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Evaluation of the Dynamic Fracture Characteristics of Aggregate in PCC Pavements

Final Report

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16. Abstract

One area that has received limited investigation is the dynamic fracture of concrete and in particular coarse aggregate. Research has shown that many materials increase their strength and stiffness as the strain rate increases. This behavior is known as rate sensitivity and is generally defined as the ratio of the dynamic strength to static strength (D/S). This report investigated the rate sensitivity of aggregate, mortar, and concrete. In addition, the report investigated aggregate interlock since it can be viewed as a dynamic loading process. The results of the investigation indicated that aggregate, mortar and concrete are all rate sensitive. In addition to the D/S ratio, a strain rate parameter λ was defined as the difference between dynamic and static strength divided by the difference in the respective strain rates at failure. The research results indicate that the D/S ratio appears to be primarily a function of the material's microstructure while the λ appears to be a function of the microstructural grain-to-grain strength. The D/S ratio also provided an interesting differentiation between limestone and dolomites suggesting variations in their respective microstructures. The D/S ratio for concrete was strongly controlled by the microstructure of the mortar and that there was a significant difference between dry and moist concrete suggesting that the dynamic strength is affected by moisture in the air-void system. The strain rate parameter λ provided a relatively broad variation in aggregate types and in particular the carbonate aggregates. An aggregate interlock test system was constructed to investigate the effect of the coarse aggregate strength on the efficiency of concrete joint performance. The system was design and constructed using a closed loop servo-hydraulic control system. The response of the testing system to aggregate interlock was found to be a function of the stiffness of the joint interface and the coarse aggregate strength. Although hydraulic and control problems limited the number of successful tests conducted, the results indicate that at close joint spacing aggregate interlock was very efficient regardless of coarse aggregate type. However, at larger joint spacing the strength of the coarse aggregate became important for joint efficiency.

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Executive Summary

Traditionally, strength is one of the primary performance criteria used to assess the quality of concrete used in pavements. For most pavements, concrete strength is determined by the uniaxial compressive strength test and in some cases the indirect tensile strength test. However, a factor in determining concrete's strength is the loading rate at which the concrete is tested. ASTM testing standards, therefore, narrowly limit the loading rate that can be applied to a concrete specimen in determining its strength. In general, the allowable loading rate is relatively slow and is considered to be quasi-static (static). For many materials, though, as the loading rate increases the strength and stiffness of the material also increases. Materials that exhibit this behavior are considered to be rate sensitive. To determine a material's rate sensitivity the material is dynamically loaded to failure at very high strain rates on the order of 100 strain (in/in)/second and then compared to its quasi-static failure strength, which is tested at approximately 10^{-7} strain/second. Research on the dynamic strength of concrete has shown that it is also rate sensitive. The main explanation for concrete's rate sensitivity is that at quasi-static loading rates concrete's larger defects such as shrinkage cracks tend to dominate the fracture process. At dynamic strain rates the fracture strength is associated with microstructural inhomogenieties such as pores, micro-cracks, and impurities that exists along grain boundaries as opposed to the larger macrostructural defects. Although the strength of the grain boundaries are involved in the overall fracture process of brittle materials, the resistance to crack growth from microstructural inhomogenieties varies with strain rate. Therefore, during quasi-static loading the macrostructural defects dominate the fracture strength while at dynamic loading rates the microstructural grain boundary defects dominate the fracture strength. This is mainly due to the dynamic loading pulse moving through the material at a rate in which the larger defects do not have time to respond, thereby allowing the microstructural strength of the grain boundaries to control the material's strength and stiffness. Rate sensitivity is generally defined as the ratio of a material's dynamic strength to its static strength (D/S) and is sometimes referred to as the dynamic increase factor. An additional rate sensitivity measure that was developed in this research is the strain rate parameter λ, which is defined as the difference in the dynamic and static strength divided by the difference in the respective strain rates at failure. While the dynamic strength of concrete is important for design considerations involving dynamic loading,

it also provides a unique way of better characterizing concrete, which is a very complex material involving a porous solid structure and with, in many cases, fluid inclusions. It is able to characterize a material's fracture strength as a function of the grain boundary strength, which is unique to each material type.

The primary objective of this research was to investigate the dynamic fracture characteristics of coarse aggregate, mortar, and Portland Cement Concrete (PCC) and how the strength of the coarse aggregate affects overall PCC strength. Additionally, the relationship between coarse aggregate strength and the efficiency of aggregate interlock in concrete joint performance at various joint or crack widths was also investigated. The final objective was to assess if the dynamic properties of aggregate can be used as a classification system to better predict concrete pavements performance. In this research the following four coarse aggregate types were investigated: igneous, carbonate, blast furnace slag and natural gravel.

All of the coarse aggregates investigated in this research were found to be rate sensitive with an increase in the dynamic to static strength ratio of 1.9 to 2.7. The D/S ratio was found to closely correspond to the aggregate's crystalline structure. The igneous and blast furnace slag aggregates had similar D/S ratios in both dry and moist states. The carbonates, however, had a noticeable difference in D/S ratio - but were unaffected by moisture content. The difference in the D/S ratio between limestone and dolomite was believed to be due to the differences in geologic formation of the two carbonates. It is generally understood that limestone develops as a primary precipitate while dolomite forms as a secondary replacement product of limestone. This occurs when migrating ground waters moving through limestone replace calcium with magnesium in the limestone, thus forming dolomite. It is hypothesized that the replacement process disrupts the limestone's crystalline microstructure producing larger grain sizes but developing weaker grain boundaries, thus decreasing the dynamic strength of dolomite from that of limestone. However, it is also possible that the replacement process also heals the larger microstructural defects in the dolomite, thus improving its static strength. In fact, the strength results show that dolomite has a higher static strength than the limestone - but the opposite occurs in dynamic strength with limestone having a significantly higher dynamic strength than dolomite. The results of the aggregate's strain rate parameter λ revealed a very wide range of values from a low of 1.2 for blast furnace slag to a high of 31.30 for the igneous aggregates. The carbonates ranged from a low of 4.52 to a high of 25.52. The average λ for dolomite was 8.6

while the average for limestone was 16.4. Inspection of the microstructure of the carbonates indicated that for limestone the dynamic strength increased with decreasing grain size and for the dolomites the dynamic strength increased with increasing grain size.

Mortar blocks were cast, cored and tested over an eighteen-week period (once a week) to investigate the mortar's rate sensitivity. The mortar was found to be rate sensitive with the D/S ratio ranging from 1.5 to 3.0. However, the mortar strength and D/S ratio were also found to vary over the testing period with two maximums strength peaks and corresponding minimums occurring. Interestingly, the first maximum strength peak occurred during the fourth week or around the 28-day strength.

PCC specimens were prepared with the coarse aggregate the only variable in the mix. In addition to fresh PCC (30-day), existing (aged) PCC specimens were also tested. The static and uniaxial compression testing of the fresh PCC indicated that all PCC mixes had adequate strength and that there was no statistical correlation between coarse aggregate strength and PCC strength at either static or dynamic strain rates. The testing confirmed the generally held belief that the mortar controls the primary strength of PCC while the shape and surface texture appears to be a secondary control. However, the D/S ratio and λ values for the PCC showed a significant difference between dry and moist (30-day cure) conditions with the moist values higher than the dry values. Clearly, moisture in the air-void system affects the dynamic response of the PCC. Interestingly, both the dry D/S ratio and λ results were surprisingly consistent for all PCC tested. However, for the moist D/S ratio and λ results there was one PCC mix that was noticeably lower then the rest. It was believed that this mix had been improperly made. While the slump and air content values were within established limits for this PCC, it was the dynamic response that clearly indicated that the mix had a problem. It is speculated that the microstructure of the concrete's mortar and its corresponding air-void system responded differently to the dynamic loading resulting in a lower D/S ratio and λ values than the other concrete mixes tested.

The λ results of the aggregates were also compared to the LA abrasion and freeze-thaw durability index values. There was a general linear correlation of λ with LA abrasion, although the correlation was not strong, and the carbonates appear to be somewhat counter to the correlation. However, there appeared to be a relatively strong correlation with λ and the freeze-thaw durability index for the carbonates when separating the carbonate aggregates into high and low groups based on their strain rate parameters λ.

An aggregate interlock system was designed, constructed and tested to determine how coarse aggregate type affects the performance of concrete joints and to determine if the dynamic strength of the coarse aggregate is a factor in the joint performance since the loading of a joint can be viewed as dynamic event. The system tested nine by nine by 18-inch PCC blocks, which were fractured using a pure tension fracture device. The PCC blocks were fractured after 18-hours of curing and then additionally cured for a minimum of two months prior to testing. The initial aggregate interlock tests used a compressive load without a corresponding uplift load. While the tests were successfully completed it was realized that the loading did not replicate realistic field loading conditions. Consequently, a compressive load followed by a tension uplift load was applied to the PCC specimens to better simulate joint aggregate interlock. However, the frequency of the loading and difficultly in the hydraulic closed loop control system caused significant problems in the testing. In addition, it was realized after the testing was completed that a significant factor in the response of the hydraulic testing system was the stiffness of the concrete joint interface itself. Although these problems limited the results of the interlock research, it was found that at larger joint widths, the interface stiffness was clearly a function of the coarse aggregate strength and stiffness.

