texas. The residual effect of freeze-thaw is contraction, which causes transverse cracking whether in east or west Texas.

PREDICTION OF REFLECTION CRACKING IN THE ASPHALT CONCRETE SURFACE LAYER

The work of Carpenter and Lytton (1977) was extended at TTI in subsequent work by Jayawickrama, Lytton and Smith (Jayawickrama et al.,1986), Tirado-Crovetti et al. (1987) and Jayawickrama and Lytton (1987). This work evolved into a mechanistic-empirical overlay design methodology which predicts reflection cracking in asphalt concrete overlays in flexible pavements. In this approach the principles of fracture mechanics and beam-on-an-elastic-foundation theory were incorporated. Using these models, the mechanistically computed pavement responses were regressed against the observed distress of pavement sections located in various parts of the State of Texas and stored in the computerized data base at Texas Transportation Institute. This data base includes forty flexible pavement sections with bituminous concrete overlays.

The basic mechanisms generally assumed to lead to reflection cracking are the vertical and horizontal movements of the underlying pavement layers. These damaging movements may be traffic (a moving wheel load) or may be thermally induced. Figure 2.1 illustrates this combined mechanism. As seen from this figure, three pulses of high stress concentrations occur at the tip of the crack as the wheel passes over it. In addition to the influence of traffic loads, contraction and expansion of the pavement and the underlying layers with changes in temperature contribute to the growth of reflection cracks. Jayawickrama and Lytton (1987) introduced a technique to logically evaluate the effects of the combination of traffic load-induced shearing stresses and stresses induced by volume changes in the base layer on the propagation of reflection cracks.

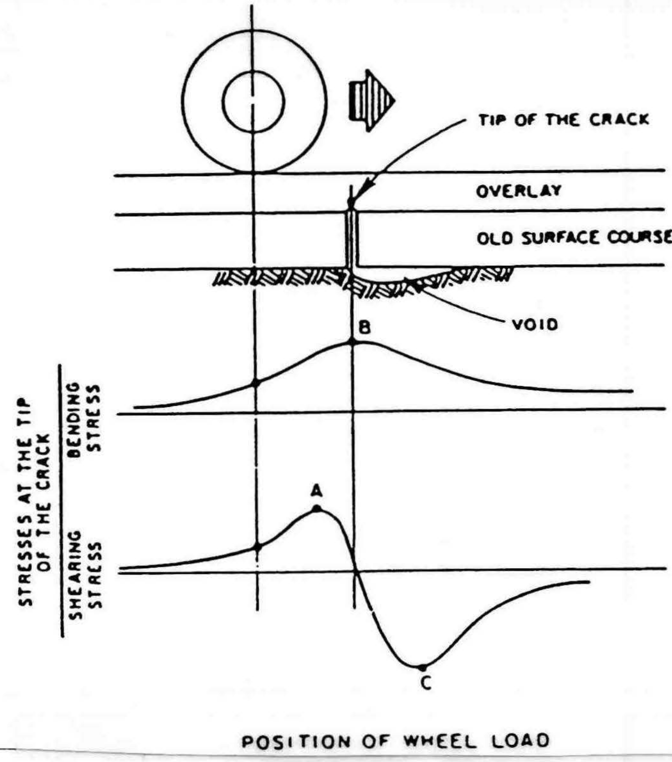


Figure 2.1. Stresses Induced at the Cracked Section Due to a Moving Wheel Load.

AGGREGATE BASE PROPERTIES (DETERMINED IN NON-DESTRUCTIVE TESTING) RELATED TO AGGREGATE BASE PERFORMANCE

Recent work at TTI by visiting Research Engineer Timo Saarenketo of the Finnish National Road Administration has revealed the practical use of Ground Penetrating Radar (GPR) to nondestructively evaluate the conditions of base materials. In this TTI study, Saarenketo (1995) evaluated eight different Texas aggregates and two different Finnish aggregates in order to relate their dielectric value and electrical conductivity at different moisture contents and densities to their strength and deformation properties. The dielectric value and electrical conductivity were measured using a dielectric and conductivity meter in the lab. However, these properties along with the moisture content of the base can be measured at highway speeds using GPR. The GPR technique is based on the measurement of travel time and reflection amplitude of a short electromagnetic pulse transmitted through a medium and then reflected partly from electrical interfaces like the base-subgrade interface. The two most important factors affecting the propagation of radar pulses in a medium are the dielectric value and the electrical conductivity, both of which are related to moisture content.

Saarenketo (1995) determined that dielectric constant and conductivity can be effectively used to predict deformation potential and strength of base materials. He further found that the dielectric constant correlates better with the California bearing Ratio (CBR), a measure of shear strength of compacted base, than does moisture content. This is illustrated in figures 2.2 and 2.3. Low dielectric constants (5.5 to 6.5) in compacted samples indicate the presence of small amounts of adsorption water and optimum strength properties. Higher values indicate that the material is sensitive to moisture. Dielectric values over 10 are identified by Saarenketo as "alarm values" as they indicate the threshold of significant potential for loss of strength and deformation potential. If the dielectric value is greater than 16, the base will become plastic and deform substantially under traffic. High electrical conductivity values indicate high concentrations of dissociated ions in the free water which can cause positive pore water pressures resulting in a rapid loss of strength.

The hysteretic effects of wetting and drying on strength and deformation of aggregates and soils in general are part of the literature (e.g., Yong and Wartkinson, 1966 and Fredlund and Rahardjo, 1993 and Lytton, 1994). Saarenketo demonstrated this hysteretic effect in the TTI study on carbonate aggregates. Figure 2.4 presents these CBR v. dielectric constants during wetting and drying cycles. These clearly defined hysteretic effects establish why substantially higher resilient moduli are measured in dry summer months than in the wetter months of the year. It is not simply a function of moisture content but also whether the soil is going through a wetting period or a dry period.

The detrimental effects of water which give rise to volume changes within the aggregate system and hence aggravate cracking stem from water in the aggregate system in the form of (a)

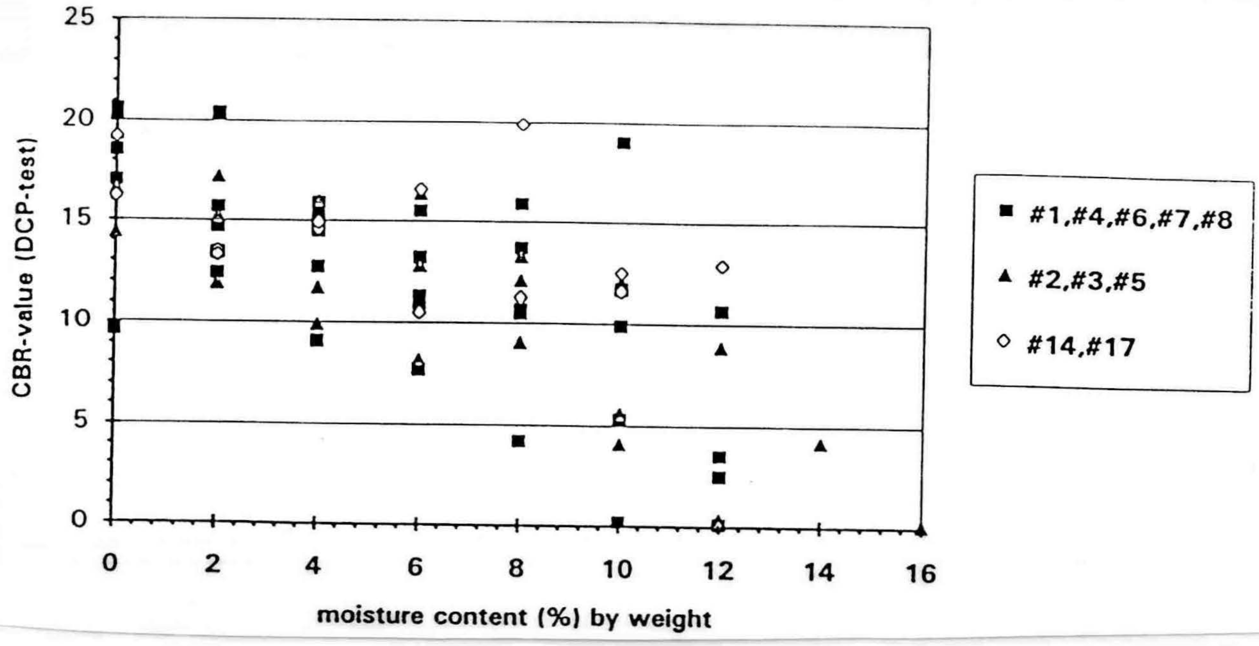


Figure 2.2. Correlation Between CBR-Value and Gravimetric Moisture Content of Texas and Finnish Aggregates.

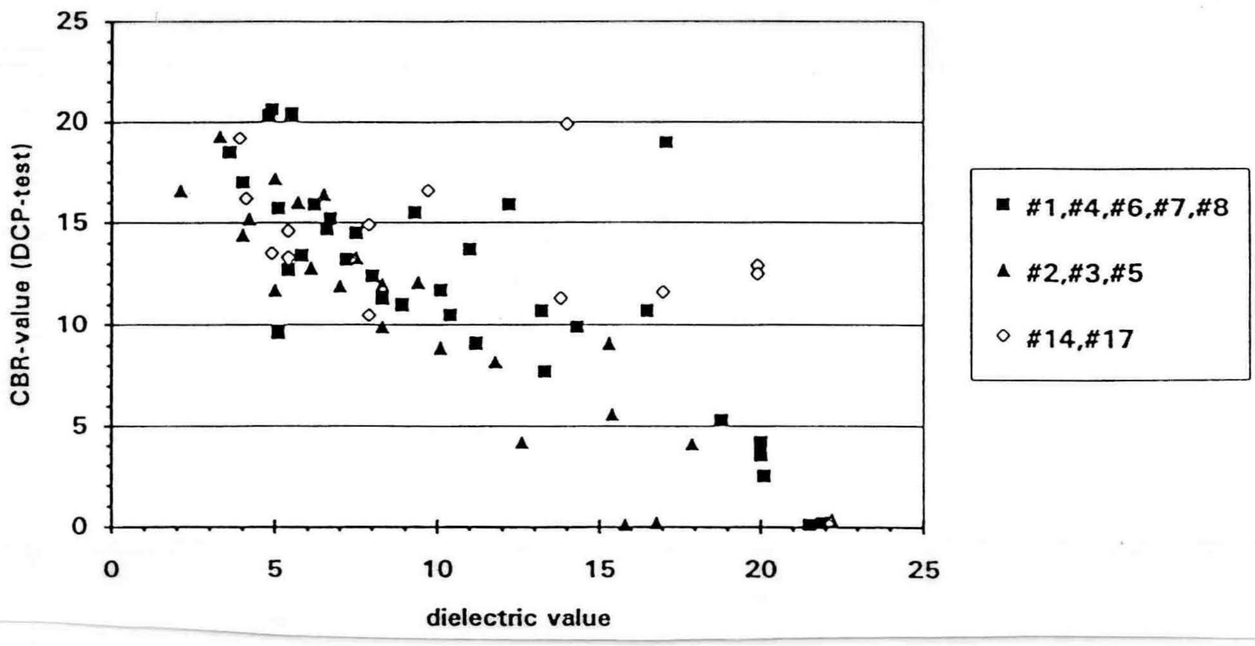


Figure 2.3. Correlation Between CBR-Value and Dielectric Value of Texas and Finnish Aggregates.

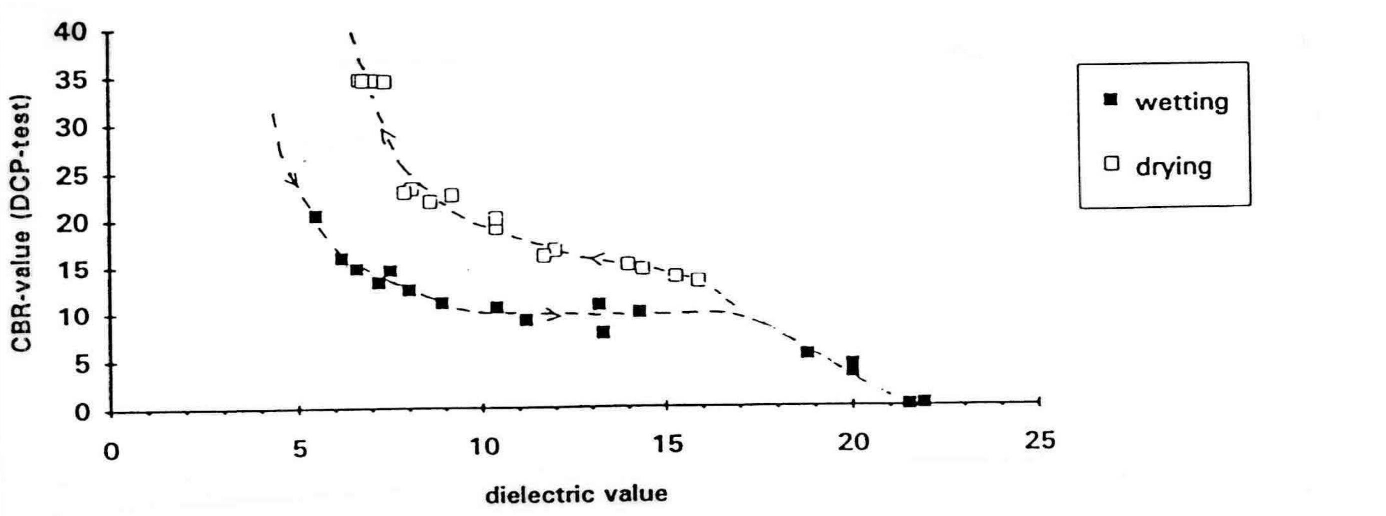


Figure 2.4. The Effect of Drying and Saturation in the Strength Properties and Dielectric Value of Some Texas Carbonate Aggregates.

water within the mineral structure, (b) free water or (c) bound water or adsorbed water. In adsorbed water, the dipole water molecules closest to the mineral surface are systematically arranged toward the mineral surface which has a negative charge. The most tightly held water layer, approximately 10 Å thick, consists of about three molecular layers of water (Mitchell, 1992) which has a higher density than free water and is much more tightly held (Mitchell, 1992 and Urry, 1992). The thickness of the adsorbed water layer can extend to about 100 Å under the right conditions and depending on the mineralogy of the aggregate fines and the specific surface area of the aggregate fines. In a Finnish study, Saarenketo (1995) related the specific surface area of selected Finnish aggregates to water adsorption which is, in turn, related to performance. More highly adsorptive aggregates exhibited substantially poorer performance. This is especially true in the cold climate of Finland. This relationship between water adsorption and specific surface of the fine aggregate is illustrated in figure 2.5.

When the soil temperature drops below 0^0 C, free water forms hexagonal crystals and thus expands. During the freezing process water molecules add one by one to the growing ice crystals, but they remain separated from the mineral surface by the thin adsorption layer (Anderson, 1989). This relatively narrow region below the nominal base of the ice lens is called the "frozen fringe" (Ladanyi and Shen, 1989). At this same time suction causes liquid water to migrate to the ice lens from the unfrozen soil through this unfrozen water layer (Konrad and Morgenstern, 1980).

As the temperature in the soil continues to decrease, the bound water starts to freeze but the tightly bound water remains unfrozen. At a temperature of -5^0 C, the amount of unfrozen water is still 12 percent of the total volume of unfrozen water (Anderson, 1989). The amount of the frozen adsorption water decreases with decreasing temperature until the water movement to the frozen fringe is significantly reduced. Small amounts of unfrozen water in soil have been measured even at temperatures of -40^0 C (Anderson, 1989). This distribution of water in frozen solid is illustrated in figure 2.6.

The freezing process is also controlled by the amount of dissolved salts, by products of hydrolytic reactions, which according to Kujlala (1991), lower the free energy and thus the freezing temperature of the aggregate-water system. On the other hand, many fine-grained base aggregates, such as argillaceous carbonates, volcanites, sandstones and chert and shale impurities degrade with repeated wetting and drying and with freezing and thawing, especially under the influence of de-icing salts (Hudec, 1994) to worsen the destructive effects of both moisture and the freeze-thaw phenomenon.

DETAILED TESTING AT TTI TO EVALUATE FREEZE-THAW POTENTIAL

Saarenketo's work at TTI comparing the eight Texas aggregates with the two Finnish aggregates demonstrated that all eight of the Texas aggregates were inferior to the two Finnish aggregates in terms of freeze-thaw resistance. The primary cause of this poor performance was the amount of plastic fines present in the Texas bases. These conclusions certainly support the findings of study 2-8-733-18 which pointed out the relationship between thermal cracking and freeze-thaw activity and clay content.

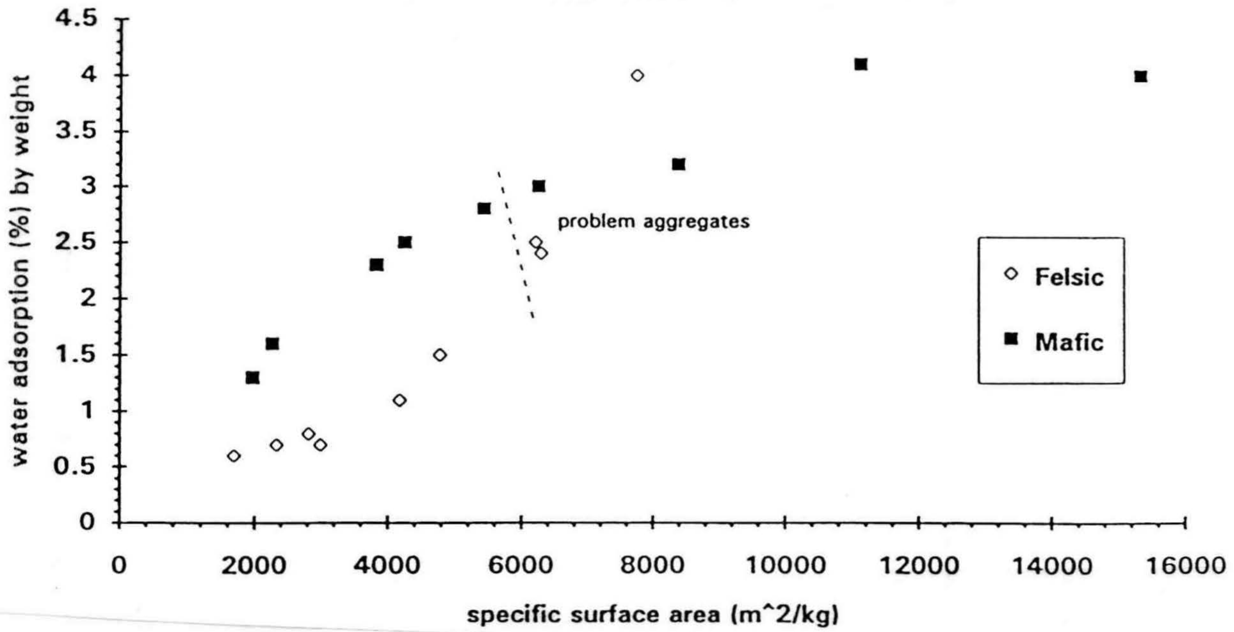


Figure 2.5. Correlation Between Water Absorption and Specific Surface Area of Fine Fractions (<0.074 mm) of Some Finnish Base Course Materials.

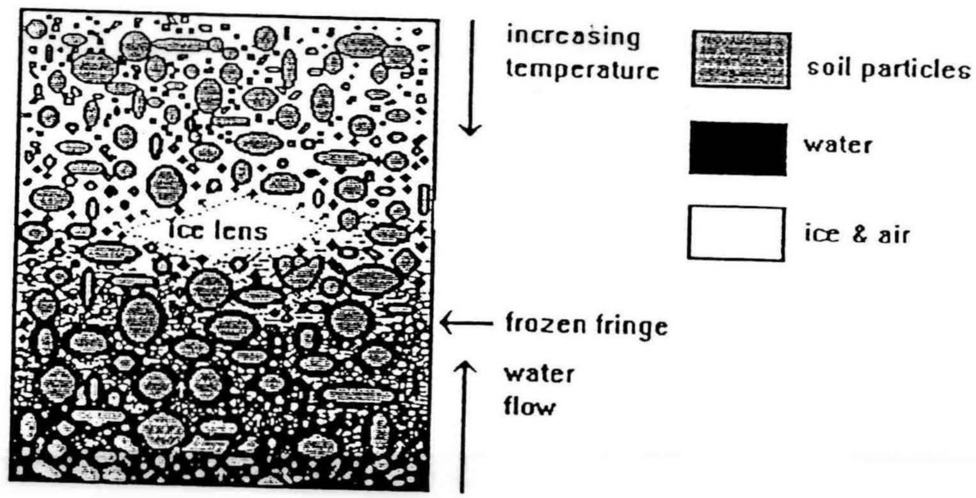


Figure 2.6. A Schematic Model of the Distribution of Water in Frozen Fringe.

Saarenketo performed the following non-standard tests to evaluate freeze-thaw potential of the aggregates:

> *Strength versus Moisture Content:* Samples were constructed in a mold at different moisture contents and the cone penetrometer test was performed to evaluate the strength of the aggregates.
>
> *Suction Tests:* This test is simple and is schematically illustrated in figure 7. It measures the suction of the aggregate base. The compacted cylinder of aggregate is contained in a plastic cylinder and allowed to stand in 2.5 cm of water. The test involves simply measuring the moisture content of the top of the sample against time. The test measures the ability of the base to attract and hold moisture. It is this trapped water within the base that causes freeze-thaw damage.

The graph in figure 2.7 contrasts the performance of good and bad performing aggregates. Bad performers have high suction and will attract and hold moisture entering the base from either the shoulders of the pavement or from the subgrade. This test is preformed in both a wetting and drying cycle. The amount of suction is dictated by the clay content and the type of clay mineral present in the base.

TXDOT STUDY 1287

Under TxDOT study 1287, Little, Scullion, Kota and Bhuijan (1994) investigated the performance of stabilized bases and subbases throughout the State of Texas. The investigation included laboratory testing but focused primarily on in situ testing in the Atlanta, Bryan, Houston, Lufkin, Yoakum, Corpus Christi, Austin and Beaumont Districts. In this study the resilient moduli of unstabilized, lightly stabilized and heavily stabilized aggregate bases were determined during two periods of the year ("wet and dry") from backcalculated deflection measurements of the Falling Weight Deflectometer (FWD).

The 1287 study revealed that highly stabilized bases in the Houston District resulted in

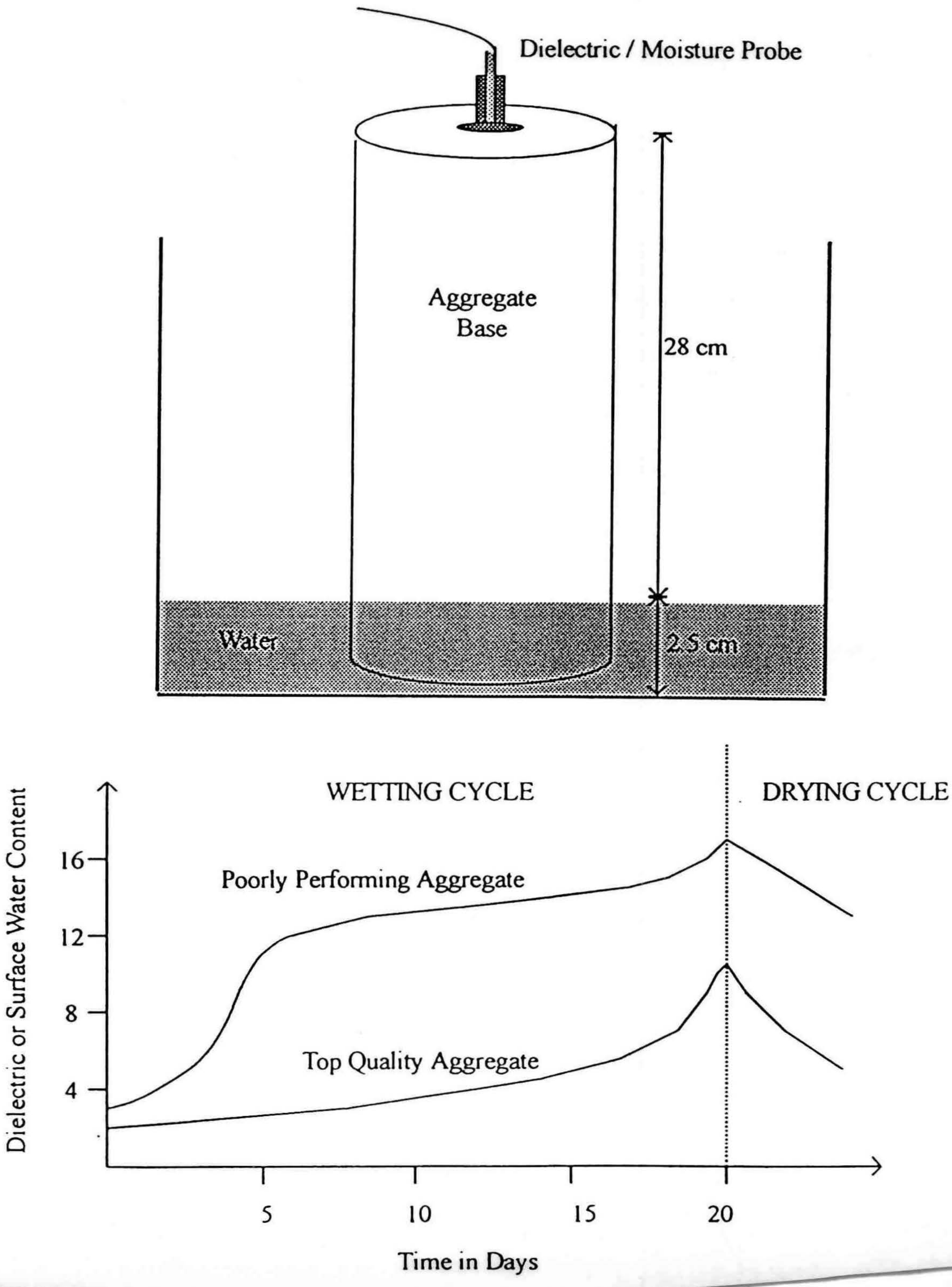


Figure 2.7. Schematic of the Suction Test and the Results of this Testing in Terms of Dielectric Constant Versus Time for Wetting and Drying Sequences.

severe reflection cracking and deterioration of the stabilized base as a result of water infiltration through reflection cracks in the asphalt concrete surface layer. Moderately stabilized bases in the Houston District performed generally better than highly stabilized bases. From the 1287 study results, Little et al. (1994) suggested a range of resilient moduli for the base that would provide the most successful performance life. The study essentially demonstrated that the very high moduli stabilized bases (above 7,500 MPa) suffered from excessive reflection cracking and subsequent moisture related deterioration. Moderately stabilized bases with moduli in the range of 1,000 MPa and 5,000 MPa performed well with little reflection cracking and adequate stability. In the 1287 study, Scullion suggested a mechanism of failure of the highly stabilized bases. This mechanism, illustrated in figure 2.8, was verified through extensive field evaluation.

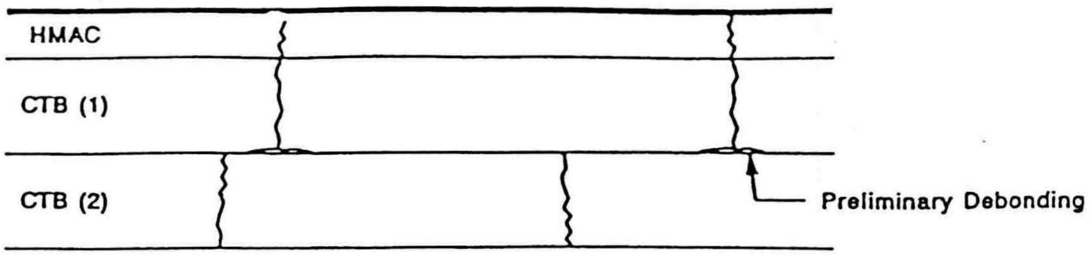
The 1287 study also evaluated lightly stabilized bases in the Yoakum and Corpus Christi Districts. Of particular interest were limestone bases in the Corpus Christi District stabilized with 1.5 to 2.0% hydrated lime. These bases have generally performed well in terms of high triaxial strength and good in situ moduli. However, the extent of cracking in these bases as compared to cracking in similar limestone bases without additives has not be evaluated.

EVALUATION OF CEMENT TREATED BASE FAILURE - TXDOT PROJECT 2919

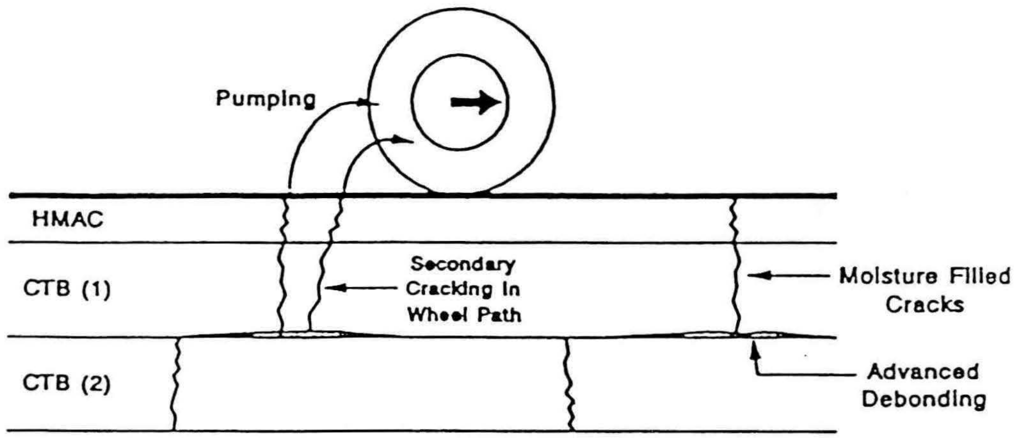
Scullion (1996) used the soil surface dielectric value test as a function of time of capillary soak to identify the mechanism of failure in cement treated bases near Conroe, Texas. The high dielectric values recorded demonstrated that the cause of distress was the ability of moisture to flow freely through the material and induce a rapid and severe disintegration of the base directly beneath surface shrinkage cracks. On SH 36 near Orchard, Texas, the problem was found to be related to the high percentage of clay in the fine material. On FM 3083, the problem was attributed to the lightweight sandstone coarse aggregate.

DRAFT

Phase 1 - Crack formation and initial debonding caused by differential shrinkage and thermal mismatch of CTB layer.



Phase 2 - Water ingress and deterioration under load.



Phase 3 - Final stages.

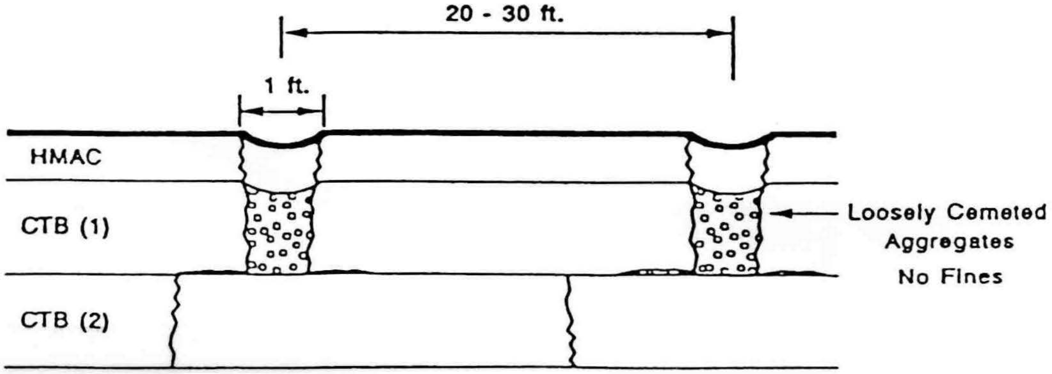


Figure 2.8. Deterioration Cycle of CRB on SH-36.

CHAPTER 3: LABORATORY TESTING PROTOCOL

GENERAL

The purpose of the lab testing program is to determine the susceptibility of aggregates used in Texas to moisture and thermal damage which may lead to cracking and other forms of distress. The literature, particularly the work of Carpenter and Lytton (1975), establishes that soil suction, a measure of the energy state of moisture, directly relates the influence of the environment to the engineering behavior of soils and aggregates. Carpenter and Lytton (1975) developed a test to measure the volume changes that occur during freezing in an aggregate system. In this test, the aggregate is compacted at a specified energy and at a specified molding moisture content in a modified Proctor compaction mold. The sample is encapsulated with metal foil and wax. A psychrometer is installed within the samples prior to subjecting the sample to freeze-thaw testing. The psychrometer measures the suction within the sample continuously during the cyclic freezing and thawing. Volume change is also monitored continuously during freeze-thaw cycling. Figure 3.1 shows the freeze coefficient versus suction (in psi) for a material tested by Carpenter and Lytton (1975). This plot demonstrates the sensitivity of the freeze coefficient to density and molding moisture content. Of course low molding moisture contents are associated with high soil suctions, and high molding moisture contents are associated with low soil suctions. The relationship between soil suction and freeze coefficient is unique for each aggregate or soil type as the suction v. moisture content relationship is unique for each aggregate or soil type. Table 3.1 presents maximum freeze coefficients for six aggregates tested by Carpenter and Lytton (1975). The researchers feel that the determination of the freeze coefficients (related to volume change between freeze-thaw cycles) and residual deformation (total volume change that is not recovered after a number of freeze-thaw cycles - approximately six) under conditions that mimic those typically encountered in the field will provide valuable information on selected aggregate systems.

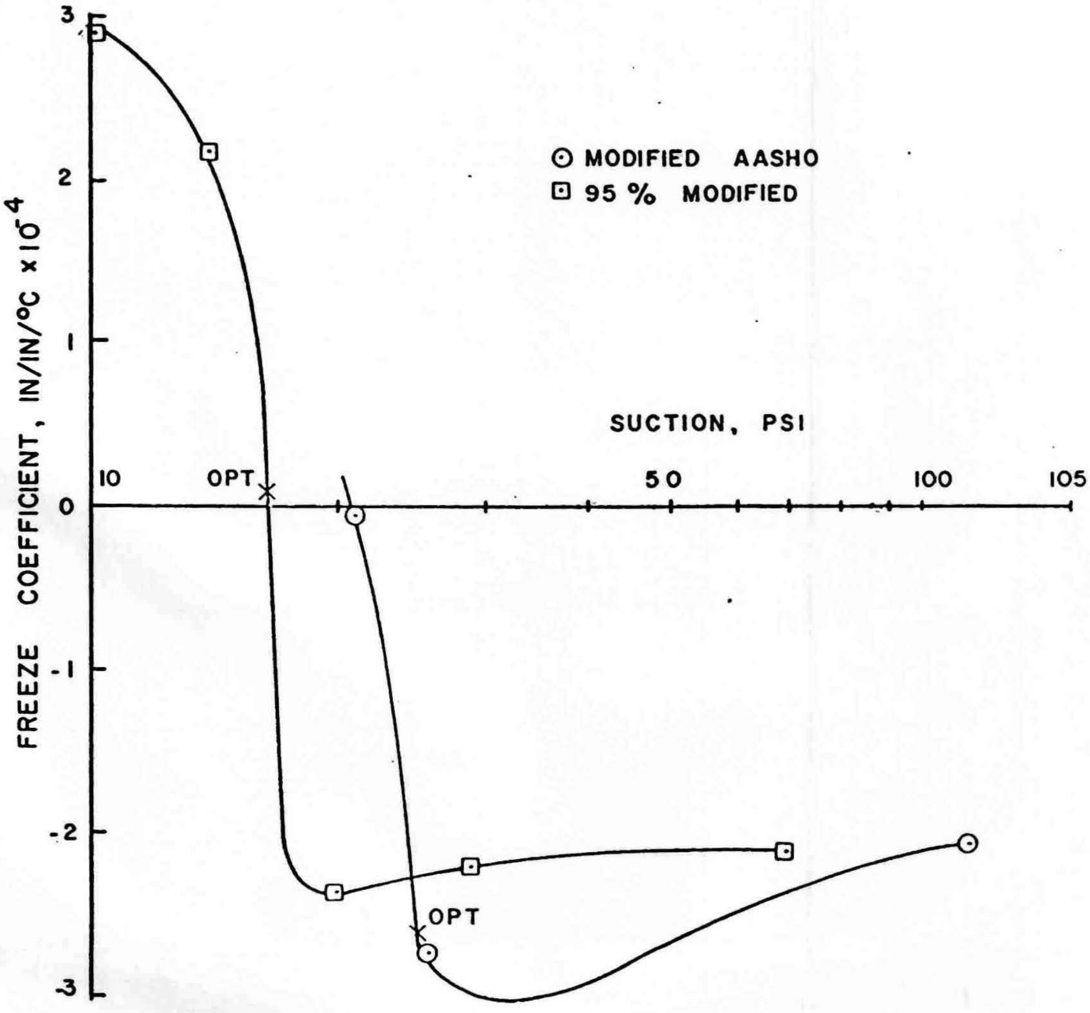


Figure 3.1 Freeze Coefficient as a Function of Suction for Material 4. After Carpenter and Lytton (1975).

Table 3.1. Properties of Base Course Materials Tested (after Carpenter and Lytton, 1975)

Material Number	Specific Gravity	Percent Fines (-#200 sieve)	Liquid Limit (%)	Plastic Limit (%)	Percent Clay (-2μ)	Maximum Freeze Coefficient ($\frac{in/in}{°C}$)
4	2.65	10	30	21	9.0	-2.7×10^{-4}
5	2.68	9	32	21	7.7	-2.5×10^{-4}
6B	2.67	10	27	17	6.2	-1.3×10^{-4}
6JD	2.69	10	22	18	4.3	-4.0×10^{-4}
6FS	2.66	9	17	16	1.6	-0.5×10^{-4}
7SA	2.68	10	22	12	6.5	-1.85×10^{-4}

Saarenketo and Scullion (1995) showed that the dielectric value and electrical conductivity relate to both strength and deformation properties and frost susceptibility of course aggregates. They showed that the dielectric value correlates well with the California Bearing Ratio (CBR) of compacted materials. Low dielectric values (5.5 to 6.5) in compacted samples indicate the presence of thin and well-arranged adsorption water and optimum strength properties. Higher values indicate that material is sensitive to moisture, dielectric values over 9 to 10 are "alarm values" because they have unfrozen water in their structure when the material freezes. If the dielectric value is greater than 16, the base material will become plastic and deformation will occur in the structure. High electrical conductivity values indicate high amounts of ions dissociated to the free water, and this can cause positive pore pressure in base materials. Saturation hysteresis also has a substantial effect on the base strength.

DESCRIPTION OF LABORATORY TESTING PROGRAM

Based on the work of Carpenter and Lytton (1975) and of Saarenketo and Scullion (1995), the dielectric value test was selected in this study as a screening test for aggregate bases. In addition, the Texas triaxial test (TEX 117-E) was selected to monitor strength of each aggregate when compacted at optimum moisture conditions and subjected to a 10-day capillary soak.

The nature of the binder portion of the aggregate matrix has a substantial influence on the behavior of the aggregate system. Carpenter and Lytton (1975) showed that the suction of the binder (minus No. 40 sieve fraction) is highly dependent on the nature of the fines: mineralogy, surface area and fabric. With this in mind, the researchers have identified several tests to characterize the nature of the fines fraction and its contribution to the moisture and freeze-thaw or thermal susceptibility of the aggregate system. Table 3.2 presents the laboratory testing protocol to be performed on the aggregates from the selected field projects. These tests are divided into tests that will be performed on the aggregate system - part one, tests performed on the binder fraction (minim No. 40 sieve size fraction, 420 microns) - part two, tests performed on the fines fractions (minim No. 200 sieve, 74 microns) - part three, and tests performed on the aggregate stabilized with selected additives which are expected to improve performance - part four.

Materials for testing come from selected pavement case study projects and material derived from source pits or quarries within selected districts. The pavement case study projects are discussed in chapter 3 of report 1432-1. All projects selected for laboratory testing, pavement case histories and reference sources, are summarized in table 3.3. Material is being collected from the pavement case histories for lab testing. These materials will be subjected to the protocol established in part one of table 3.2 to determine whether or not the dielectric value and electrical conductivity parameters are effective in screening for performance. Materials from the source pits and quarries will be subjected to more intense testing. Each of these materials will be subjected to the protocol in part one of table 3.2, but selected materials will also be subjected to testing on the binder and fines aggregate fraction (parts two and three of table 3.2). Furthermore, selected aggregates will be modified with various stabilizers to evaluate the effect of stabilization on mineralogical, strength and durability properties (phase four of table 3.2).

The link between laboratory testing results and field performance will be provided by the field test protocol discussed in chapter 4. The link between the complete laboratory testing protocol (phases one through four) on approximately four to six selected aggregates and field performance will be provided by selecting a "young" (one to two year old) pavement for performance monitoring. Performance monitoring of these pavements is discussed in chapter 4.

EVALUATION OF THE EFFECTS OF ADDITIVES ON THE PERFORMANCE OF MOISTURE SUSCEPTIBLE, PROBLEMATIC BASES

Carpenter and Lytton (1975) had moderate success in improving the freeze coefficient as a function of molding moisture content by using selected additives (hydrated lime, KCl and calcium sulfate ($CaSO_4$)). However, the freeze coefficient response was found to be very sensitive to the amount of additive. Nevertheless, the researchers feel that it is imperative to evaluate the influence of low levels of additives on the performance of the bases being evaluated in study 1432. We believe that relatively low levels of chemical additives can substantially influence the mineralogy of the aggregates fines fraction, CEC of the fines, and can provide an improved binder matrix through cementitious and pozzolanic reactions which can not only improve moisture resistance and increase strength, but can also potentially lessen the damaging effects of moisture during a freeze-thaw environment.