For the closed-loop testing system to function properly the digital control system's auto feedback system, termed the PID settings, had to be adjusted for the PCC joint interface stiffness. That is, at least one PID setting would be needed for very stiff PCC joint interfaces and a significantly different PID setting for relatively limited PCC joint interface stiffness. However, to maintain consistency between tests, a single PID setting was used in this research assuming that the difference between PCC joints would not be that large and that a single PID setting would be sufficient. Consequently, the PID setting selected was based on the least stiff PCC joint interface, which was PCC made with blast furnace slag as the coarse aggregate. While the blast furnace slag PCC joint interlock tests ran well, the higher stiffness PCC interface joint tests did not. For the higher stiffness PCC interface joint tests it was very difficult to maintain the stability of the hydraulic system when testing. In fact, a number of test blocks became unstable during testing resulting in a premature failure of the aggregate interlock. The higher stiffness joints consisted of the higher strength coarse aggregates. Therefore, due to system control and hydraulic problems only a limited number of tests were successfully completed. The results, while not conclusive, indicate that aggregate interlock at a 0.028-inch joint (crack) width was very efficient regardless of coarse aggregate type. However, at larger crack widths (0.035-inch and greater) the efficiency of the aggregate interlock becomes a function of the coarse aggregate strength and stiffness. These results were consistent with a recently completed aggregate interlock study at the University of Illinois where three coarse aggregate types (basalt, natural gravel, and limestone) were tested. The University of Illinois test results indicate that at larger crack widths the basalt coarse aggregate was the most efficient in aggregate interlock followed by the natural gravel while the limestone coarse aggregate had relatively poor aggregate interlock. The LA abrasion test was used to differentiate the coarse aggregate types in regards to an existing performance criterion.

The development of an aggregate classification system was not accomplished in this research. However, the results of this research strongly suggest that the dynamic strength of aggregate, mortar, and concrete as measured by the D/S and λ parameters, indicate unique material properties that can be used to better understand their performance and subsequently used as a classification system for aggregate and concrete. Additional research is highly recommended to continue the research into the dynamic properties of these materials and their relationship to field performance.

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SECTION 1

Introduction and Background

1.1 Introduction

There are significant costs associated with premature distress to PCC pavements, including increased maintenance and repair costs, increased vehicle wear and tear, and the loss of use during maintenance and replacement. While many factors affect the performance of PCC pavements, such as the design and construction procedures used, one of the most important factors is the material quality of the PCC pavement itself. Consequently, the economic viability of PCC pavements depends on selecting quality materials that will consistently produce durable and long-lasting PCC. However, a major challenge in selecting quality materials is having available laboratory tests that successfully predict field performance.

In general, strength has been a key criteria used in determining the suitability of PCC for pavement structures. The strength of PCC in turn has been primarily considered a function of the water-to-cement ratio of the paste. While the quality of water, cement and to a lesser degree the fine aggregate can be relatively well controlled through selection and processing, the remaining component of PCC coarse aggregate has generally not been as well controlled. Consequently, the coarse aggregate, which is considered a filler material, is selected from a wide variety of materials, ranging from naturally occurring earth materials to industrially derived materials. Since coarse aggregate is considered a filler material, less emphasis has been placed on developing quality tests for coarse aggregate. The main reason for this lack of emphasis is that when fracture occurs in PCC it has been assumed that the primary fracture process is for cracks to propagate around the coarse aggregate and through the cement matrix. Consequently, the strength of PCC is largely controlled by the strength of the cement matrix. For high-strength PCC, however, the fracture process is for cracks to go through both the coarse aggregate and the cement matrix since the strength of the matrix may be as high as the coarse aggregate strength. Accordingly, for high strength concrete the strength of the aggregate does plays an important role in the overall strength of the PCC. However, strength testing of the coarse aggregate is generally not conducted for either normal strength PCC or high strength PCC. Currently, the most common tests conducted on coarse aggregate that are related to strength are the LA Abrasion test; freeze-thaw testing and aggregate wear index (AWI) testing. Although the LA abrasion test and to some degree the AWI are related to the aggregate strength, it is at best only an indirect measure of strength.

In addition to PCC strength a unique feature of PCC pavements is that it also experiences continuous dynamic vehicle loading. The majority of research on the dynamic loading of PCC has been is in the area of earthquake engineering and in military applications of the effects of explosives on structures while only limited research has been conducted on the dynamic characteristics of PCC used in pavements. It is known, however, that PCC is a rate dependent material. That is, the strength and stiffness of PCC are a function of the rate of loading and in general, the faster the rate of loading, the higher the PCC strength and stiffness. It is also known that some materials do not exhibit rate dependency. The practical issues of rate sensitivity will be further addressed later in the report.

An additional area of interest is the response of cracks in PCC to dynamic loading. Since cracks are an integral part of PCC structures, their ability to transmit shear and compressive loading becomes important and must be considered in analyzing and designing the structures for dynamic loading. This is especially important in earthquake engineering where research has been conducted to model and predict the response of cracks to dynamic loading. In general, this research should be directly related to the performance of pavement cracks and particularly construction and expansion joints in PCC pavements. However, a significant difference between earthquake engineering and the loading of PCC pavements is in the width of the crack itself. In earthquake engineering the cracks are generally tightly closed or are held closely together through reinforcement. In PCC pavements, however, the joint widths are significantly wider with the joint opening being a function of temperature as well as to long-term loading. As cracks and precut joint form in PCC, the ability of the crack to efficiently transmit shear loading is a function of the crack width with small openings having good load transfer capability and larger openings with poorer load carrying ability. At larger crack widths it is generally believed that the coarse aggregate plays an important role in transmitting loads across the crack. When the load transfer across a crack is maintained by the coarse aggregate the load transfer mechanism is referred to as *aggregate interlock.*

In considering the dynamic behavior of cracks, an additional consideration is the magnitude and rate at which strains develop and diminish at the points of contact between the surfaces of the crack during loading. When the surfaces of the crack are relatively smooth and close together there is significant surface area contact and thus for a given shear load a minimum strain on the contact points, with the points of contact being between the coarse aggregate, cement matrix or a combination of the two materials. However, as the crack width increases the contact surface area decreases and for the same shear load the strain levels will significantly increase. It can be further assumed that as the strain levels increase with larger crack openings that coarse aggregate and cement matrix crushing will occur resulting in load transfer to other contact points at the joint interface. This effect, the transfer of shear load at the contact points between the surfaces, will ultimately result abrasion and breakage that leads to an increase in displacement or faulting of the PCC pavement. Consequently, the strength of the coarse aggregate may also play an important role in maintaining the performance of a joint - especially for PCC pavements that experience large crack width openings.

1.2 Research Program

This research presented in this report was initiated in November of 1997 under the title *Evaluation of the Dynamic Fracture of Aggregates in PCC Pavements* and completed in December of 2001. The primary focus of the research was three-fold. The first focus area was to investigate the static and dynamic strength of the materials used in PCC materials focusing primarily on the coarse aggregate. The second focus area was to develop an aggregate interlock test system and to conduct aggregate interlock tests on PCC made with various types of coarse aggregate. The third focus area was to assess whether the strength of the coarse aggregate on PCC strength and aggregate interlock and if so whether strength testing of aggregate can be used as an index test for the performance of PCC pavements.

Based on these focus areas the following four objectives were developed for this research:

1) The first objective was to test the static and dynamic strength of the coarse aggregate, cement matrix, and PCC. The difference in the static and dynamic

strength in turn determines the rate sensitivity of the PCC materials. The rate sensitivity as well as the static and dynamic strength of the coarse aggregate was then compared to the 28-day uniaxial compressive strength and the indirect tensile strength of the PCC and the aggregate interlock tests.

2) The second objective was to develop an aggregate interlock testing system to investigate the performance of the PCC made with different types of coarse aggregate. A key part of this objective was to simulate (as close as possible) true aggregate interlock, but to also develop a system that would be efficient to use with a minimum amount of specimen set up time.

3) The third objective was to conduct aggregate interlock tests investigating different coarse aggregate types.

4) The fourth objective was to determine if the dynamic properties of coarse aggregate could be used as a selection criterion for coarse aggregate.

The remainder of this chapter provides background information used in this investigation as well as the format of the report.

1.3 Research Program Background

1.3.1 Aggregate Tests for PCC

There has been a significant amount of research conducted on PCC. However, due to the large variation in material properties used in PCC and lack of follow up studies there has not been good correlation of laboratory tests with field performance. This is particular true for aggregates. An evaluation of aggregate tests related to the performance of PCC was recently conducted by the National Cooperative Highway Research Program (NCHRP) and reported by Meininger (1998). While the report provided a comprehensive study of aggregate tests used for PCC, it did not provide an evaluation on how these tests are related to field performance. The report, however, provides the following recommendation: " A laboratory study is proposed to evaluate selected aggregates tests and to verify the relationship between the selected aggregate test and concrete performance. Then, the next step is to validate the set of tests and performance relationships with laboratory concrete research and PCC pavements performance in the field." Meininger cites a significant need to validate either current aggregate tests or to develop new tests to better predict PCC field performance since it is unclear how existing laboratory testing of aggregate relate to field performance.