Table 3.2 lists the tests that will be used to assess the effect of additives on the performance of selected aggregates. Candidate aggregates for testing following the introduction of stabilizers will be identified in the testing of the untreated aggregates from various sources. Screening testing will be performed (on selected aggregates) using the dielectric value and conductivity tests. This will be followed by thin film petrographic analysis, triaxial testing and freeze coefficient determination at optimum, dry and wet molding moisture states.

Table 2.3. Description and Purpose of Tests Selected for Evaluation of Aggregates Systems

Test Method	Description	Purpose
Tests on the Aggregate System		
Gradation	The representative gradation of each aggregate source is determined and the aggregate is reporportioned in order to meet the gradation requirements for a representative aggregate base for the location in question	To document the gradation of the aggregate being evaluated and to compare with acceptable TxDOT standards for gradation
Moisture-Density Determination	The moisture-density relationship is determined for each aggregate source in accordance with TEX 103-E and 113E	To document the sensitivity of aggregate densification to molding moisture content and to provide the necessary information for fabrication of samples for dielectric values testing and Texas triaxial strength testing
Dielectric Value	Aggregate samples of a representative gradation are fabricated at optimum molding moisture contents. The sample is then oven dried for ** hours at **° C and then subjected to capillary soak by placing the sample in a reservoir of water 20 mm above the base of the compacted sample. The surface dielectric value of the compacted samples is measured as a function of time until approximate equilibrium conditions are reached. Conditions are monitored at 0.5, 1, 2, 4, 8,16 and 24 hours and then every 24 hours until approximately equilibrium is reached.	The dielectric value as a function of time of capillary soak is used to screen the sensitivity of the aggregate to absorb and hold water and for the water to be deleterious to the strength and thermal susceptibility of the aggregate mixture

Table 3.2 Description and Purpose of Tests Selected for Evaluation of Aggregates Systems (continued)

Test Method	Description	Purpose
Electrical Conductivity v. Time of Capillary Soak	Performed concomitantly with the dielectric value test	Electrical conductivity in soils is mostly affected by temperature and concentrations and sizes of the ions in concentrations in the pore water. This information is important as it may help determine the influence of cation concentration in the pore fluid. This information is potentially beneficial especially when judging the effects of stabilizer additives
Moisture Content v. Time of Capillary Soak	Performed concomitantly with the dielectric value test	To monitor moisture content changes in the sample and to compare with dielectric values v. time of testing
Texas Triaxial Strength Test (TEX 117-E)	Aggregate samples of representative gradation are fabricated at optimum moisture conditions and subjected to ten days of capillary soak and then tested a 7 kPa of confining pressure in accordance with TEX 117-E	The triaxial test following capillary soak provides a direct measure of the strength of the aggregate system and can be used to evaluate the suitability of the aggregate in pavement design applications
Freeze Coefficient	Aggregate samples are compacted at specified moisture contents and to specified densities in a foil and wax-sealed system. An imbedded psychrometer continuously monitors soil suction. Volume change is also continuously monitored using a dial gage mounted on a tripod assembly and a micrometer	The freeze coefficient is a measure of volume change within the sample as the temperature drops from 0 to 6.8° C. The volume change may be contraction or expansion depending on the nature of the aggregate, the molding moisture content and the level of compaction. The freeze coefficient and the residual deformation will be determined in this mode of testing only on selected aggregates

Table 3.2. Description and Purpose of Tests Selected for Evaluation of Aggregates Systems (continued)

Test Method	Description	Purpose
Tests on the Binder (Minus No. 40 Sieve, Minus 420 Microns)		
Atterberg Limits	Determine plastic limit, liquid limit and plasticity index in accordance with TEX ****	Atterberg limits data provides basic information about clay mineralogy, swell potential and general mineralogical activity
X-Ray Diffraction (XRD) of the Bulk Minus 40 Sieve Size Sample	Screen the binder fraction of the aggregate with X-ray diffraction	Provide a general "finger print" of the mineralogy of the binder fraction
Surface Area Determination	Test performed using the QUANTASORBE JR.BET surface area analyzer in the Department of Geology at Texas A&M University	Provide a general determination of the available surface area of the binder fraction
Tests on Fines (Minus No. 200 Sieve, 74 Microns)		
Surface Area Determination	As discussed above	Provide a determination of the available surface area of the fines fraction
XRD of Clay Fraction	Screen the clay (minus 2 micron) size fraction of the aggregate with X-ray diffraction	Provide a "finger print" of the mineralogy of the clay fraction
Cation Exchange Capacity		Determine capacity for cation exchange defines surface activity and potential for effective stabilization of the fines fraction
Tests on Selected Aggregate System with Selected Additives		
Dielectric Value	Discussed previously	Discussed previously
Texas Triaxial Strength	Discussed previously	Discussed previously

Table 3.2. Description and Purpose of Tests Selected for Evaluation of Aggregates Systems (continued)

Test Method	Description	Purpose
Freeze Coefficient	Discussed previously	Discussed previously. The effects of stabilizers on the freeze coefficient will be monitored at selected compaction efforts and molding moisture contents on selected aggregates and compared with results on these aggregates without stabilizer addition
Petrographic and EPMA Analysis of Thin Sections	Thin sections of the stabilized aggregate are carefully prepared and vacuum impregnated with blue, fluorescent epoxy to enhance visibility of pores and fractures for light microcopy analysis. This is followed by probing the sample with an EPMA energy source	The petrographic analysis will identify how the additives have changed the fabric of the fines and any minerals that have been produced as a result of the stabilization (both helpful and potentially deleterious). The electron probe analysis (EPMA) utilizes wavelength dispersive analysis to evaluate the type of chemical reaction products produced during stabilization and the extent of stabilization product development. The petrographic and EPMA probe analysis will be performed on thin sections before and following stabilization

Table 3.3 Aggregates Selected for Laboratory Testing from Field Sources and Actual Pavement Case Histories

Project Code	Designation of Material: Pavement Case History or Source Material	District/Highway/ Direction	Pavement Structure of Other Source Description
AM1	Pavement Case History	Amarillo, US 87/287, NB Divided, two lanes in each direction	37 mm Asphalt 200 mm Gravel Base 100 mm Gravel Subbase 150 mm Lime Stab. Subgrade (1.5% Lime)
AM2	Pavement Case History	Amarillo, FM 1541, NB/SB Undivided, one lane in each direction	37 mm Asphalt 280 mm Base, Caliche
AB1	Pavement Case History	Abilene, US 83, SB Divided, two lanes in each direction	50 mm Asphalt 200 mm Limestone Base 150 Subbase
AB2	Pavement Case History	Abilene, US 83 BU, NB Undivided, two lanes in each direction	37 mm Asphalt 300 mm Base, Limestone
SA1	Pavement Case History	San Angelo, US 67, NB Divided, two lanes in each direction	76 mm Asphalt 320 mm Base, Limestone 200 mm Subgrade, 2% Limestone
SA2	Pavement Case History	San Angelo, SH 208, NB Undivided, two lanes in each direction	37 mm Asphalt 165 mm Base, Limestone 100 mm Subgrade

Table 3.3 Aggregates Selected for Laboratory Testing from Field Sources and Actual Pavement Case Histories (continued)

Project Code	Designation of Material: Pavement Case History or Source Material	District/Highway/ Direction	Pavement Structure of Other Source Description
AT1	Pavement Case History	Atlanta, SH 08, NB/SB Undivided, one lane in each direction	50 mm Asphalt 250 mm Base, Gravel with 1% Lime and 2% Fly Ash
YO1	Pavement Case History	Yoakum, US 290, WB Divided, two lanes in each direction	63 mm Asphalt 200 mm Base, Limestone 300 mm Subbase, Gravel, with 1.5% Lime 150 Subgrade, with 4% Lime
YO2	Pavement Case History	Yoakum, LP 463, EB/WB Undivided, one lane in each direction	37 mm Asphalt 355 mm Base, Gravel with 2% Lime 200 mm Subgrade, with 5% Lime
PH1	Pavement Case History	Pharr, US 281, NB Divided, two lanes in each direction	50 mm Asphalt 250 mm Base, Caliche 255 mm Subgrade with 3% Lime
PH2	Pavement Case History	Pharr, FM 2128, EB Undivided, two lanes in each direction	50 mm Asphalt 200 mm Caliche, with 1% Lime 255 mm Subgrade with 3% Lime
AMB	Source material from the Amarillo District - Buckles Pit	FM 297 Two lanes from US 287 to Dallas County Line	37 mm Asphalt 250 mm Base (Caliche)

Table 3.3 Aggregates Selected for Laboratory Testing from Field Sources and Actual Pavement Case Histories (continued)

Project Code	Designation of Material: Pavement Case History or Source Material	District/Highway/ Direction	Pavement Structure of Other Source Description
AMC	Source material from the Amarillo District - Coon Pit	SH 136 Four lanes from Fritch to FM 1319	37 mm Asphalt 250 mm Base (River Gravel)
AMJ	Source material from the Amarillo District - Johnson Pit	Coulter Street Amarillo between 45th and Hillside SB lanes	River Gravel
AML	Source material from the Amarillo District - Linsey Pit	North Hughes Street Amarillo CSJ 904-2-19	River Gravel
ABC	Source material from the Abilene District - Clements Pit	NA	Caliche
ABJ	Source material from the Abilene District - Jordan Pit	NA	Caliche
ABK	Source material from the Abilene District - Kemper Pit	NA	Caliche
ABP	Source material from the Abilene District - Parmley Pit	NA	Caliche
ABT	Source material from the Abilene District - Tubbs Pit	NA	Caliche
SAM	Source material from the San Angelo Mayer Pit	NA	Limestone source

Table 3.3 Aggregates Selected for Laboratory Testing from Field Sources and Actual Pavement Case Histories (continued)

Project Code	Designation of Material: Pavement Case History or Source Material	District/Highway/ Direction	Pavement Structure of Other Source Description
YOV	Source material from the Yoakum District - Victoria	NA	Silicious gravel with clay binder treated with hydrated lime
LSB	TTI Control limestone base - TTI	NA	Standard limestone used for aggregate and asphalt mixture testing at TTI
ABL (1)	Pavement Case History	US 83	Caliche
ABL (2)	Pavement Case History	US 83	Caliche
ABL (3)	Pavement Case History	US 83	Caliche

CHAPTER 4: FIELD TESTING PROTOCOL

GENERAL

Pavement case histories have been selected for laboratory testing and field observation of their performance. The pavements selected for case history evaluation were based on considerations of temperature and moisture conditions. Subgrade type was also included in the factorial since it has an indirect effect on the occurrence and propagation of cracks in aggregate bases. The experiment design used to screen districts within Texas for candidate pavement sections is explained in chapter 3 of report 1432-1. Based on the experiment design discussed in report 1432-1 and screening parameters of temperature (freeze v. no freeze), moisture condition (wet v. dry) and subgrade soil type, the potential districts are presented in figure 4.1. Figure 4.2 presents the number of case history pavement projects selected for each cell. These pavement case history projects are summarized in the first eleven rows of table 3.2.

SELECTION OF THE FIELD TESTING PROGRAM

The purpose of the field testing program is to determine in situ properties of the aggregate bases at the selected sites and to record pavement distress by visual mapping at the selected sites. The condition survey analysis is explained in report 1432-1 and the results of the initial condition surveys for the eleven case history projects are presented in report 1432-1. In addition to the visual condition surveys it is necessary to determine certain properties of the in situ materials: moisture content profiles of the base material during extreme periods of the year (wettest and driest - as determined by Ground Penetrating Radar (GPR)), material properties during the extreme periods of the year (i.e., Falling Weight Deflectometer (FWD)-derived resilient moduli, FWD-derived load transfer efficiency across transverse cracks in the cold period of the year and strength as determined by an in situ device such as the Dynamic Cone Penetrometer (DCP).

Table 4.1 presents the selected field testing protocol which supplements the laboratory testing program for the selected pavement case histories. We should note that in addition to these field case history sites, we may select between five and ten additional pavement case history sites which have very new pavements constructed. The new pavement sections will allow us to monitor early distress in the aggregate bases.

Table 4.1. Field Testing Procedures Selected to Monitor and Evaluate Pavement Case Histories

Test Identification	Description of Test	Purpose of Test
Ground Penetration Radar (GPR) Analysis	———————	Determine the moisture content profile within the granular base layers during wet and dry seasons
FWD Evaluation	The FWD analysis is determined during the wet and cool period of the year.	Determine in situ layer moduli of the base materials and the load transfer effeciency across transverse cracks in the pavement section
DCP Evaluation	Determine DCP profiles throughout the thickness of the granular base layer during the wet and dry seasons of the year	Identify the strength profile within each pavement layer

Temperature	Freeze		No Freeze	
Moisture	Wet	Dry	Wet	Dry
Soil Classification				
Clay	1 Paris	5 Waco	9 Yoakum Houston	13 Pharr Austin
Clay Loam	2 Paris Atlanta	6 Amarillo	10 Yoakum Houston Bryan	14 Yoakum Pharr
Loam	3 *	7 Amarillo San Angelo Abiline	11 *	15 Pharr Austin
Sandy Loam	4 *	8 El Paso	12 *	16 El Paso

Note: * No district in these cells

Figure 4.1 Potential Texas Districts for Case Study Projects Selection.

Temperature / Moisture / Soil Classification	Freeze		No Freeze	
	Wet	Dry	Wet	Dry
Clay	1 **	5 **	9 One Project	13 **
Clay Loam	2 One Project	6 **	10 **	14 One Project
Loam	3 *	7 Six Projects	11 *	15 Two Projects
Sandy Loam	4 *	8 **	12 *	16 **

Note: * No district in these cells

** Case study projects are not available

Figure 4.2 Factorial Cells Showing the Number of Selected Case Study Projects.

CHAPTER 5: INTERIM RESULTS AND FINDINGS OF THE LABORATORY TESTING PROGRAM

GENERAL

The laboratory testing program is discussed in chapter 3. The protocol for testing and the purpose of the tests are summarized in table 3.2. Figures 5.1, 5.2 and 5.3 illustrate the flow and sequence of the laboratory testing program.

The researchers have gathered aggregates from four districts and from fourteen different sources. These sources and aggregate types are summarized in table 3.2. In addition to the selected aggregate sources, we have identified selected pavement sections within the four districts for field testing, performance survey evaluation and monitoring. These sections are also summarized in table 3.2. Details on how the source materials and pavement monitor sections were selected are presented in Report 1432.1. We will identify additional young pavement sections (besides those presented in table 3.2) for field monitoring. We will endeavor to monitor as many pavement sections as possible that are less than two years old and whose bases are constructed of material from the fourteen different sources. These pavements are now being identified. It is imperative to monitor more young pavement sections in order to evaluate the rate of occurrence and severity of early pavement cracking.

Lab Testing of Source Materials

Figure 5.1 presents the laboratory testing sequence for source materials. These materials are divided into three systems (as presented in table 3.2): total aggregate system, binder fraction (minus No. 40 sieve fraction) and fines fraction (minus No. 200 sieve fraction). All of the aggregate tests shown in figure 5.1 will be performed on the total aggregate from each source. X-ray diffraction (XRD) on the bulk sample, Atterberg limits and cation exchange capacity (CEC) will be performed on the finder fraction of each aggregate system. These will be identified following subsequent to XRD (bulk), Atterberg limits and CEC testing. XRD testing will be performed on the clay fraction of each aggregate system. However, surface area analysis will only

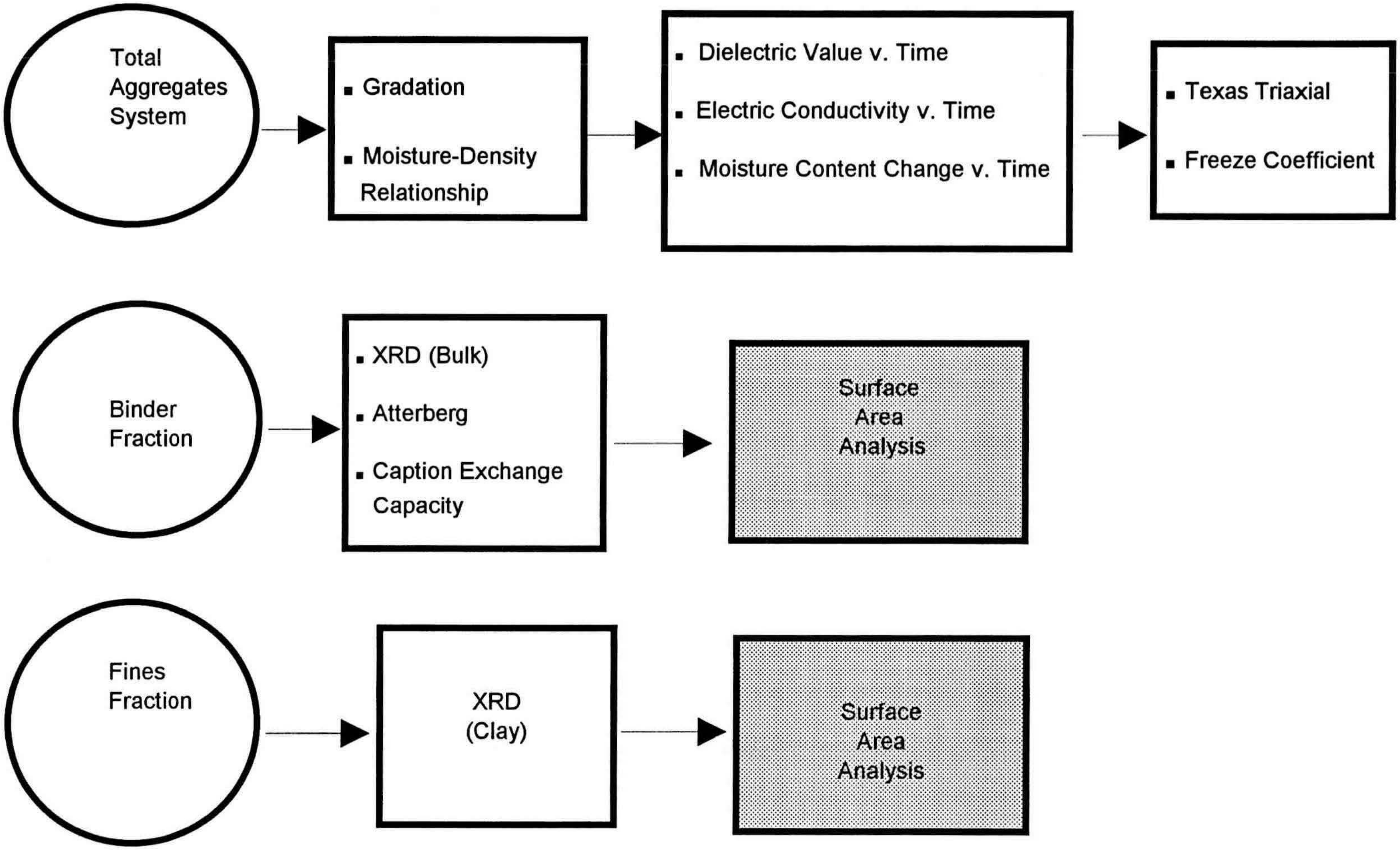


Figure 5.1 Laboratory Testing Sequence on Source Sample (From Amarillo District (4), Abilene (6), San Angelo (2) and Yoakum (2)).

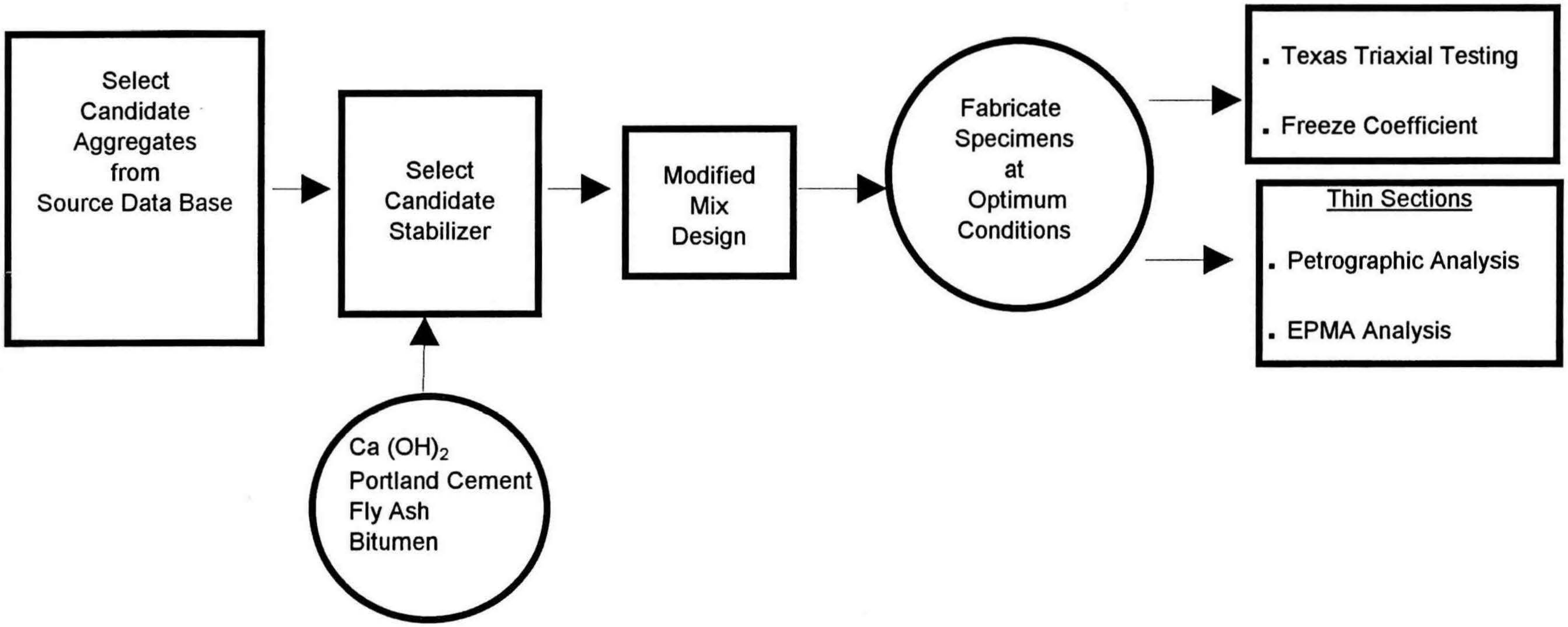


Figure 5.2 Laboratory Analysis of Stabilized Aggregate System.

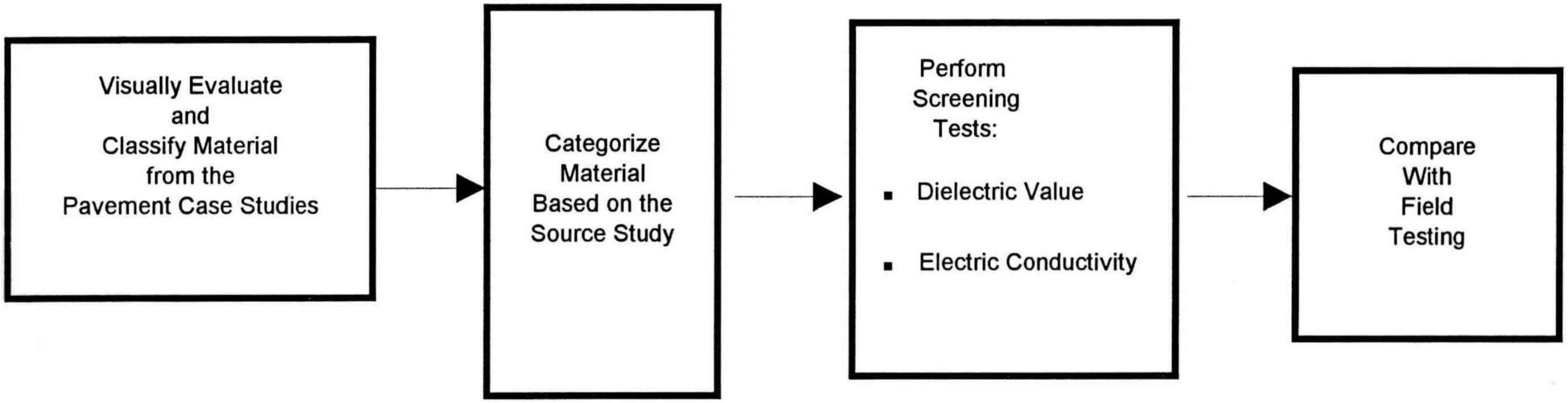


Figure 5.3 Flow Chart for Laboratory Testing of Pavement Case Studies.

be performed on the binder and clay fractions of selected source material. The purpose for testing of the source materials is to develop a data base of pertinent strength, mineralogical and volumetric change properties which will provide the basis for correlations with field performance and for the development of specifications and guidelines for improved aggregate base performance, especially with respect to early-life cracking.

Laboratory Analysis of Stabilized Aggregate Systems

Stabilization with traditional chemical and bituminous-based stabilizers offers an attractive and cost effective potential to reduce improved strength, reduce the potential for catastrophic strength and stiffness loss due to moisture and freeze-thaw activity and reduce volumetric change problems related to thermal activity. Figure 5.2 presents the scheme of testing for selected aggregate systems from the 14 source aggregates. Traditional stabilizers including hydrated lime, portland cement, fly ash (perhaps with lime) and bitumen will be added at relatively low levels to enhance performance of the aggregate systems. A modified mixture design will be used in an effort to provide enough of the chemical and calcium-based stabilizers to the system to insure permanency (durability) yet not so much as to produce a rigid system. Optimum for lime will be based on a pH test to ascertain the minimum amount of stabilizer to satisfy initial and secondary cation exchange-based reactions yet provide enough residual lime for pozzolanic, strength gain reactions.

Texas triaxial strength tests, freeze coefficient volumetric tests during the freezing operation and petrographic evaluation and elemental analysis (EPMA) of thin sections of the stabilized material will be performed following stabilization and curing. These tests will identify the ability of the stabilizers to reduce strength loss upon moisture intake (Texas triaxial), reduce deleterious volumetric effects (freeze coefficient) and promote development of a pozzolanic, cementitious or bituminous matrix within the aggregate system (petrographic and EPMA analysis).

Laboratory Test Support for Pavement Case Histories

Figure 5.3 presents the laboratory testing scheme for material collected from specific pavement case histories which will be monitored for performance. The sequence will be to visually evaluate and classify which source material it came from or which source material to which it is most similar. Screening tests will include: gradation, Atterberg limits and XRD (bulk). The moisture-density relationship will be determined for each material. Samples will then be fabricated at optimum conditions. Dielectric values and electric conductivity will be determined on each sample. These data will be compared to the results of field testing. Field testing will include FWD testing to determine moduli in wet and dry periods of the year, GPR analysis to determine the amount of moisture held by the base in the wet and dry periods of the year, DCP analysis of the base strength in the wet and dry periods of the year and field condition surveys during wet and dry periods of the year.

ILLUSTRATION OF THE USE OF DIELECTRIC VALUE SCREENING TESTS FOR AMARILLO AND ABILENE DISTRICTS

One of the main objectives of study 1432 is to evaluate if the proposed suction/dielectric test is a good indicator of an aggregate's ability to withstand freeze/thaw damage. To accomplish this, the researchers propose to test a range of base course material from Districts in the freeze/thaw areas of Texas, then to locate recently constructed projects containing these aggregates and monitor their performance. This monitoring will include distress surveys as well as nondestructive testing with Falling Weight Deflectometers. The first step in this process is to rank aggregates in the laboratory.

Laboratory test results from the Amarillo and Abilene Districts are shown in figures 5.4 and 5.5. Results from four pits in the Amarillo District, (Coon, Lindsey, Johnson and Buckles Pit) are shown in figure 5.4. The Coon and Johnson materials are sandy river gravels, the Buckles and Lindsey are caliche/limestone materials. The criteria dielectric/suction proposed to evaluate materials is as follows;

a) If the final asymptotic dielectric value is greater than 16 then this material will be prone to freeze/thaw damage,

b) If the dielectric value increases by over a factor of 2 in the first 24 hours then this material has high suction and will draw in any available moisture,

The most critical requirement is a), as it is a measure of how tightly bound free water is within the aggregate base. If the asymptotic dielectric is high then the amount of water available to form ice lenses will also be high. The worst case scenario is for a material to fail both criteria. In this case, the base course will have the capability to "self-destruct".

With the material from Amarillo, the best performers were the two river gravel materials from the Coon and Johnson pits. Both materials passed the criteria; they have very low suction values and absorb little moisture. However, both of the Caliche materials failed criteria a) with final dielectric values of above 20. Both materials passed criteria b). For the Amarillo aggregates it is concluded that the Coon and Johnson materials should be good performers and not prone to moisture-induced damage. On the other hand, the Buckles and Lindsay pit may have a tendency to deteriorate if moisture is freely available to enter the base layer. This moisture could be from snow melt, high ground water table or unsealed surface cracks. Both the Buckles and Lindsay materials are good candidates for chemical treatment.

The results for the Abilene materials are shown in figure 5.5, the ranking of best to worst performance for these materials is as follows;

- Kemper Pit (best),
- Clements Pit,
- Tubbs Pit,
- Jordan Pit, and
- Parmley Pit (worst).

The Parmley pit material failed both criteria so this material will attract and hold moisture from the subgrade, surface cracks or the shoulder. It is anticipated that this material will exhibit a high dielectric if tested in the field with GPR. The most interesting material is the Jordan Caliche aggregate. The surface dielectric was very low for the first 89 hours of the test. The measured value then rose dramatically to over 40, and the top of the base became plastic. This implies that if this material is used in a dry environment with good drainage, the satisfactory performance may be obtained but if there is any chance of the base becoming saturated then rapid failure may occur. For the Abilene materials, the benefit of chemical stabilization should be studied for the Tubbs, Jordan and Parmley materials.

Appendices A through E present gradations, density v. molding moisture content relationships, dielectric value v. time of capillary soak, electric conductivity v. time of capillary soak and moisture content v. time of capillary soak, respectively, for the source aggregate materials for this study. Figures 5.6, 5.7 and 5.8 summarize the ultimate (maximum values after capillary soak) dielectric values, electric conductivities and moisture contents, respectively, for each source binder.

MINERALOGICAL PROPERTIES

The mineralogy of each aggregate is critically important and influential in determining the properties of the aggregates. XRD analysis was performed on bulk and clay fractions of each source aggregate. XRD analysis allows us to identify the important minerals within the binder and clay-size fraction. Appendices F and G will present these data for source aggregates tested to date using XRD.

Appendix H presents CEC data on source aggregates tested to date, and appendix I presents Atterberg limits data on aggregates tested to date.

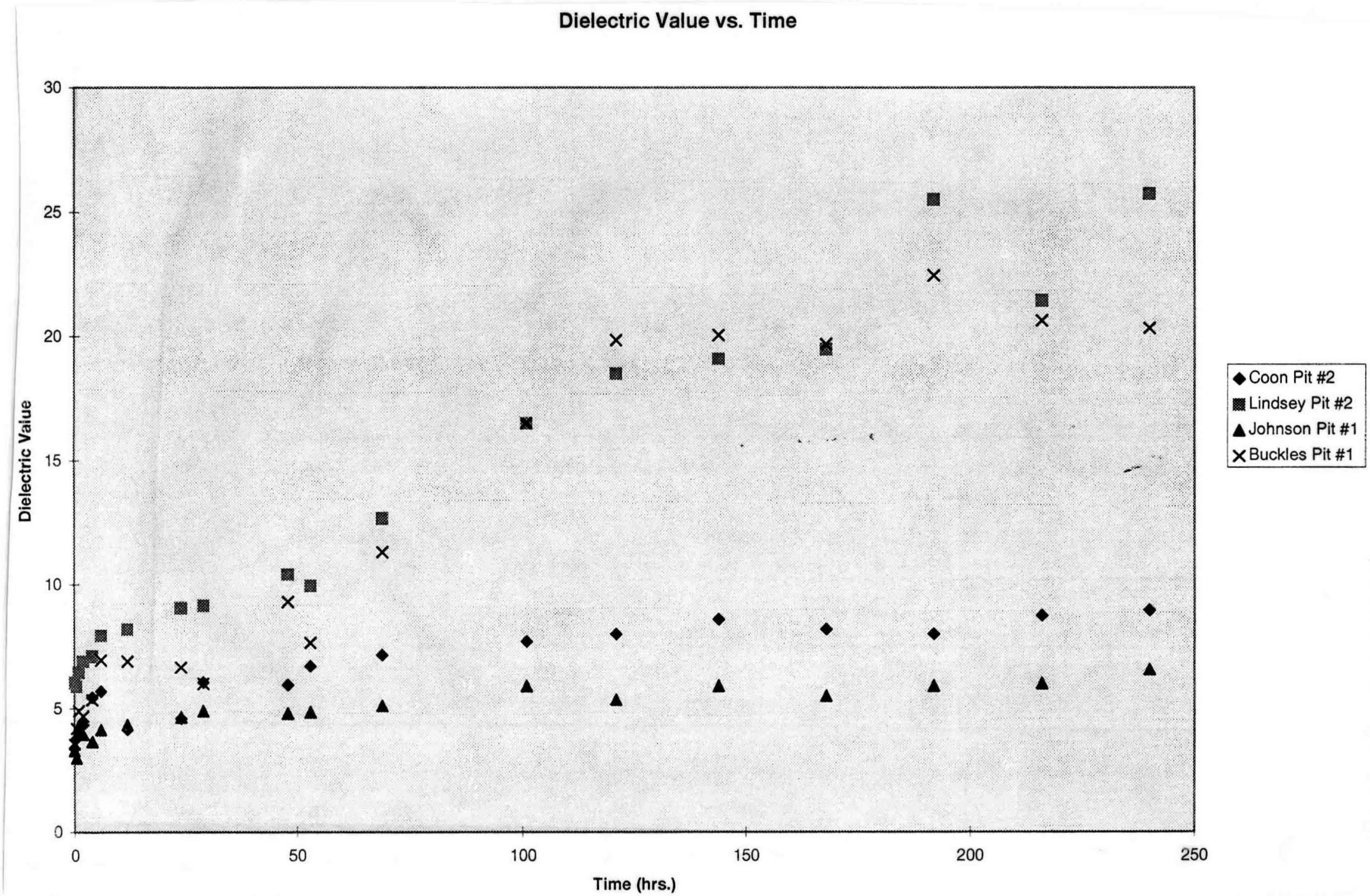


Figure 5.4 Dielectric Value v. Time of Testing for Amarillo Aggregates.

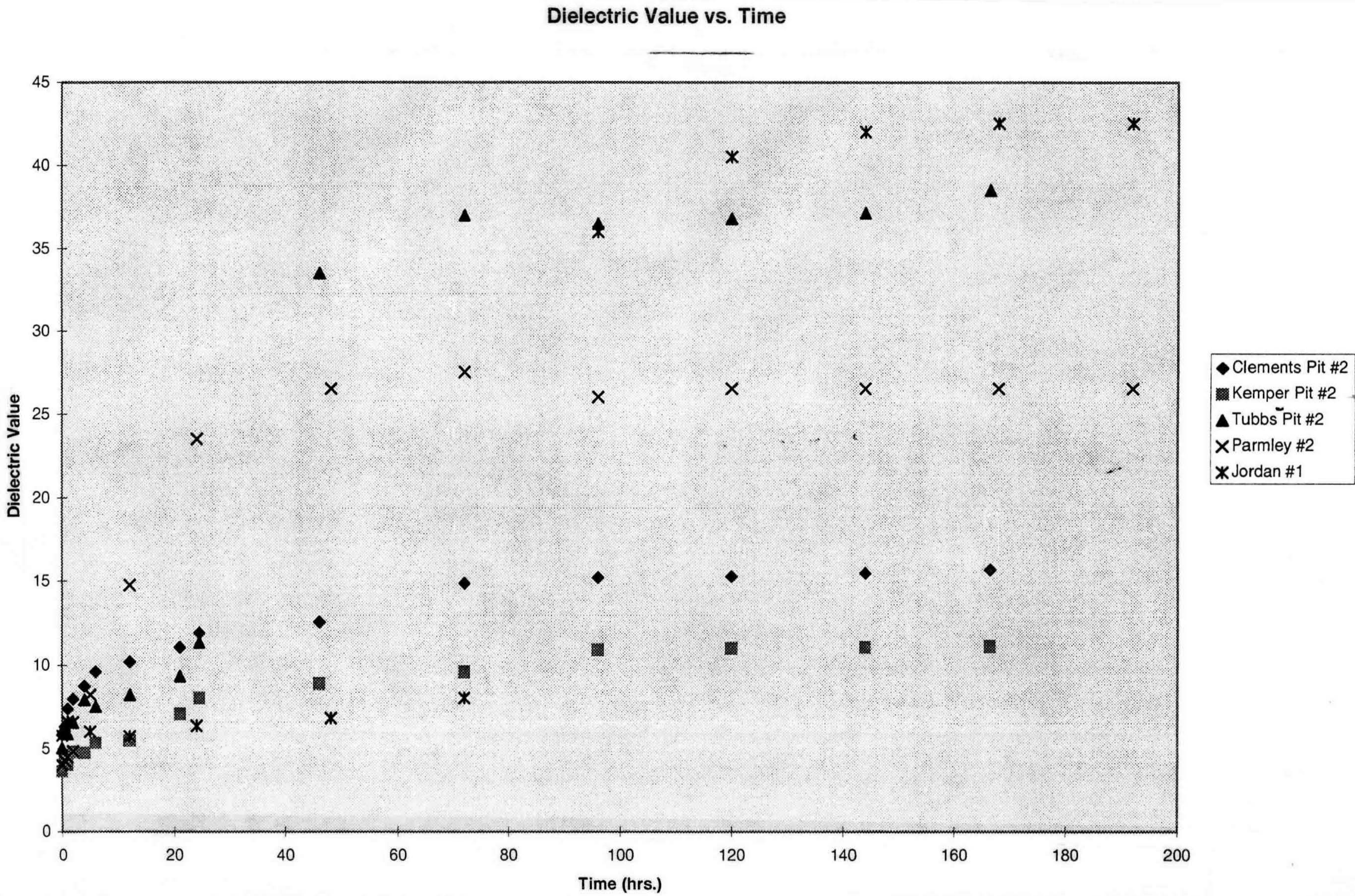


Figure 5.5 Dielectric Value v. Time of Testing for Abilene Aggregates.

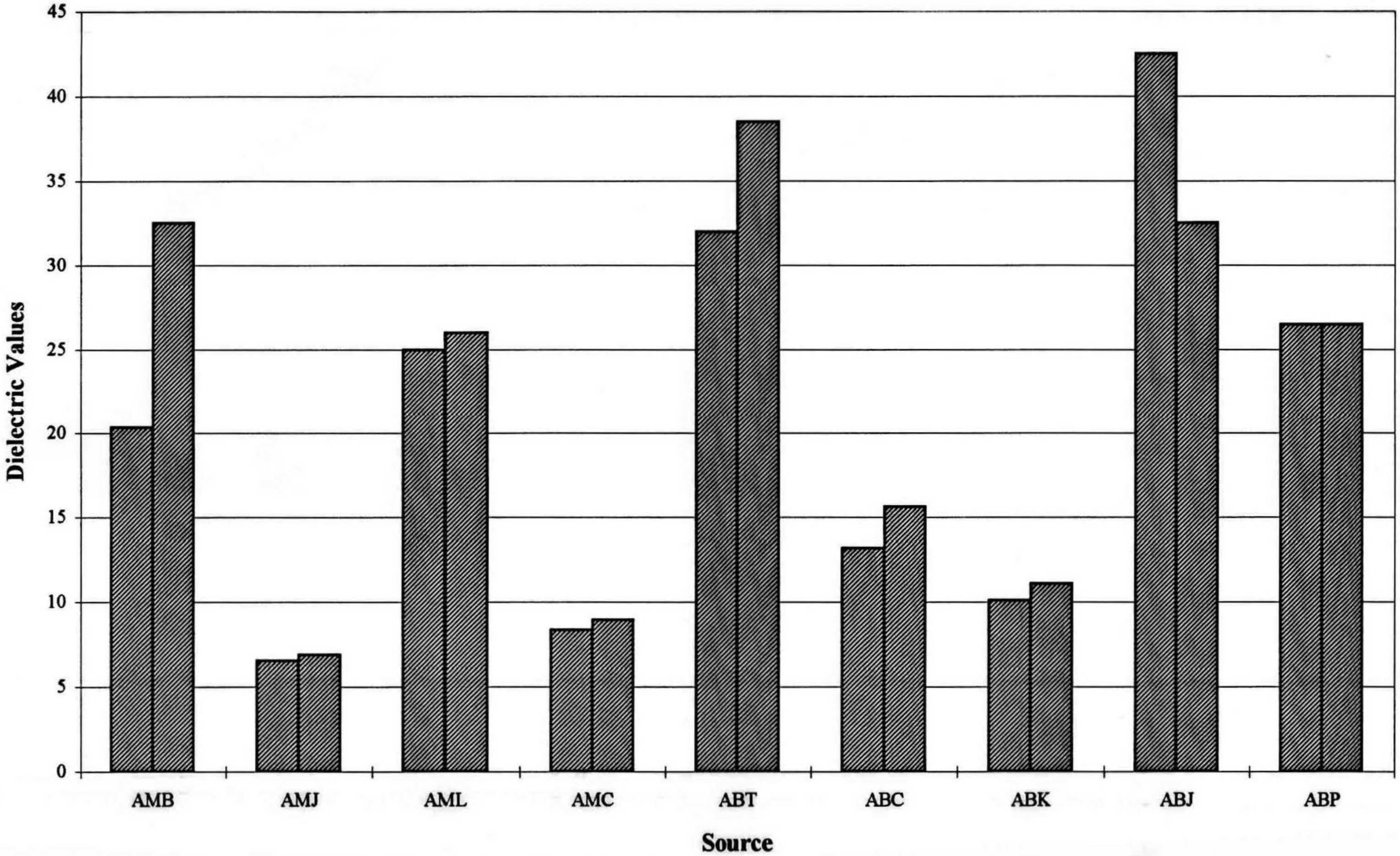


Figure 5.6 Summary of Ultimate Dielectric Values for Amarillo and Abilene Sources.

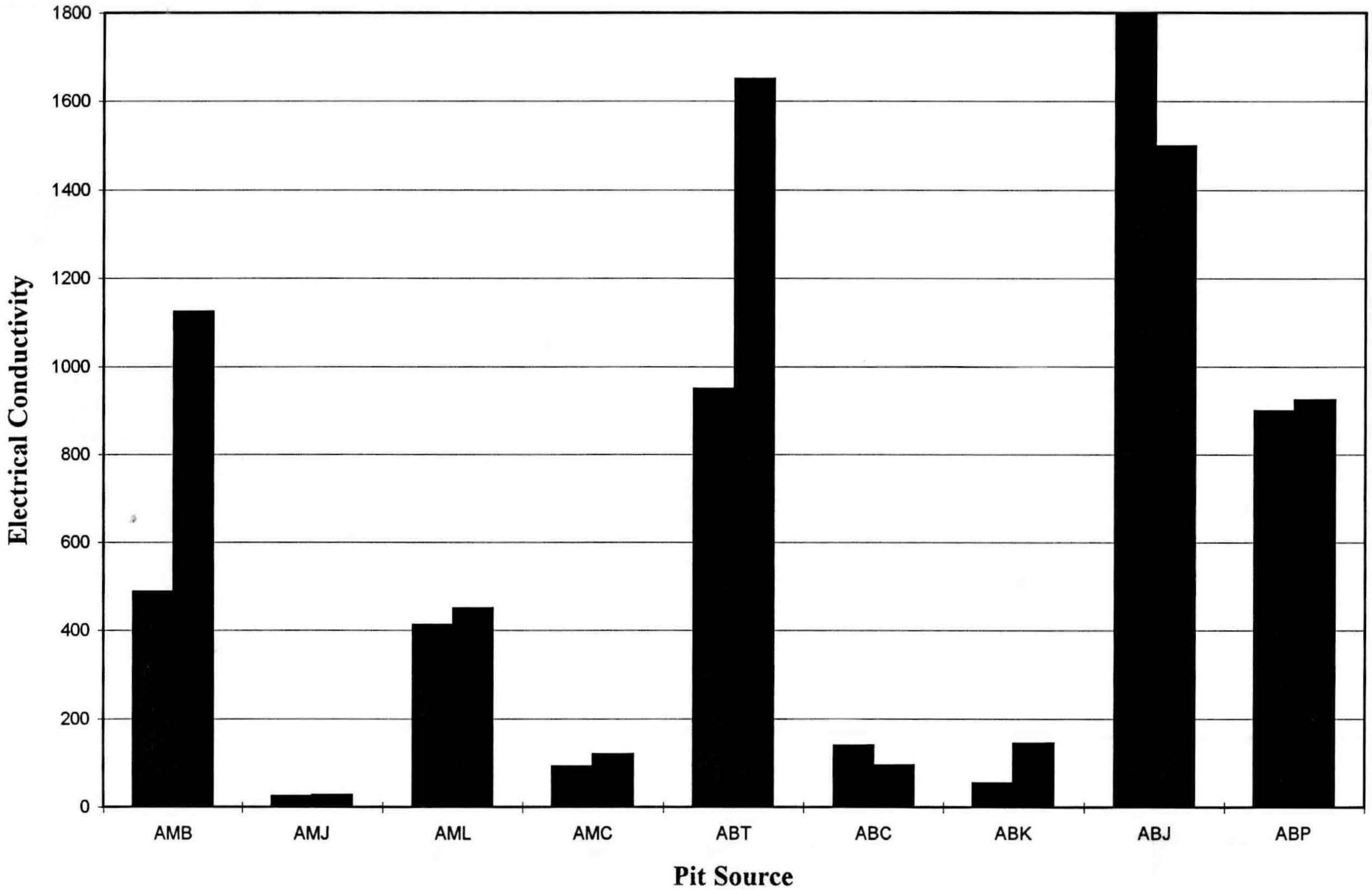


Figure 5.7 Ultimate Electrical Conductivity for Amarillo and Abilene Source Aggregates.

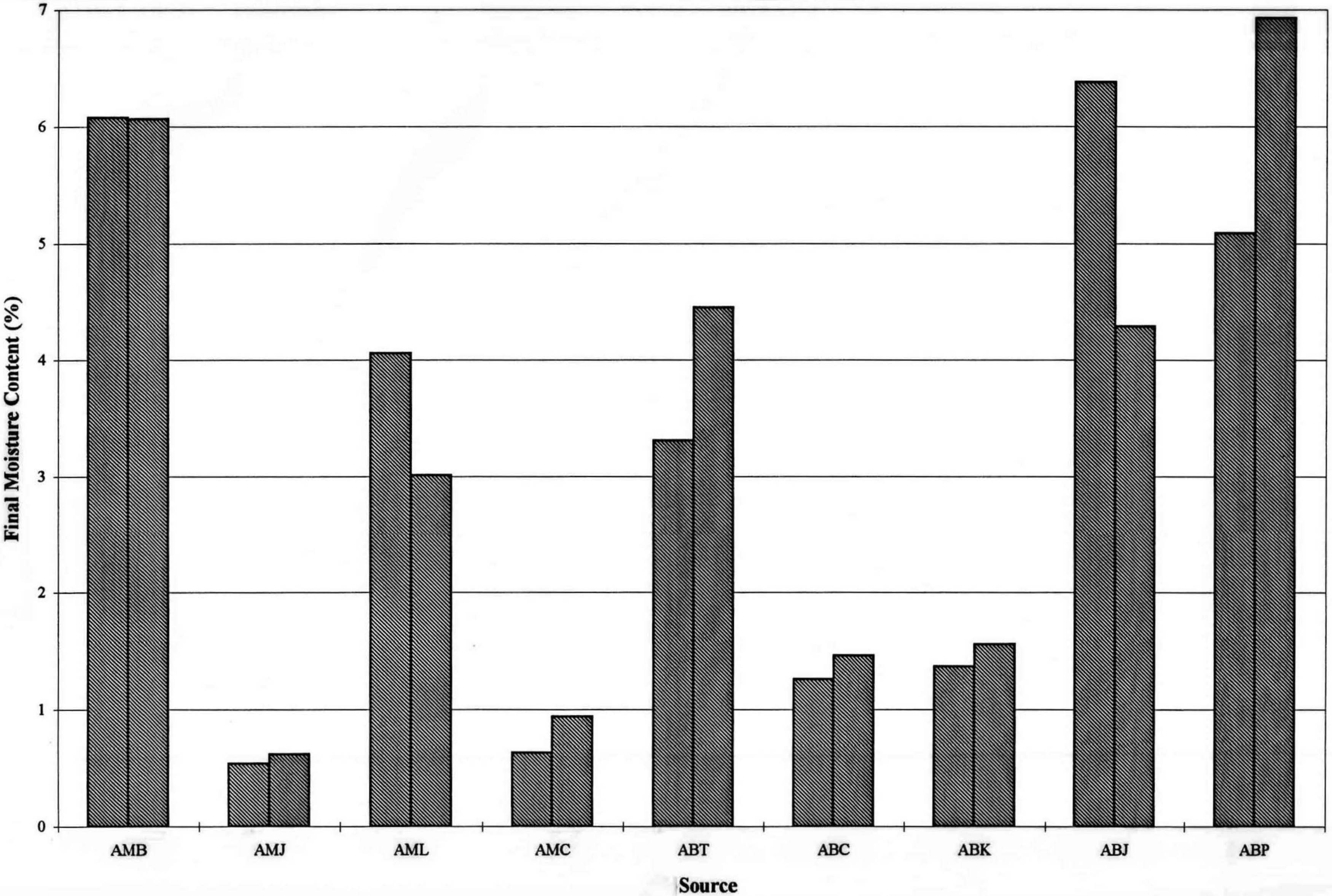


Figure 5.8 Ultimate Moisture Contents for Amarillo and Abilene Sources.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Both the lab and field testing programs are well underway. The lab and field testing program for aggregates taken from pavement case history sites and from aggregate source pits and quarries is very comprehensive and will provide the data on mineralogical, strength, moisture and freeze-thaw susceptibility necessary to evaluate the moisture sensitivity and thermal volumetric change sensitivity of these aggregates.

The dielectric value test is a good screening technique by which to identify the sensitivity of aggregates to moisture, freeze-thaw activity and thermally induced volumetric changes. This technique has been able to differentiate between poorly and well-performing aggregates in the Amarillo and Abilene Districts.

The correlations between field performance and laboratory characterization of the aggregates selected for study will allow the researchers to develop design specifications which will minimize the effects of moisture on strength and stiffness loss and will minimize thermally induced cracking. The correlations will also allow the researchers to develop improved construction specifications for flexible bases, which account for such factors as moisture content, compaction density and other construction design parameters.

The determination of the effect of stabilizer type and amount on strength, dielectric value, electrical conductivity, petrography, mineralogy, freeze coefficient and strength will allow the researchers to prescribe rehabilitation methods and new construction methods to minimize the deleterious effects caused by strength loss due to moisture-and thermally induced volume changes within aggregate bases.

RECOMMENDATIONS

The research approach established on pages 1.2 to 1.4 is sound and should be followed until completion. The detailed laboratory and field testing approach established in chapters 3 and

4 should be followed as outlined as these tests are theoretically appropriate, sensitive to material variation and can be performed on a reasonable sample set to provide sufficient data for the successful accomplishment of the objectives of project 1432.

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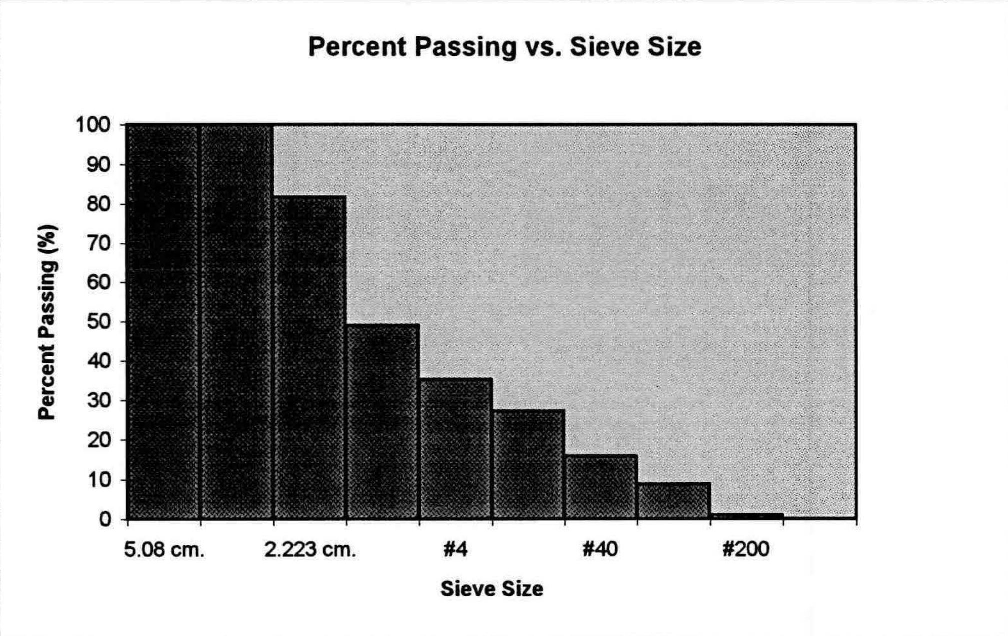
APPENDIX A:

GRADATIONS OF THE VARIOUS SOURCE MATERIALS EVALUATED IN THE PHASE 1 TESTING PROGRAM

Sieve Analysis
for
Buckles Pit

Weight of sample(g) - 2375.1

Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	0	100
2.223 cm.	431.7	81.82
0.953 cm.	780.2	48.97
#4	324.9	35.30
#10	192.4	27.19
#40	265.1	16.03
#80	170.3	8.86
#200	184.3	1.10
<#200	23.5	0.11

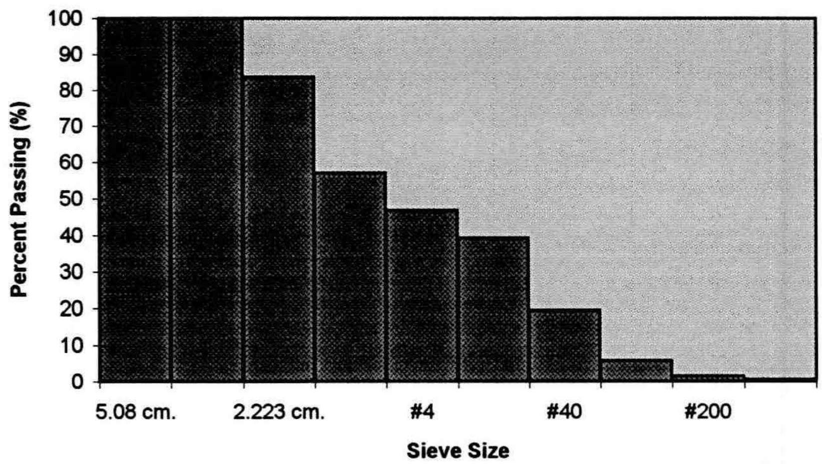


Sieve Analysis
for
Coon Pit

Weight of sample(g) - 2746

Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	0	100
2.223 cm.	444.9	83.80
0.953 cm.	731.2	57.17
#4	282.1	46.90
#10	210.2	39.24
#40	545.2	19.39
#80	371.5	5.86
#200	116.3	1.62
<#200	25.3	0.70

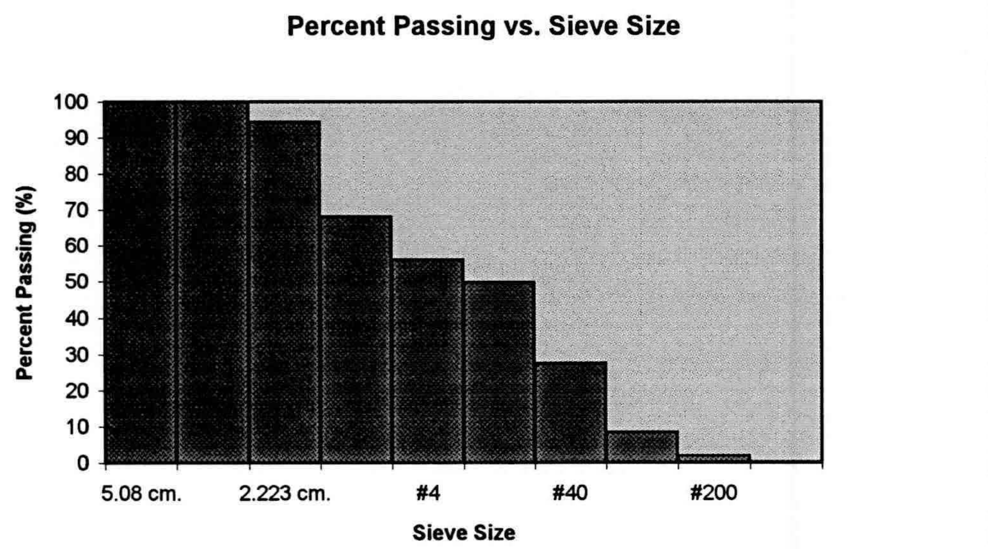
Percent Passing vs. Sieve Size



Sieve Analysis
for
Johnson Pit

Weight of sample(g) - 2145

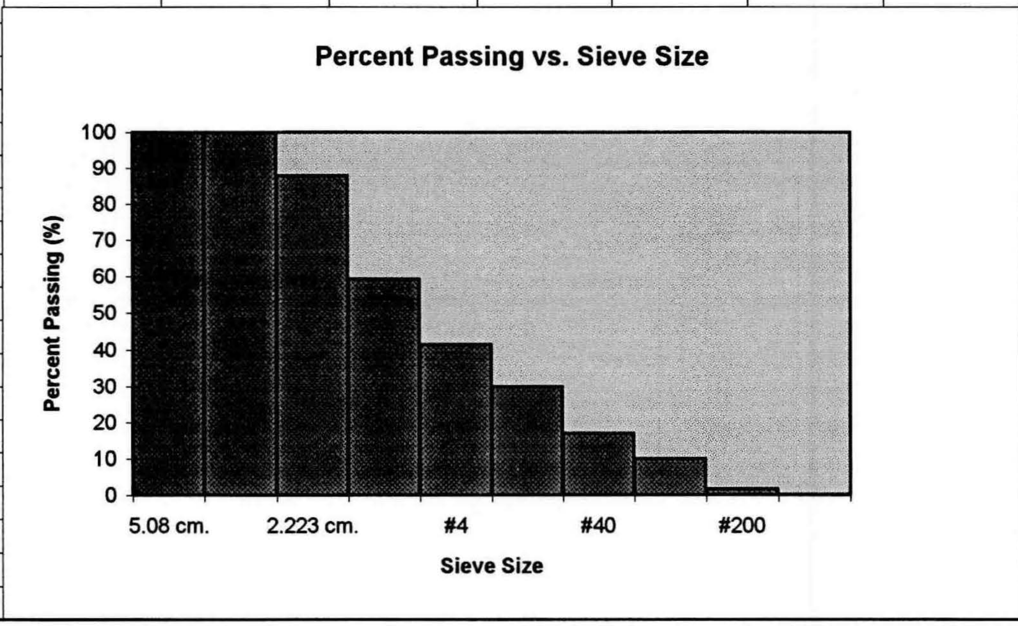
Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	0	100
2.223 cm.	121.7	94.33
.0953 cm.	561.3	68.16
#4	259.3	56.07
#10	136.4	49.71
#40	477	27.47
#80	409.1	8.40
#200	139.7	1.89
<#200	35.6	0.23



Sieve Analysis
for
Lindsey Pit

Weight of sample(g) - 2325.8

Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	0	100
2.223 cm.	279.6	87.98
0.953 cm.	666.1	59.34
#4	413.4	41.56
#10	269	30.00
#40	302.1	17.01
#80	163.7	9.97
#200	192.9	1.68
<#200	36.7	0.10

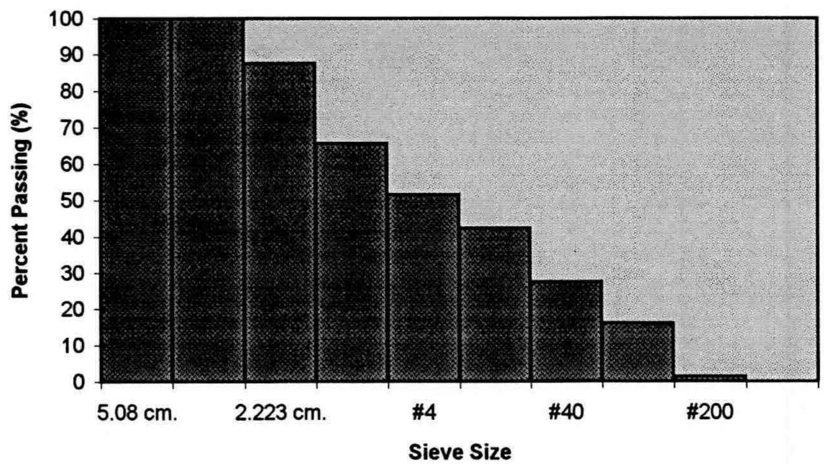


Sieve Analysis
for
Clements Pit

Weight of sample(g) - 2411.7

Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	0	100.00
2.223 cm.	298.2	87.64
0.953 cm.	527.8	65.75
#4	341.1	51.61
#10	221.8	42.41
#40	358.4	27.55
#80	277.4	16.05
#200	354.2	1.36
<#200	32.5	0.01

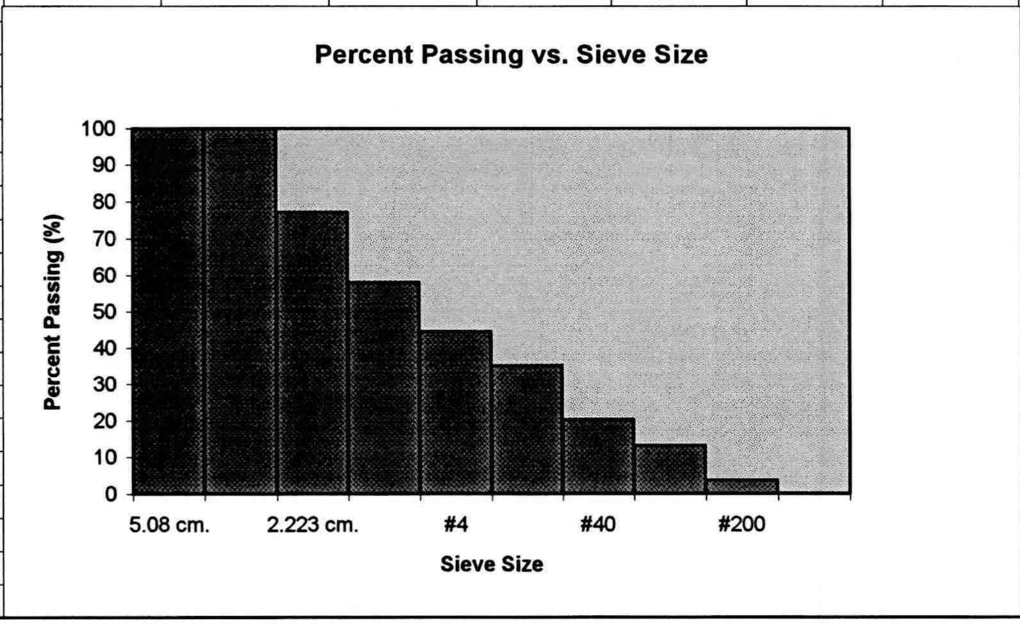
Percent Passing vs. Sieve Size



Sieve Analysis
for
Jordan Pit

Weight of sample(g) - 2477.9

Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	0	100.00
2.223 cm.	561.3	77.35
0.953 cm.	476.8	58.11
#4	334.5	44.61
#10	231.8	35.25
#40	371	20.28
#80	175.8	13.18
#200	233.6	3.76
<#200	90.5	0.10

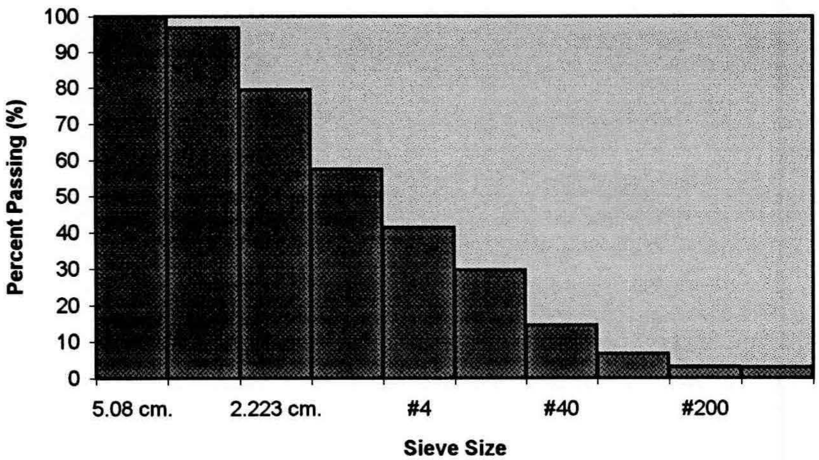


Sieve Analysis
for
Kemper Pit

Weight of sample(g) - 2467.3

Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	68.9	96.79
2.223 cm.	507.3	79.44
0.953 cm.	540.2	57.54
#4	401.8	41.26
#10	289.9	29.51
#40	369.6	14.53
#80	196.2	6.58
#200	86	3.09
<#200	4	2.93

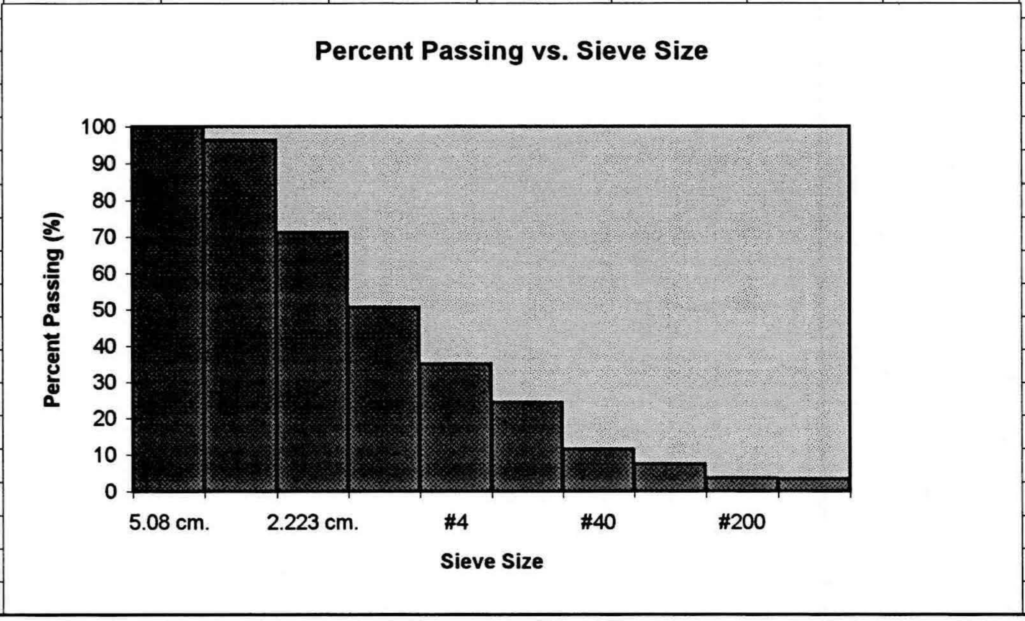
Percent Passing vs. Sieve Size



Sieve Analysis
for
Parmley Pit

Weight of sample(g) - 2482.1

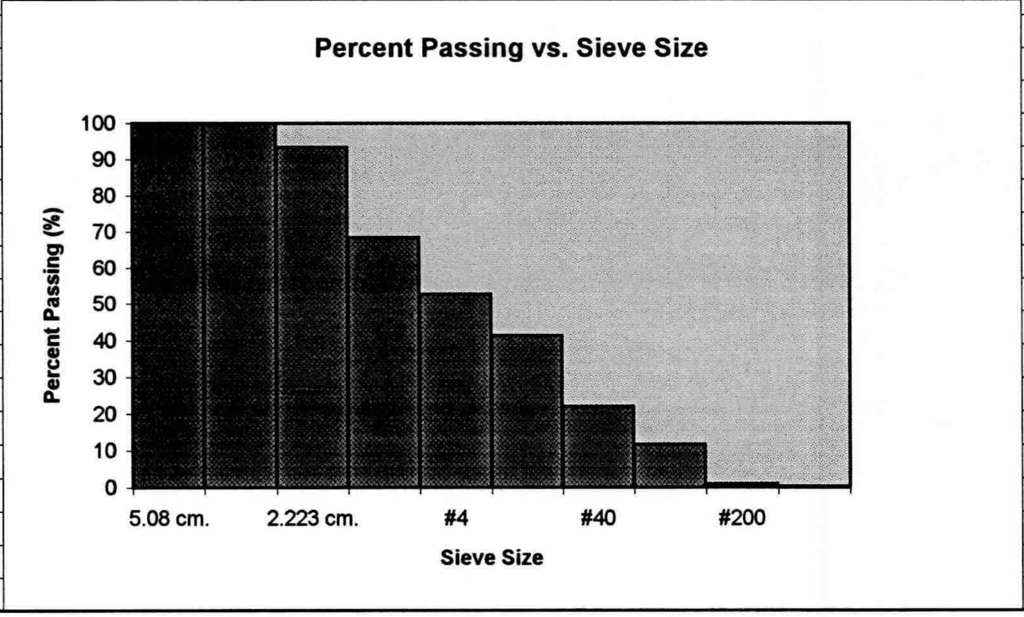
Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	80	96.27
2.223 cm.	713.7	71.25
0.953 cm.	509.8	50.71
#4	389.8	35.00
#10	264.3	24.35
#40	317.7	11.55
#80	100.6	7.50
#200	96.2	3.63
<#200	1.6	3.56



Sieve Analysis
for
Tubbs Pit

Weight of sample(g) - 2450.5

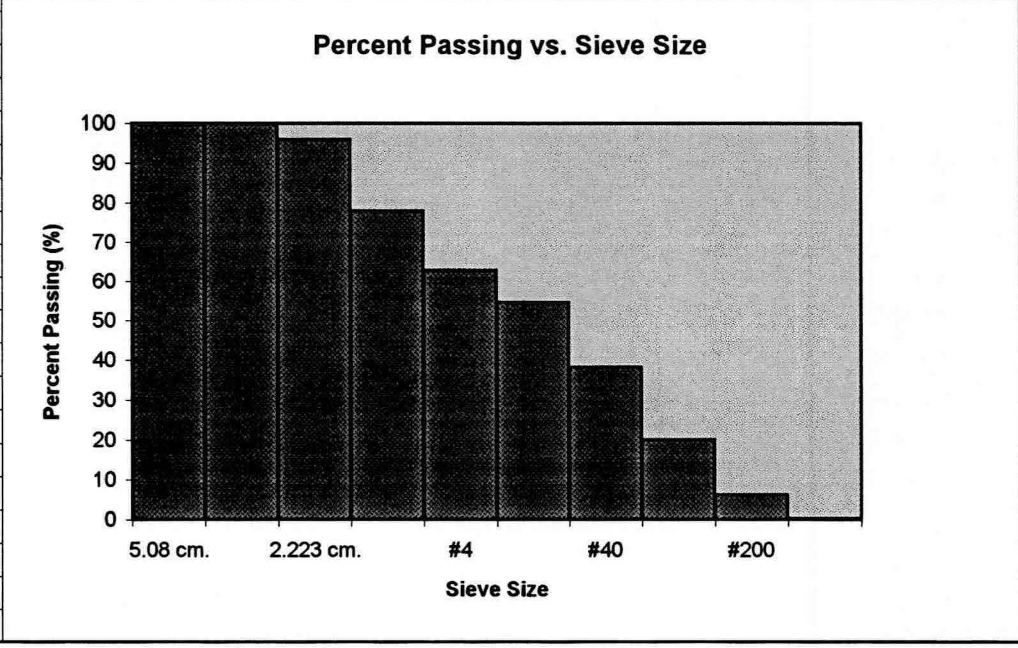
Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	0	100.00
2.223 cm.	163.5	93.33
0.953 cm.	608.1	68.51
#4	385.6	52.78
#10	282.3	41.26
#40	470	22.08
#80	254.6	11.69
#200	259.4	1.10
<#200	19.9	0.29



Sieve Analysis
for
Abilene

Weight of sample(g) - 2650.5

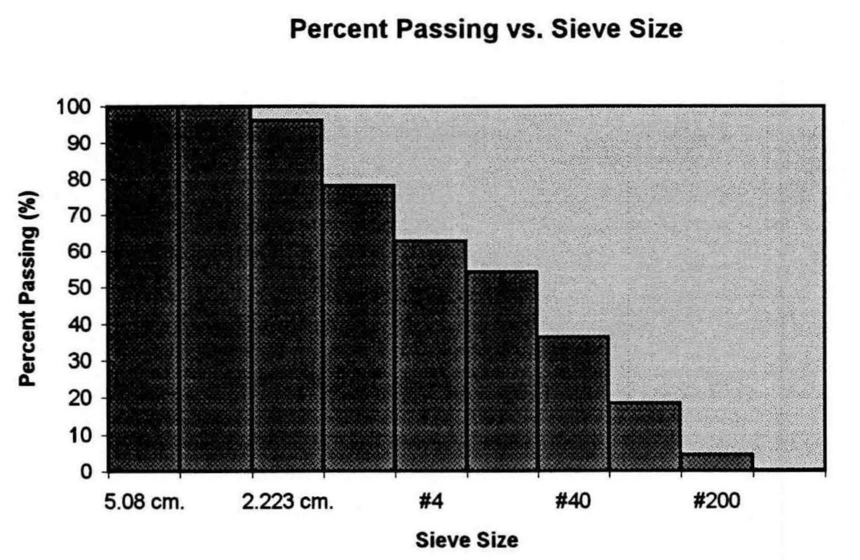
Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	0	100
2.223 cm.	107.6	95.94
0.953 cm.	475.2	78.01
#4	402.1	62.84
#10	215.3	54.72
#40	435.2	38.30
#80	478.9	20.23
#200	370.2	6.26
<#200	159.3	0.25



Sieve Analysis
for
Victoria

Weight of sample(g) - 2577.7

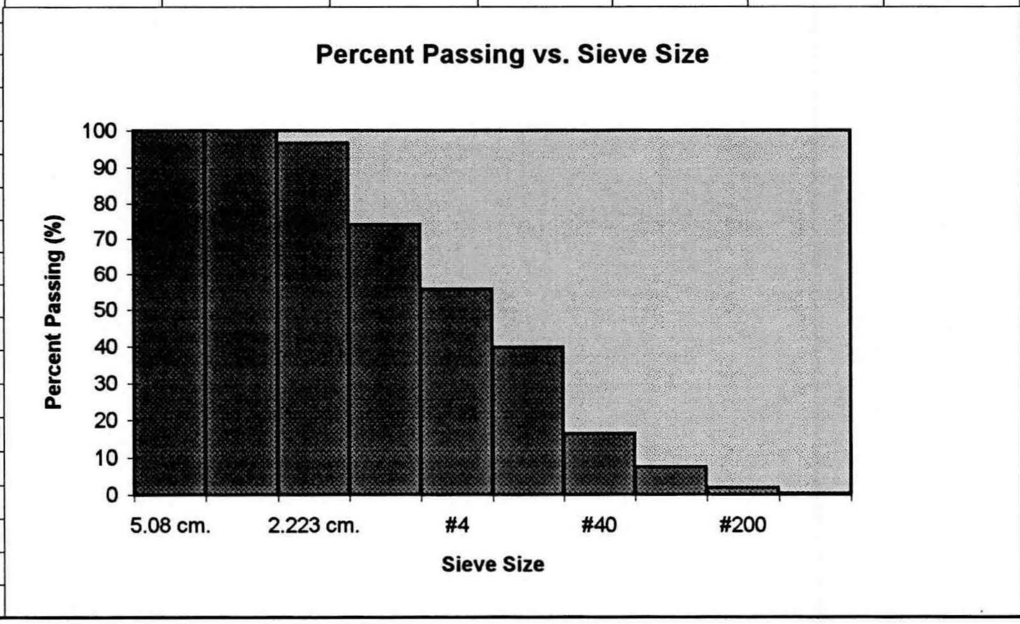
Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	0	100
2.223 cm.	95.8	96.28
0.953 cm.	466.2	78.20
#4	393.1	62.95
#10	221.9	54.34
#40	461.8	36.42
#80	466.5	18.33
#200	354.8	4.56
<#200	108.9	0.34



Sieve Analysis
for
Crushed Limestone

Weight of sample(g) - 2145

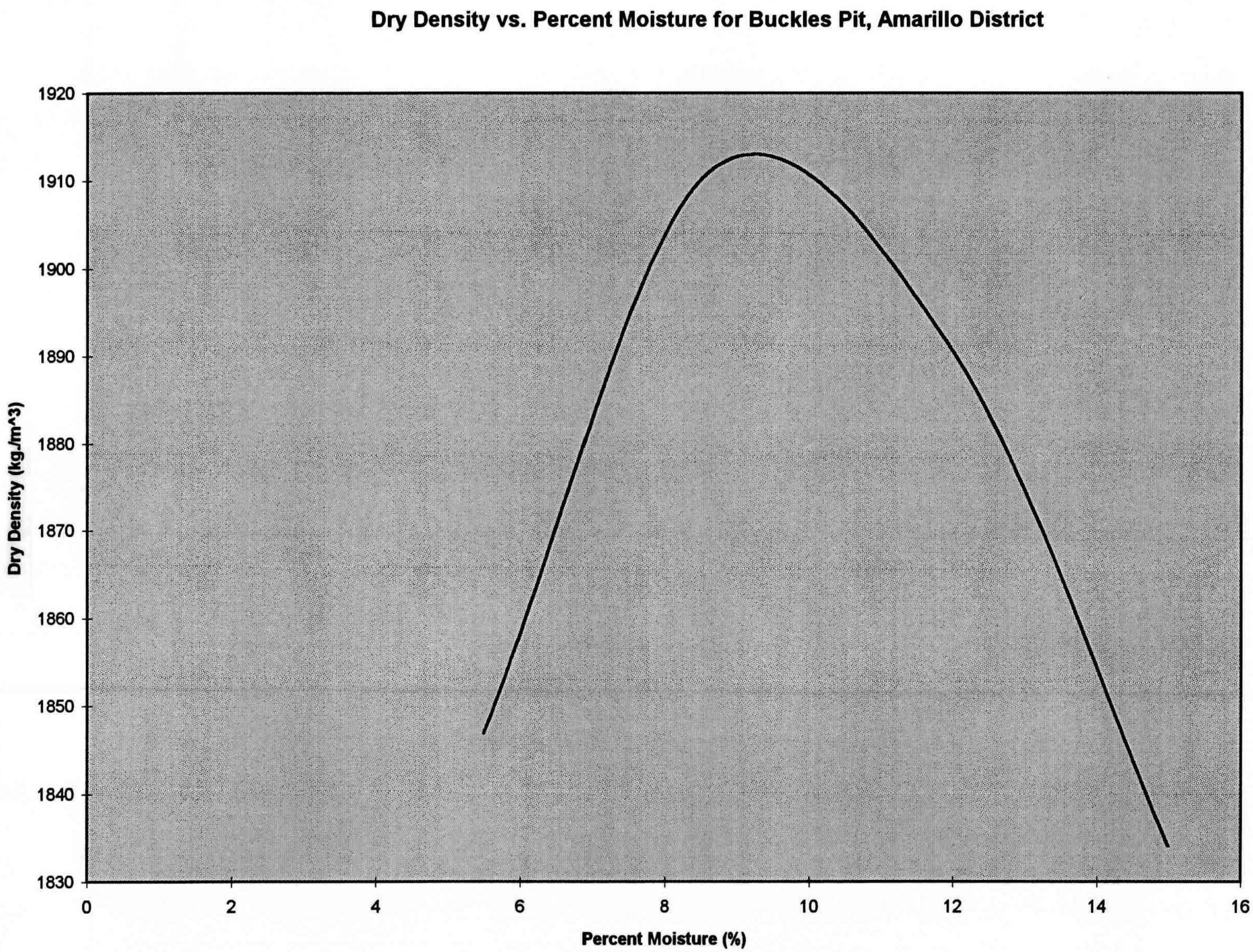
Sieve Size	Weight Passing(g)	Percent Passing (%)
5.08 cm.	0	100
3.81 cm.	0	100
2.223 cm.	70	96.74
0.953 cm.	486.8	74.04
#4	390.5	55.84
#10	344	39.80
#40	503.9	16.31
#80	187.5	7.57
#200	121.6	1.90
<#200	33	0.36



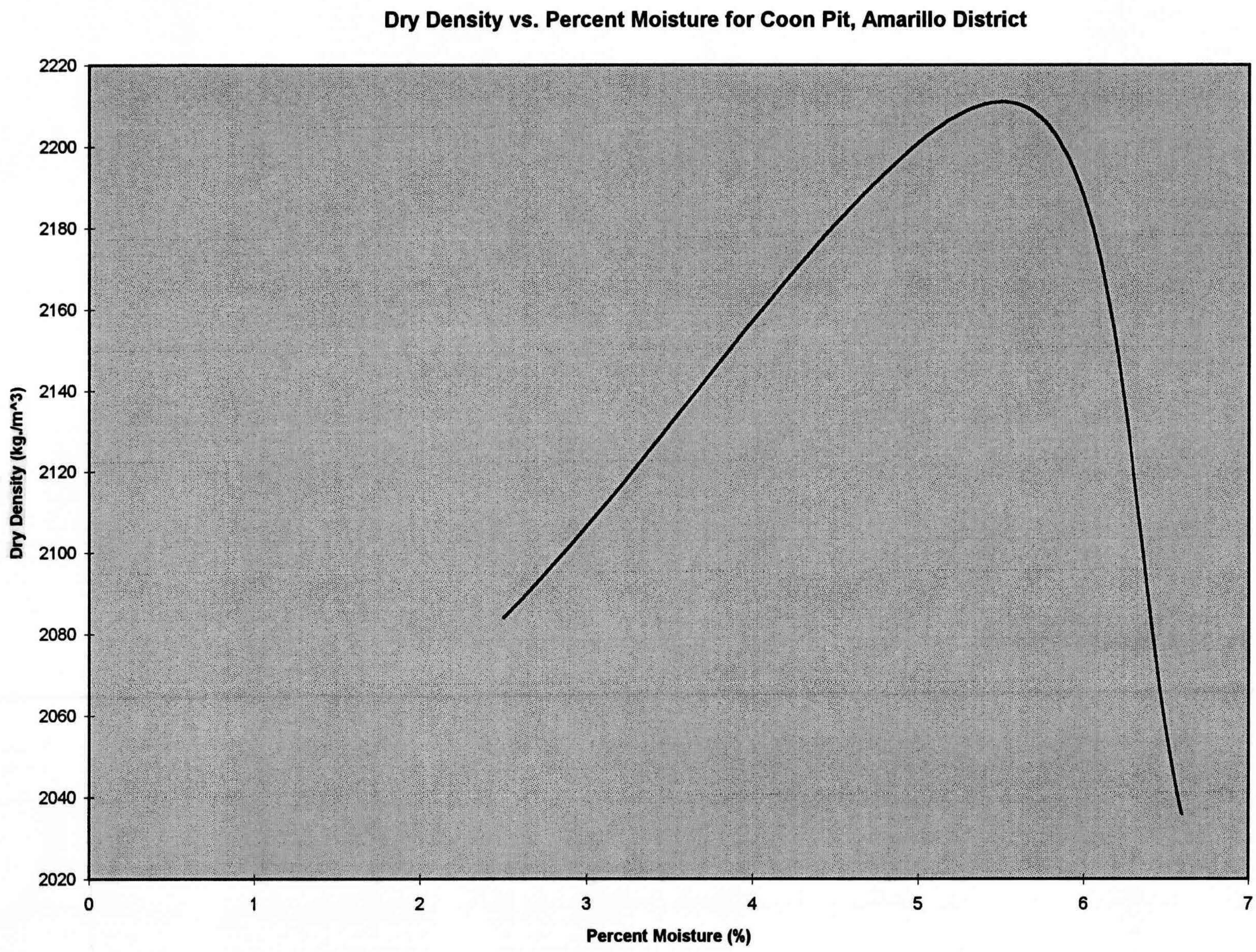
APPENDIX B:

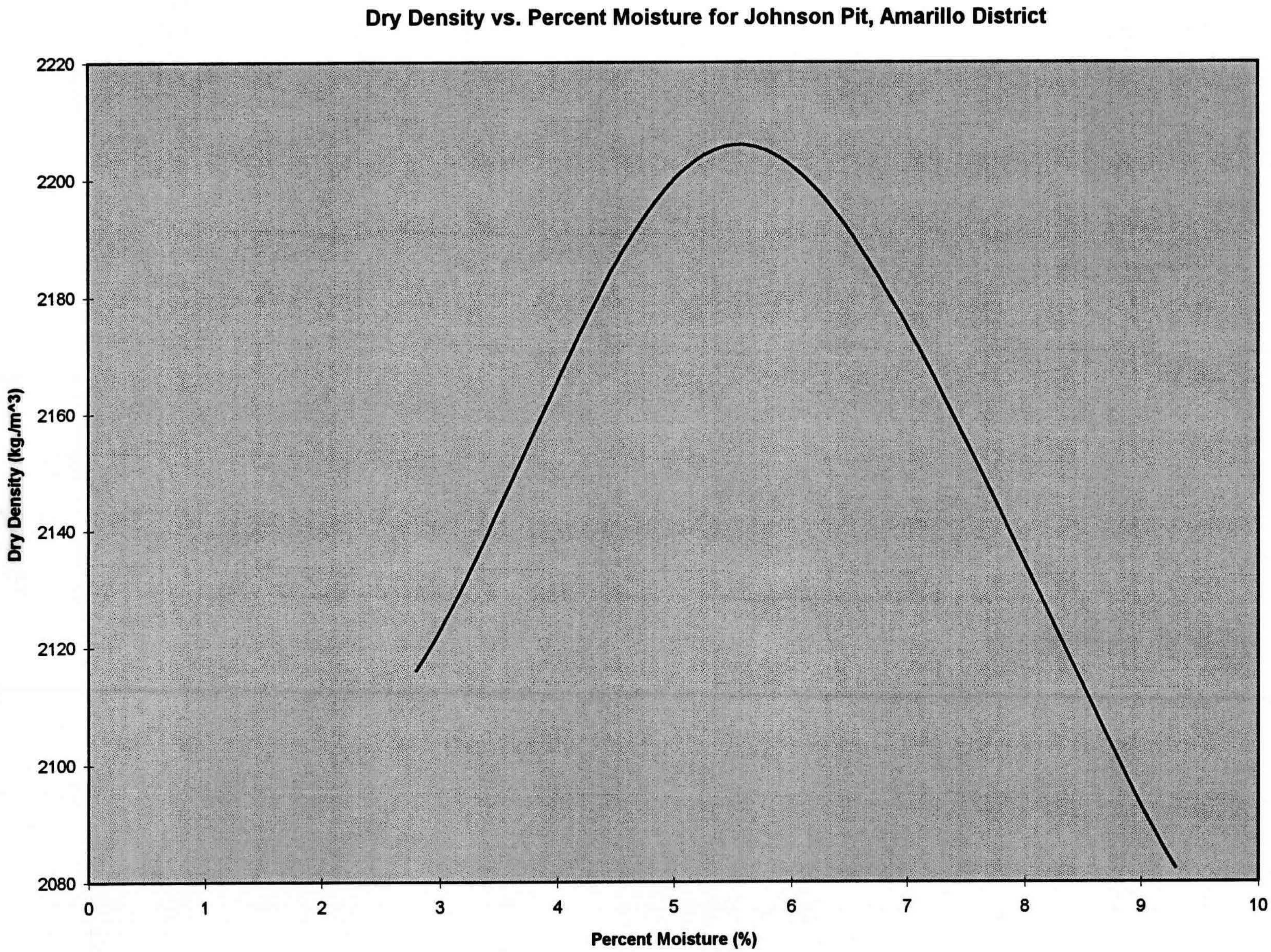
PLOTS OF DRY DENSITY v. MOLDING MOISTURE CONTENT FOR THE VARIOUS SOURCE MATERIALS EVALUATED IN THE PHASE 1 TESTING PROGRAM

B.2.