The Meininger report also cited carbonate aggregate for special consideration. According to Meininger, "a better understanding of carbonate aggregates for use in concrete is needed." He states a number of questions such as why do many weaker, absorptive carbonates perform better than expected both in strength, while some do not. Or why do some carbonates with high absorption values have good durability, while some do not. Currently, there are no tests available that provide an indication as to how carbonates will behave. This is an important question for Michigan, since a significant amount of carbonate aggregates are used in the state.

According to Meininger (1998) the following four properties are currently investigated to evaluate aggregates used in PCC: physical, mechanical, petrographic, and chemical. Below are the specific tests used to investigate each of the four properties. Unfortunately, to date there is only limited information on how these specific tests are related to PCC field performance.

Physical Properties

Gradation and Minus #200 Determination
Characteristics of Minus No. 200 Material
Fine Particle Shape and Angularity
Coarse Aggregate Particle Shape and Angularity
Absorption, Porosity, and Specific Gravity
Pore Properties: Pore Size Distribution, Surface Area, Pore Volume
Coarse Aggregate Durability and Soundness
Thermal Coefficient of Aggregate
Coarse Aggregate Drying Shrinkage

Mechanical Properties

Fine Aggregate Breakdown and Slaking in Wet Attrition
Coarse Aggregate Breakdown and Slaking in Wet Attrition
Coarse Aggregate Breakdown in Dry Impact, Attrition, and Abrasion
Coarse Aggregate-Mortar Bond
Stress-Strain Response of CA, Modulus of Elasticity, Creep, and Impact
Strength of Coarse Aggregate

Petrographic/Chemical Properties of Aggregate

Mineral Structure of Coarse Aggregate and Fine Aggregate
Elements Present in Aggregate
Compounds Present in Aggregate
Fine Aggregate Mica Content
Organic Impurities
Chlorides in Aggregate
Reactivity with Alkalis in Concrete

1.3.2 Characteristic of Dynamic Problems

In general, dynamic problems are characterized when forces are applied to an object in a very short period of time resulting in very high strain per unit of time. Although the loading on PCC can be described as dynamic, strength testing of the PCC is conducted using static tests such as the uniaxial compression test, which tests the material at slow loading rates. In fact, the ASTM standard for uniaxial compression testing recognizes the problem of varying loading rates for concrete and requires that the loading rate be conducted within a narrow quasi-static range. A distinguishing feature in many dynamics problems, such as in wave propagation, is the relatively small strain levels considered. For example, dynamic strain level as low as 10^{-6} (e.g., in/in) would be disregarded in conventional material testing but when the strain level is dynamic the inertial forces that are generated must be considered. According to Ishihara (1996) inertia forces play an increasing significant role when the deformation occurs over a short period of time. Further, when considering sinusoidal loading, the inertia force increases in proportion to the square of the frequency at which soils deform. Consequently, even at very low levels of strain, the inertia forces become significantly larger with increasing speed of loading.

The rates of loading for various engineering applications are shown in Figure 1.1. The data presented in Figure 1.1 separate the time of loading into dynamic and static problems as well as by shock, wave, vibration, and fatigue. It can be seen that traffic loading is considered a dynamic problem as well as a wave and fatigue problem. However, this is considering only the loading rate from wheel loads traveling over smooth pavement. When wheel loads encounter bumps or cross an uneven joint the loading on the pavement can better be described as a shock loading. This is especially true for a concrete joint where the nature of the rough interface can result in high levels of localized strain as well as transfer of loading from one interface contact to another during dynamic loading.

In reviewing test results from dynamic testing it is common to use the units of strain per unit time as opposed to load per unit time. The primary reason for this is that loading does not provide a measure of material deformation, which is important in characterizing material behavior. Since strain is defined as deformation over total length, both of which are in units of length, strain is a dimensionless quantity, e.g., in/in. Consequently, the units used to report dynamic test results are commonly given as strain/second, e.g., 10^{-5}/sec and referred to as the strain rate. For example, a strain rate of 10^{-3}/sec would be a dynamic load that caused an average 10^{-3} strain per second in the material. As mentioned previously the uniaxial testing of concrete cylinders is generally a quasi-static test. Knowing the loading rate of a test the strain rate of the test can be determined. For example, assume that the minimum loading rate of a conventional ASTM uniaxial compressive test is 34,000 lbs/min, while the time to failure for a 12-inch long by six-inch diameter specimen is 5.3 minutes and that the failure load is 180,000 lbs. Further, assuming that the total vertical deformation of specimen prior to failure was 0.0315 inches (0.8 mm), the total strain at failure was 0.0315 in/12 in or 2.6^{-3} in/in. The average strain rate of this test is 2.6^{-3}/5.3 min or 8.8^{-6}/sec, which is a very slow strain rate and considered a quasi-static test. All of the quasi-static tests conducted in this research were conducted at strain rates of 10^{-5}/sec or slower, while the dynamic tests were conducted at strain rates of 10^{2}/sec or higher.

The main feature of dynamic testing, as discussed above, is that the load is applied over an extremely short period of time. In general, most dynamic testing of materials use loading duration of 100 to 400 microseconds, which is sufficient to fail most material. Failure loads (at a given strain rate) from the dynamic testing are then compared to the static failure strength to determine whether the material is rate sensitive. For many materials higher strain rates result in higher strength and stiffness. Although the fracture process is very complicated, a practical aspect of dynamic testing is that due to the velocity at which the loading is applied the larger material defects, which typically control failure (and subsequently compressive strength), do not have time to fail. Thus, more of the material experiences the loading, resulting (for rate sensitive materials) in higher strength and stiffness since more material can distribute the loading.

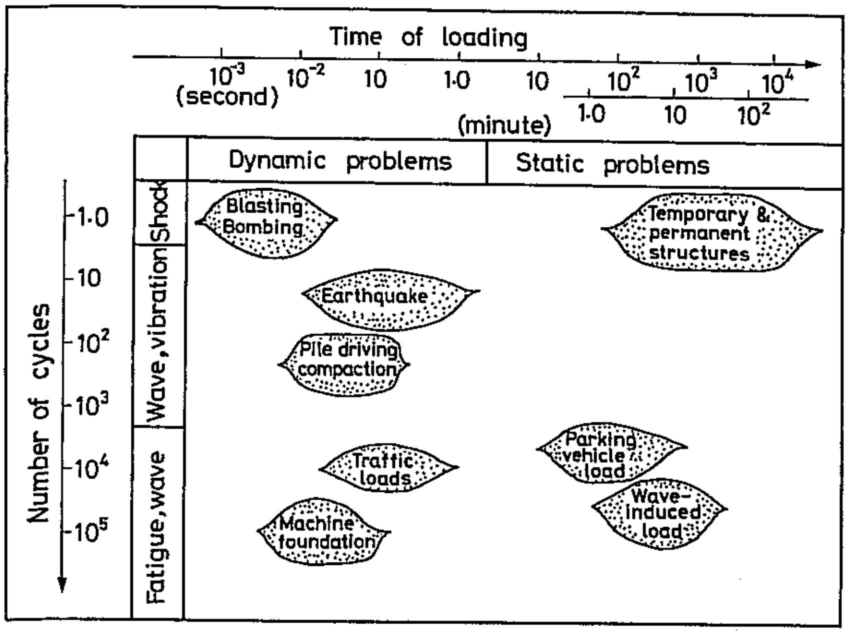

Figure 1.1 Classification of dynamic problems (Ishihara, 1996).

1.4 Report Format

The main objectives of this research were to investigate the dynamic strength of the coarse aggregate, mortar, and PCC and its relationship to aggregate interlock load transfer in PCC joint/crack performance. The research described in this research report is divided into seven sections. Section One provides the background and introduction into the research, while Section Two provides a summary of the research including the conclusions and recommendations of the research. A description of the aggregate types used in the research, which include the geologic properties of the aggregates, is provided in Section Three. Due to the importance of concrete mixing, Section Four details the concrete mixing procedures used in the research as well as the initial unconfined compression and indirect tension results for concrete with different aggregate types. Following this section, Section Five provides the results and discussion of the dynamic and quasi-static testing of the aggregate, cement matrix, and PCC. Sections Six and Seven details the aggregate interlock research with Section Six providing the details of the development, construction and initial testing of the aggregate interlock test device. The final section in the report, Section Seven, which details the test results and discussion of the aggregate interlock research. Section's four, six, and seven are based upon a graduate master thesis's and report work and follow the general outline of a master's thesis. However, each section is formatted into chapters so the number system adopted will be to start each chapter in a section as chapter one, proceeding to two and so on. References are provided at the end of each section.

1.5 References

Ishihara, K., *Soil Behavior in Earthquake Geotechniques*, Oxford Science Publications, Oxford, England, 1996, p.350.

Meininger, R.C., "Aggregate Tests Related to Performance of Portland Cement Concrete Pavement," National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Report 4-20A, 1998, p.84.