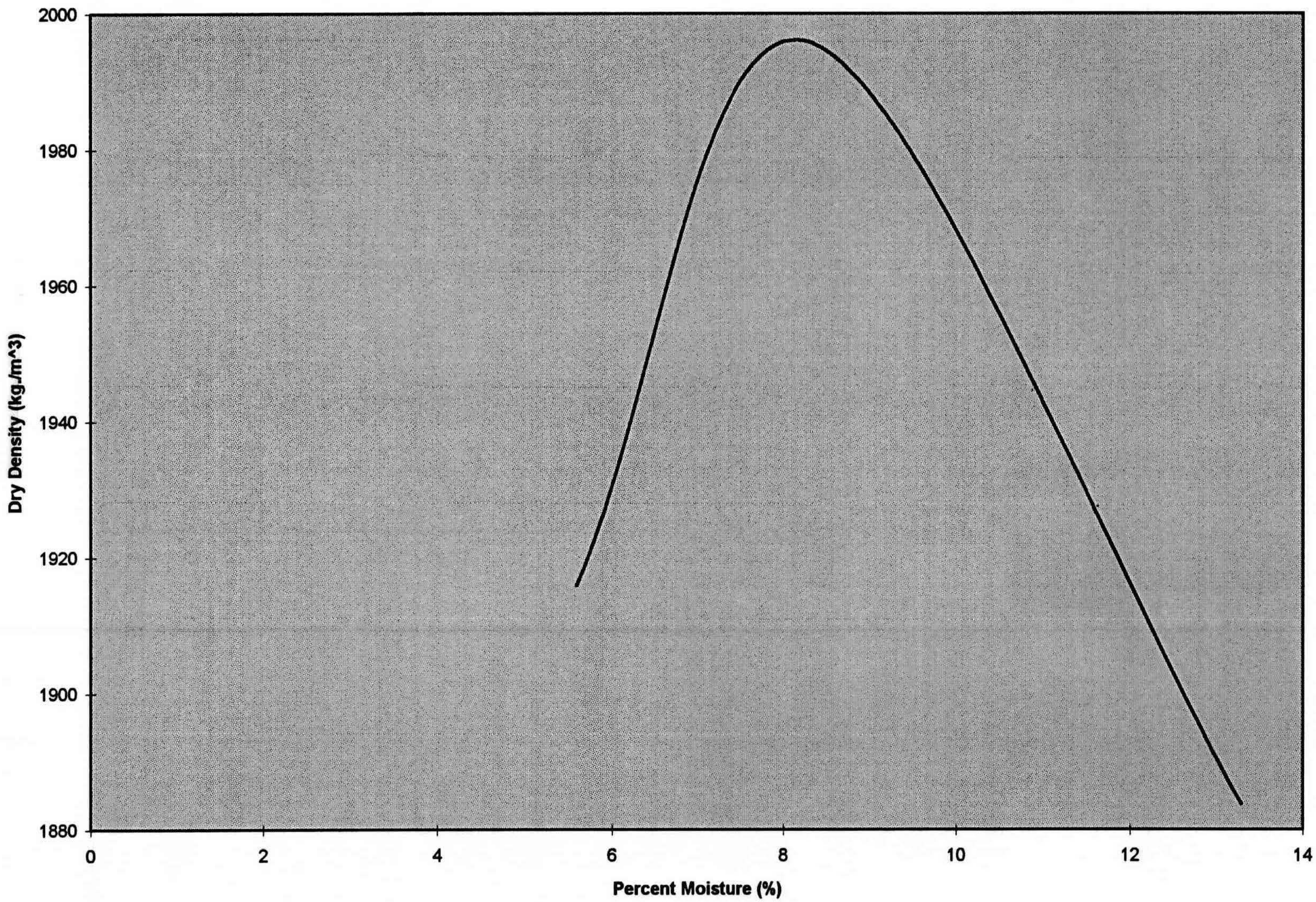


Dry Density vs. Percent Moisture for Buckles Pit, Amarillo District
1920
1910
1900
1890
1880
1870
1860
1850
1840
1830
Dry Density (kg./m^3)
0
2
4
6
8
10
12
14
16
Percent Moisture (%)

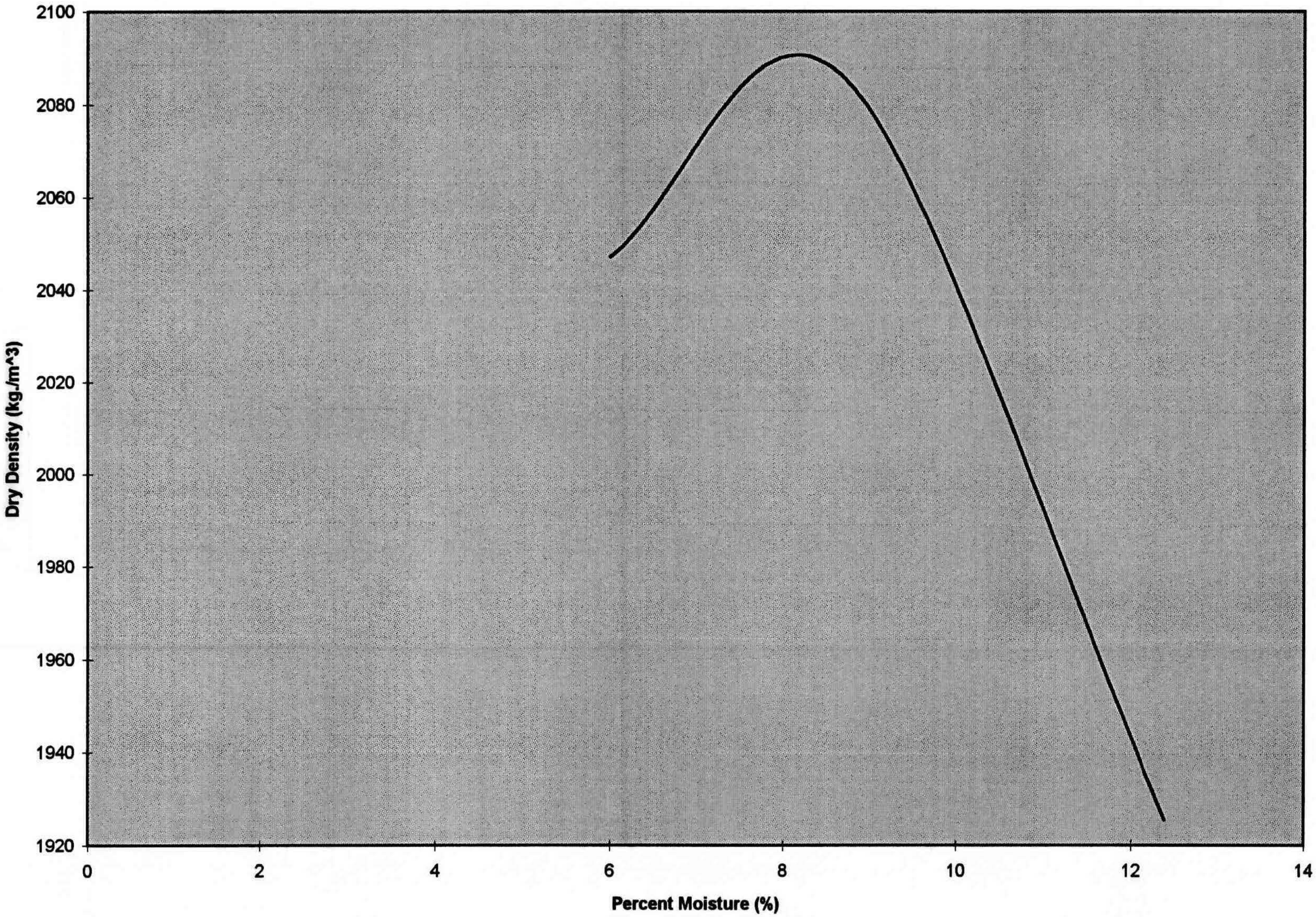

Dry Density vs. Percent Moisture for Coon Pit, Amarillo District
Dry Density (kg./m^3)
2220
2200
2180
2160
2140
2120
2100
2080
2060
2040
2020
0
1
2
3
4
5
6
7
Percent Moisture (%)


Dry Density vs. Percent Moisture for Johnson Pit, Amarillo District
Dry Density (kg./m^3)
2220
2200
2180
2160
2140
2120
2100
2080
0
1
2
3
4
5
6
7
8
9
10
Percent Moisture (%)

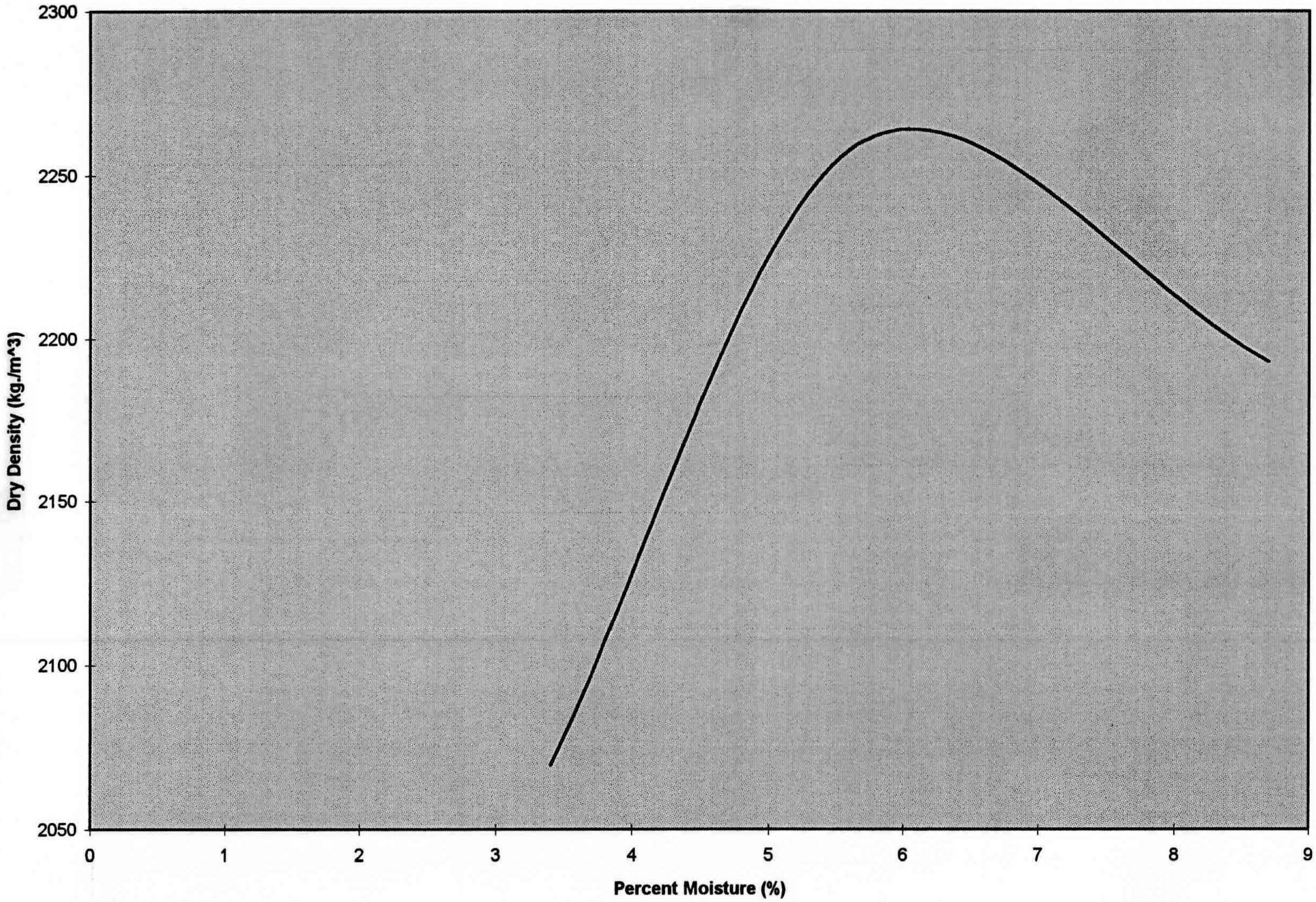
Dry Density vs. Percent Moisture for Lindsey Pit, Amarillo District



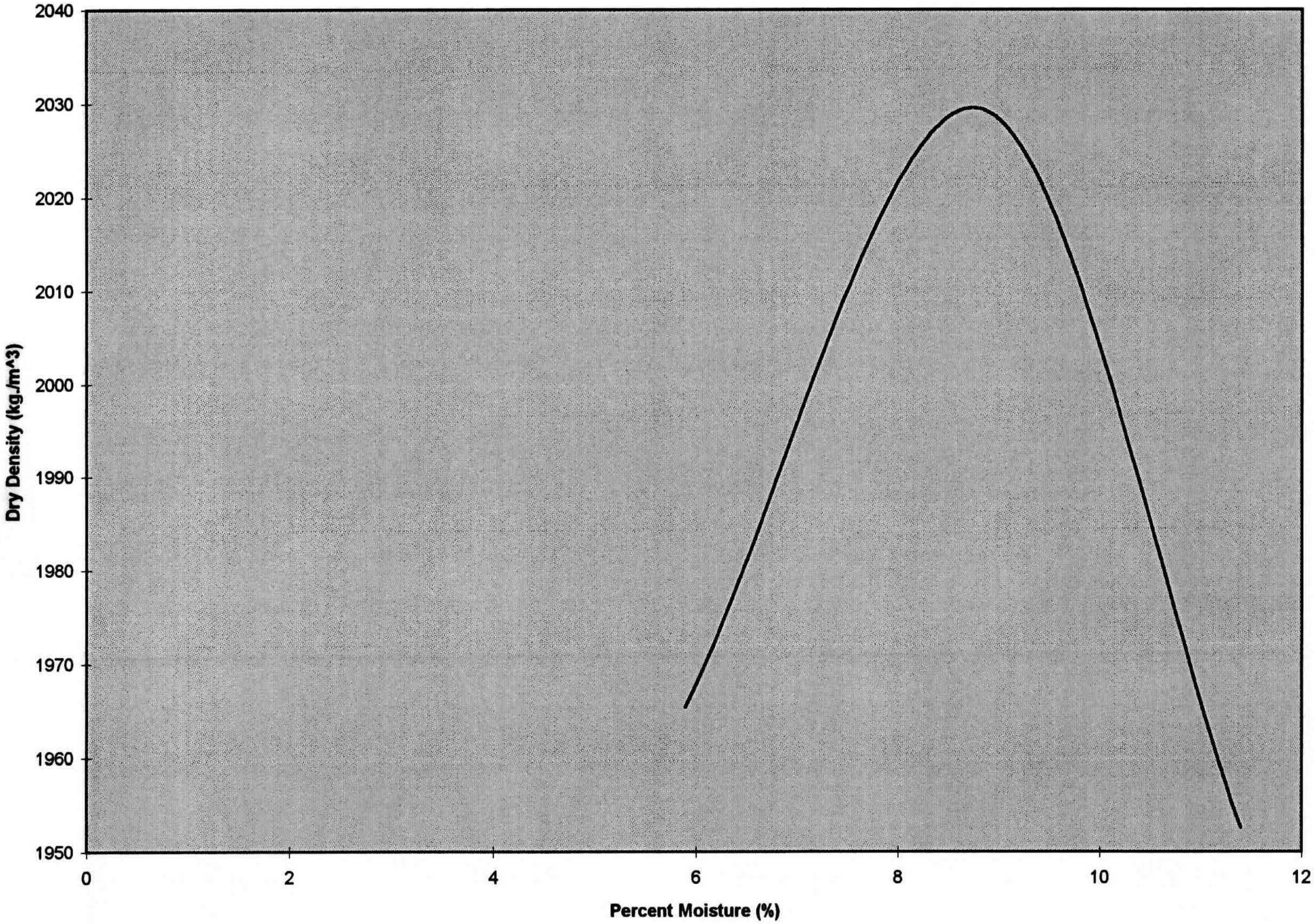
Dry Density vs. Percent Moisture for Clements Pit, Abilene District



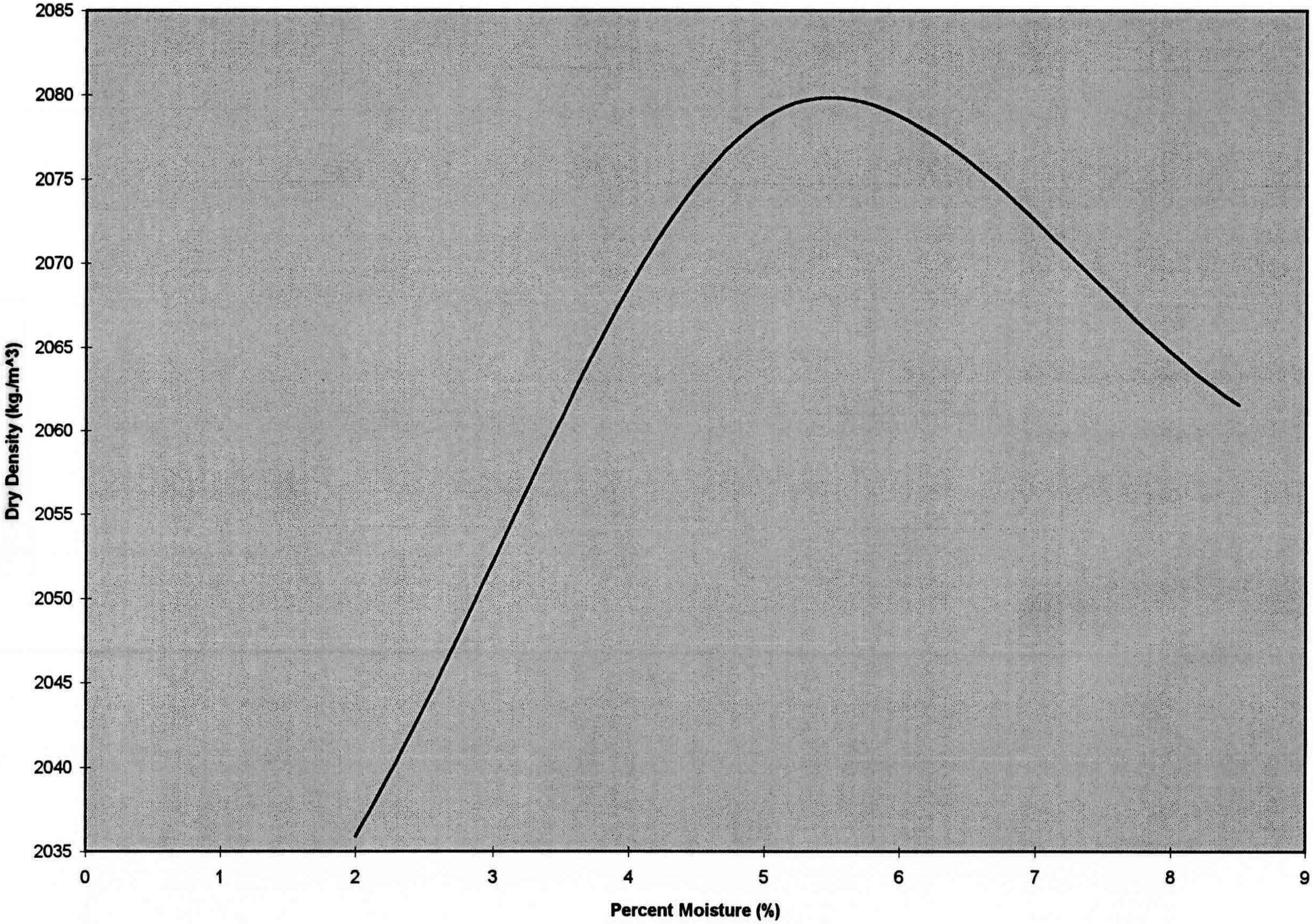
Dry Density vs. Percent Moisture for Jordan Pit, Abilene District



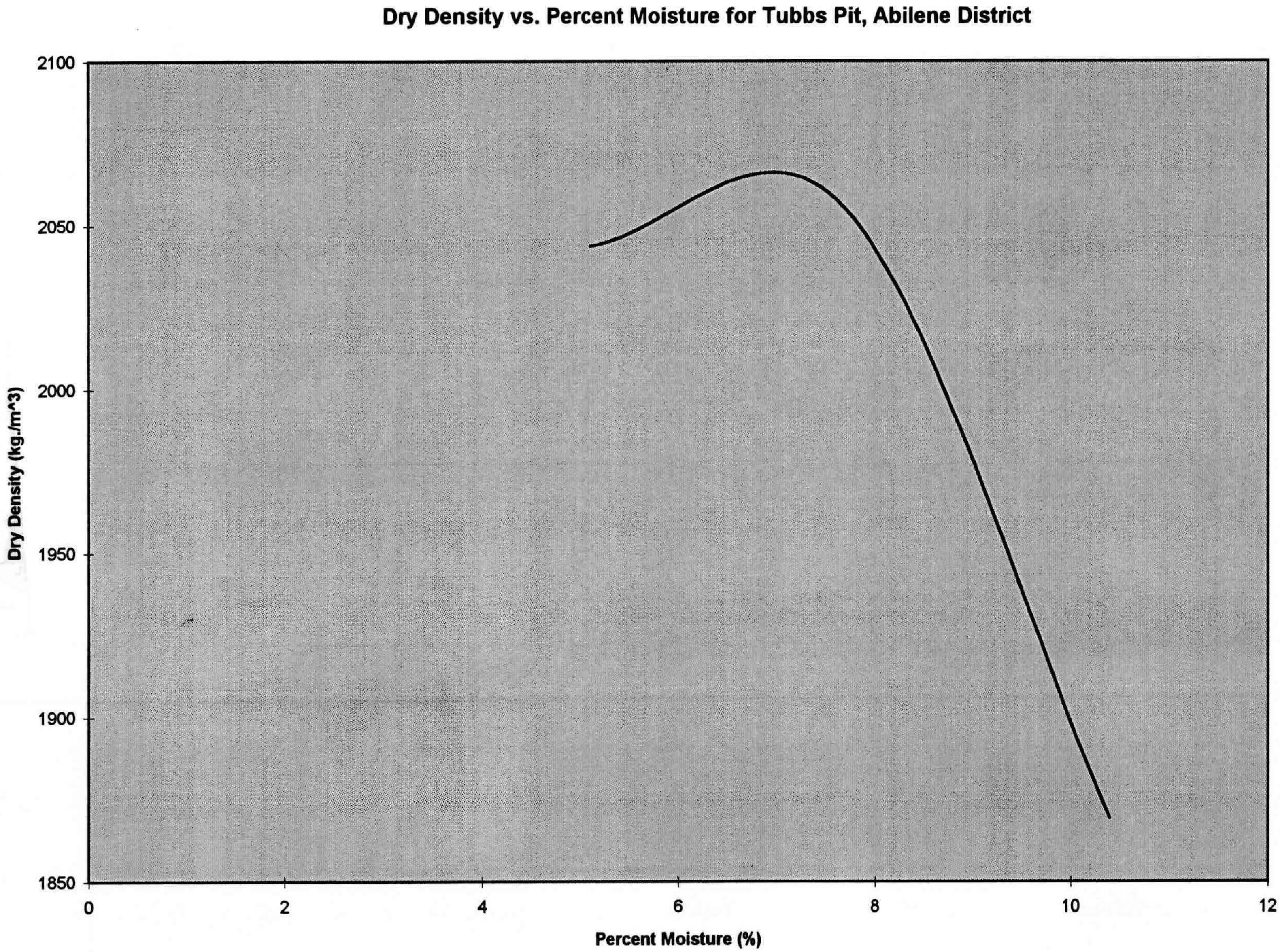
Dry Density vs. Percent Moisture for Kemper Pit, Abilene District



Dry Density vs. Percent Moisture for Parmley Pit, Abilene District



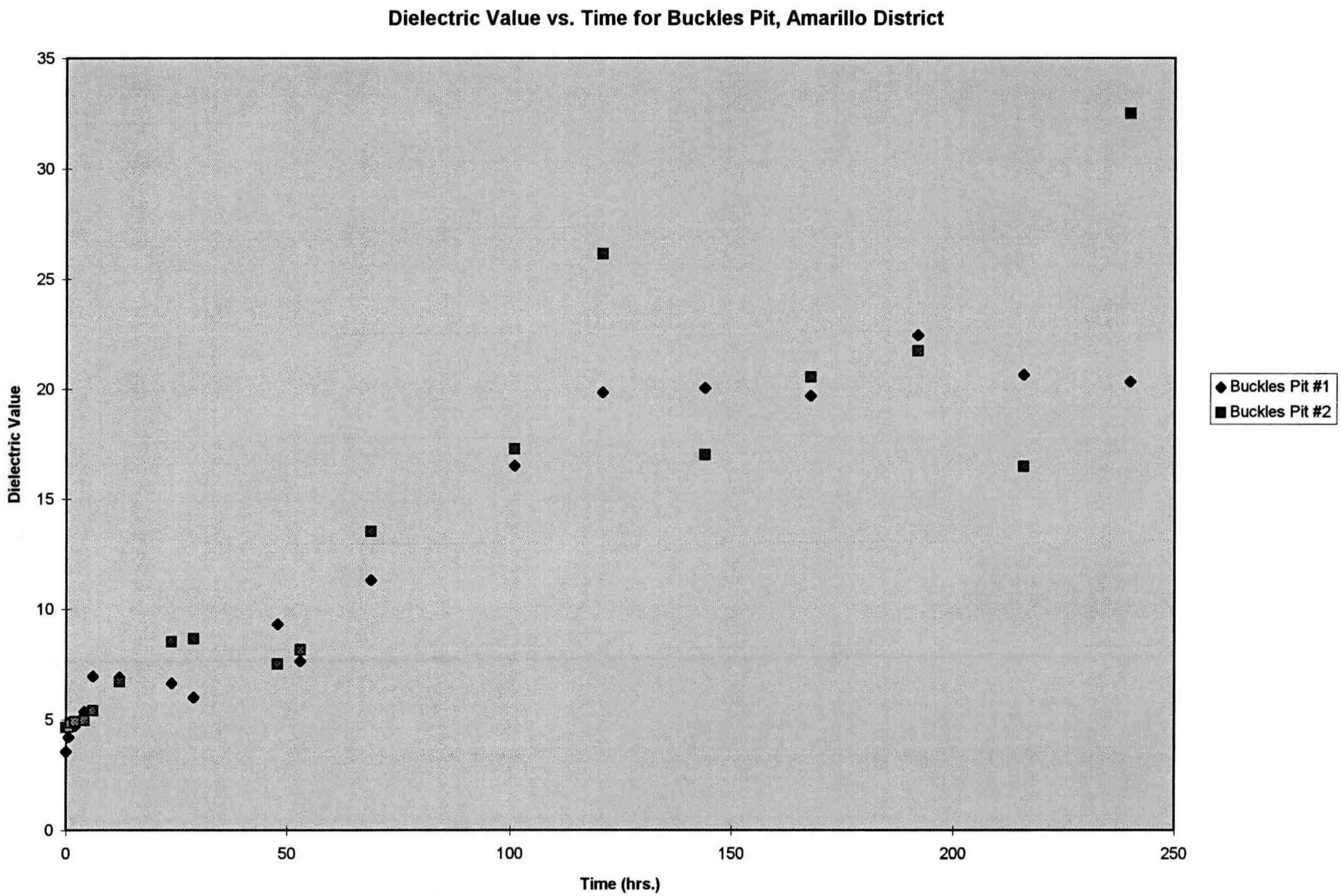
B.9.


Dry Density vs. Percent Moisture for Tubbs Pit, Abilene District
2100
2050
2000
1950
1900
1850
Dry Density (kg./m^3)
0
2
4
6
8
10
12
Percent Moisture (%)

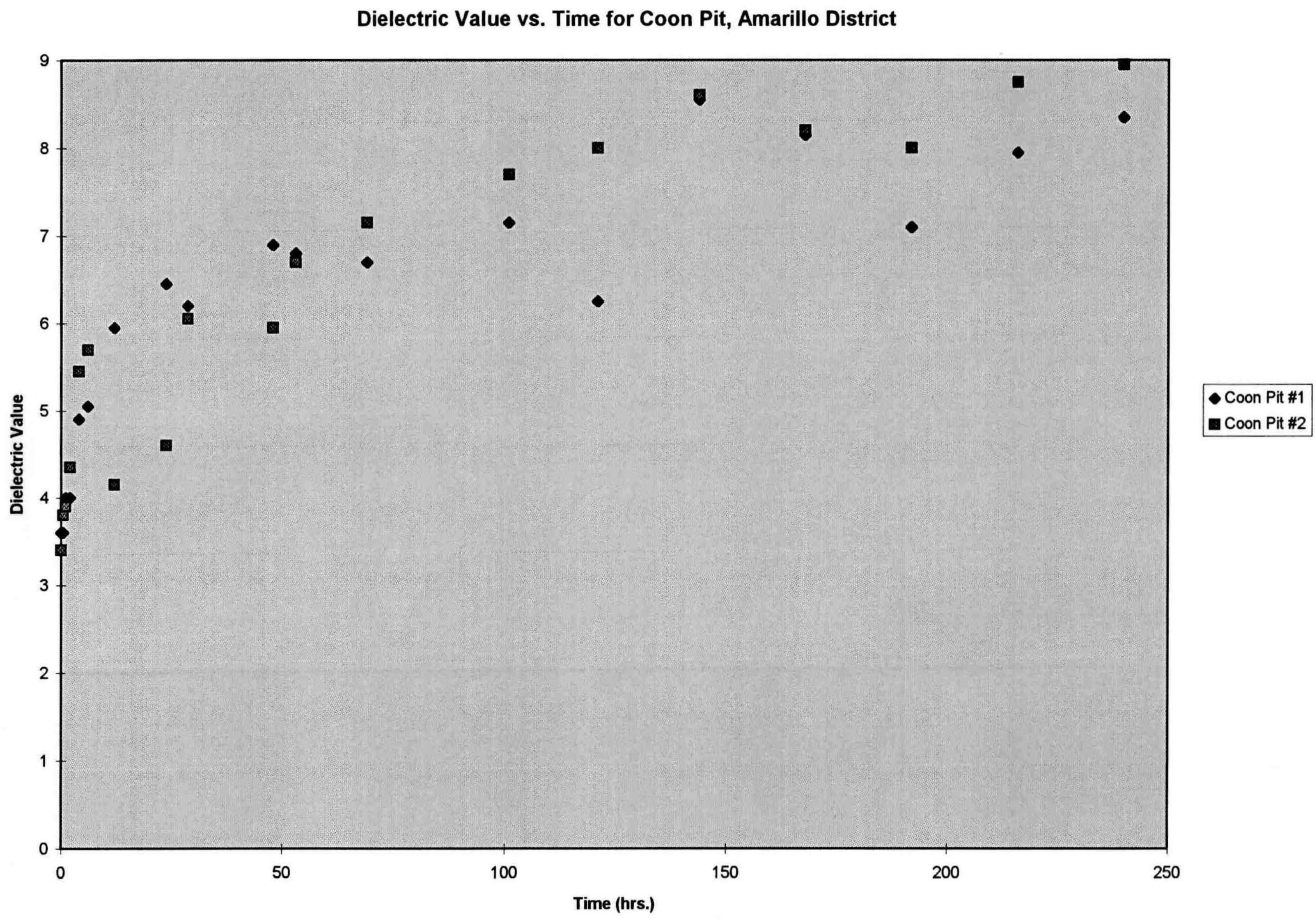
APPENDIX C:

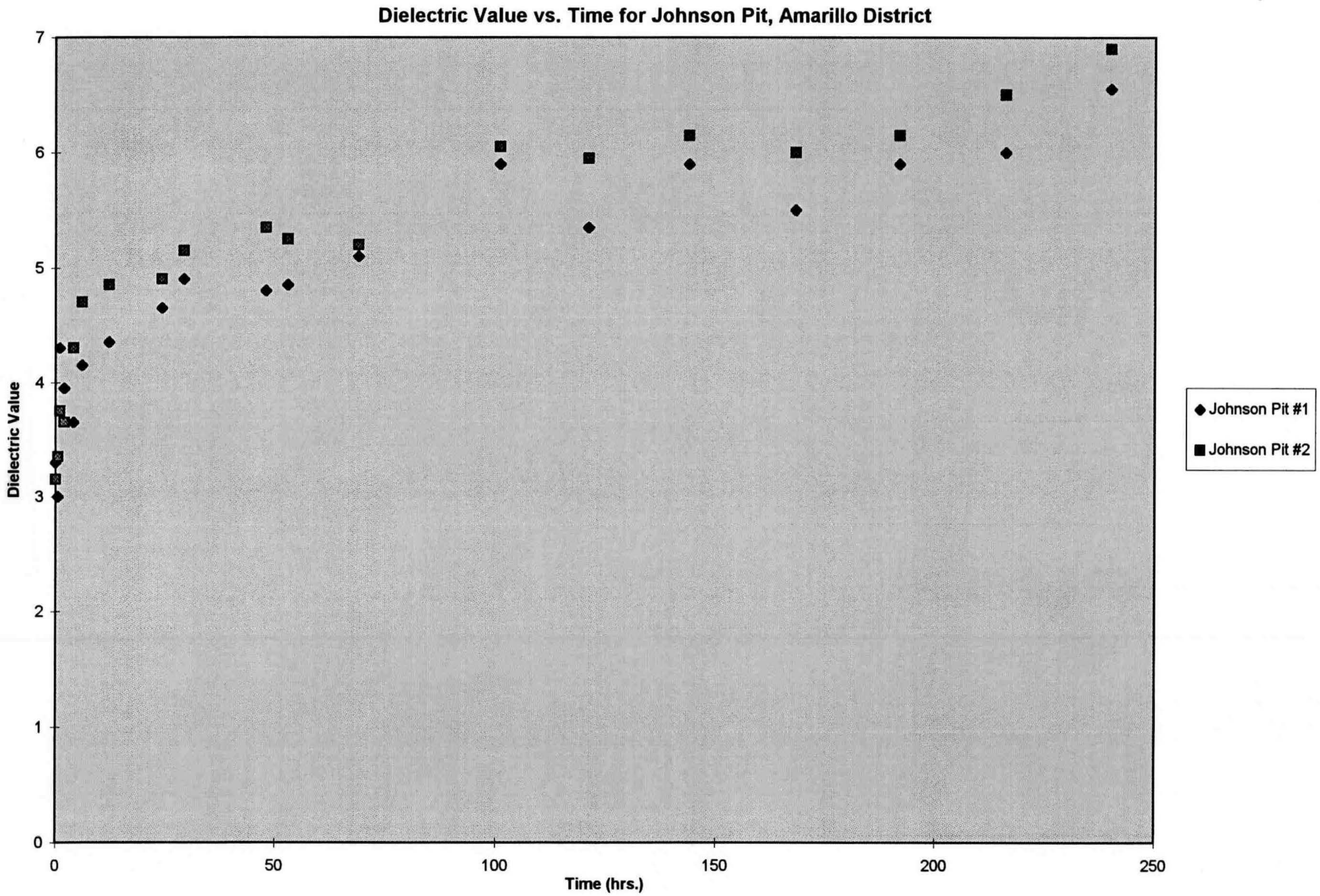
PLOTS OF THE DIELECTRIC VALUE v. TIME OF TESTING (CAPILLARY SOAK) FOR THE VARIOUS SOURCE MATERIALS EVALUATED IN THE PHASE 1 TESTING PROGRAM

C.2.