SECTION 2

Major Conclusions and Recommendations

1 Major Conclusions and Observations

The research conducted in this report was initiated in November 1997 under the project title *Evaluation of the Dynamic Fracture of Aggregates in PCC Pavements* and completed in December 2001. The research project, which investigated the dynamic strength of Portland Cement Concrete (PCC) and its components as well as aggregate interlock, had four overall objectives. The first objective was to determine the static and dynamic strength of coarse aggregate, mortar and concrete. The static and dynamic strengths in turn provided a measure of the rate sensitivity of these materials. The rate sensitivity was then investigated to determine its relationship with existing performance tests for concrete and to assess whether it can be used for a possible classification system. Since concrete joint behavior is a significant dynamic loading event, the dynamic performance of the coarse aggregate was also investigated in regards to aggregate interlock, which is an important load transfer mechanism in jointed concrete pavement. The second objective, therefore, was to develop an aggregate interlock testing system that simulated, as closely as possible, actual field loading conditions. After completing this objective the third objective was to conduct aggregate interlock tests of PCC with different coarse aggregate strengths. Finally, the fourth objective was to assess the research results to determine whether dynamic testing might provide a means to better classify coarse aggregates for use in PCC and to possibly provide performance criteria for concrete pavements.

The major conclusions and observations developed from this research are summarized in the following three sections. The first section provides the conclusions and observations for the preparation of the PCC along with the standard strength testing to determine the effect of varying the coarse aggregate type. The second summarizes the results of the dynamic and static strength of the coarse aggregate and PCC, where the results of this research were then used to determine the coarse aggregate types to investigate in the aggregate interlock research. The third section presents the major conclusions and observations for the aggregate interlock research.

1.1 PCC Preparation and Static Strength Results

An important aspect of this research was to prepare PCC with uniform properties. Therefore, a concerted effort was made to properly utilize the MDOT mortar voids method in preparing the concrete tested in this research. The ability to prepare concrete was necessary to observe possible strength variations when different coarse aggregate types were investigated. All of the concrete in this research was prepared following the MDOT P1 mix design keeping the components as consistent as possible while only varying the coarse aggregate. For this phase of the research three different coarse aggregate types were used; basalt, gravel, and blast furnace slag. These three coarse aggregates provided a broad range of aggregate characteristics such as texture, specific gravity and strength. The following conclusions and observations from this phase of the research are summarized below.

(1) Automated test methods were used to determine the coarse aggregate's apparent specific gravity, bulk dry specific gravity and absorption. The results had excellent agreement with the standard ASTM test methods for the basalt and gravel aggregates. However, there was significant variation in the blast furnace slag aggregate values. It is believed that the conventional method of determining apparent specific gravity does not provide an accurate measure for the aggregates solid volume due to the inability of the water to fully penetrate the aggregate's void system. On the other hand, the helium gas, which was used by the automated method using a helium pycnometer, better penetrates the aggregate's void system providing a more accurate measure of the aggregate's true solid volume, which is needed to calculate the apparent specific gravity.

(2) All P1 mixes generated adequate (static) strength independent of coarse aggregate type compared to the design strength of 24 MPa (3500 psi) at 28 days.

(3) There were, however, strength variations up to 10% for uniaxial compression tests and 12% for indirect tension with the blast furnace slag PCC having the highest strength and the basalt PCC the lowest in both compression and tension while the gravel PCC fell in between these two.

(4) It was observed that the surface of the slag aggregate had significantly more surface voids that allowed mortar to penetrate into the particles. It was also observed in the *yield data* that the slag PCC had an overall decrease in volume per batch. It is believed that the capacity of the mortar to penetrate into the blast furnace slag coarse aggregate provided the increase in strength observed in the slag PCC over the other PCC tested. This is, the pores provide a better interlock with the mortar.

(5) The strength test results suggest that the overall strength of PCC is controlled primarily by the mortar's strength while the coarse aggregate's shape and texture has a secondary influence.

1.2 Static and Dynamic Strength of Aggregate, Mortar and PCC

As discussed in the Introduction, strength testing of aggregates and PCC is conducted at a relatively slow loading rate based on ASTM and AASHTO standards and is considered in the dynamic testing literature as being a static or quasi-static test. Further, a material's static strength is controlled by the presence of larger defects within the materials such as cracks, bedding planes or weak zones, while during dynamic loading the loading pulse travels through the material at a rate in which the macroscale defects do not have time to react subjecting the entire material to full loading prior to failure. As a result, the material's microstructure has a greater control over its dynamic strength. Consequently, materials where the dynamic strength and stiffness are greater than its static strength and stiffness are considered to be rate sensitive, which is the focus of this section.

The following three sections, therefore, provide the major conclusions and observations of the results of the static and dynamic testing of the aggregate, mortar and PCC tested in this research. To provide a better understanding of these conclusions and results, some of the research results, e.g., presented in figures and tables, which are more fully described and discussed in the following chapters, are repeated in the following sections. However, not all of the research details describing these results are provided and therefore the reader is advised to consult the following chapters for more detailed information concerning the research results presented in these sections.

1.2.1 Aggregates

Uniaxial compression testing was conducted on the following aggregate types: (1) three blast furnace slags, (2) three limestones, (3) four dolomites and (4) two igneous aggregate for a total of 12 aggregate types tested. Both static and dynamic testing was conducted under dry and moist conditions; with the dynamic tests being conducted on a one-half-inch split Hopkinson pressure. While the aggregates were placed in water for approximately two days prior to testing, they were not vacuum saturated and therefore were not considered fully saturated when tested. The conclusions and observations, based on the static and dynamic uniaxial compression test results, are summarized as follows.

(1) The static uniaxial compression results for the igneous and carbonates had excellent agreement with the commonly used Deere & Miller rock strength classification system, verifying the static uniaxial compression testing procedures used in this research. The Deere & Miller classification system along with the test results is shown in Figure 2.1.

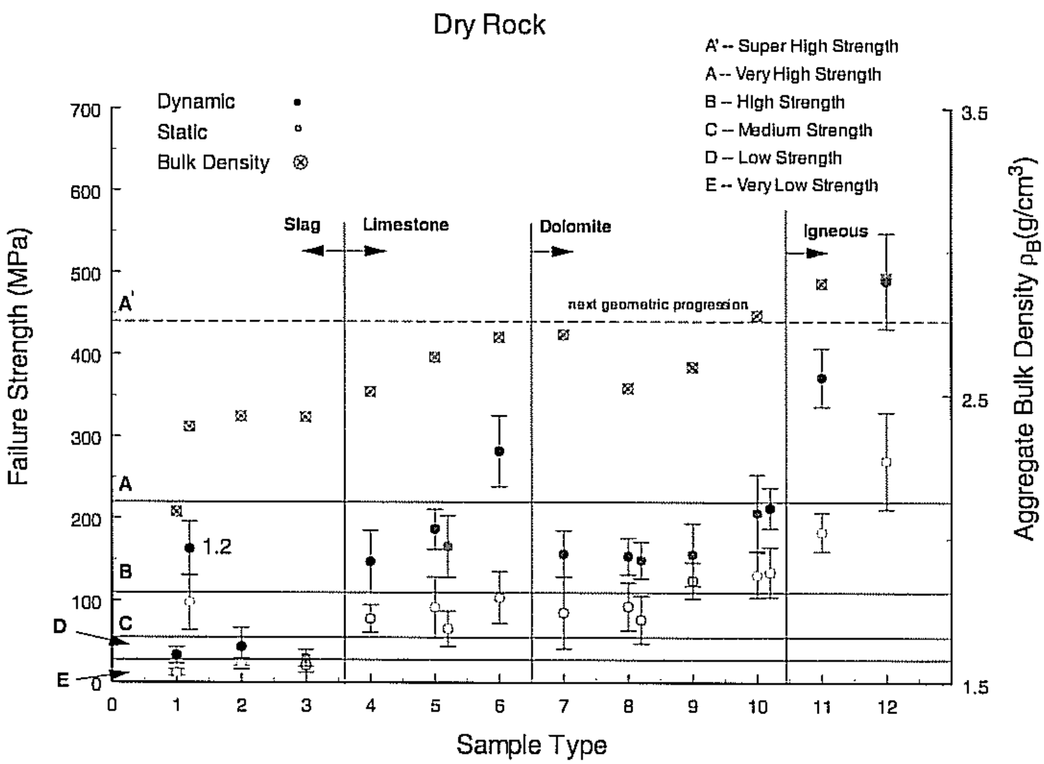

Figure 2.1 The Deere & Miller Strength Classification System along with the test results.

a. The mafic igneous aggregates had the highest uniaxial compressive strength under static loading conditions and are rated as "high strength" (Category A) according to the Deere and Miller Rock Classification System. The carbonate aggregates had average strength and are rated as "medium strength" (Category C), although two of the dolomites carbonates are in the next higher category "high strength" (Category B). The blast furnace slag's tested had the lowest strength and are rated as "very low strength" (Category E). However, the dense portion of the air-cooled slag (specimen 1.2) had significantly higher strength and is rated as "medium strength" (Category C). This strength was just below the "high strength" boundary and very close to the strength of dolomites.

b. In general, the dynamic strength for most of the aggregates increased by one strength category on the Deere & Miller Rock Strength Classification System over the aggregate's static strength. Since the static strength of the Specimen 12 (Bruce Mines aggregate, 95-010) was already in the "high strength category," an additional strength category needed to be added to the Deer & Miller classification system following the geometric progression used to generate the existing categories. The new category is termed "super high strength" (Category A') and is the category where the dynamic strength of the Bruce Mine aggregate (95-010) is in.

c. The majority of the carbonate aggregates are in the medium strength Category C with the exception of two dolomites, which are in the high strength Category B. In general, the *static* strength of the dolomites is higher than average *static* strength of the limestones, which is typically reported in the literature. However, the reverse occurs for the dynamic strengths in which the limestones have a higher average strength than the dolomites.

d. Specimens 1 (Algoma, 95-006), which was air-cooled, and 2 (Algoma, 95-006) and specimen 3 (Levy, 82-019), which were water-quenched, had the lowest aggregate strength and are rated as "Very Low Strength." However, the dense portion of the

Algoma air-cooled slag (specimen 1.2) is significantly stronger and is two strength categories higher at "Medium Strength" (Category B) and is approximately equivalent to the carbonate aggregate strength. It was also observed that even at a very low bulk density of 2.09 g/cm^3 the porous air-cooled slag had strength equal to the water-quenched slag, which had a significantly higher bulk density at 2.40 g/cm^3. It is speculated that the early crushing of the water-quenched slag may result in a more rapid cooling of the slag reducing the slag's strength.

e. There is a very good correlation with dynamic strength and bulk density for the limestone aggregate and similarly for the dolomites with the exception of the Cedarville dolomite. It appears that the random and non-uniform grain size distribution of the Cedarville dolomite (specimen 7 in Figure 2.1), which is believed due to the partial secondary replacement nature of dolomite, may account for this discrepancy.

(2) All of the aggregates tested were rate sensitive. Two measures of rate sensitivity were used in this research. The first measure was the dynamic to static strength ratio (*D/S*), which is also known as the dynamic increase factor (DIF). The second measure was the strain rate sensitivity parameter λ that provides a measure of the dynamic strength increase as an inverse function of the difference in the dynamic and static strain rate at failure. Functionally, the two parameters are described as follows:

$$\frac{D}{S} = \frac{\sigma_d}{\sigma_s} \tag{2.1}$$

$$\lambda = \frac{d\sigma_f}{d\left(\log\dot{\varepsilon}\right)} = \frac{\sigma_d - \sigma_s}{\log\left(\frac{\dot{\varepsilon}_d}{\dot{\varepsilon}_s}\right)} \tag{2.2}$$

where σ_d = Dynamic uniaxial compressive strength
σ_s = Static uniaxial compressive strength
$\dot{\varepsilon}_d$ = Dynamic uniaxial strain rate at failure
$\dot{\varepsilon}_s$ = Static uniaxial strain rate at failure

(3) The dynamic to static strengths ratios (D/S) ranged between 1.33 and 2.68 for the aggregates tested. There was a noticeable increase in the average D/S value between moist and dry conditions for the blast furnace slag and the mafic igneous aggregate with an average of 1.86 and 2.62 respectively. However, there was no noticeable difference in the carbonate aggregates between moist and dry conditions. There was, though, a significant difference in the D/S between limestones and dolomites at 2.27 and 1.74, respectively. This represents a 40% difference in the total range in the D/S of the aggregates tested. The D/S ratios obtained in this research are provided in Table 2.1. Note that the numbers listed under "ID No." correlate to those in Figure 2.1.

Table 2.1 Dynamic to static strength ratios.

<table>
<tr><th rowspan="2">ID No.
Pit ID</th><th rowspan="2">Aggregate/
(Quarry)</th><th rowspan="2">Orientation
and Batch</th><th colspan="4">Compressive Fracture Strength
Dynamic/Static Strength (D/S) Ratio</th></tr>
<tr><th>Dry</th><th>Aggregate
Average</th><th>Saturated</th><th>Aggregate
Average</th></tr>
<tr><td>1
95-006</td><td>AC Slag
(Algoma)</td><td>Batch 1
Batch 1.2</td><td>2.74
1.67</td><td rowspan="3">Slag
1.93</td><td>2.42</td><td rowspan="3">Slag
2.68</td></tr>
<tr><td>2
95-006</td><td>WC Slag
(Algoma)</td><td>Batch 2.0
Batch 2.1</td><td>1.90</td><td>2.75
3.49</td></tr>
<tr><td>3
82-019</td><td>WC Slag
(Levy)</td><td>Random</td><td>1.42</td><td>2.05</td></tr>
<tr><td>4
71-047</td><td>Limestone
(Presque Isle)</td><td>Random</td><td>1.90</td><td rowspan="3">Limestone
2.30</td><td>2.67</td><td rowspan="3">Limestone
2.23</td></tr>
<tr><td>5
06-008</td><td>Limestone
(Bay Co.)</td><td>Normal
Parallel</td><td>2.04
2.54</td><td>1.88
2.30</td></tr>
<tr><td>6
75-005</td><td>Limestone
(Port Inland)</td><td>Random</td><td>2.72</td><td>2.05</td></tr>
<tr><td>7
49-065</td><td>Dolomite
(Cedarville)</td><td>Random</td><td>1.84</td><td rowspan="4">Dolomite
1.64</td><td>2.15</td><td rowspan="4">Dolomite
1.83</td></tr>
<tr><td>8
58-009</td><td>Dolomite
(Denniston)</td><td>Normal
Parallel</td><td>1.66
1.95</td><td>1.55
2.05</td></tr>
<tr><td>9
58-008</td><td>Dolomite
(Rockwood)</td><td>Normal
Parallel</td><td>1.25</td><td>1.82</td></tr>
<tr><td>10
93-003</td><td>Dolomite
(France St.)</td><td>Normal
Parallel</td><td>1.58
1.58</td><td>1.42</td></tr>
<tr><td>11
31-076</td><td>Basalt
(Moyle)</td><td>Random</td><td>2.03</td><td rowspan="2">Igneous
1.78</td><td>2.91</td><td rowspan="2">Igneous
2.55</td></tr>
<tr><td>12
95-010</td><td>Diabase
(Ontario)</td><td>Water Cut
Oil Cut</td><td>1.81
1.51</td><td>2.18</td></tr>
</table>

(4) A strain rate sensitivity parameter λ defined above takes into account the difference in static and dynamic strength and normalizes it to the difference in strain rate between the static and dynamic loading rates. The λ parameters obtained in this research are provided in Table 2.2. The slag aggregates have the lowest λ, ranging from 1.17 to 3.00 for the water quenched slag, but 9.29 for the dense air-cooled slag (specimen 1.2) for an overall average of 4.2. The carbonates have the intermediate values ranging from 4.52 to 25.52, with an average of 11.9. The high strength igneous aggregates have the highest λ, ranging from 26.90 to 31.30, with an average of 29.1. Based on the rate sensitivity parameter λ the trend in highest to lowest rate sensitivity was as follows: diabase > basalt > limestone > dolomite > slag. In addition, there was a significant range in the λ values for the aggregate such that they can be clearly differentiated, which might potentially be used for a classification system.

Table 2.2 Strain rate sensitivity λ values

ID Number	Strain Rate Sensitivity, λ Aggregate	λ	λ Average
1.0	Algoma air cooled blast furnace slag – porous section	3.00	
1.2	Algoma air-cooled blast furnace slag – dense section	9.81	
2	Algoma water-quenched blast furnace slag	2.93	4.2
3	Levy water-quenched blast furnace slag	1.27	
4	Limestone, Presque Isle	9.97	
5	Limestone, Bay County	13.59	16.4
6	Limestone, Port Inland	25.52	
7	Dolomite, Cedarville	10.27	
8	Dolomite, Denniston	8.77	
9	Dolomite, Rockwood	4.52	8.6
10	Dolomite, France Stone	10.81	
11	Basalt, Portage Lake Lava Series, Moyle	26.90	
12	Diabase, Ontario Traprock	31.30	29.1

(5) For the aggregates tested there is a general increase in the strain rate parameter λ with increasing bulk density with a linear correlation coefficient of 0.61. This correlation, however, did not include the slag aggregates since there did not appear to be a relationship between λ and bulk density for the slag. A correlation coefficient to 0.74 was found for the limestone, but the dolomite correlation coefficient was only 0.42.

(6) There was a significant difference in the average strain rate sensitivity parameter λ between the dense air-cool slag (specimen 1.2) at 9.8 and the remaining three slags at 2.4. The rate sensitivity of the air-cooled slag was even higher than the average rate sensitivity of the dolomite aggregates at 8.6.

(7) The dynamic testing results D/S and rate sensitivity parameter λ between limestones and dolomites is relatively large compared the range of all the aggregates tested. The limestone and dolomites had a D/S of 2.30 and 1.64, respectively, while the average rate sensitivity parameter λ for limestone and dolomite was 16.4 and 8.6, respectively. Inspecting the carbonate's microstructure indicates that for limestone the rate sensitivity increases (both in D/S and λ) with decreasing grain size while the opposite occurred for dolomite where the D/S and λ decreased with decreasing grain size. It is hypothesized that the limestone and dolomite's geologic formation may help explain this difference. Basically, limestones form as a primary sedimentary rock while dolomite forms by chemically altering the structure of the original limestone. This includes recrystallizing and replacing calcium with heavier magnesium and iron ions. It is highly probable that this replacement process results in a weakening of the dolomite's grain boundaries and thus results in lower dynamic strength. However, it is also possible that the higher *static* strength of dolomite versus limestone might result from a healing process during replacement, where some of the larger defects are repaired.