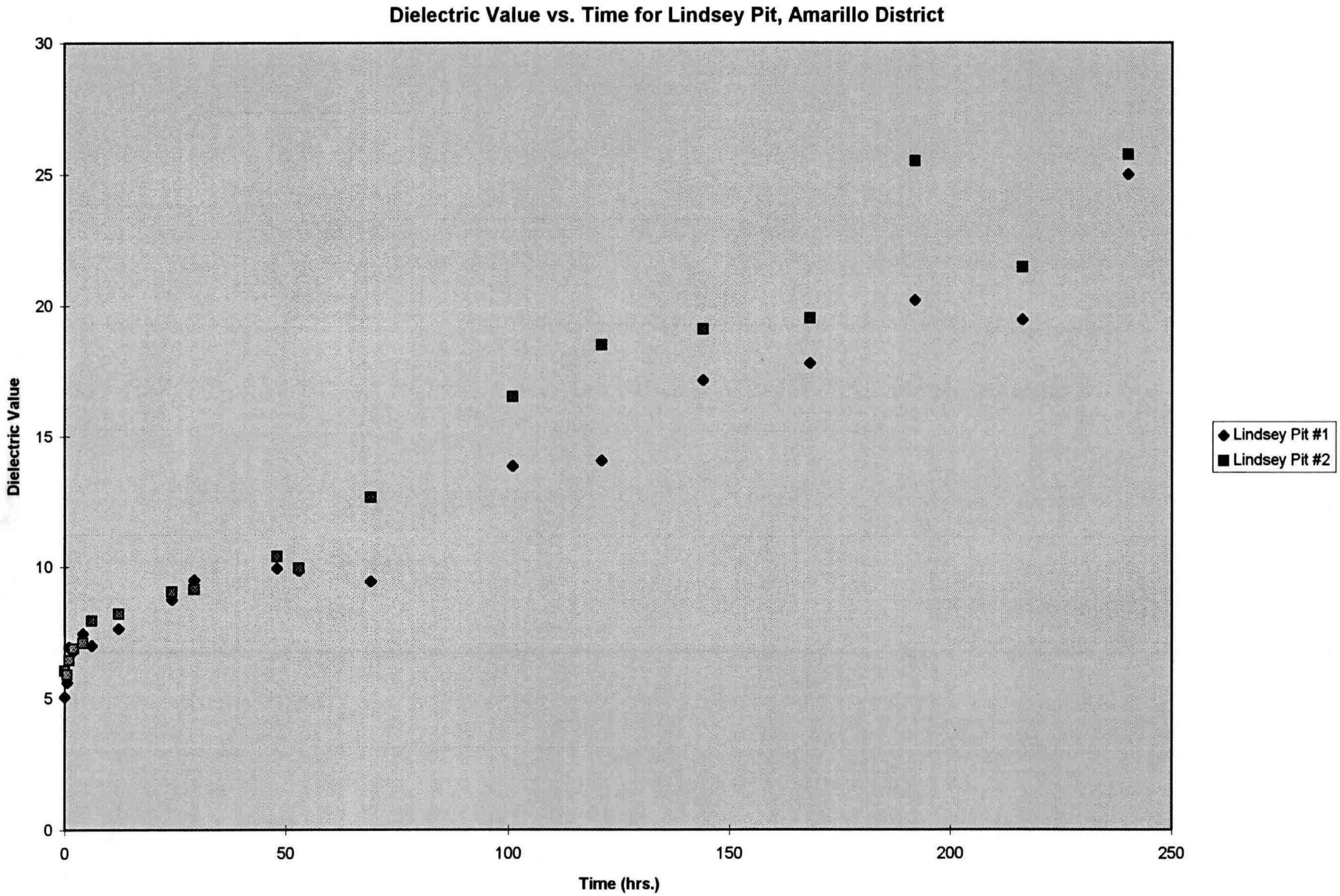


Dielectric Value vs. Time for Buckles Pit, Amarillo District
Dielectric Value
Time (hrs.)
0
5
10
15
20
25
30
35
0
50
100
150
200
250
Buckles Pit #1
Buckles Pit #2


Dielectric Value vs. Time for Coon Pit, Amarillo District
Dielectric Value
Time (hrs.)
0
1
2
3
4
5
6
7
8
9
0
50
100
150
200
250
Coon Pit #1
Coon Pit #2

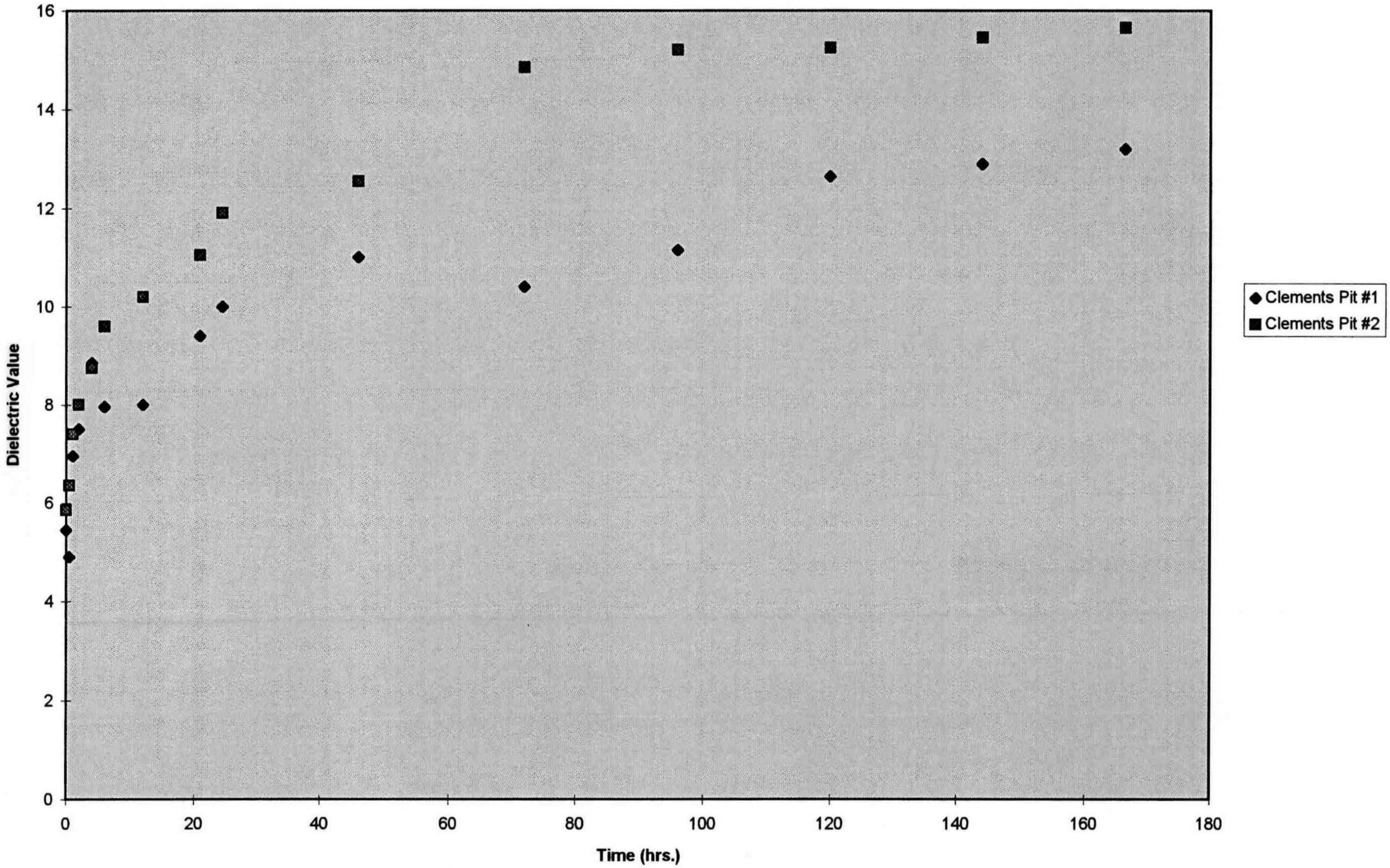

Dielectric Value vs. Time for Johnson Pit, Amarillo District
Dielectric Value
Time (hrs.)
0
1
2
3
4
5
6
7
0
50
100
150
200
250
Johnson Pit #1
Johnson Pit #2

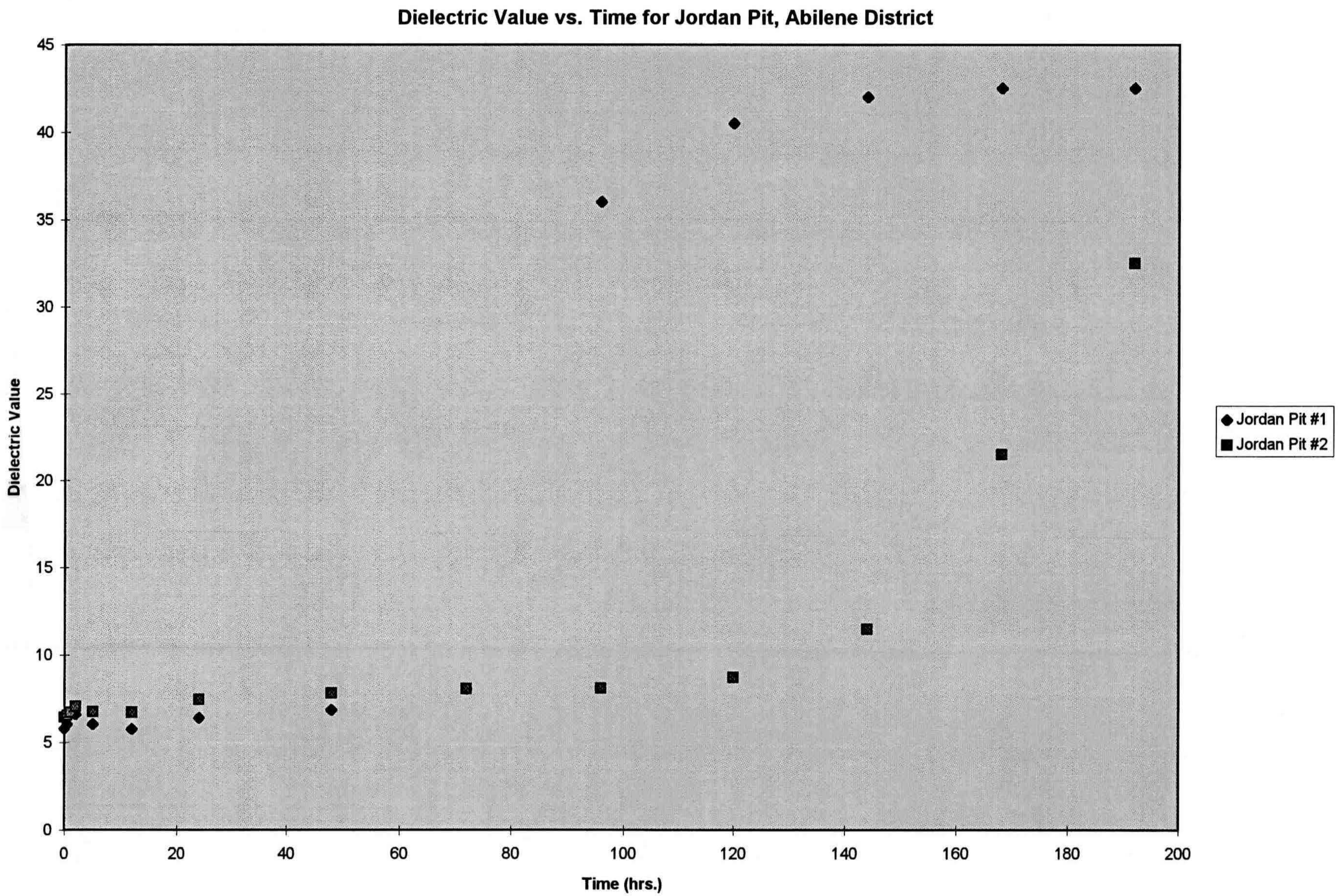
C.5.



Dielectric Value vs. Time for Lindsey Pit, Amarillo District
Dielectric Value
Time (hrs.)
0
5
10
15
20
25
30
0
50
100
150
200
250
Lindsey Pit #1
Lindsey Pit #2

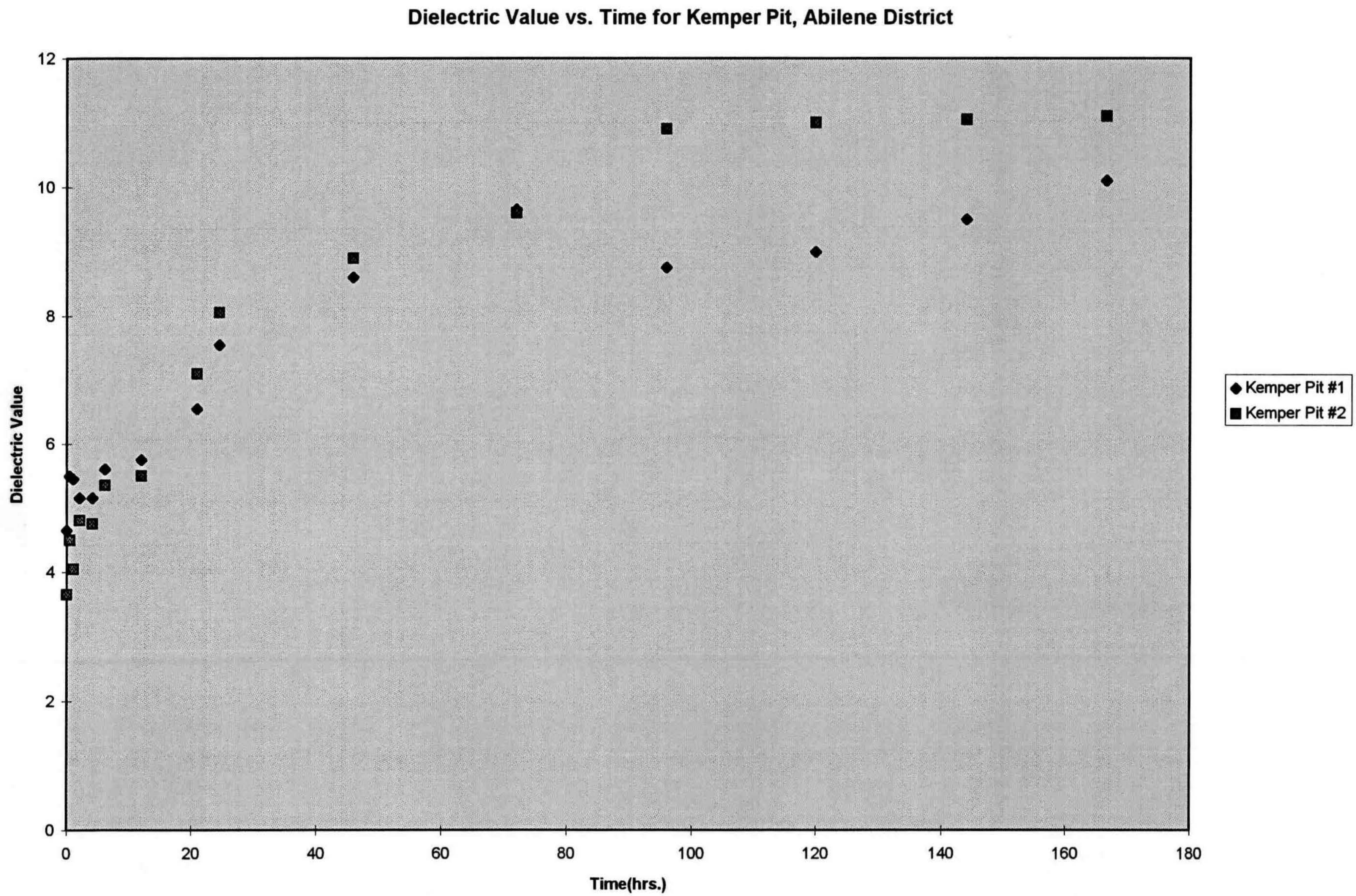
Dielectric Value vs. Time for Clements Pit, Abilene District



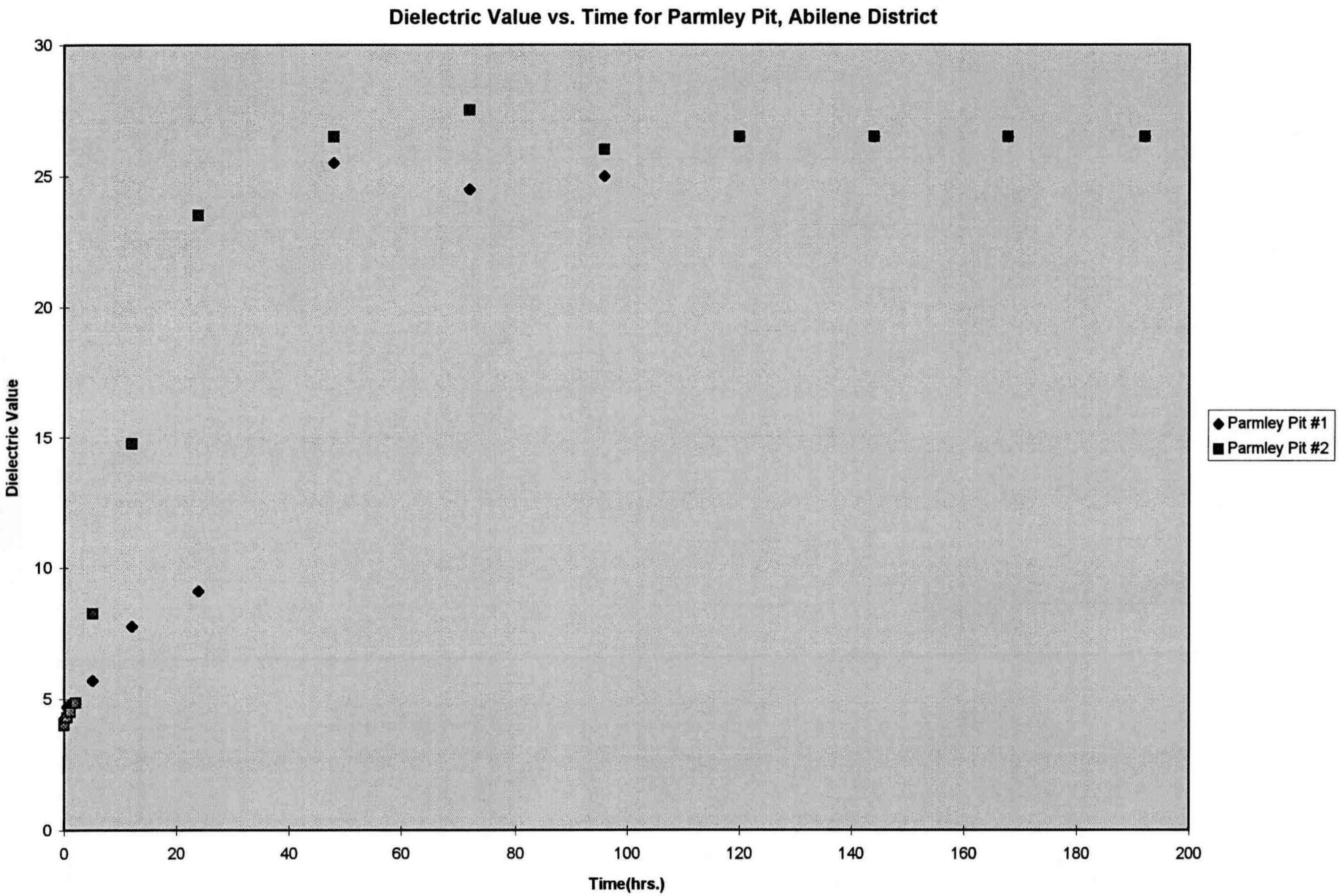


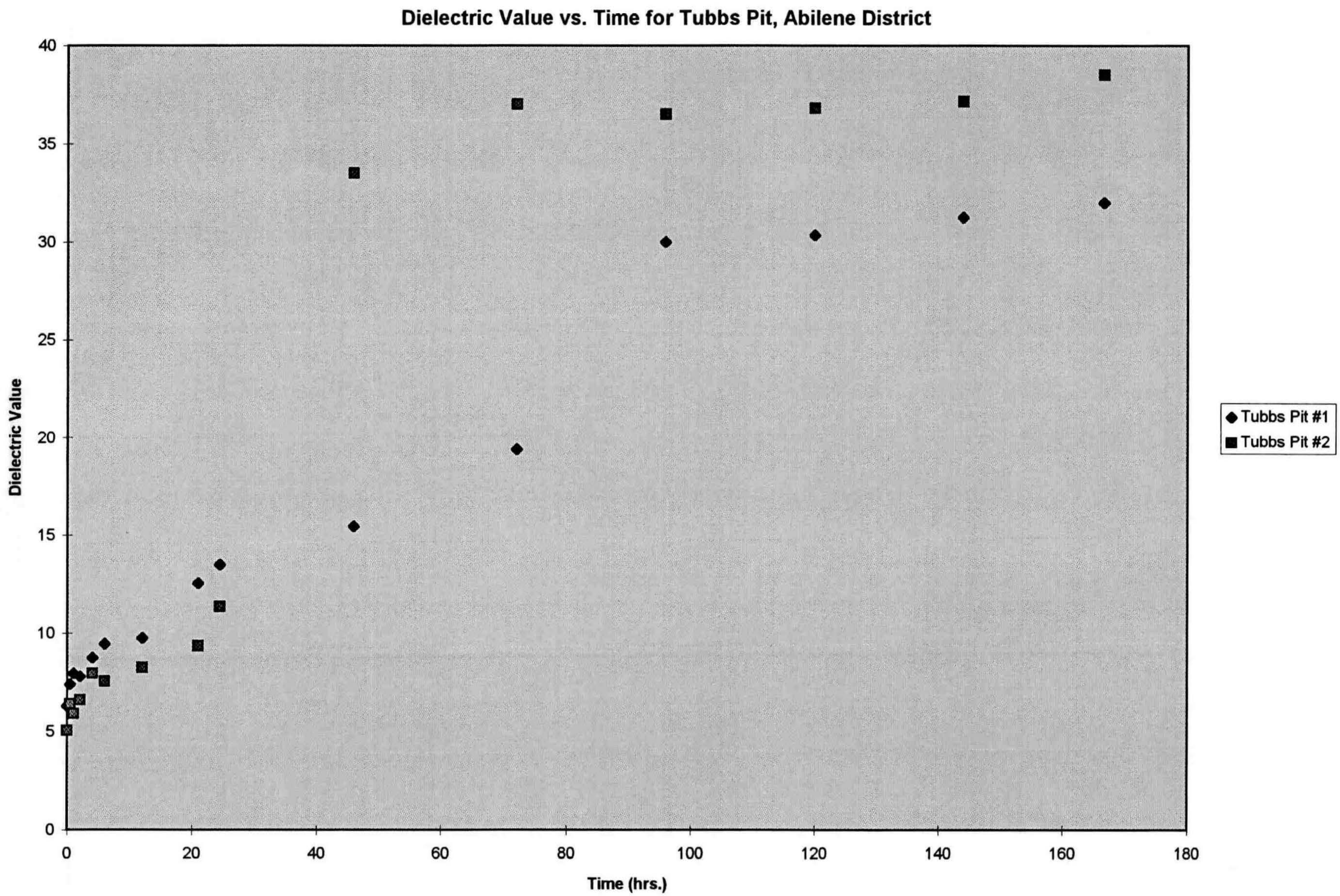
Dielectric Value vs. Time for Jordan Pit, Abilene District
Dielectric Value
Time (hrs.)
0
5
10
15
20
25
30
35
40
45
0
20
40
60
80
100
120
140
160
180
200
Jordan Pit #1
Jordan Pit #2

C.7.



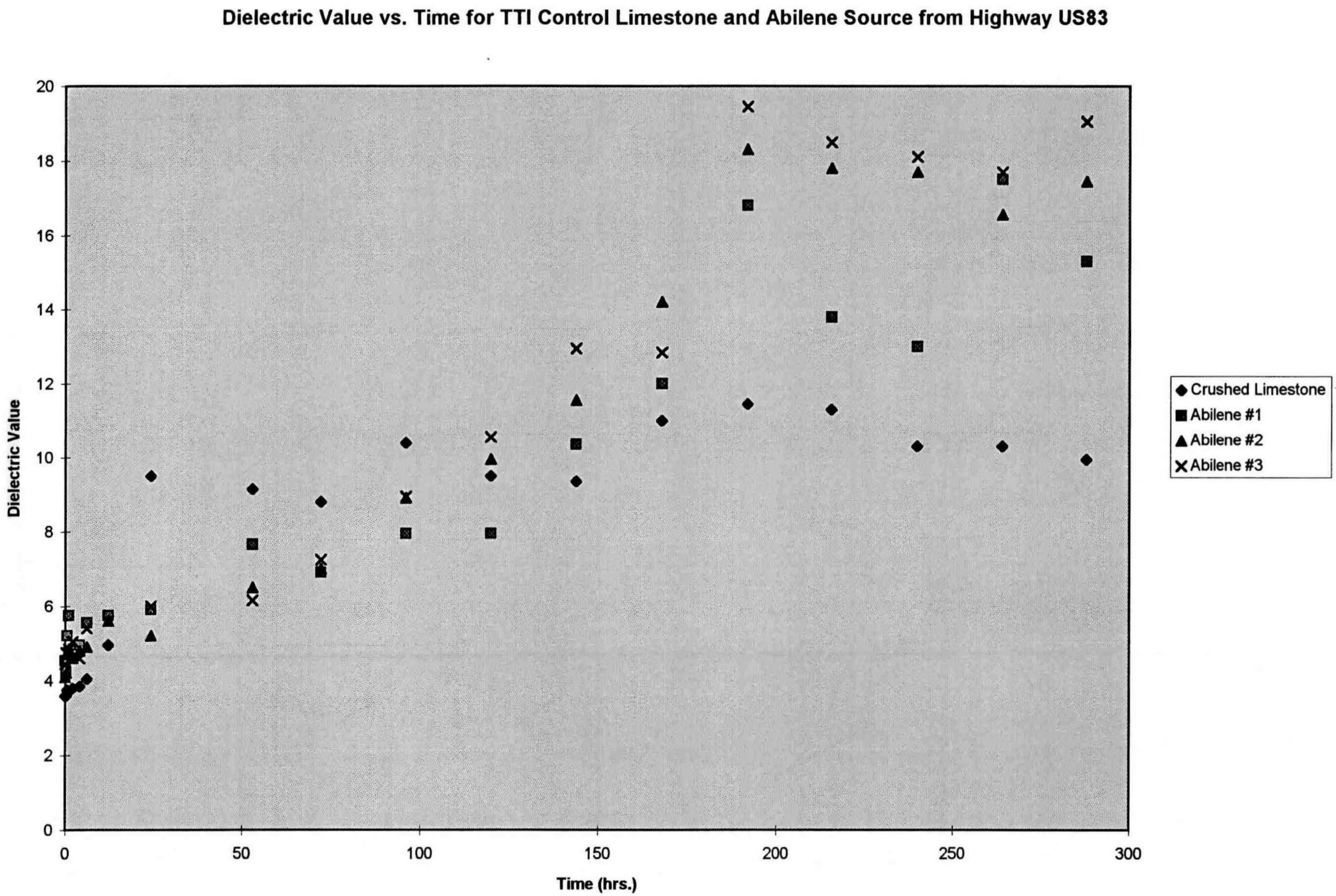
Dielectric Value vs. Time for Kemper Pit, Abilene District
Dielectric Value
Time(hrs.)
Kemper Pit #1
Kemper Pit #2
0
2
4
6
8
10
12
20
40
60
80
100
120
140
160
180


Dielectric Value vs. Time for Parmley Pit, Abilene District
Dielectric Value
Time(hrs.)
0
5
10
15
20
25
30
0
20
40
60
80
100
120
140
160
180
200
Parmley Pit #1
Parmley Pit #2



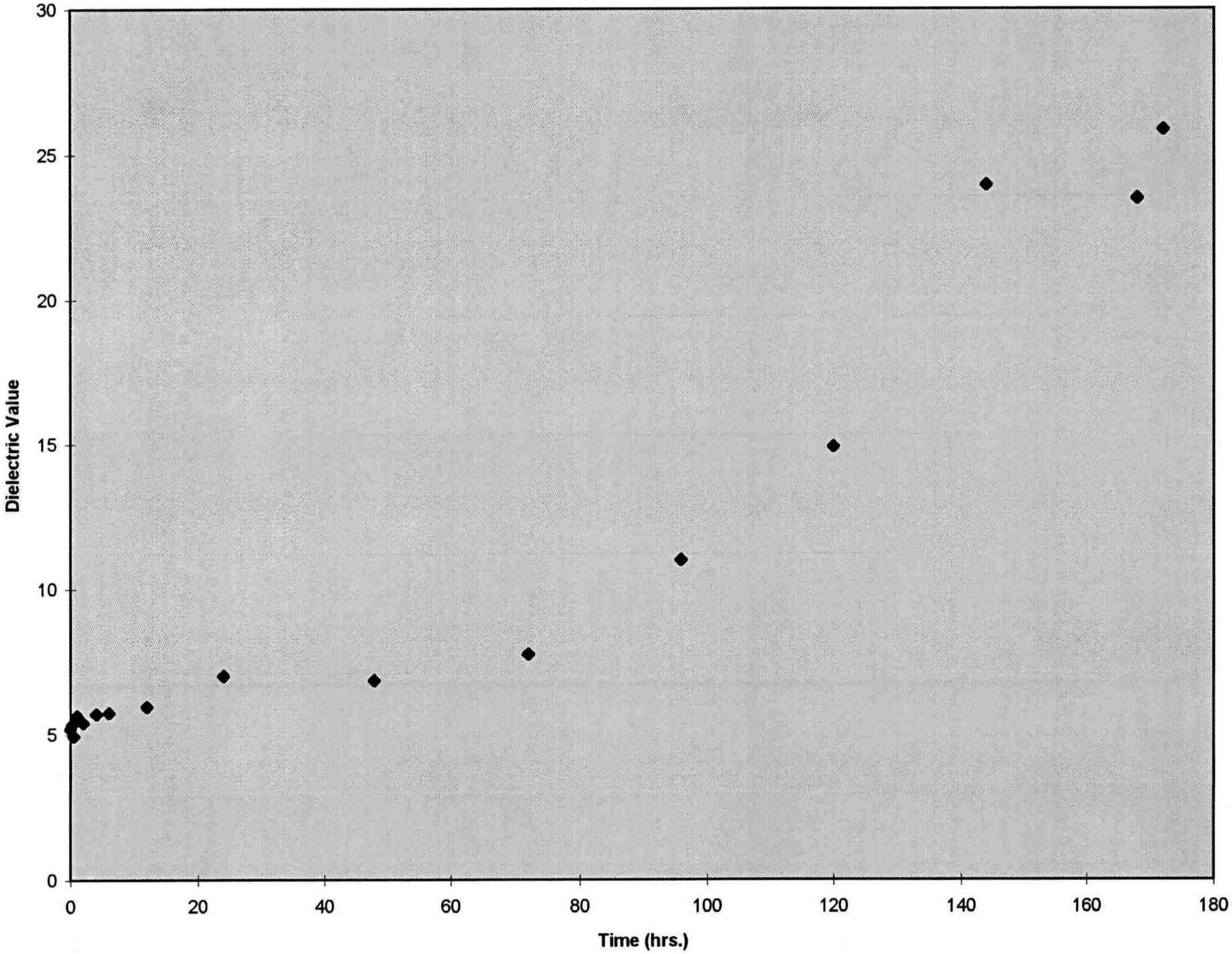
Dielectric Value vs. Time for Tubbs Pit, Abilene District
Dielectric Value
Time (hrs.)
0
5
10
15
20
25
30
35
40
0
20
40
60
80
100
120
140
160
180
Tubbs Pit #1
Tubbs Pit #2

C.10.

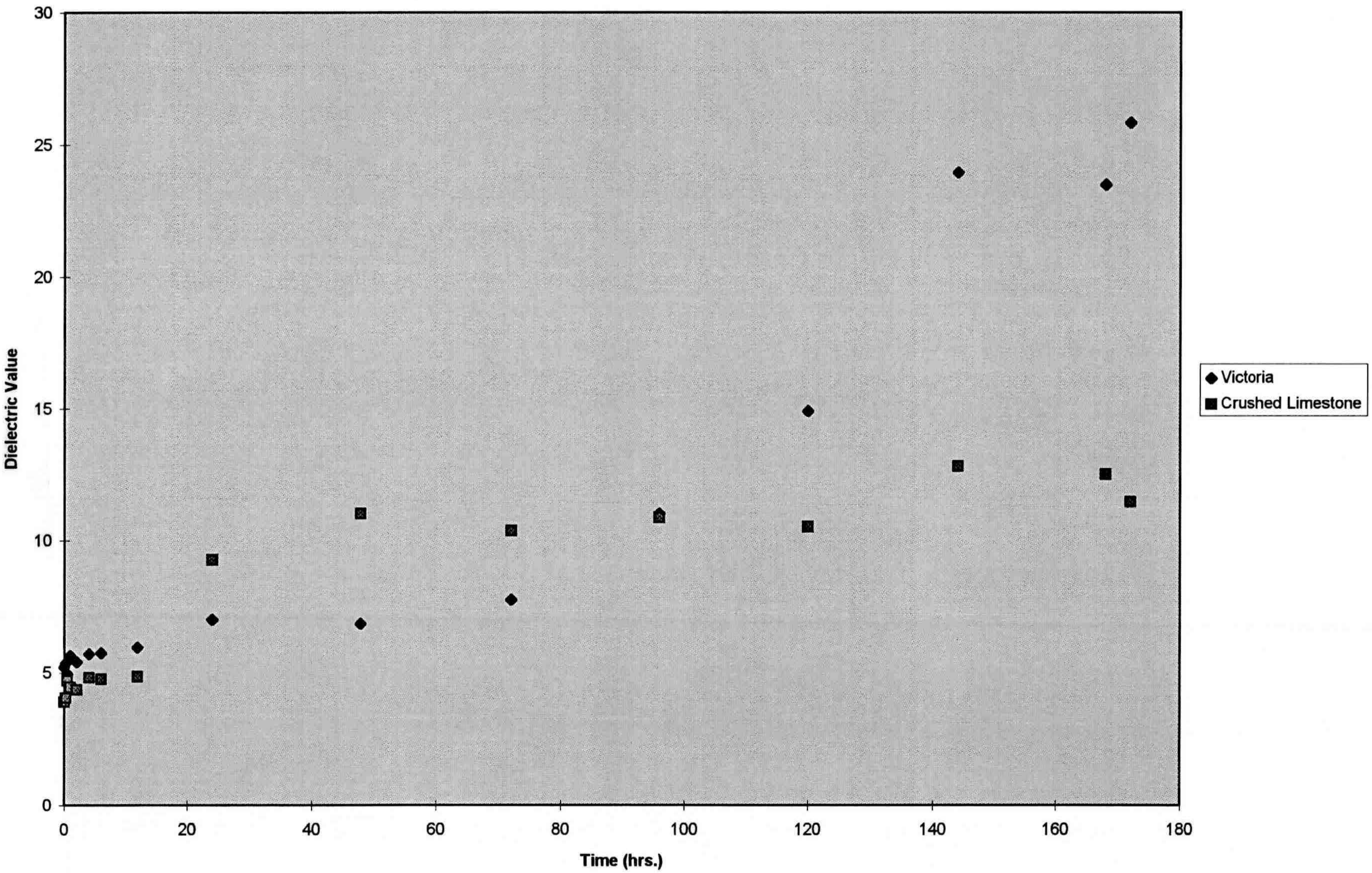


Dielectric Value vs. Time for TTI Control Limestone and Abilene Source from Highway US83
Dielectric Value
Time (hrs.)
0
2
4
6
8
10
12
14
16
18
20
0
50
100
150
200
250
300
Crushed Limestone
Abilene #1
Abilene #2
Abilene #3

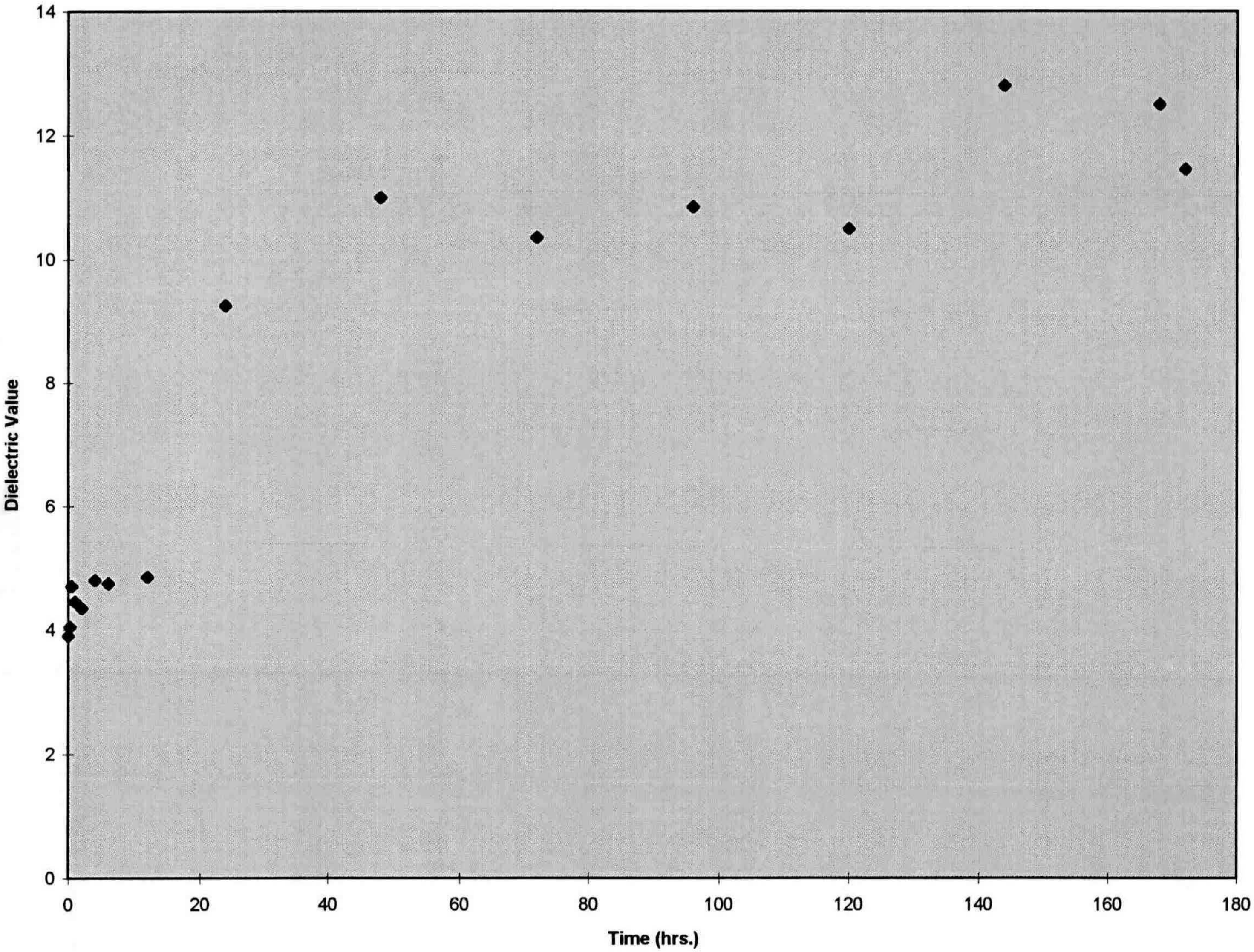
Dielectric Value vs. Time for Victoria Source, Yoakum District



Dielectric Value vs. Time for Victoria Source and TTI Control Limestone

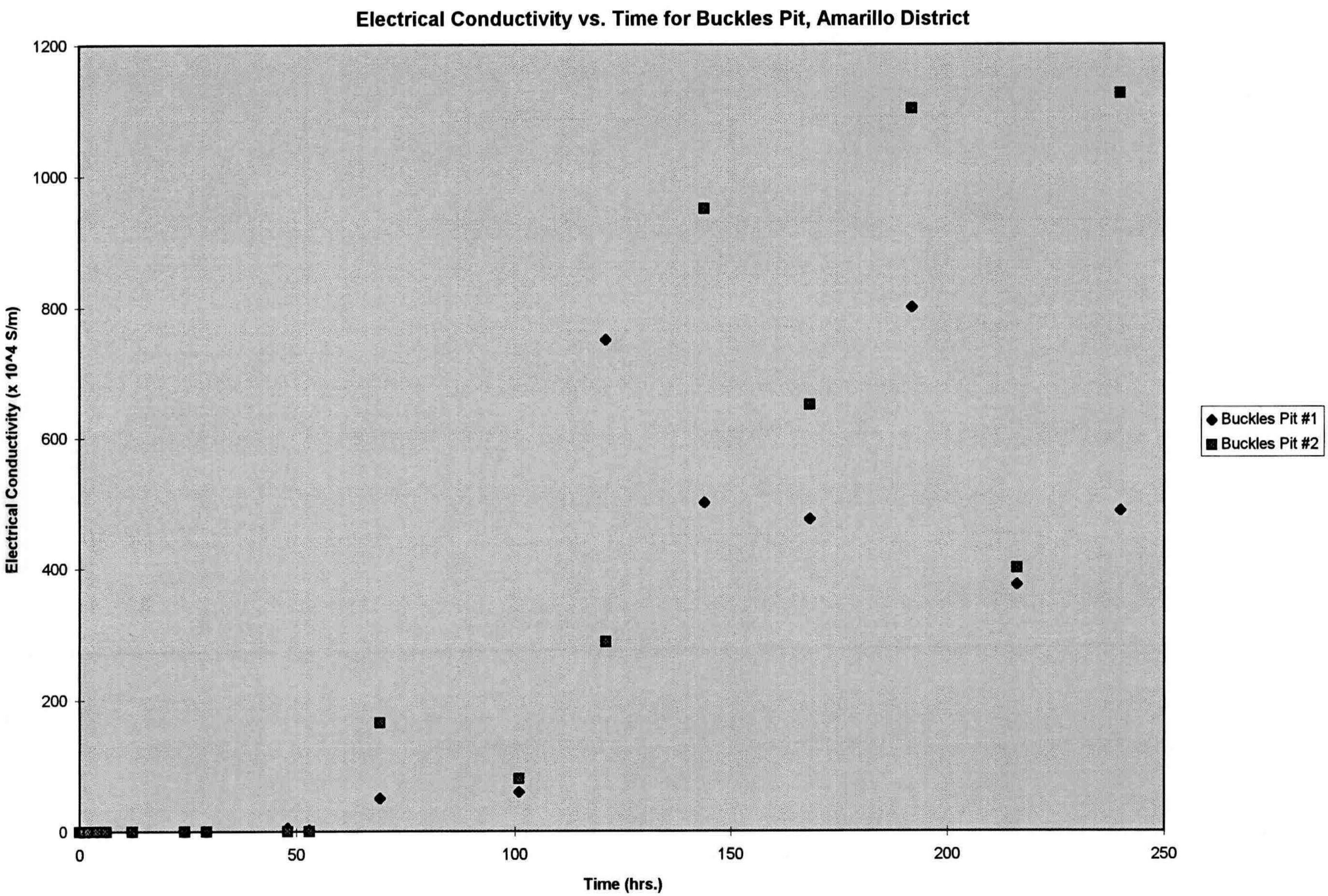


Dielectric Value vs. Time for TTI Control Limestone



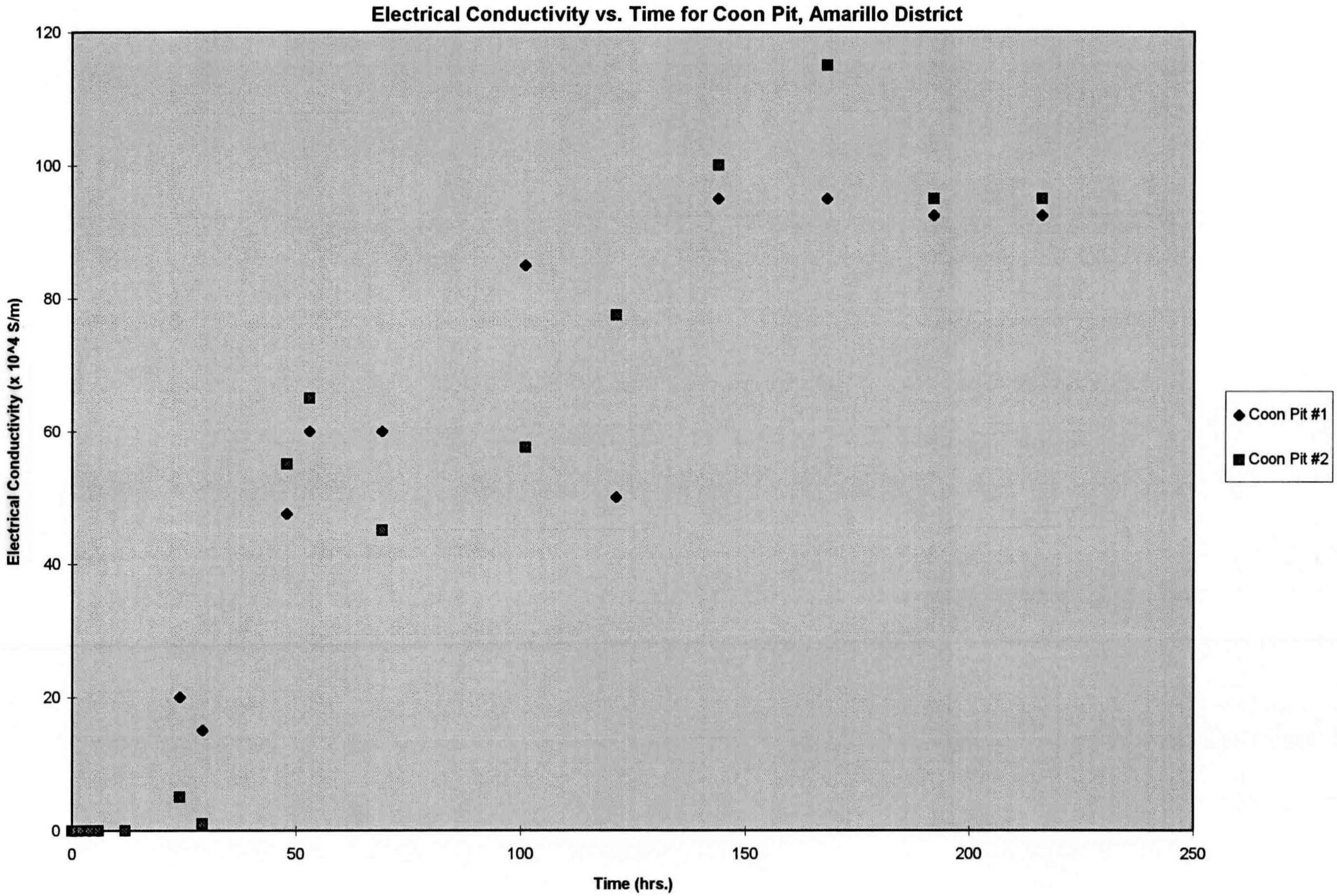
APPENDIX D:

PLOTS OF THE ELECTRICAL CONDUCTIVITY v. TIME OF TESTING (CAPILLARY SOAK) FOR THE VARIOUS SOURCE MATERIALS EVALUATED IN THE PHASE 1 TESTING PROGRAM

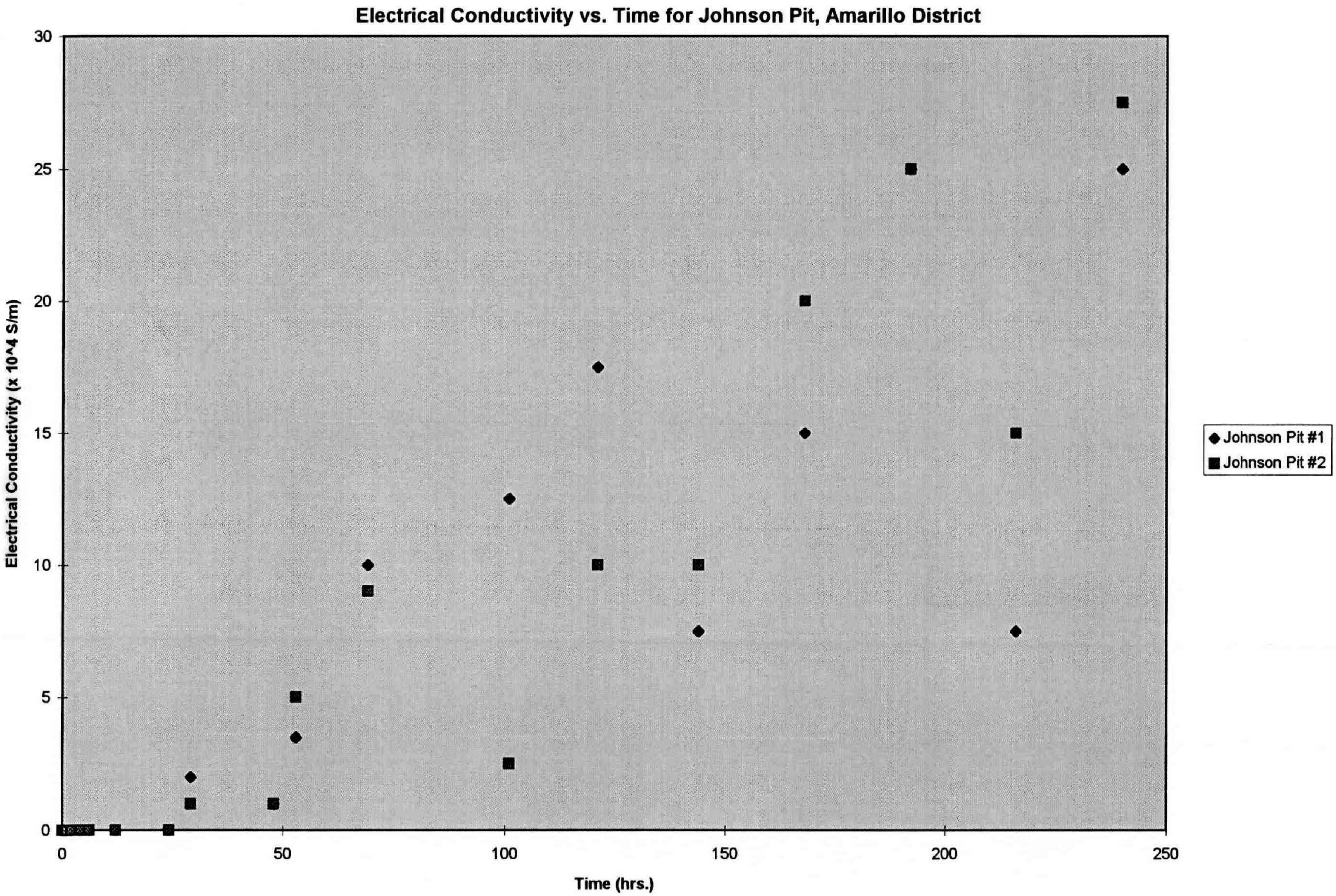

Electrical Conductivity vs. Time for Buckles Pit, Amarillo District
Electrical Conductivity (x 10^4 S/m)
Time (hrs.)
0
200
400
600
800
1000
1200
0
50
100
150
200
250
Buckles Pit #1
Buckles Pit #2

D.2.

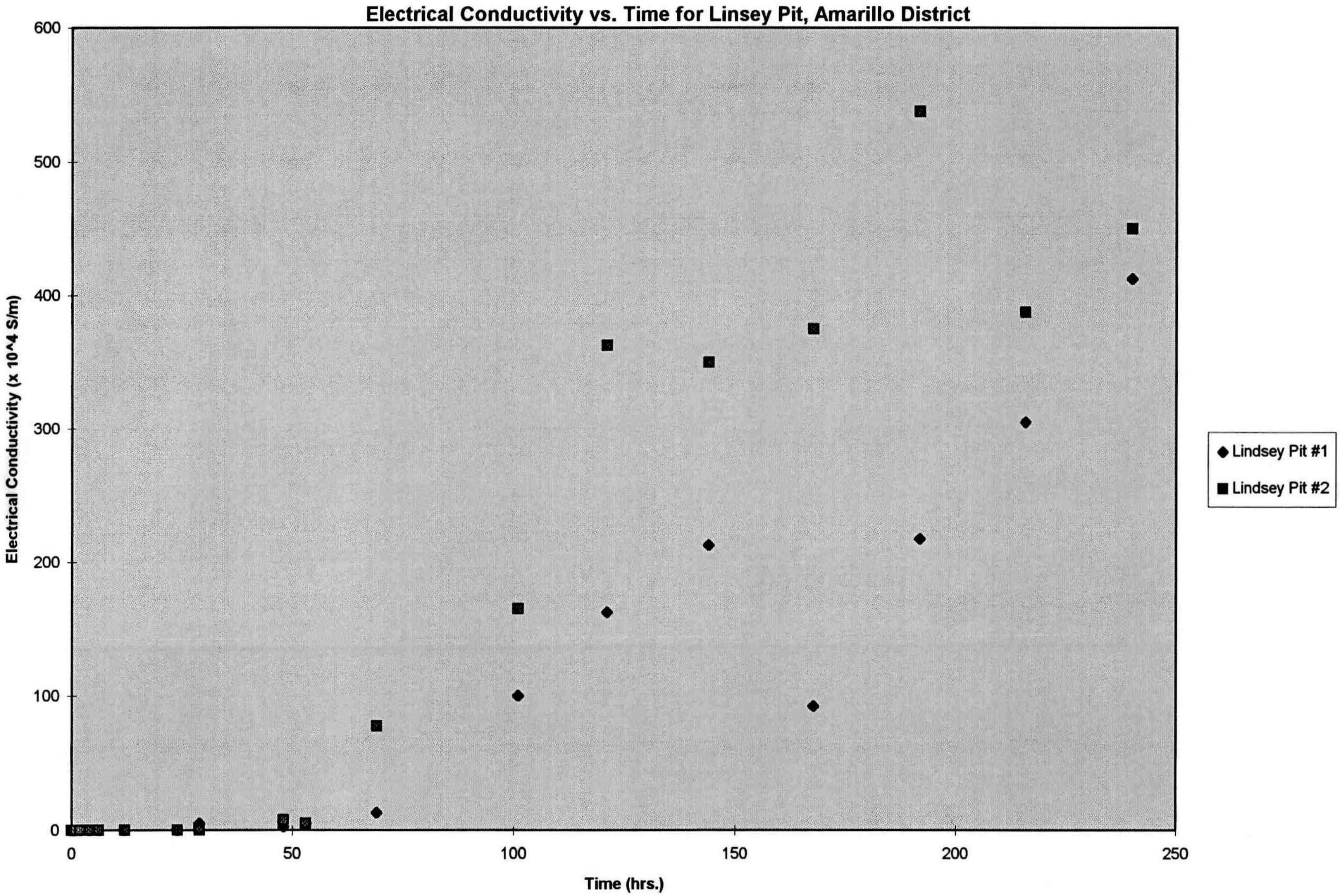
D.3.



Electrical Conductivity vs. Time for Coon Pit, Amarillo District
Electrical Conductivity (x 10^4 S/m)
Time (hrs.)
0
20
40
60
80
100
120
0
50
100
150
200
250
Coon Pit #1
Coon Pit #2

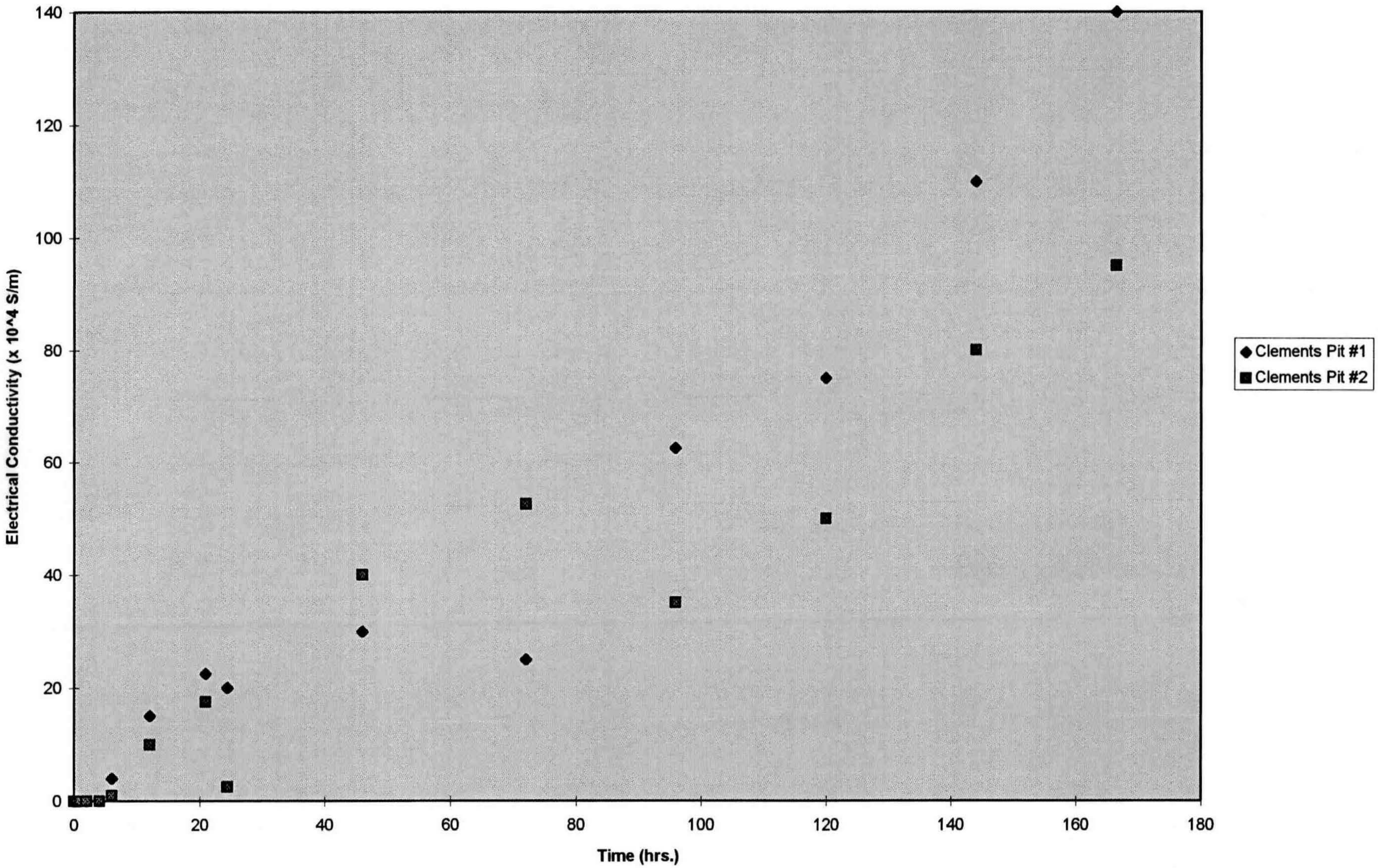

Electrical Conductivity vs. Time for Johnson Pit, Amarillo District
Electrical Conductivity (x 10^4 S/m)
Time (hrs.)
0
5
10
15
20
25
30
0
50
100
150
200
250
Johnson Pit #1
Johnson Pit #2

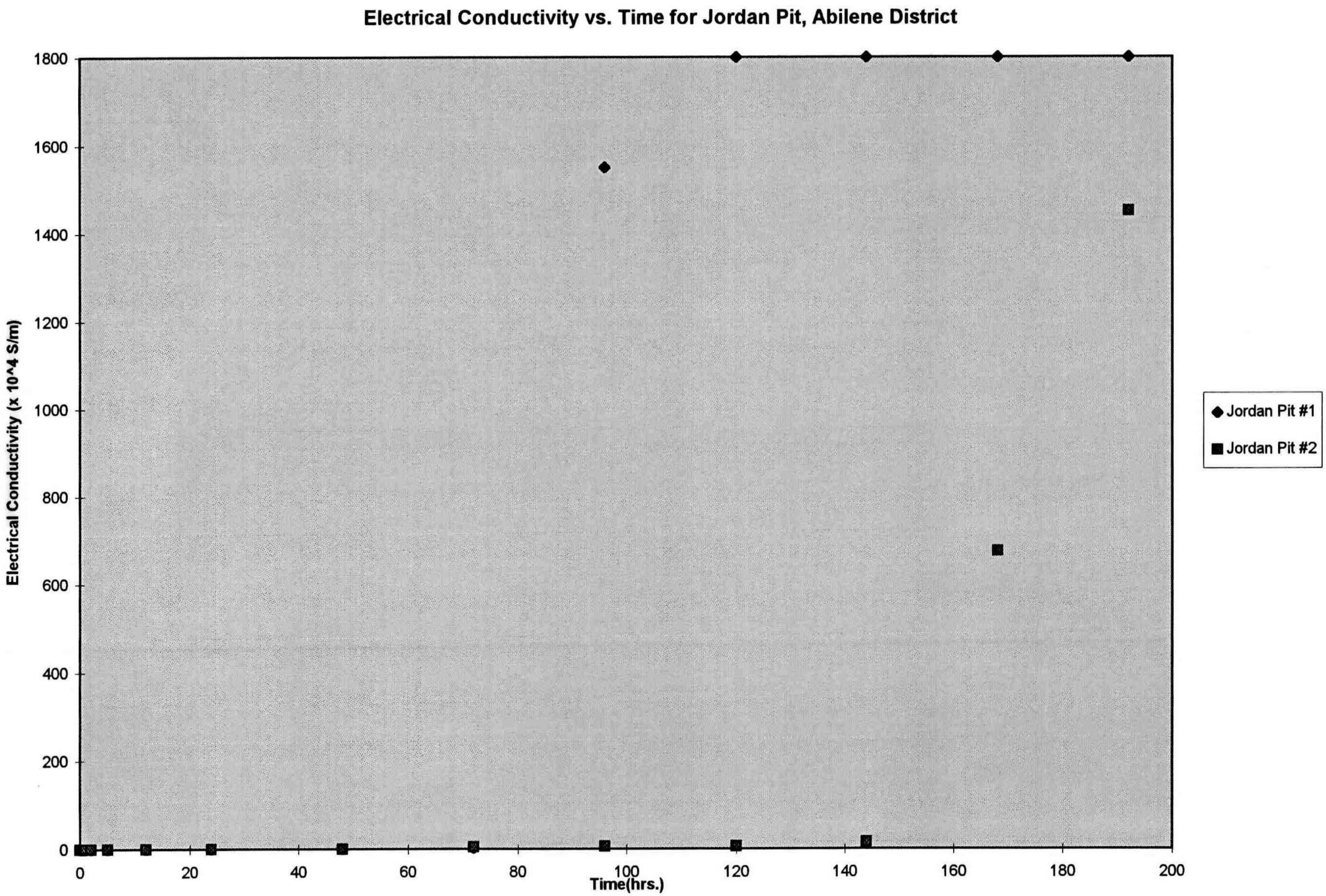
D.5.



Electrical Conductivity vs. Time for Linsey Pit, Amarillo District
Electrical Conductivity (x 10^4 S/m)
600
500
400
300
200
100
0
0
50
100
150
200
250
Time (hrs.)
Lindsey Pit #1
Lindsey Pit #2

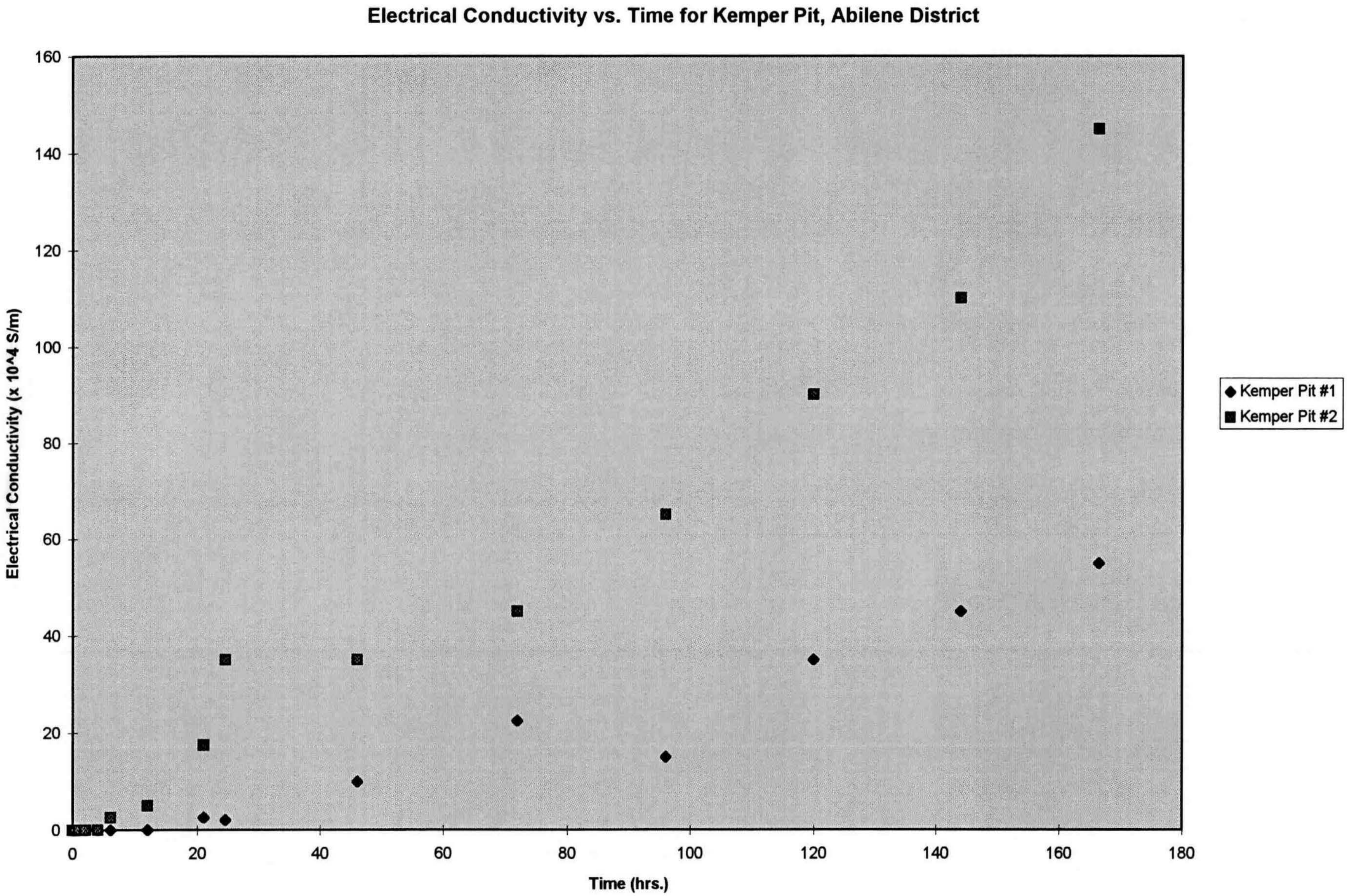
Electrical Conductivity vs. Time for Clements Pit, Abilene District

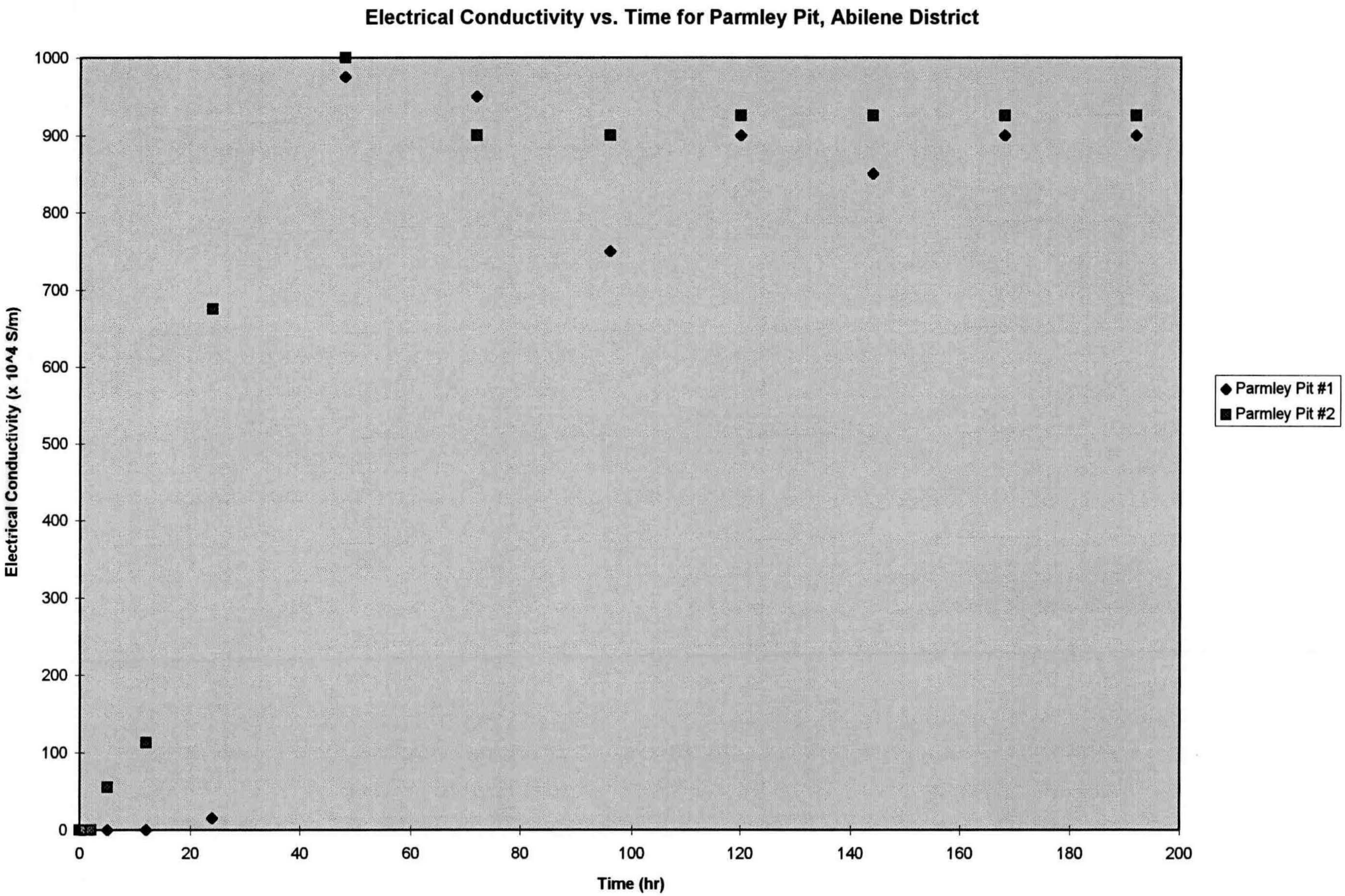


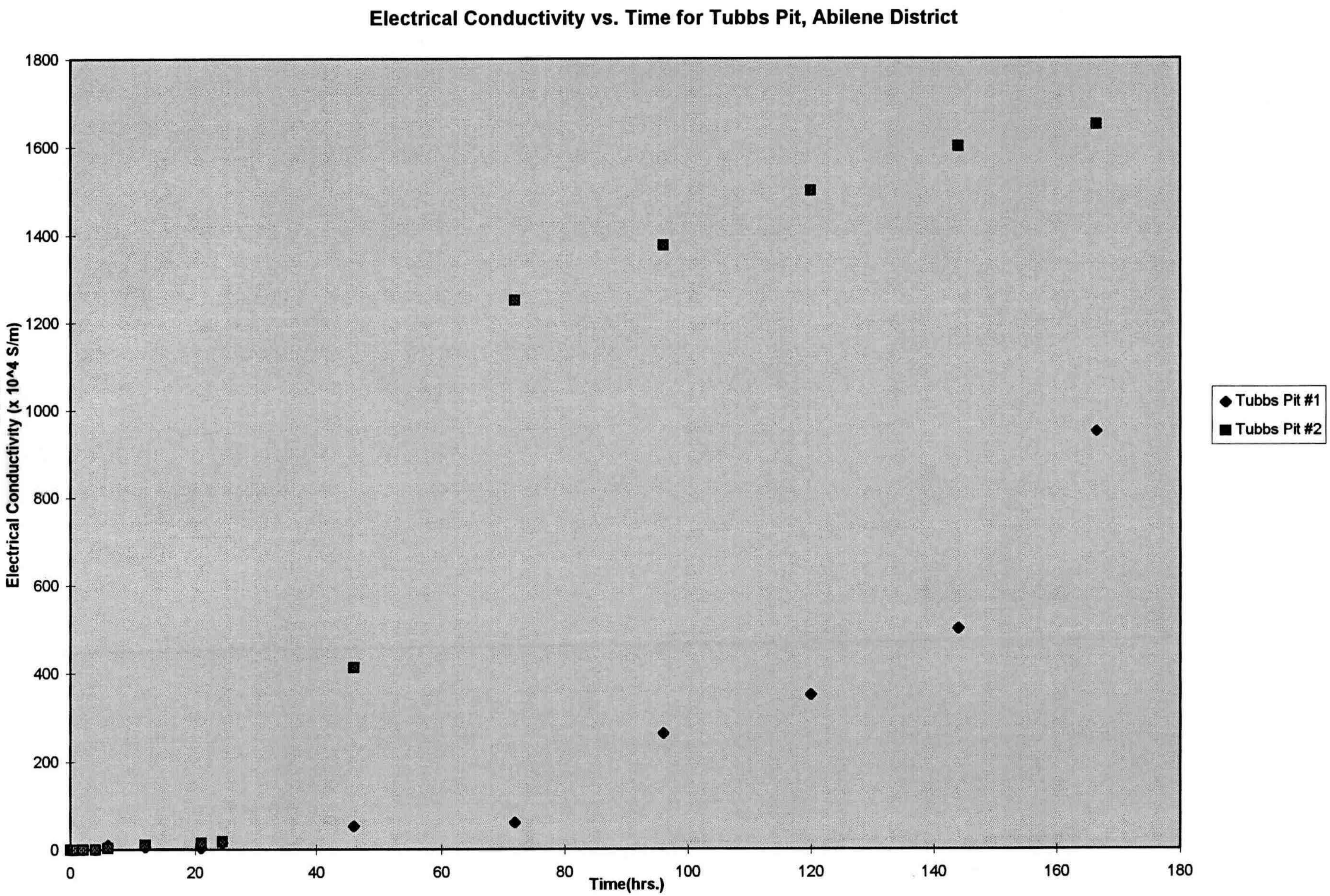

Electrical Conductivity vs. Time for Jordan Pit, Abilene District
Electrical Conductivity (x 10^4 S/m)
Time(hrs.)
0
200
400
600
800
1000
1200
1400
1600
1800
0
20
40
60
80
100
120
140
160
180
200
Jordan Pit #1
Jordan Pit #2

D.7.

D.8.


Electrical Conductivity vs. Time for Kemper Pit, Abilene District
Electrical Conductivity (x 10^4 S/m)
Time (hrs.)
0
20
40
60
80
100
120
140
160
180
Kemper Pit #1
Kemper Pit #2

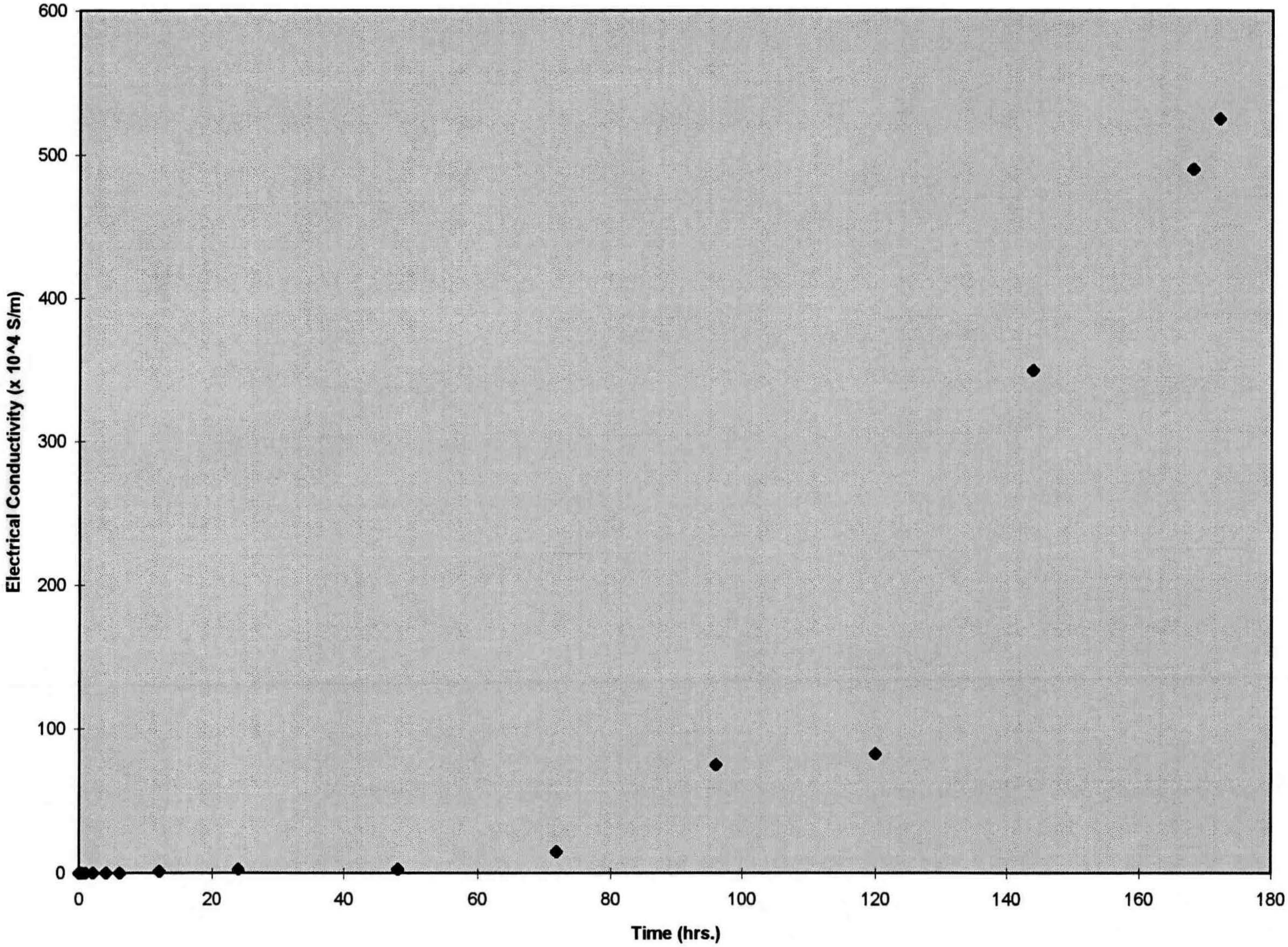

Electrical Conductivity vs. Time for Parmley Pit, Abilene District
Electrical Conductivity (x 10^4 S/m)
1000
900
800
700
600
500
400
300
200
100
0
0
20
40
60
80
100
120
140
160
180
200
Time (hr)
Parmley Pit #1
Parmley Pit #2



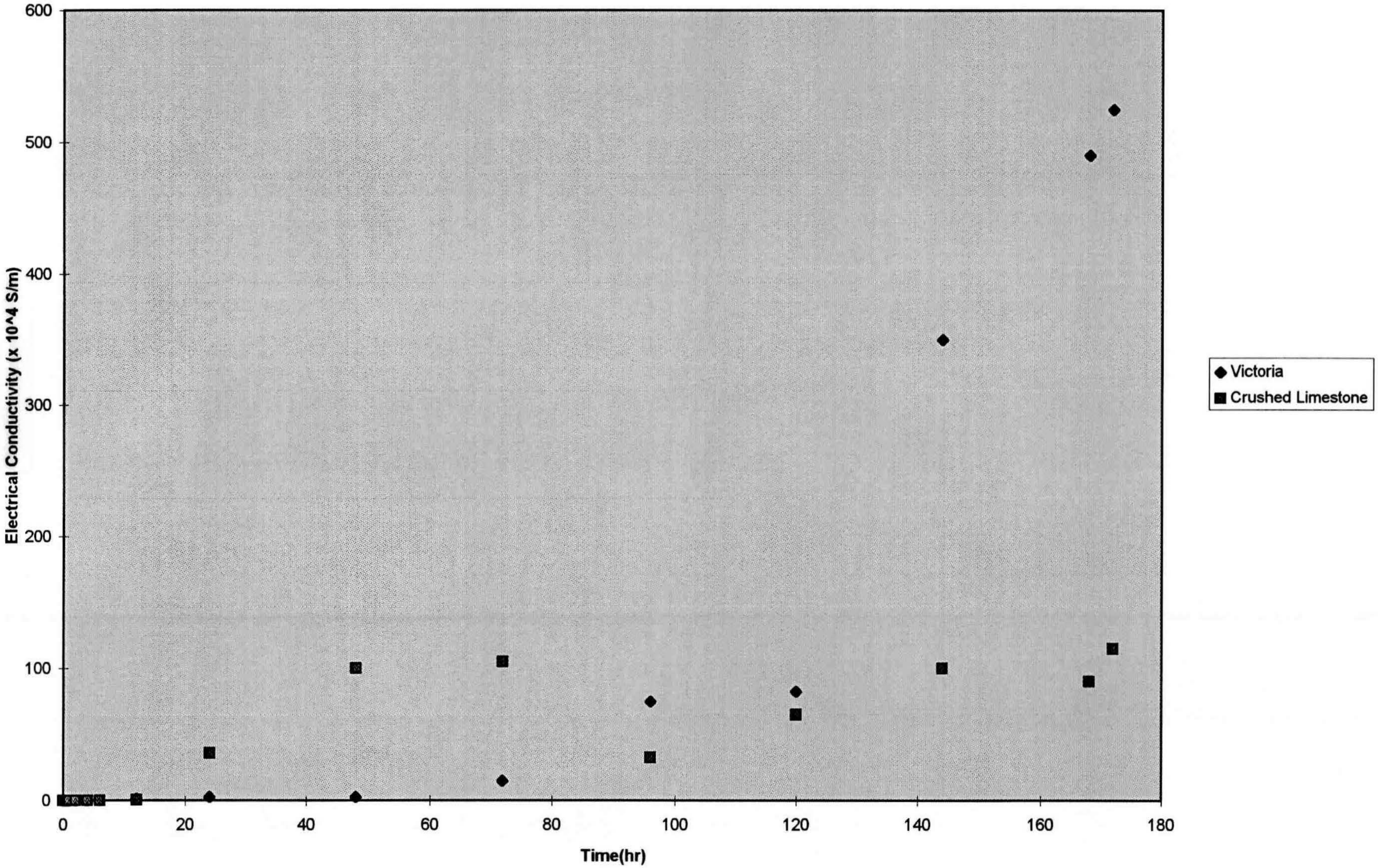
Electrical Conductivity vs. Time for Tubbs Pit, Abilene District
Electrical Conductivity (x 10^4 S/m)
Time(hrs.)
1800
1600
1400
1200
1000
800
600
400
200
0
0
20
40
60
80
100
120
140
160
180
Tubbs Pit #1
Tubbs Pit #2

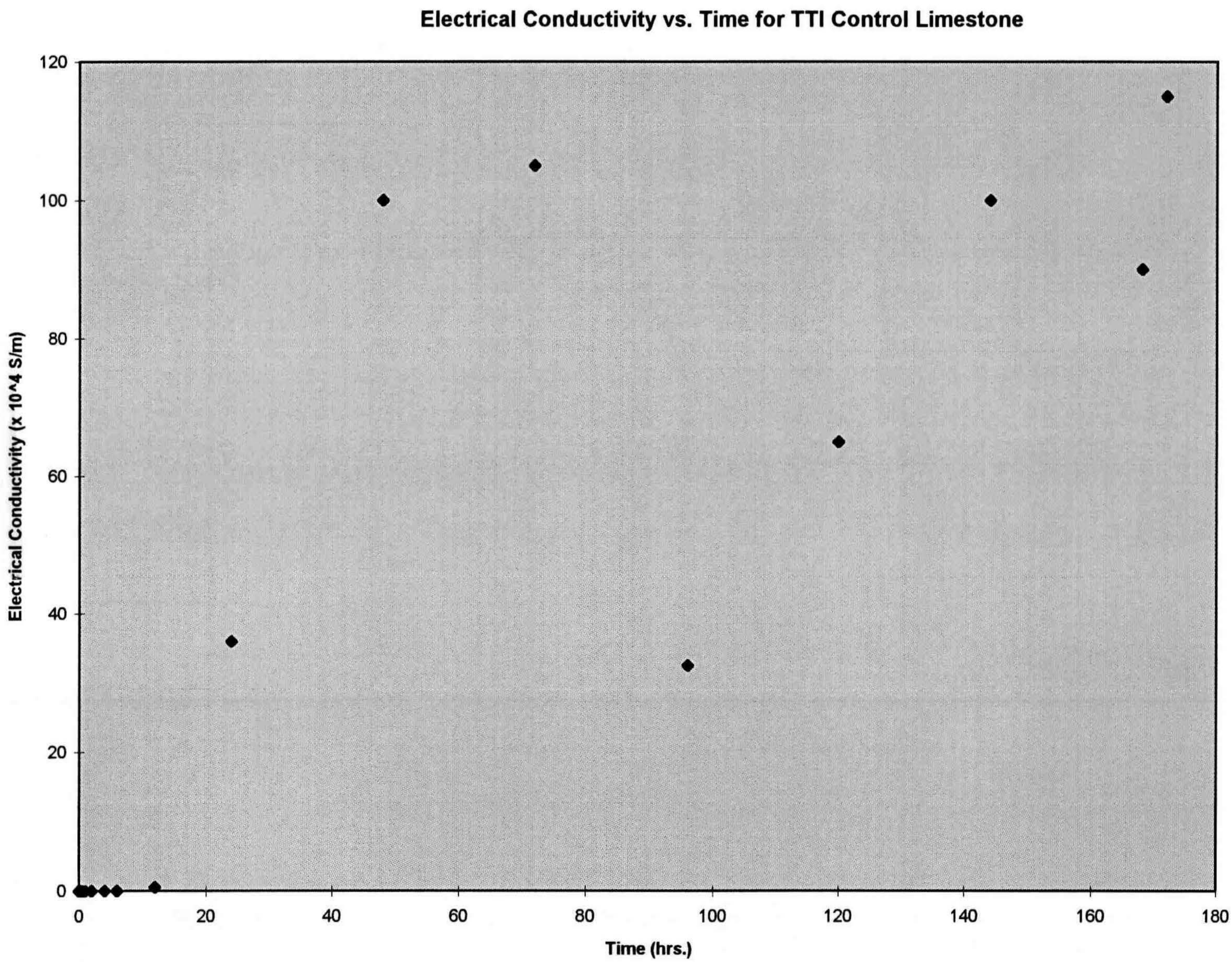
D.10.