(8) The D/S ratios for the igneous and slag aggregates were approximately equal indicating similar microstructures, but with significantly different strain rate parameter values (4.2 versus 29.1) indicating that the grain boundary strength was significantly different between the igneous and slag aggregate. It is suggested that the D/S ratio provides an indication of microstructure characteristics, while the rate sensitivity parameter provides a measure of microstructural strength.

(9) The rate sensitivity parameter λ was compared to the freeze/thaw (F/T) susceptibility dilation and durability index values for all the aggregates tested. While there was considerable scatter in the data, the aggregates were separated into two clearly

identifiable groups; those with rate sensitivity parameters greater than 25 and those less than 15. When correlating the carbonate aggregates, which had rate sensitivity parameters less than 15, but excluding the Bay County limestone, there was an excellent agreement between rate sensitivity and the F/T index values dilation and durability with a linear correlation coefficient of 0.98. The Bay County limestone was excluded because it appeared to have contradictory performance vales, e.g., LA abrasion, F/T.

(10) There was a general linear inverse relationship between the LA abrasion index values and the static and dynamic compressive strength results. The dynamic strength results had a somewhat better correlation with a linear correlation coefficient of 0.77 versus 0.66 for the static strength results. In addition, the slope of the dynamic strength versus LA abrasion results was steeper providing a broader separation of dynamic strength and LA abrasion data.

(11) There was also a general inverse relationship between the strain rate sensitivity parameter λ and LA abrasion values, with a linear correlation coefficient of 0.74 for all aggregates. However, this relationship does not necessarily hold for the carbonate aggregate. For the carbonates, it appears that the relationship is reversed with the LA abrasion values increasing with increasing rate sensitivity.

1.2.2 Mortar

The mortar was prepared in two batches at an air content of 9 to 10%. Static and dynamic tests were conducted under dry and moist conditions using the one-half-inch split Hopkinson pressure bar located in the Department of Mechanical Engineering and Engineering Mechanics. The mortar was tested over an 18-week period with testing once a week. The conclusions and observations, based on the static and dynamic uniaxial compression test results, are summarized as follows.

(1) The mortar was found to be rate sensitive with the dynamic to static strength ratio (D/S) ratio range between 1.5 and 3.5 over the 18-week testing period.

(2) The mortar's static and dynamic strength varied over the 18-week testing period with two increases followed by decreases. Interestingly, a high strength period where the D/S was approximately 3 occurred at the 28-day testing period followed by a decrease to a D/S of 1.5 in week nine.

(3) The D/S changes may indicate that the development of the mortar's microstructure during the curing process may not be constant. However, it is possible that testing procedures may have also played a role in the increase and decrease in strength over the 18-week period.

1.2.3 Portland Cement Concrete

Two separate batches of PCC were prepared for each coarse aggregate type tested. The first batch was prepared for the static and dynamic indirect tension and compressive strength testing, while the second batch was prepared for the aggregate interlock testing. The following five coarse aggregate types were tested:

Coarse Aggregate Type	MDOT Pit Number
Bruce Mines, Diabase	95-010
Port Inland #1, Limestone	75-005
Presque Isle Stone, Limestone	71-047
Superior Sand & Gravel	31-045
Levy Steel Dix #1, Slag	82-019

In addition, aged concrete from existing highway pavement sections were cored and tested. The PCC specimens were tested under static and dynamic loading conditions in both uniaxial compression and indirect tension. The static tests were conducted using a 55 kip MTS system with a TestStar II digital controller, while the high strain rate tests were conducted using a three-inch diameter Split Hopkinson Pressure Bar (SHPB) located in the Concrete Testing Laboratory at Michigan Tech shown in Figure 2.2. The system functions in basically the same way as the half-inch diameter system but had not been modified for single load testing. The conclusions and observations of the testing are presented in the following two sections.

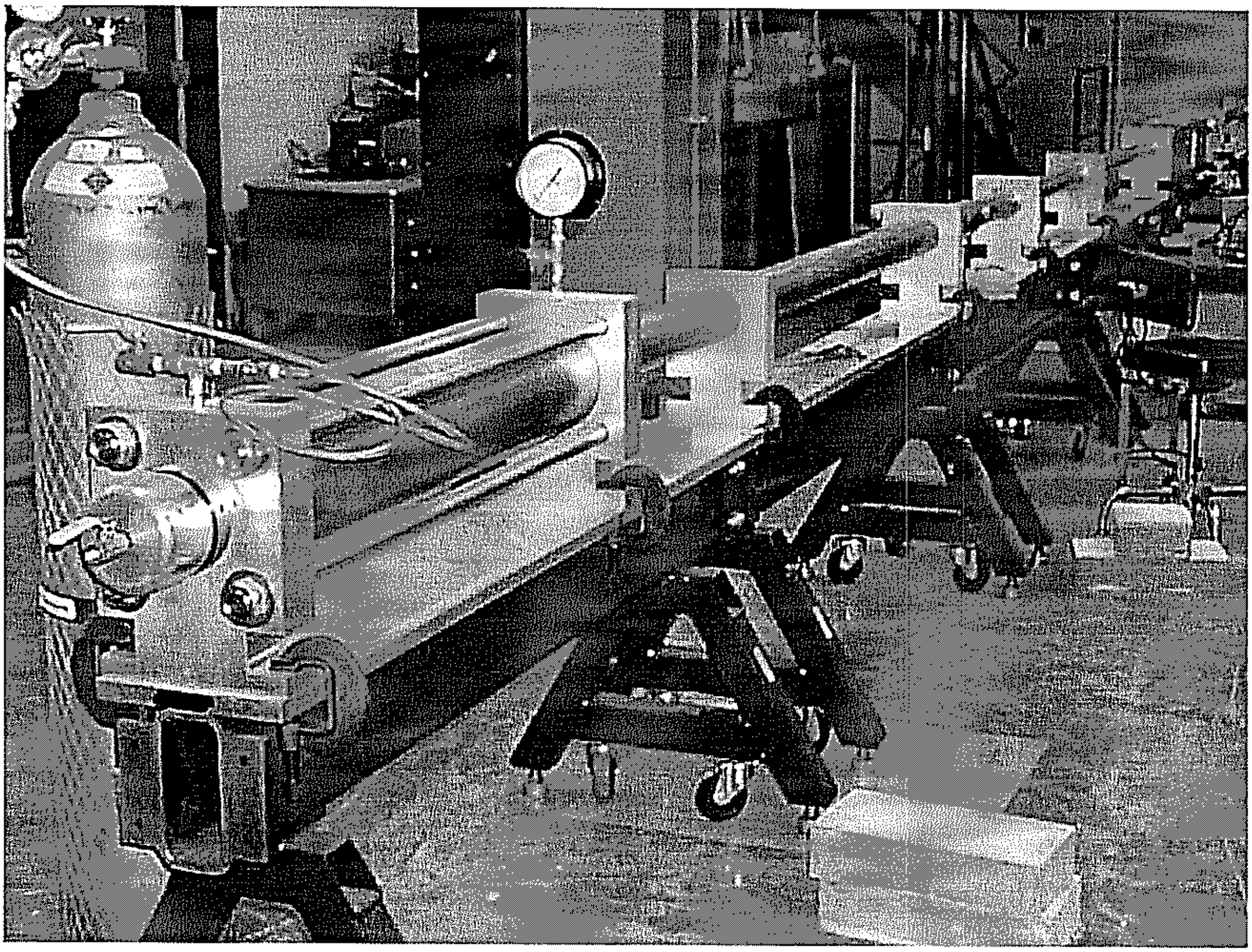

Figure 2.2 Three-inch split Hopkinson pressure bar.

1.2.3.1 Indirect Tension Test Results

(1) The PCC was found to be only slightly rate sensitive in tension. The dynamic to static strength ratio (D/S) ranged between 1 and 1.3. In addition, there was no statistical correlation between PCC strength and coarse aggregate type in either the static or dynamic indirect tension testing.

(2) The results of the indirect tension did not correlate with the results from other US researchers who found a 6 to 8 D/S ratio. However, similar research in Europe developed

a model known as the CEB model. According to the CEB model the concrete's rate sensitivity in tension at the strain rate tested in this research would have had a D/S ratio of approximately 2.2, somewhat closer to the results in this research.

(3) It is believed, however, that the results of the indirect tension testing might not have been properly conducted although the reasons for this remain unclear since other researchers used the same procedures that were used in this research. It is suspected that one reason for the lower results may be attributed to not having proper alignment of the specimen in the split Hopkinson pressure bar device.

1.2.3.2 Uniaxial Compression Test Results

(1) All of the 30-day[1] cured PCC tested in uniaxial compression was rate sensitive. In general, the PCC had an average static compressive strength of 45 MPa (6,525 psi), while the dynamic compressive strength was approximately 67 MPa (9,720 psi). The dynamic to static strength ratio (D/S) ranged between 1.4 and 1.9, which agrees extremely well with the results from other researchers.

(2) The aged pavement concrete (40 years old) was also found to be rate sensitive. The natural aggregate PCC had a static compressive strength at 80 MPa (11,600 psi), which was higher than all of the 30-day cured PCC tested, while its dynamic strength was significantly higher at 120 MPa (17,400 psi). On the other hand, the aged slag coarse aggregate PCC (25 years old) had a static compressive strength of 45 MPa (6,525 psi), which was close to the average strength of the 30-day cured PCC, while its dynamic strength was 78 MPa (11,300 psi), which was approximately equal to the static strength of the aged PCC, but somewhat higher than the dynamic strength of the 30-day cured PCC.

(3) There was generally good agreement between the six-inch diameter specimens and the three-inch diameter specimens tested in static loading conditions with a 6% difference for

[1] The PCC was tested at 30 days instead of the traditional 28-day due to a longer sample preparation time.

the Bruce Mines PCC, 2% for the Presque Isle PCC, and 2% for the Port Inland PCC. However, the Levy slag and Superior Sand & Gravel PCC had a 15% and 14% difference, respectively.

(4) There was no statistical correlation found between either the static and dynamic strength of the coarse aggregate in the PCC and the static or dynamic strength of the concrete. This suggests that the mortar controls the primary strength of concrete in both static and dynamic loading, while the coarse aggregate plays a secondary role.

(5) The dynamic strength of fresh concrete was greater than concrete that was oven dried.

(6) The results of the dynamic to static strength ratios (D/S) for the fresh (moist) and oven dried PCC are presented in Table 2.3. The results indicate that the dry PCC was relatively consistent with an average D/S of 1.41 for coarse aggregate types tested, while the moist PCC had an average D/S of 1.78. However, the average was 1.82 when excluding the Port Inland PCC. It was suspected that the Port Inland PCC might have been improperly prepared. While the D/S ratio for dry Port Inland PCC was similar to the other PCC, its moist D/S at 1.59 was well below the other moist D/S values. This is an interesting result since the air content, slump, and dry D/S ratio did not indicate any problems with the Port Inland PCC.

Table 2.3 Ratio of dynamic to static strength tests for uniaxial compression

PCC Type		Oven Dry Dynamic/Static	Fresh (moist) Dynamic/Static
Bruce Mines	(95-101)	1.50	1.86
Levy Slag	(82-019)	1.38	1.80
Port Inland	(75-005)	1.38	1.59
Presque Isle	(71-047)	1.40	1.84
Superior Sand & Gravel	(31-045)	1.40	1.79
Natural Aggregate:	Aged (40 yr)	1.50	Not Tested
Slag Aggregate:	Aged (25 yr)	1.73	Not Tested

(7) Since a material's rate sensitivity, as defined by the D/S ratio, has been found to originate from microstructural inhomogenieties such as pores, cracks and impurities that exist along the grain boundaries, the PCC D/S ratio results are believed to be a function of the concrete's microstructure and in particular the mortar. In addition, it has been shown that the presence of water in the pore space (Conclusion 5) also affects the rate sensitivity of concrete. Consequently, the D/S values for concrete may be a means by which the concrete's microstructure and air void system may be better quantified.

(8) The results of the rate sensitivity parameter λ for concrete were similar to the D/S ratio results showing a significant difference between dry and moist conditions as shown in Table 2.4. In addition, it also indicates that there was a problem with the Port Inland PCC, which had a significantly lower moist λ value than the other PCC tested. However, a significant difference between the D/S ratio and the λ values was in the strain rate parameter values between fresh (30-day cured) and aged concrete in dry conditions with an average 2.7 and 5.6, respectively. This suggests that the λ is sensitive to the concrete's maturity while the D/S ratio appears to be more a function of the concrete's microstructure. However, it is unclear at this point regarding the relationship between the D/S ratio and the λ and why the λ is sensitive to the maturity of a concrete while the D/S ratio is not. It appears, though, that both the D/S ratio and λ may provide significant information to better predict the performance of concrete.

Table 2.4 Rate sensitivity parameter λ for dry and moist conditions.

PCC Type		Dry Dynamic/Static	Moist Dynamic/Static
Bruce Mines	(95-101)	3.3	5.3
Levy Slag	(82-019)	2.8	5.2
Port Inland	(75-005)	2.3	3.6
Presque Isle	(71-047)	2.4	5.4
Superior Sand & Gravel	(31-045)	2.7	5.1
Natural Aggregate	Aged (40 yr)	6.2	Not Tested
Slag Aggregate	Aged (25 yr)	5.1	Not Tested

1.3 Aggregate Interlock

A critical component in the long-term performance of PCC pavements is the ability of joints and cracks to effectively transfer vehicle shear loading across the joint or crack interface. A key component in aggregate interlock is the joint width or crack width and the strength of the coarse aggregate. Recent research at the University of Illinois has found that the strength of the aggregate dramatically affects aggregate interlock. The Illinois research, which was conducted for the Federal Aviation Administration, constructed an aggregate interlock test system and investigated three coarse aggregate types; basalt, carbonate and a natural aggregate. A total of 34 tests were conducted over a range of crack widths. The primary finding of this study was that the coarse aggregate type dramatically affected aggregate interlock at large crack widths. The test showed that the basalt had the best results, followed by the natural aggregate, with the carbonates having the poorest results.

The first objective of the aggregate interlock research was to design and construct an aggregate interlock test system that was relatively easy to use, where tests could be set up in a minimum of time and where the tests could be very accurately controlled. The second objective was to conduct aggregate interlock tests on concrete using different coarse aggregate types with a range of static and dynamic strengths. The third objective was to determine how different coarse aggregates strengths affect aggregate interlock and if the dynamic strength of the aggregate could be used as a classification system to predict PCC long-term field performance. The conclusions and observations from the aggregate interlock research are divided into two sections. The first section is the development of the aggregate interlock system while the second section provides the conclusions and observations on the aggregate interlock test results.

1.3.1 Aggregate Interlock Test Equipment Development Conclusions

(1) The design, fabrication and construction of the structural frame and sample holders functioned well for their intended purpose, providing a frame that was structurally sound and adaptable for other research requirements. In addition, the sample holders performed very well, securely restraining the concrete blocks in place during testing. It was also found that the utilization of the holders, such as inserting the concrete blocks and

alignment could be accomplished in an efficient manner. A picture of the test system is shown in Figure 2.3.

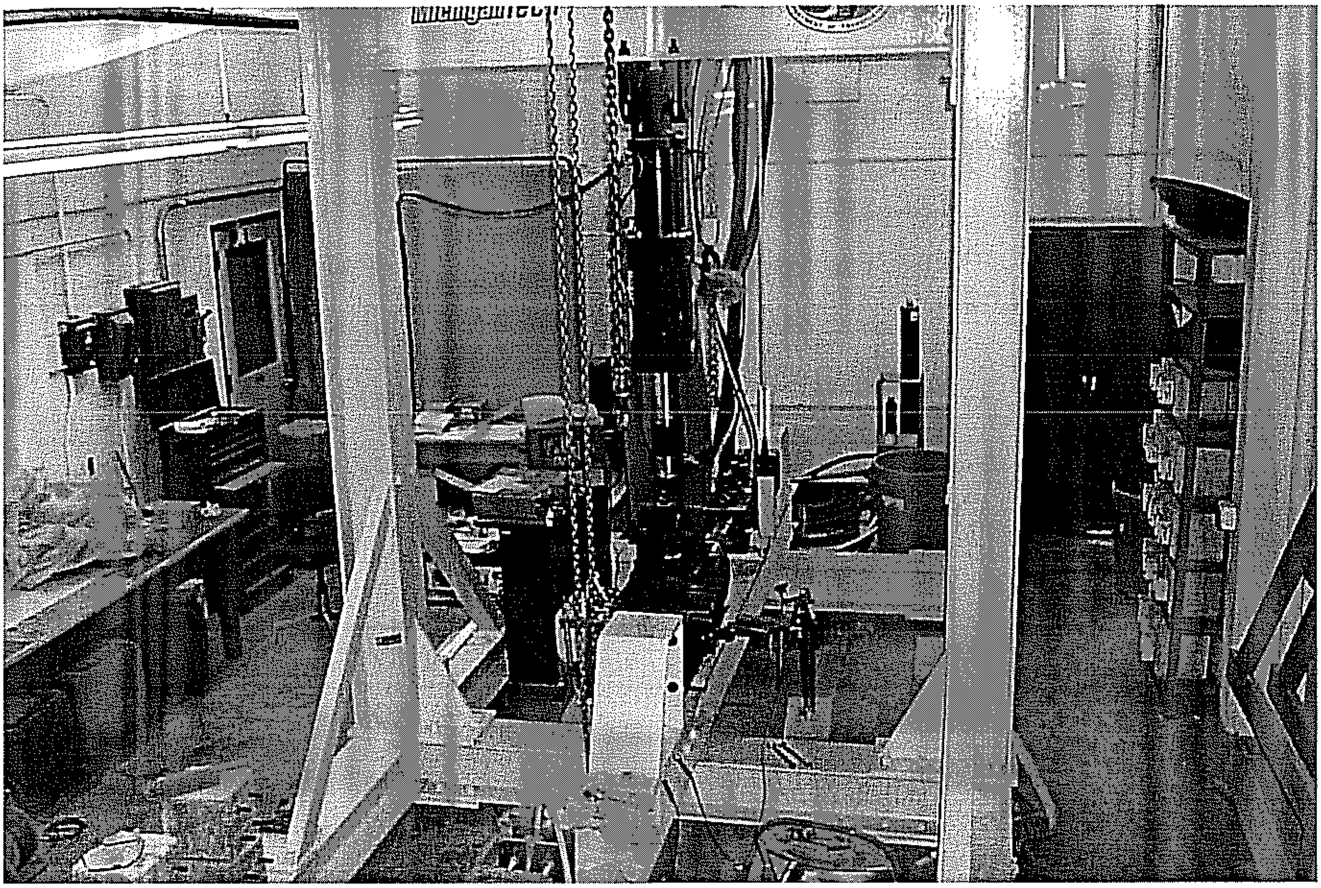

Figure 2.3 Aggregate interlock test system.