Electrical Conductivity vs. Time for Victoria Source, Yoakum District



Electrical Conductivity vs Time for Victoria Source and TTI Control Limestone

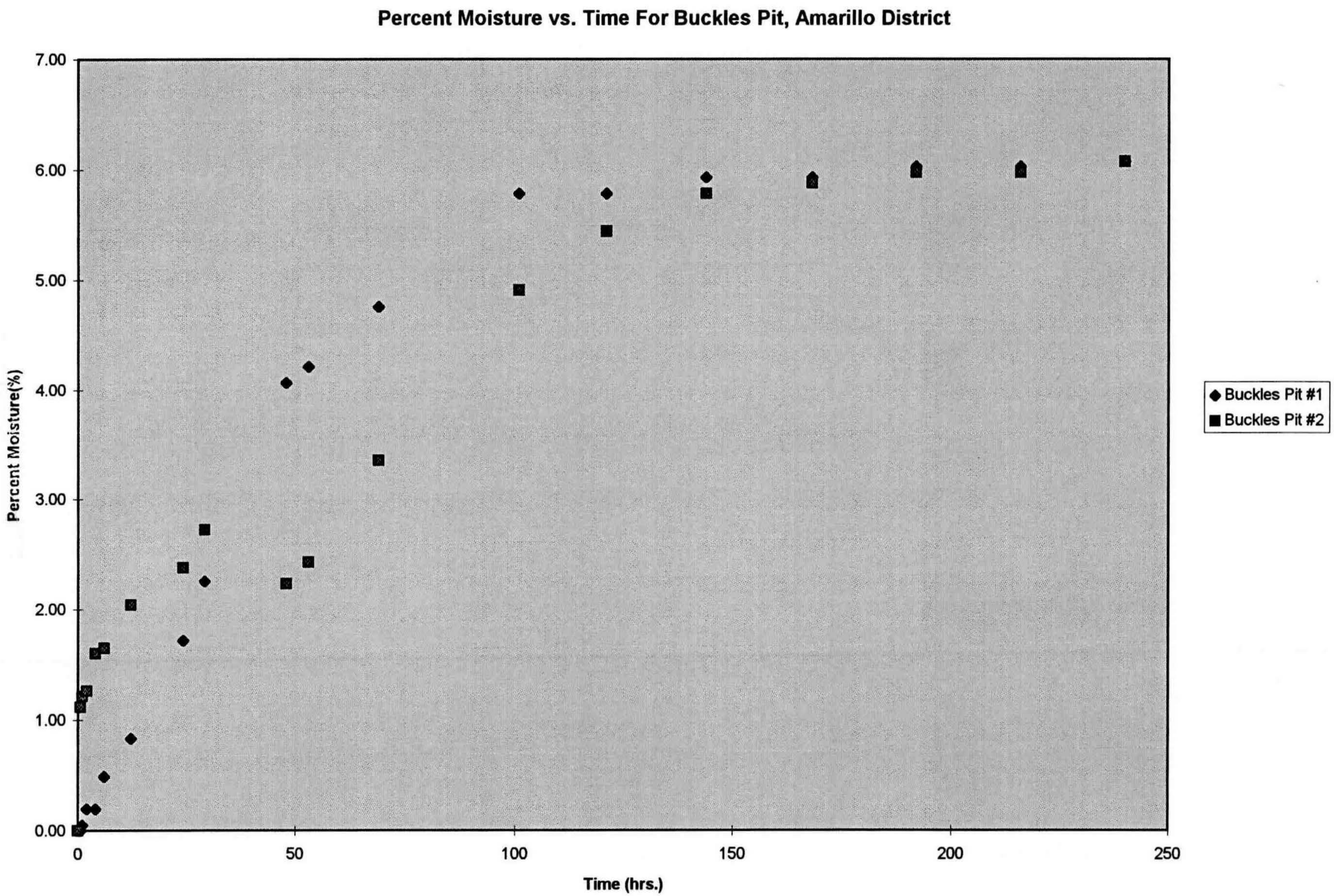



Electrical Conductivity vs. Time for TTI Control Limestone
Electrical Conductivity (x 10^4 S/m)
Time (hrs.)
0
20
40
60
80
100
120
140
160
180

APPENDIX E:

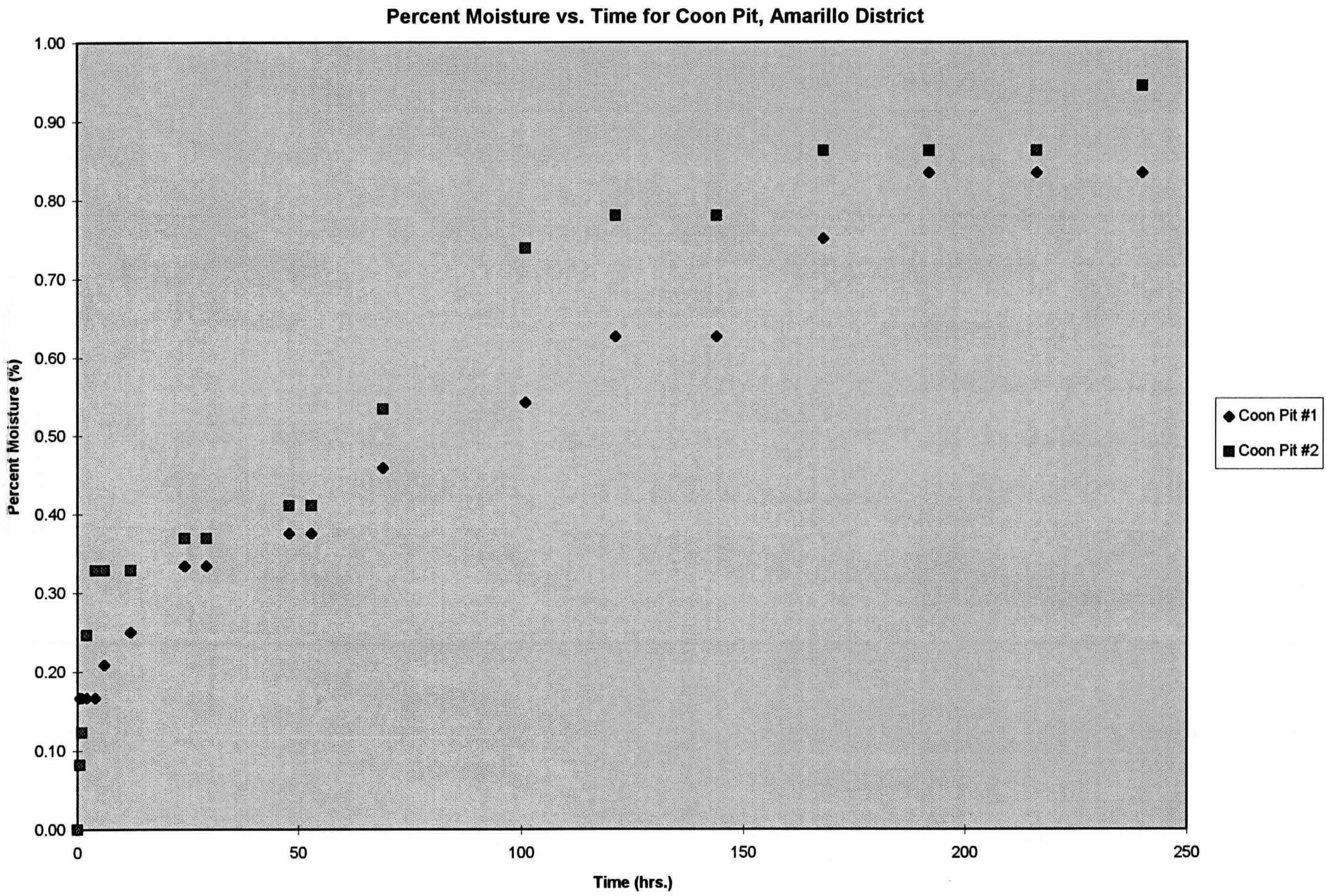
PLOTS OF MOISTURE CONTENT v. TIME OF TESTING (CAPILLARY SOAK) FOR THE VARIOUS SOURCE MATERIALS EVALUATED IN PHASE 1 TESTING PROGRAM

E.2.

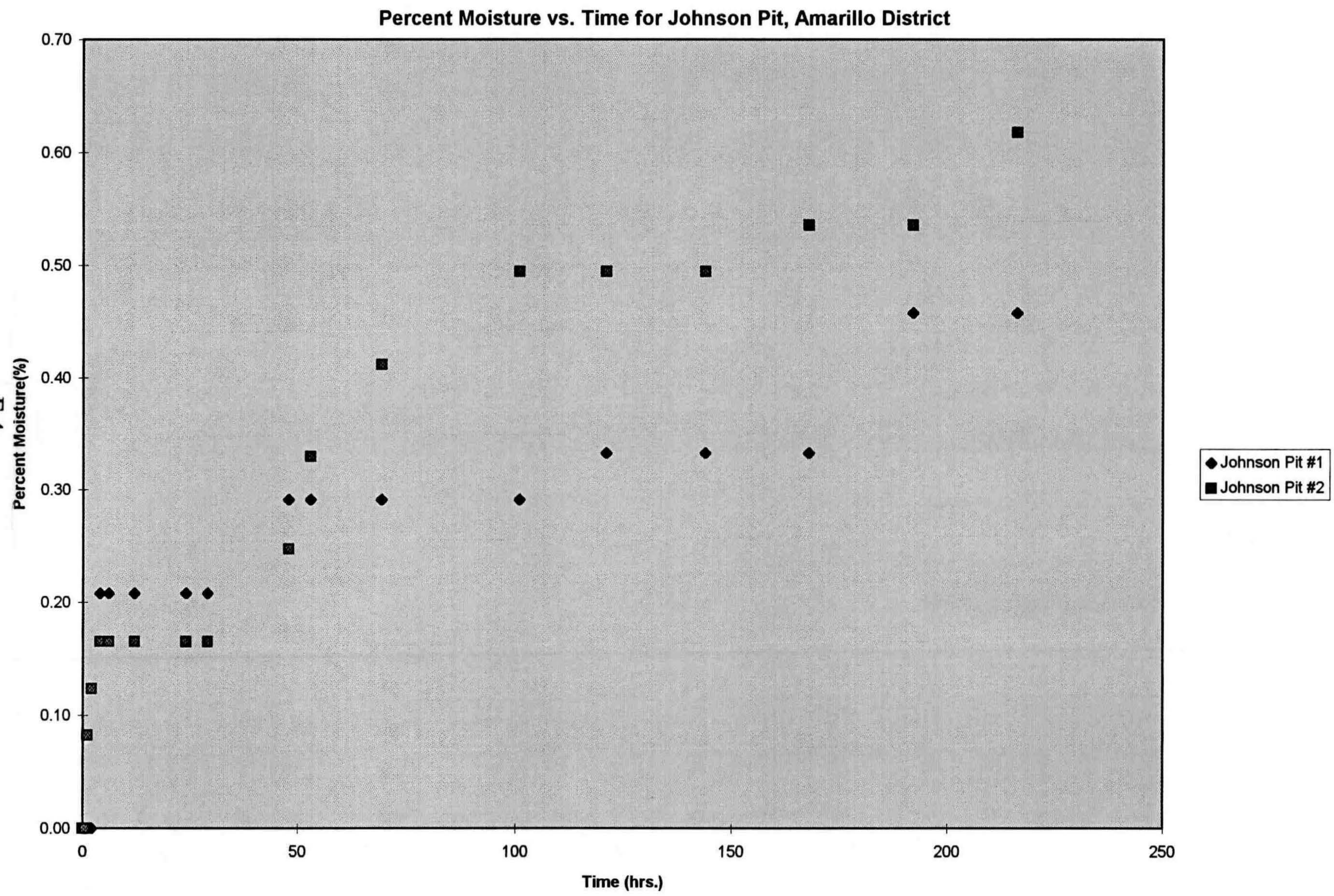


Percent Moisture vs. Time For Buckles Pit, Amarillo District
Percent Moisture(%)
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
50
100
150
200
250
Time (hrs.)
Buckles Pit #1
Buckles Pit #2

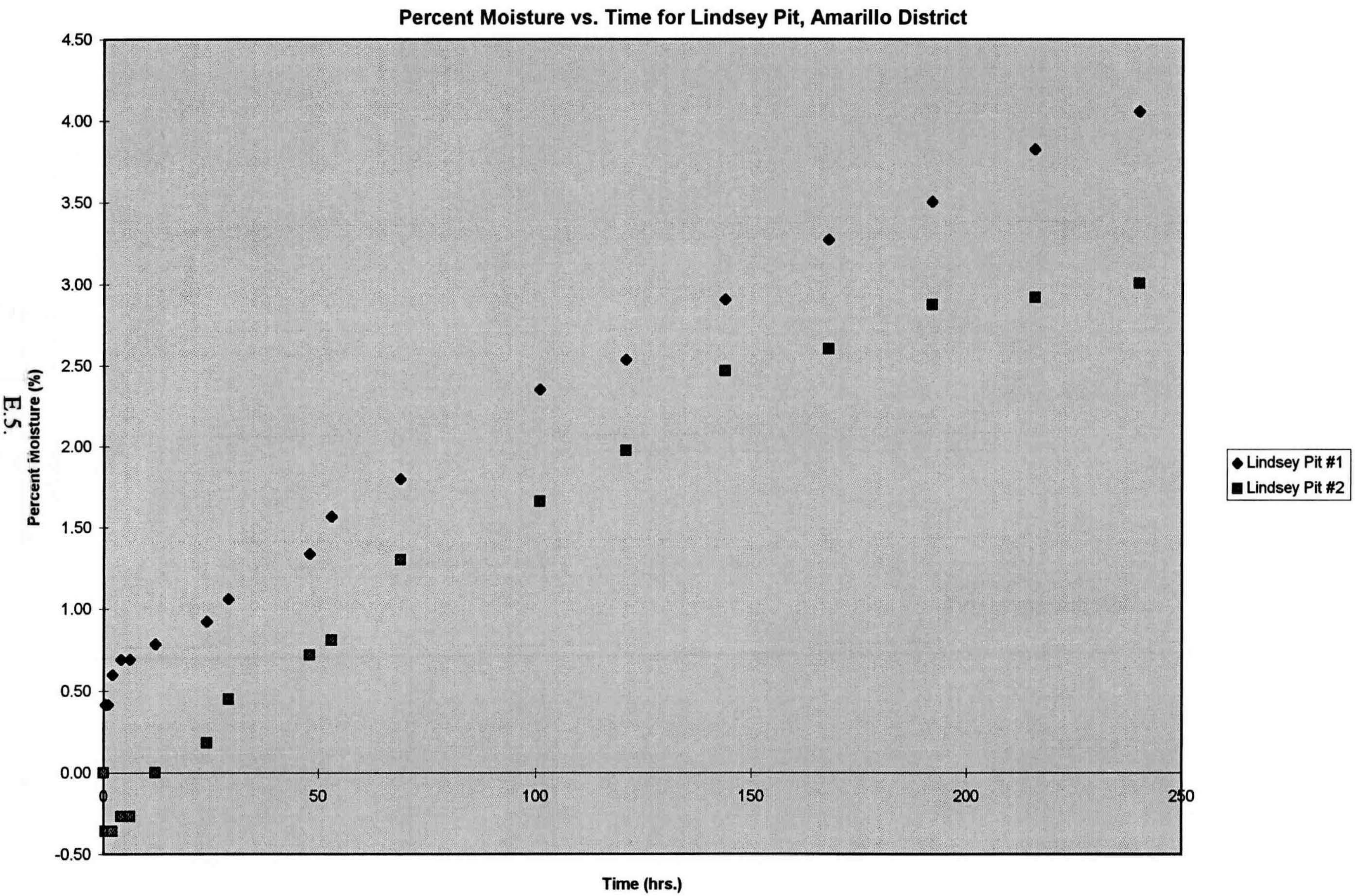
E.3.



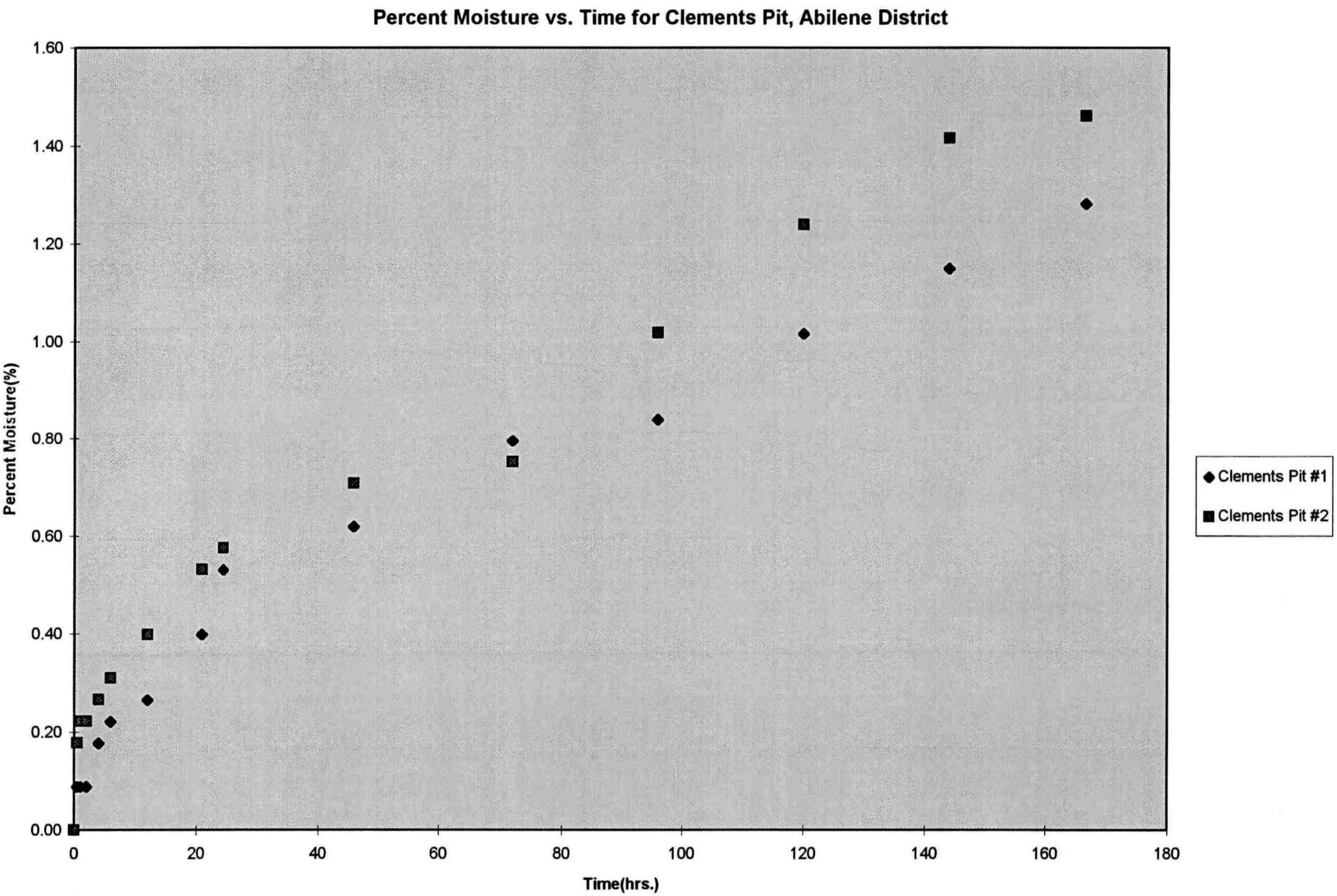
Percent Moisture vs. Time for Coon Pit, Amarillo District
Percent Moisture (%)
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
50
100
150
200
250
Time (hrs.)
Coon Pit #1
Coon Pit #2

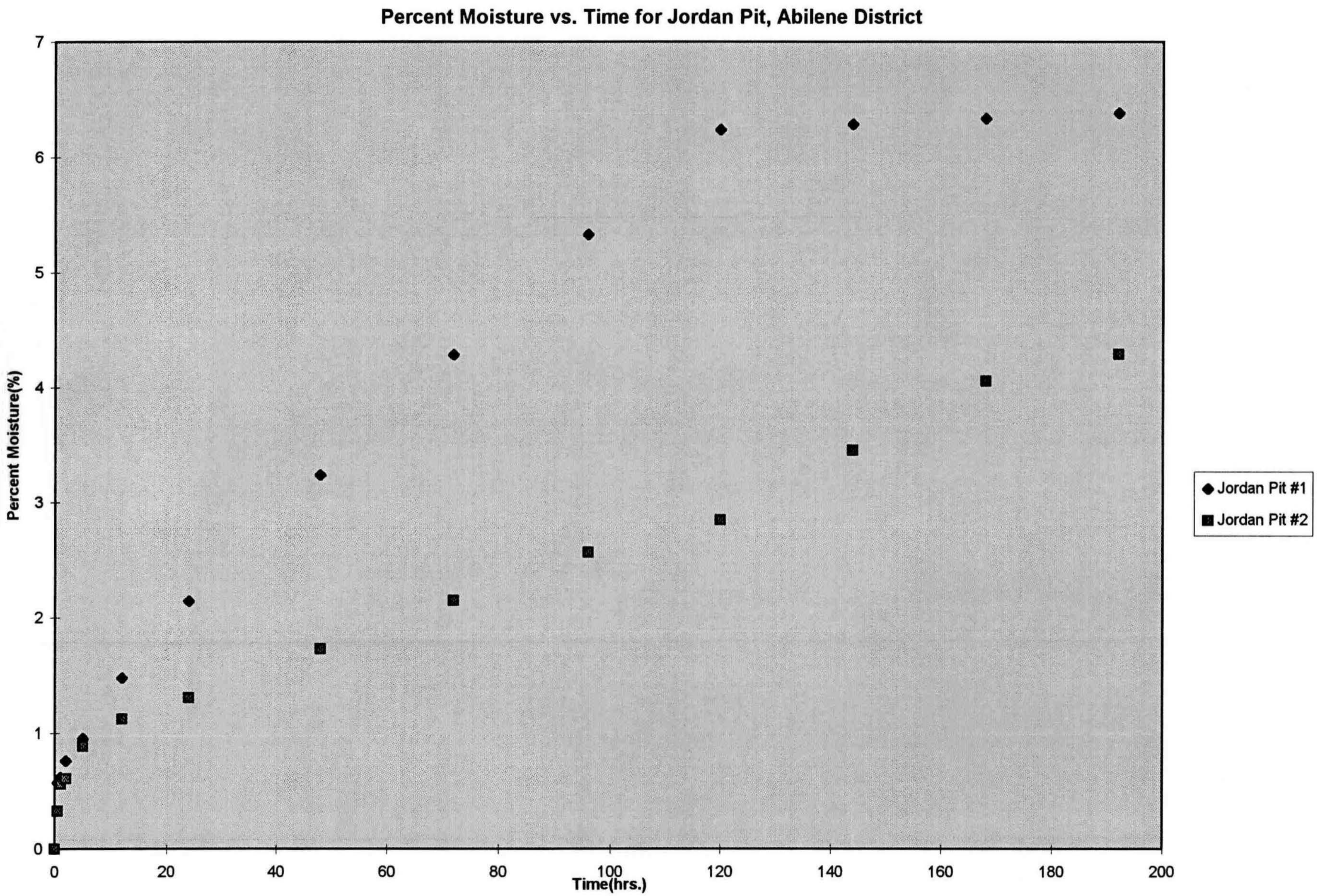

Percent Moisture vs. Time for Johnson Pit, Amarillo District
Percent Moisture(%)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
50
100
150
200
250
Time (hrs.)
Johnson Pit #1
Johnson Pit #2

E.5.

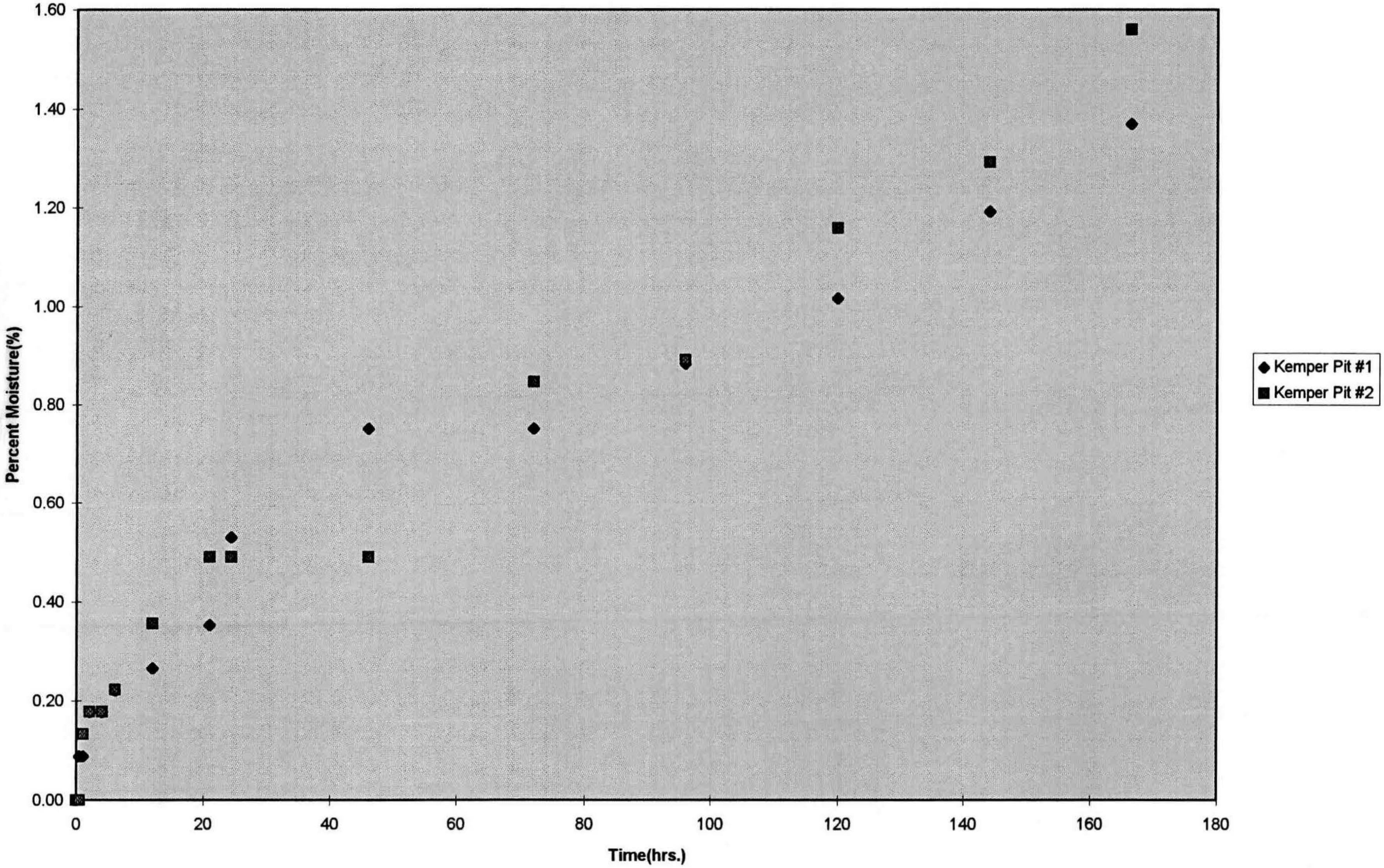


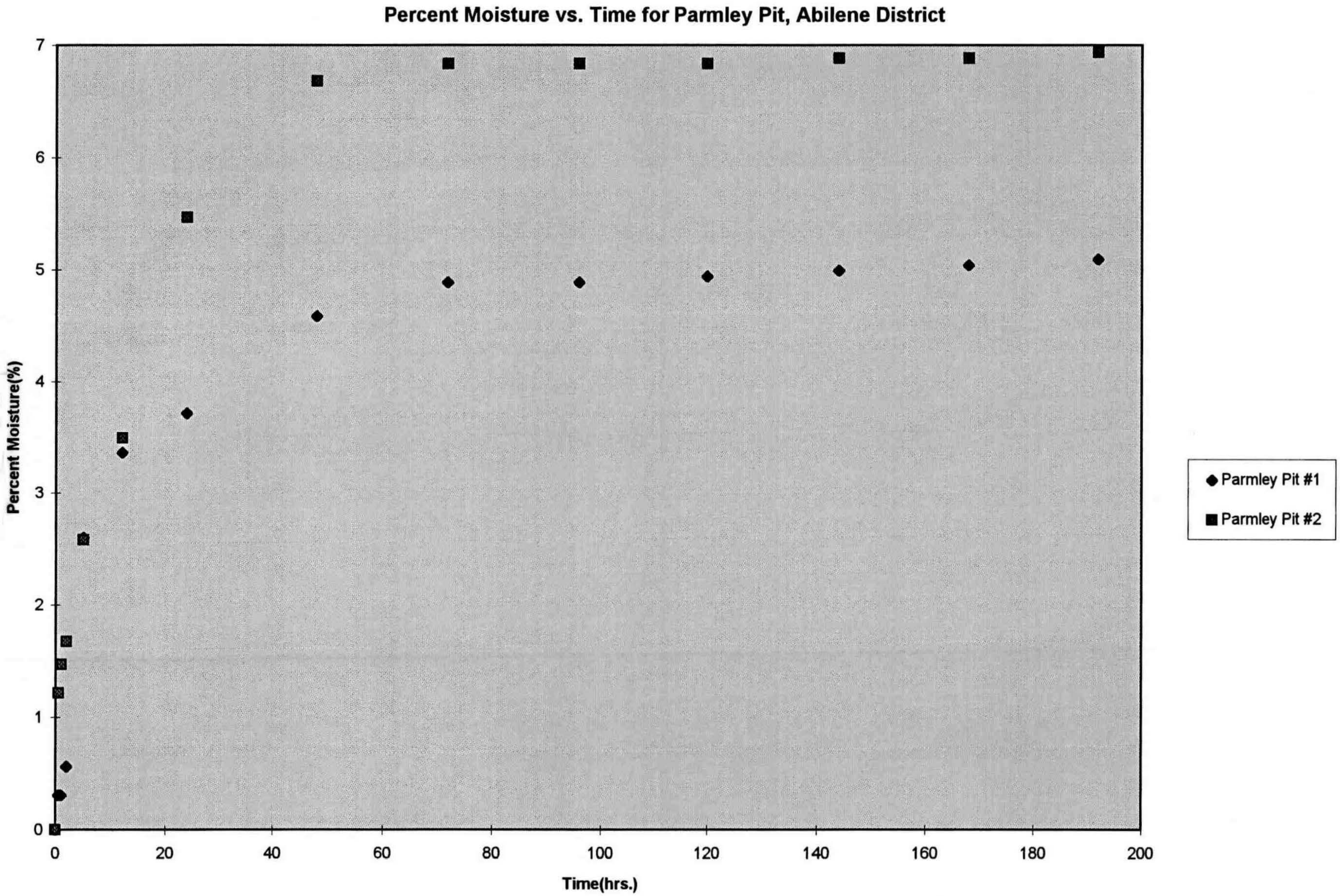
Percent Moisture vs. Time for Lindsey Pit, Amarillo District
Percent Moisture (%)
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
0
50
100
150
200
250
Time (hrs.)
Lindsey Pit #1
Lindsey Pit #2


Percent Moisture vs. Time for Clements Pit, Abilene District
Percent Moisture(%)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
Time(hrs.)
0
20
40
60
80
100
120
140
160
180
Clements Pit #1
Clements Pit #2

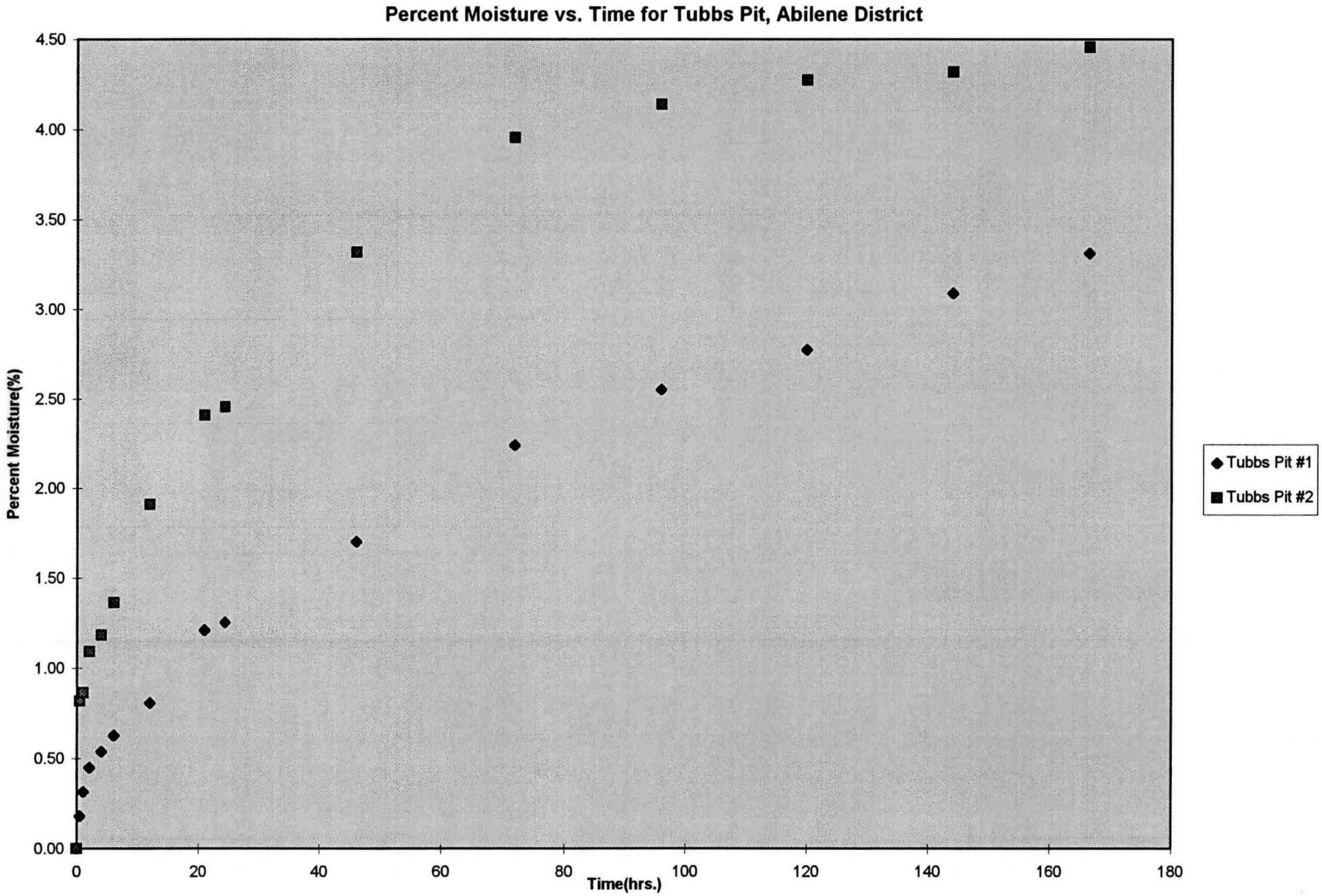

Percent Moisture vs. Time for Jordan Pit, Abilene District
Percent Moisture(%)
0
1
2
3
4
5
6
7
Time(hrs.)
0
20
40
60
80
100
120
140
160
180
200
Jordan Pit #1
Jordan Pit #2

Percent Moisture vs. Time for Kemper Pit, Abilene District



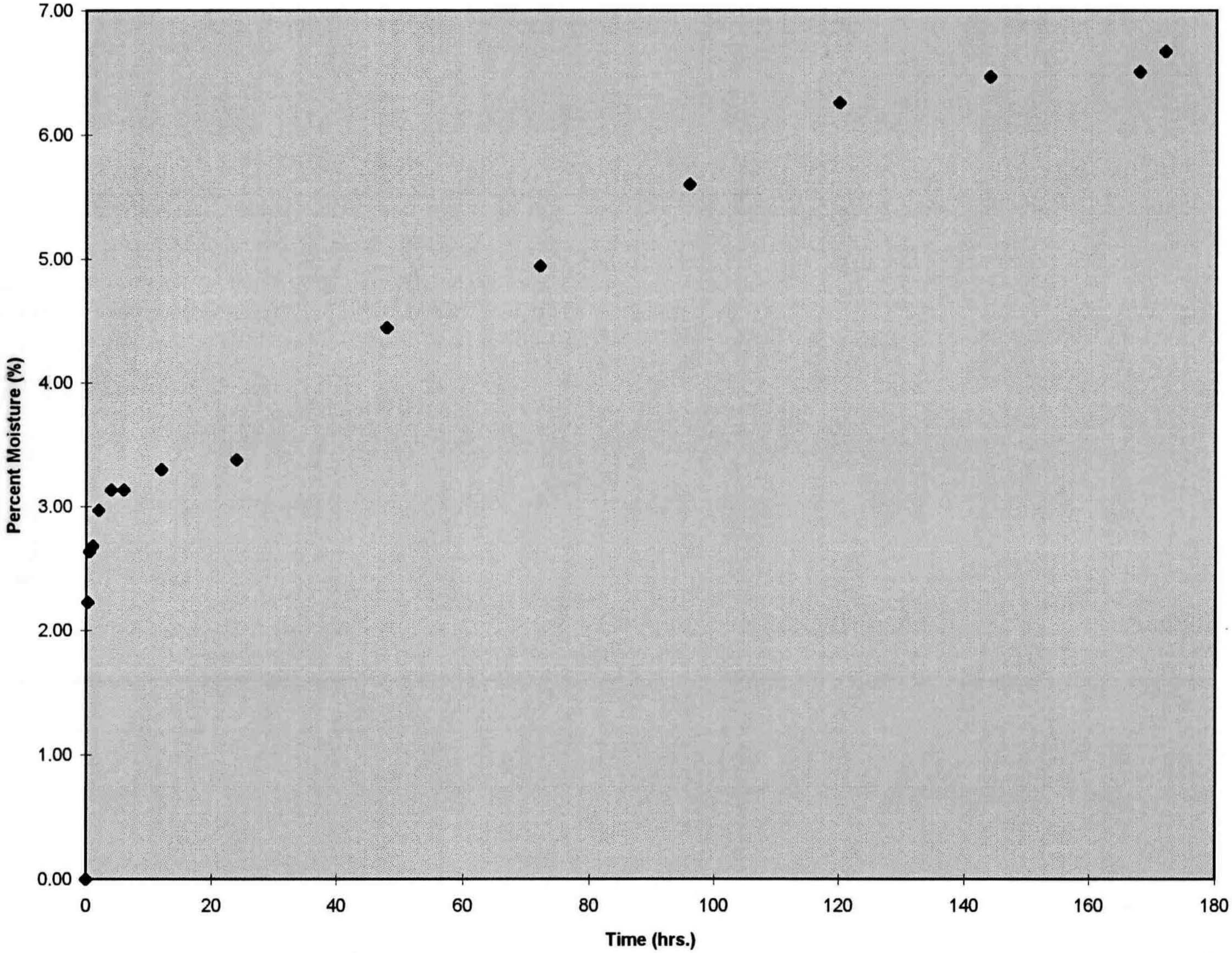

Percent Moisture vs. Time for Parmley Pit, Abilene District
Percent Moisture(%)
Time(hrs.)
Parmley Pit #1
Parmley Pit #2
0
1
2
3
4
5
6
7
0
20
40
60
80
100
120
140
160
180
200

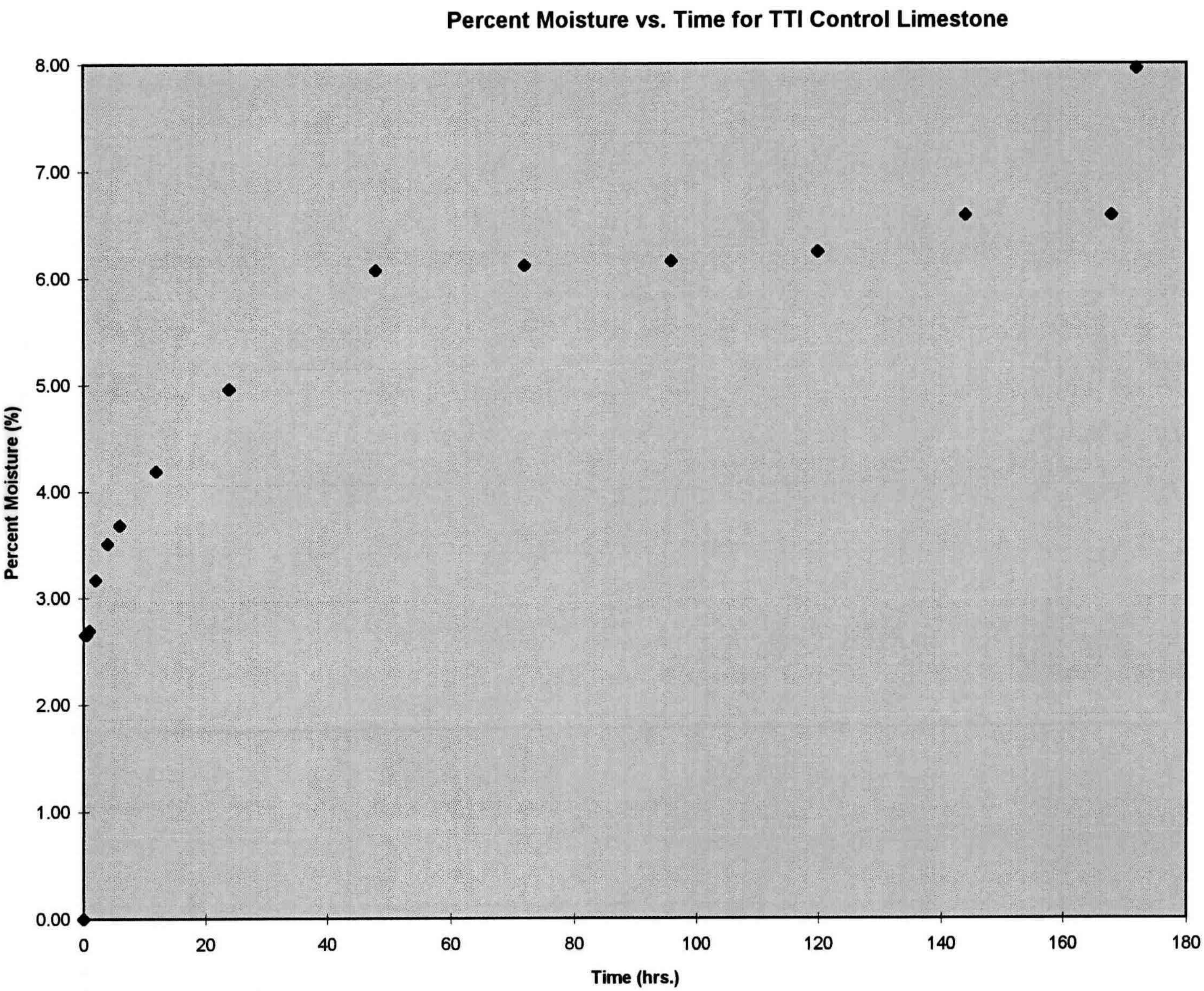
E.10.



Percent Moisture vs. Time for Tubbs Pit, Abilene District
Percent Moisture(%)
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0
20
40
60
80
100
120
140
160
180
Time(hrs.)
Tubbs Pit #1
Tubbs Pit #2

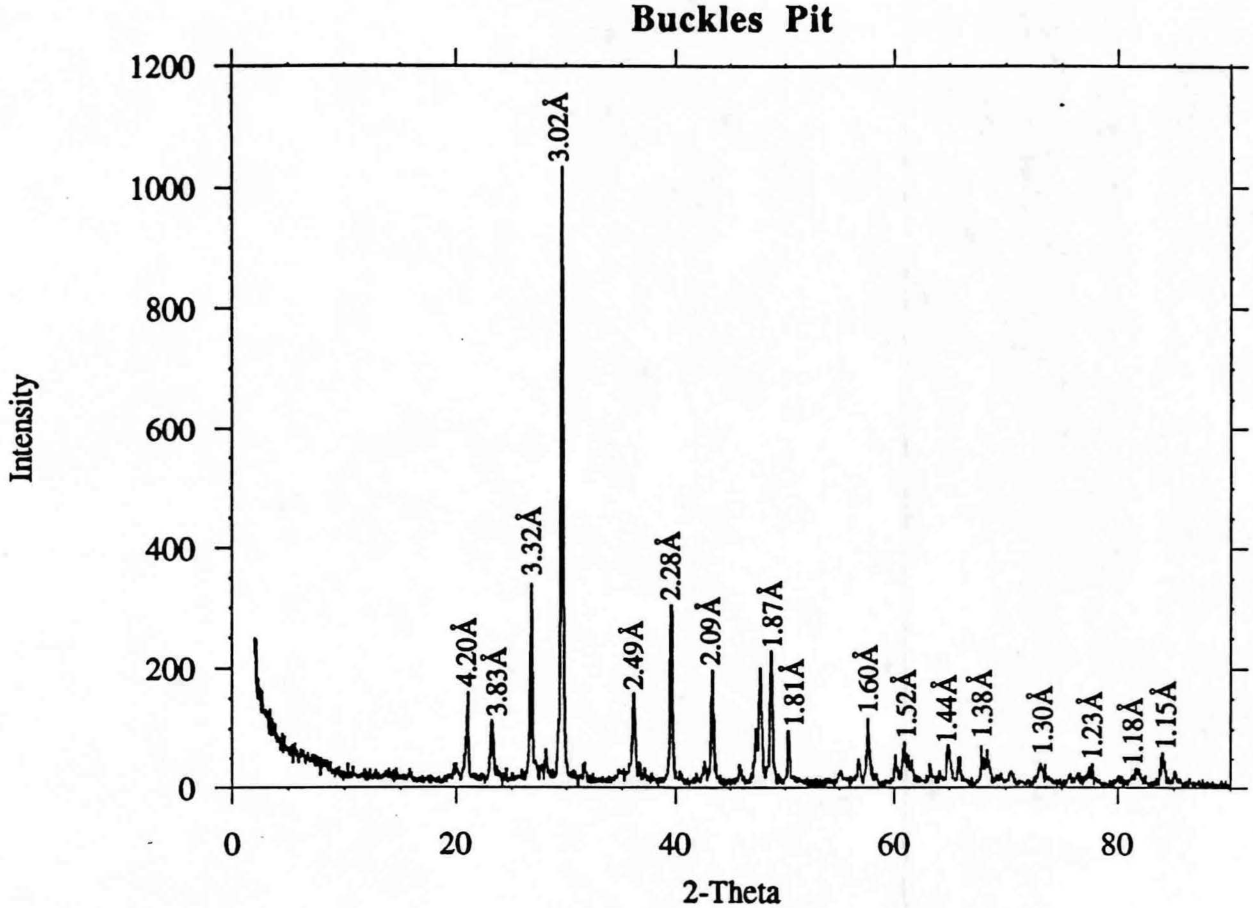
Percent Moisture vs. Time for Victoria Source, Yoakum District




Percent Moisture vs. Time for TTI Control Limestone
Percent Moisture (%)
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
20
40
60
80
100
120
140
160
180
Time (hrs.)

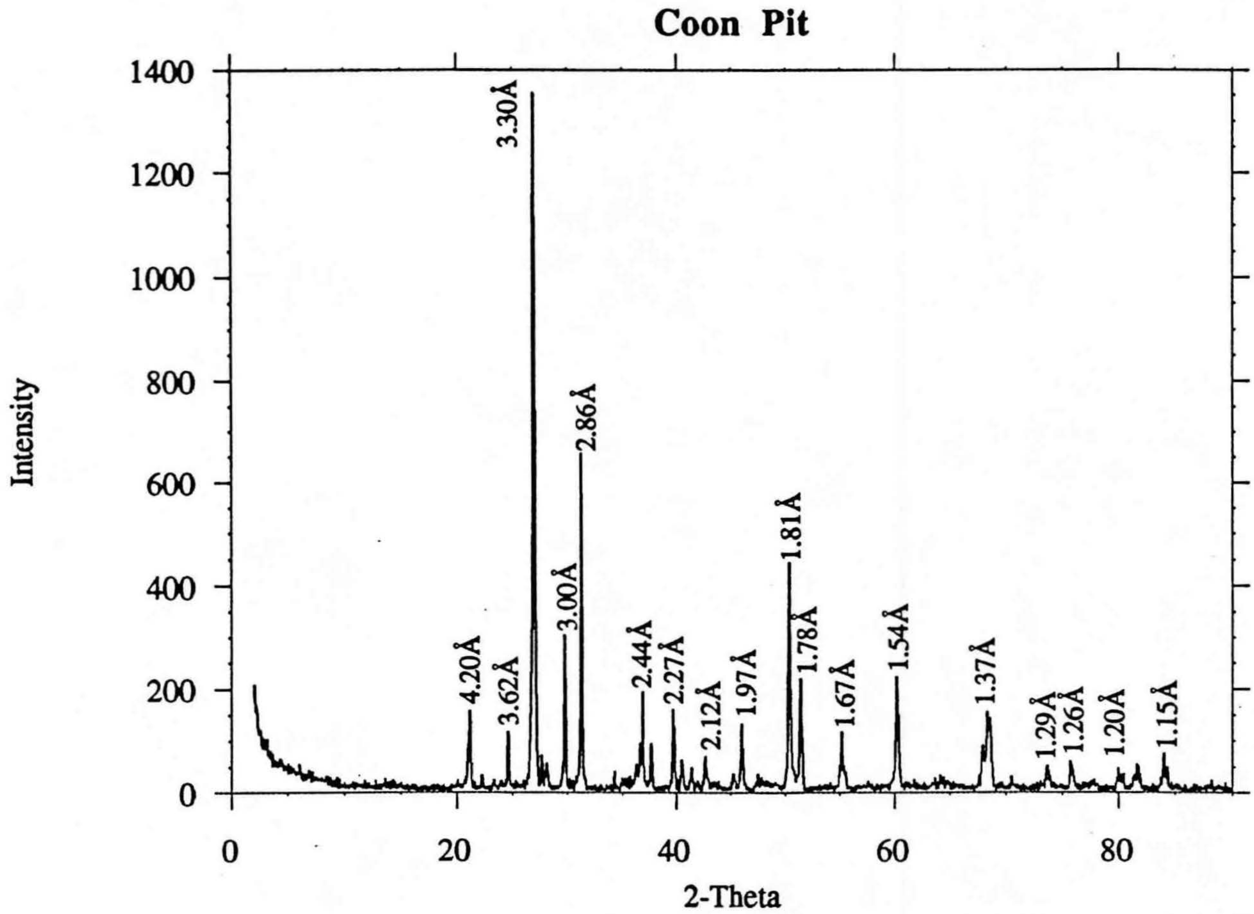
APPENDIX F:

RESULTS OF XRD ANALYSIS OF THE BULK SAMPLES (MINUS No. 40 SIEVE SIZE MATERIALS) FOR THE VARIOUS SOURCE MATERIALS EVALUATED IN THE PHASE 1 TESTING PROGRAM



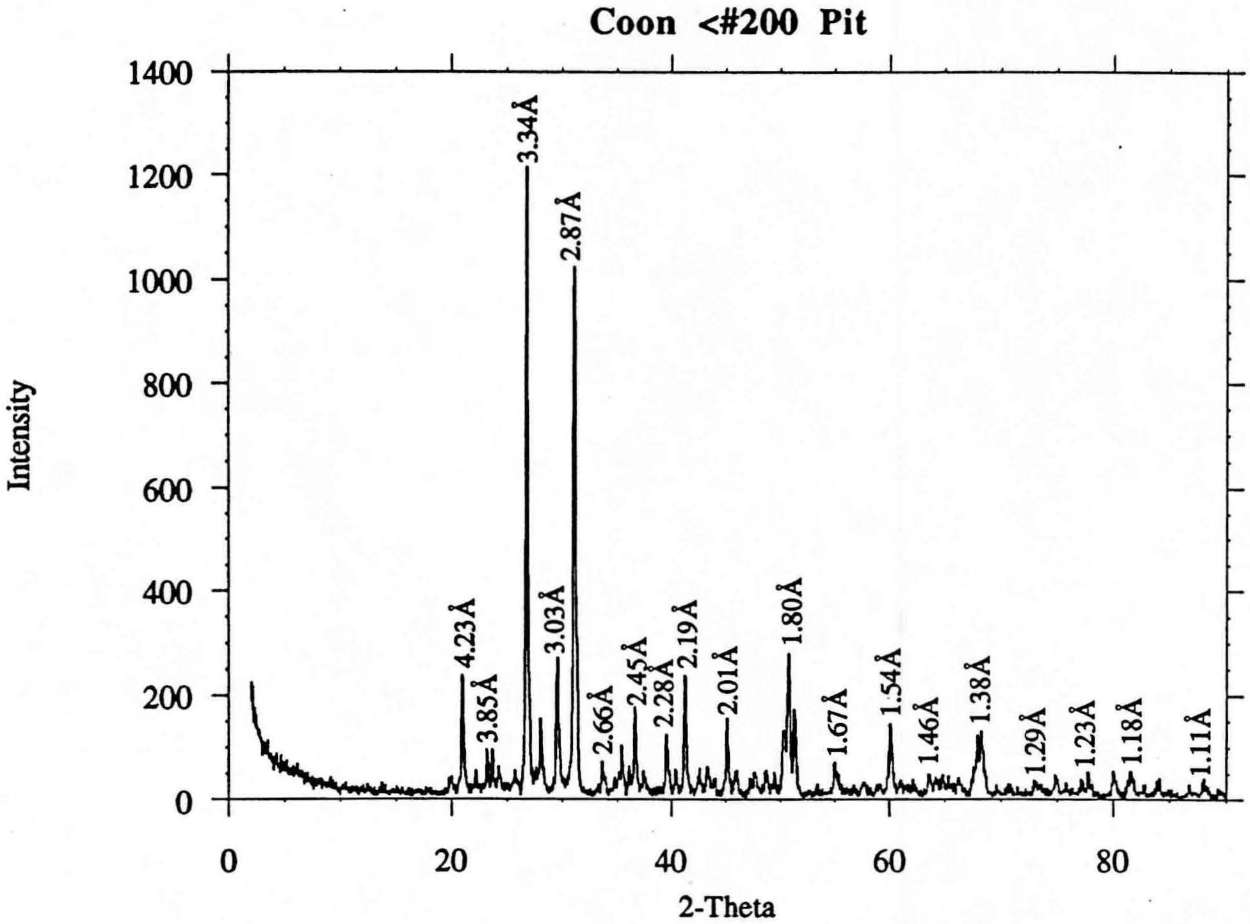
BUCKLES PIT

X-ray diffraction, utilizing random powder mounts, of the #200 sieve fraction of the Buckles sample reveals mineralogy dominated by low Mg calcite and minor quartz. Peaks diagnostic of calcite are located at 3.83, 3.02, 2.49, 2.28, 2.09, and 1.87 angstroms. Diagnostic peaks for quartz are located at 4.20, 3.32, 1.81, and 1.52 angstroms.



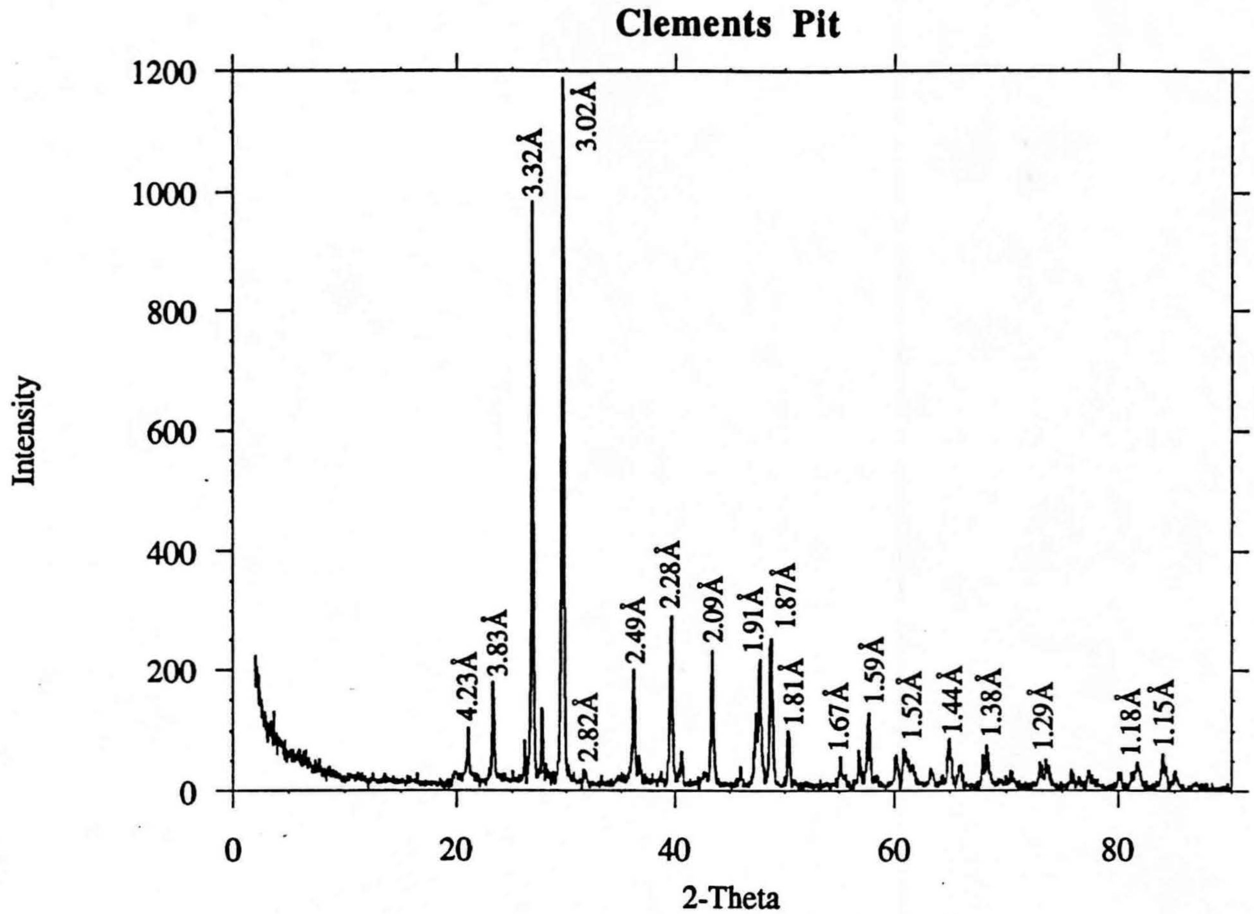
COON PIT

X-ray diffraction, utilizing random powder mounts, of the #200 sieve fraction of the Coon sample reveals mineralogy dominated by quartz, dolomite, and minor low Mg calcite. The peaks are shifted in this sample relative to the finer grained sample due to sample thickness. Peaks diagnostic of calcite are located at 3.00, and 2.27 angstroms. Quartz is also present in about the same proportion as the other Coon sample. Diagnostic peaks for quartz are located at 4.20, 3.30, 2.18, 1.81, 1.67, and 1.54 angstroms. Dominant dolomite peaks are 3.62, 2.86, and 1.78 angstroms.



COON PIT

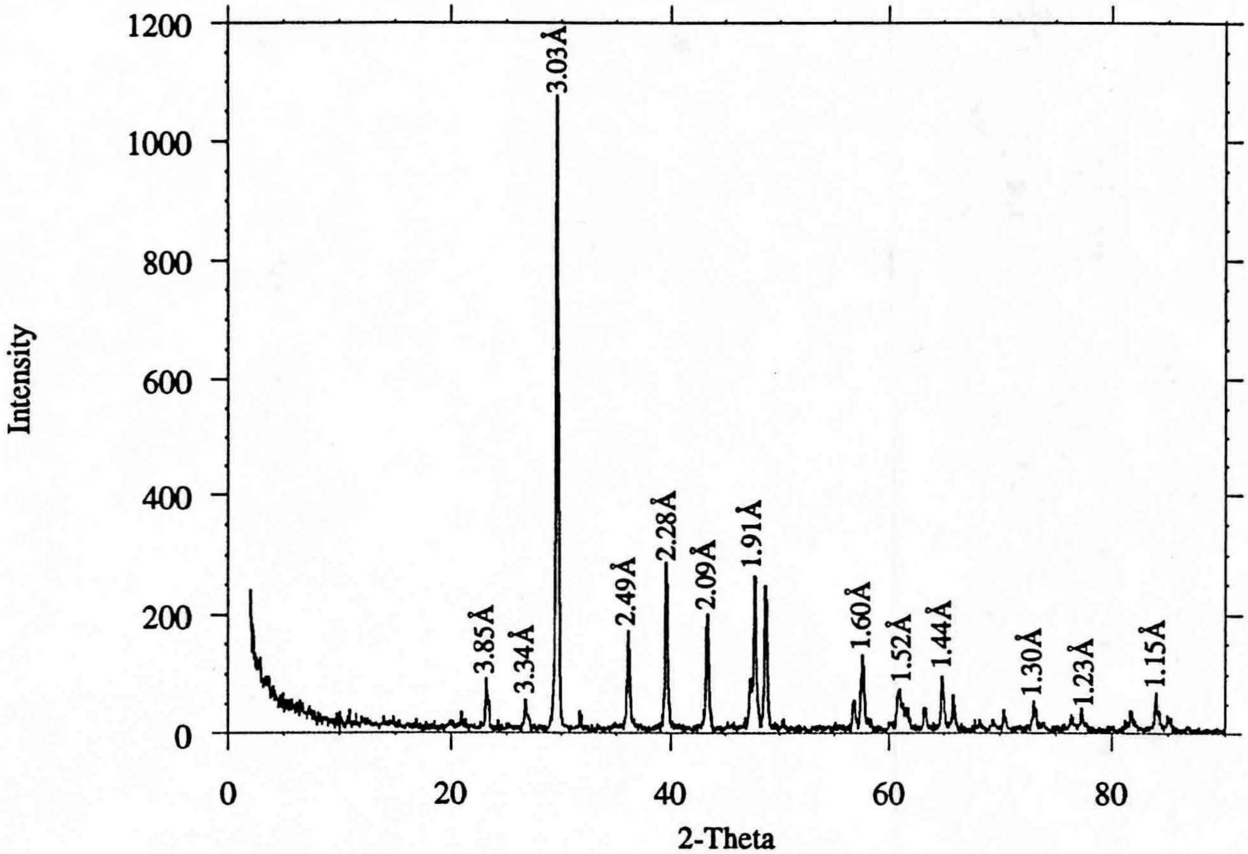
X-ray diffraction, utilizing random powder mounts, of the <#200 sieve fraction of the Coon sample reveals mineralogy dominated by quartz, dolomite, and minor low Mg calcite. Peaks diagnostic of quartz are located at 4.23, 3.34, 2.45, 2.28, and 1.67, angstroms. Dolomite is present as a major mineral phase as well. Peaks at 2.87, 2.66, 2.19, 2.01, 1.80, and 1.46 angstroms are diagnostic of dolomite. Low Mg calcite is also present as a minor constituent. Diagnostic peaks for calcite are located at 3.03, 2.28, and 1.46 angstroms.



CLEMENTS PIT

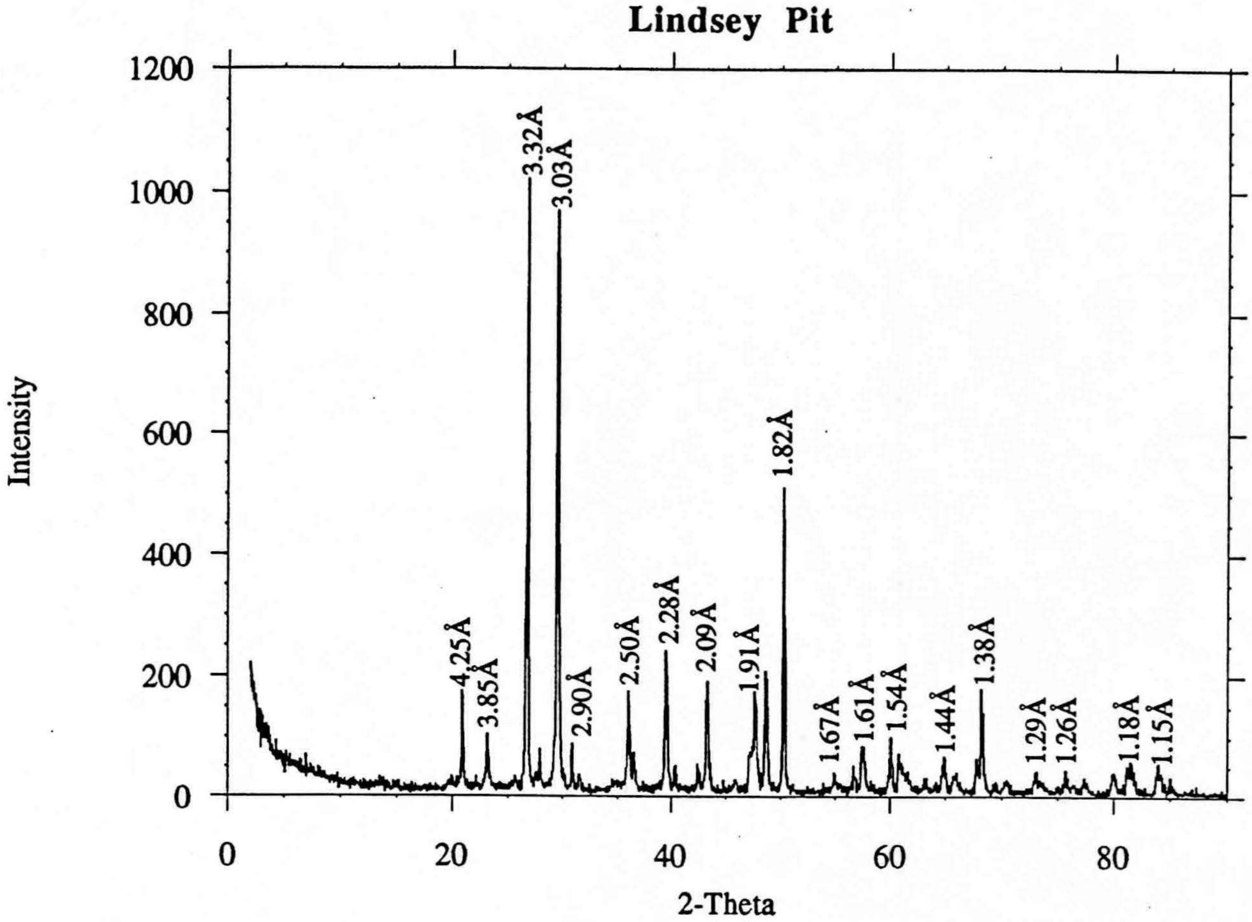
X-ray diffraction, utilizing random powder mounts, of the #200 sieve fraction of the Clements sample reveals mineralogy dominated by low Mg calcite and quartz. Peaks diagnostic of calcite are located at 3.83, 3.02, 2.49, 2.28, 2.09, and 1.87 angstroms. Diagnostic peaks for quartz are located at 4.23, 3.32, 1.81, and 1.52 angstroms.

Kemper Pit



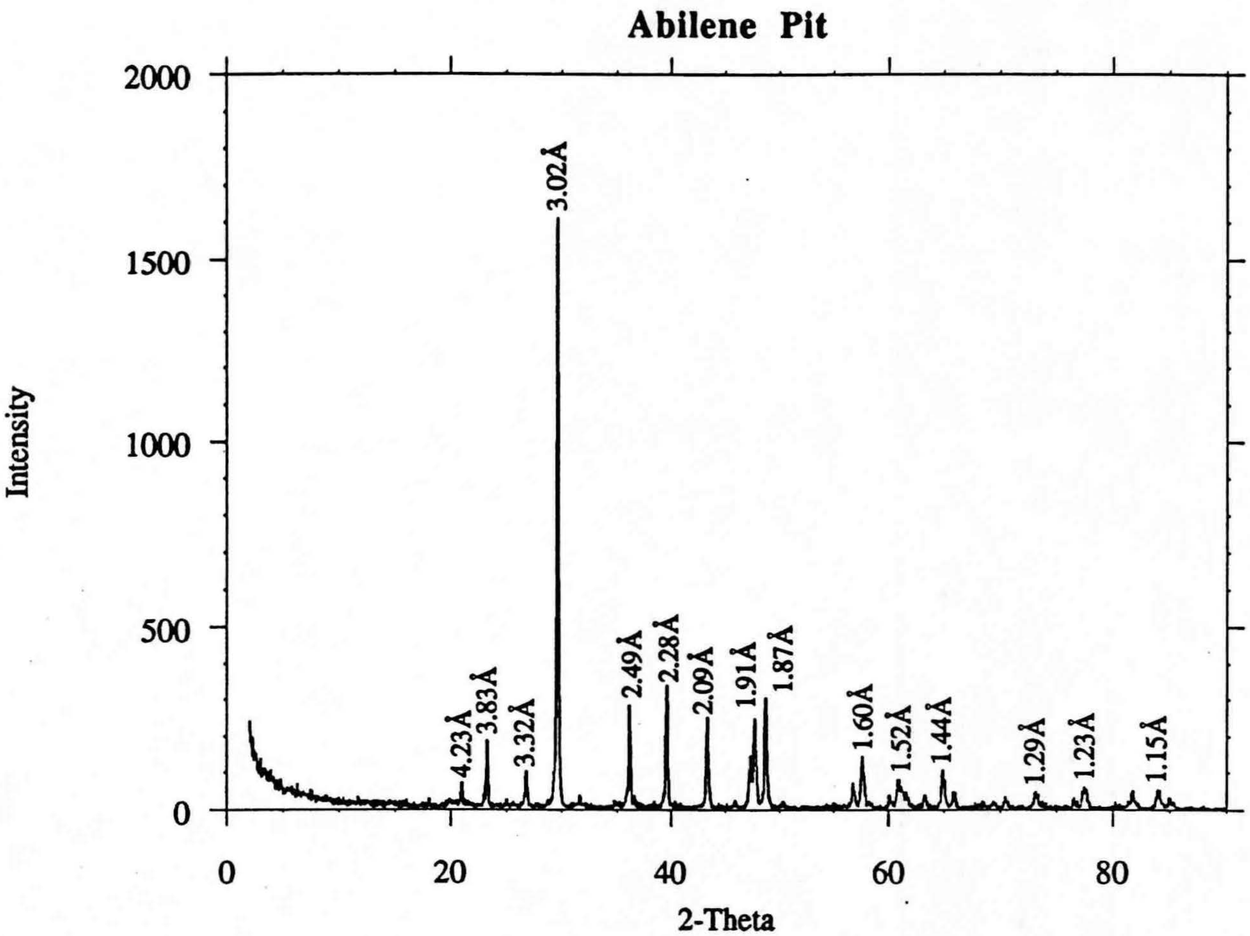
KEMPER PIT

X-ray diffraction, utilizing random powder mounts, of the #200 sieve fraction of the Kemper sample reveals mineralogy dominated by low Mg calcite. Peaks diagnostic of calcite are located at 3.85, 3.03, 2.28, 2.09, 1.91, and 1.87 angstroms. Quartz is also present as a minor constituent. Diagnostic peaks for quartz are located at 3.34, 2.28, and 1.60 angstroms.



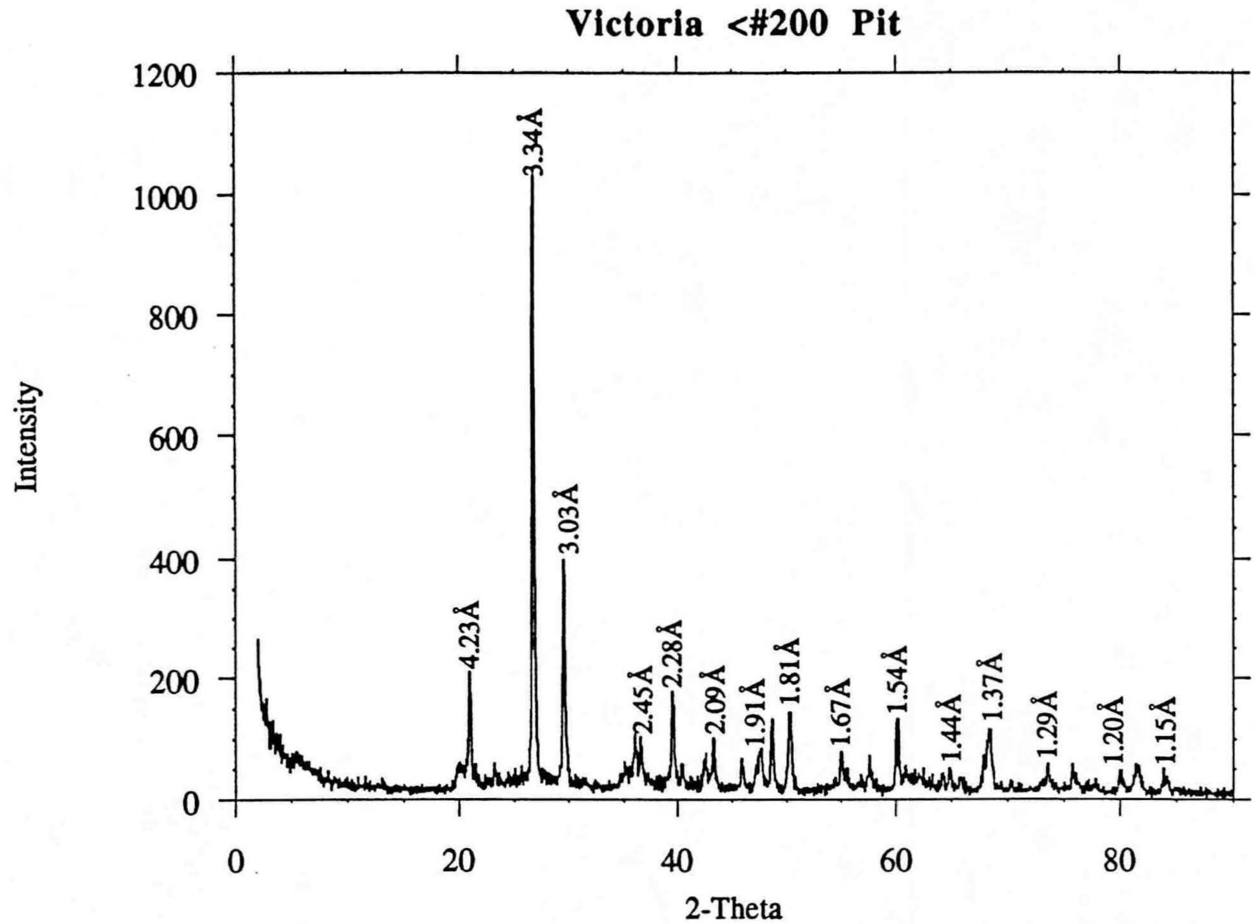
LINDSEY PIT

X-ray diffraction, utilizing random powder mounts, of the #200 sieve fraction of the Lindsey sample reveals mineralogy dominated by quartz and low Mg calcite with minor feldspars. Peaks diagnostic of quartz are located at 4.25, 3.32, 2.28, 1.82, 1.67, and 1.38 angstroms. Diagnostic peaks for calcite are located at 3.03, 2.28, 2.09, 1.91, and 1.44 angstroms. Feldspar peaks of 3.19, and 2.90 angstroms indicate the presence of small amounts of feldspar minerals.



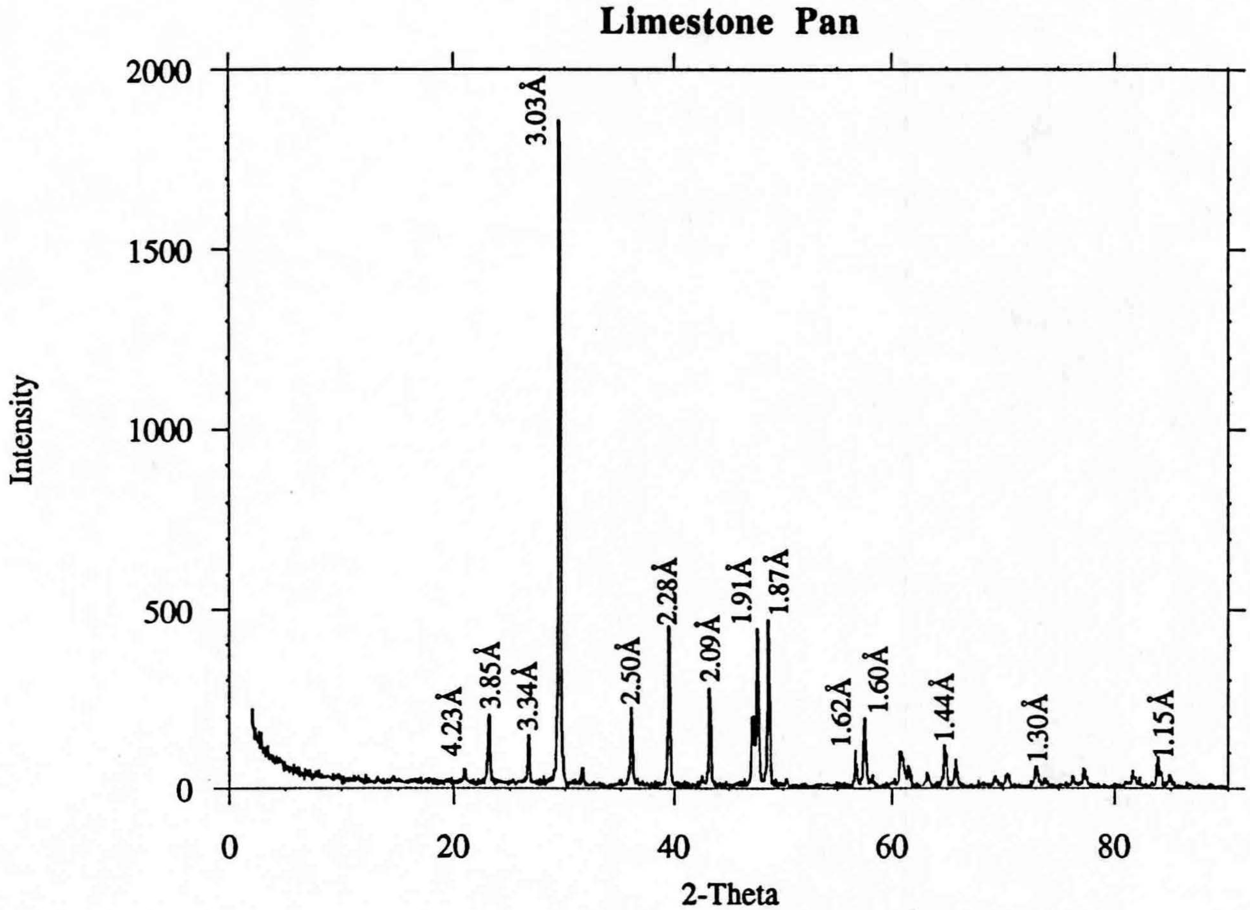
ABILENE PIT

X-ray diffraction, utilizing random powder mounts, of the #200 sieve fraction of the Abilene sample reveals mineralogy dominated by low Mg calcite with minor quartz. Peaks diagnostic of calcite are located at 3.83, 3.02, 2.49, 2.28, 2.09, and 1.87 angstroms. Diagnostic peaks for quartz are located at 4.23, 3.32, 1.81, and 1.54 angstroms.



VICTORIA PAN

X-ray diffraction, utilizing random powder mounts, of the <#200 sieve fraction of the Victoria sample reveals mineralogy dominated by quartz. Peaks diagnostic of quartz are located at 4.23, 3.34, 2.45, 2.28, 1.81, and 1.67, angstroms. Low Mg calcite is also present as a minor constituent. Diagnostic peaks for calcite are located at 3.03, 2.28, 2.09, 1.91, and 1.44 angstroms. Peaks in the region from 2 to 10° 2-Theta indicate the presence of clay minerals.



LIMESTONE PAN

X-ray diffraction, utilizing random powder mounts, of the <#200 sieve fraction of the Limestone sample reveals mineralogy dominated by low Mg calcite. Peaks diagnostic of calcite are located at 3.85, 3.03, 2.28, 2.09, 1.91, and 1.87 angstroms. Quartz is also present as a minor constituent. Diagnostic peaks for quartz are located at 4.23, 3.34, 2.28, and 1.60 angstroms.

F.11.

surface area

					L.S.					
Sample I.D.	**Run #**	**P (bars)**	**T (K)**	**A**	**A c**	**Vc N2**	**m (g)**	*S (m2/g)*	Mean	S.D.
Limestone Agg.	1	1.0088	295	855	1078	0.06	0.03498	3.6671		
10/15/96	1	1.0088	295	855	1072	0.06	0.03498	3.6877		
degas with 10% Nitrogen	1	1.0088	295	855	1068	0.06	0.03498	3.7015		
200°C for 15 hrs	2	1.0088	295	1081	1079	0.06	0.03498	4.6322		
Prepared by Pat Harris	2	1.0088	295	1081	1074	0.06	0.03498	4.6537		
	2	1.0088	295	1081	1062	0.06	0.03498	4.7063		
attn= 4	3	1.0088	295	850	1081	0.06	0.03498	3.6356		
	3	1.0088	295	850	1052	0.06	0.03498	3.7358		
	3	1.0088	295	850	1063	0.06	0.03498	3.6971		
									4.0130	0.53668057
					TUBBS					
Sample I.D.	**Run #**	**P (bars)**	**T (K)**	**A**	**A c**	**Vc N2**	**m (g)**	*S (m2/g)*	Mean	S.D.
Tubbs Agg.	1	1.0088	295	2345	2439	0.145	0.0595	6.3158		
10/15/96	1	1.0088	295	2345	2447	0.145	0.0595	6.2952		
degas with 10% Nitrogen	1	1.0088	295	2345	2447	0.145	0.0595	6.2952		
200°C for 15 hrs	2	1.0088	295	2419	2410	0.145	0.0595	6.5935		
Prepared by Pat Harris	2	1.0088	295	2419	2430	0.145	0.0595	6.5393		
	2	1.0088	295	2419	2452	0.145	0.0595	6.4806		
attn= 4	3	1.0088	295	2446	2490	0.145	0.0595	6.4529		
	3	1.0088	295	2446	2470	0.145	0.0595	6.5052		
	3	1.0088	295	2446	2430	0.145	0.0595	6.6123		
									6.4544	0.13417935