(2) The concrete fracturing device shown in Figure 2.4 performed moderately well in the initial development of the system. During later testing, however, a torque wrench was used to place a constant tension load on the threaded rods, which were secured in the concrete, prior and during sample fracture. In addition, anchor nuts were repositioned to the ends of the embedded threaded rods to provide better anchorage. These changes improved the performance of the sample fracture device, produced accurate data, and effectively produced cracks in all of the test blocks.

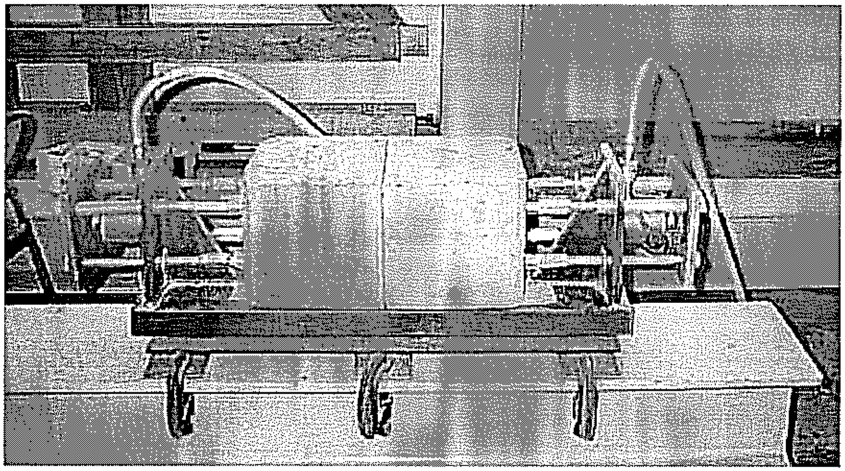

Figure 2.4 Concrete fracture device.

(3) Initially, a half sine wave in compression was applied to the concrete sample interface at a 9 kip load over a 0.1 second loading period as shown in Figure 2.4. However, it was clear from the initial test results that the nine kip load was too high for the concrete block size used. In addition, applying only a compressive load did not correctly simulate aggregate interlock. To better replicate aggregate interlock a sine-loading wave in both compression and tension was used as shown in Figure 2.5. Also, stress calculation indicated that a 3 kip load was more realistic for the concrete blocks being tested than the original 9 kips. However, the rest period between the sine wave load pulses was very difficult to control with the closed-loop control systems used in this research. After a number of attempts at trying to produce the pulse with a delay in the test sequence shown in Figure 2.5 a continuous sine-loading wave, without a rest period was used.

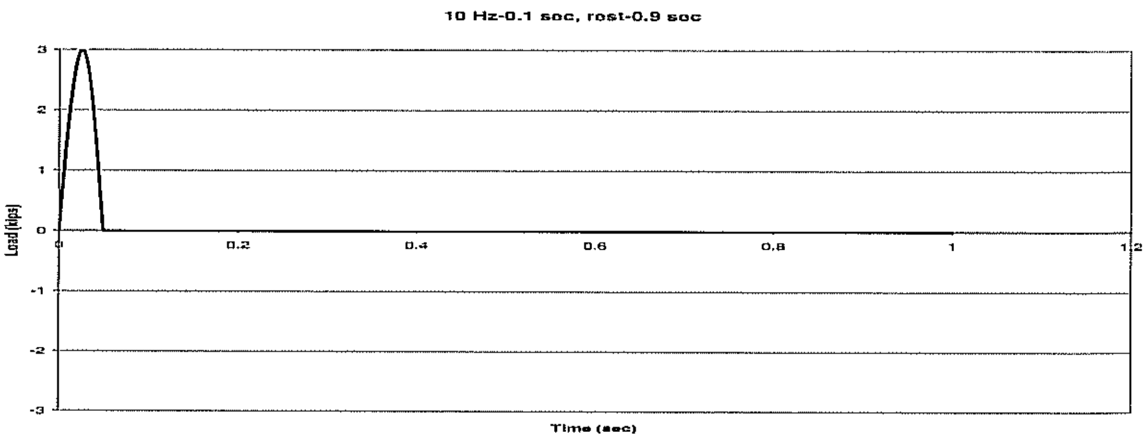

Figure 2.4 Compressive loading signal.

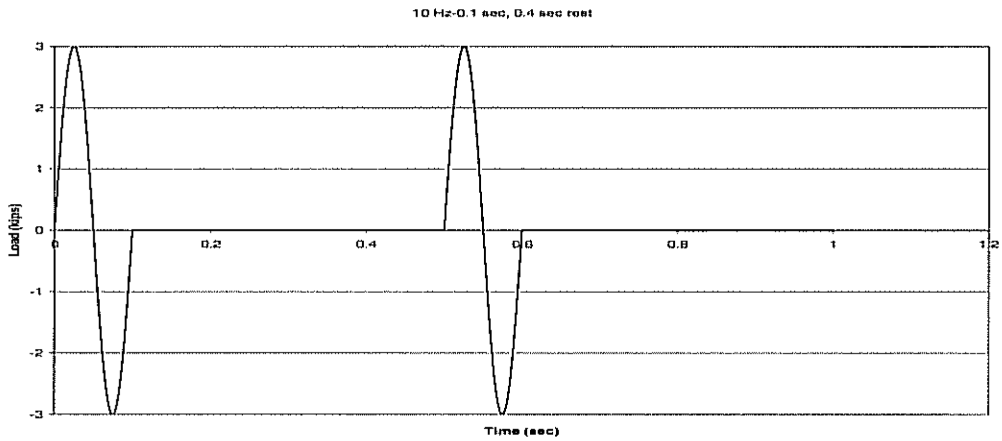

Figure 2.5 Compressive and tensile loading signal.

(4) A CNC mill with a LVDT displacement gauge was used to measure the surface texture of the concrete both before and after interlock testing. The results of this work were only partly successful. The main obstacle was in collection and analysis of the data. However, surfaces analyzed indicated, both visually and numerically, that the difference in roughness was a function of the coarse aggregate's strength with the stronger aggregate showing less wear than the weaker aggregates after testing.

1.3.2 Aggregate Interlock Testing Results

As discussed in the development of the aggregate interlock testing system, it became clear that the applied load was too large and that the method of loading needed to provide both a compressive load to the interface, as well as a tension load, to better simulate aggregate interlock. While lowering the applied load was straightforward, the application of a cyclical load between compression and tension was more difficult for the hydraulic and control system to handle. In addition, a significant factor in the close-loop control of the test was the stiffness of the joint interface itself. Basically, the stiffer the interface the easier it is for a close-loop control system to control the test. On the other hand, when the interface is not stiff it becomes more difficult to control, since more displacement is required for a given load and subsequently more hydraulic fluid is required to move the hydraulic piston. It was found that two significant factors controlled the concrete's joint stiffness; (1) the joint or crack width and (2) the coarse aggregate type at larger crack widths. All of the tests conducted at narrow crack widths of 0.028 inches performed well. However, when the crack width was increased the system had difficulty controlling the test for some of the PCC tested. The main problem in many of the tests was that the hydraulic system was back pressuring the return lines causing the hydraulic fluid to cavitate, i.e., gas bubbles forming in the fluid, causing the hydraulic system's emergency shut off system to stop the test. Unfortunately, this problem was not discovered until after the testing was complete and significant delays had been incurred. A second factor in the aggregate interlock testing was that the stiffness of the interface was controlled by the strength and stiffness of the coarse aggregate at larger crack widths. It was learned in the initial testing that the stronger coarse aggregate provided the stiffest interface while the weaker coarse aggregate provided the lower stiffness interface. Consequently, the control system was "fine-tuned" using the weaker coarse aggregate concrete, i.e., the closed-loop control response or ability to control the test was based on a the lower stiffness interface. A result of this is that all of the tests on the lower stiffness interface concrete worked well, while the higher stiffness interface concrete the system was unable to control the test due in part to the cavitation of the hydraulic fluid in the return line; but also in some cases the control system became so unstable the interface was damaged the resulting in premature failure. This can be observed in Figure 2.6 where the test results for the following three coarse aggregates are provided: (1) blast furnace slag, (2) limestone and (3) basalt. These tests were conducted at a crack width of 0.035 inches. All three aggregates types performed well between 1 and 100 cycles. Between 100 and 10,000 cycles the blast furnace slag develops significant displacement, while the limestone and basalt remain relatively constant. However, at approximately 10,000 cycles the basalt specimen started to have significant vertical displacement between the two PCC blocks indicating a rapid degradation of the aggregate interlock and ultimately resulting in premature failure of the specimen. Failure was selected as a maximum displacement of 0.5 inches between the two specimen blocks. The limestone, which was less stiff, became unstable after 50,000 cycles.

Although not all of the aggregate interlock tests were successfully completed, a number of conclusions and observations were still obtained from the successful tests. These conclusions and observations are provided as follows:

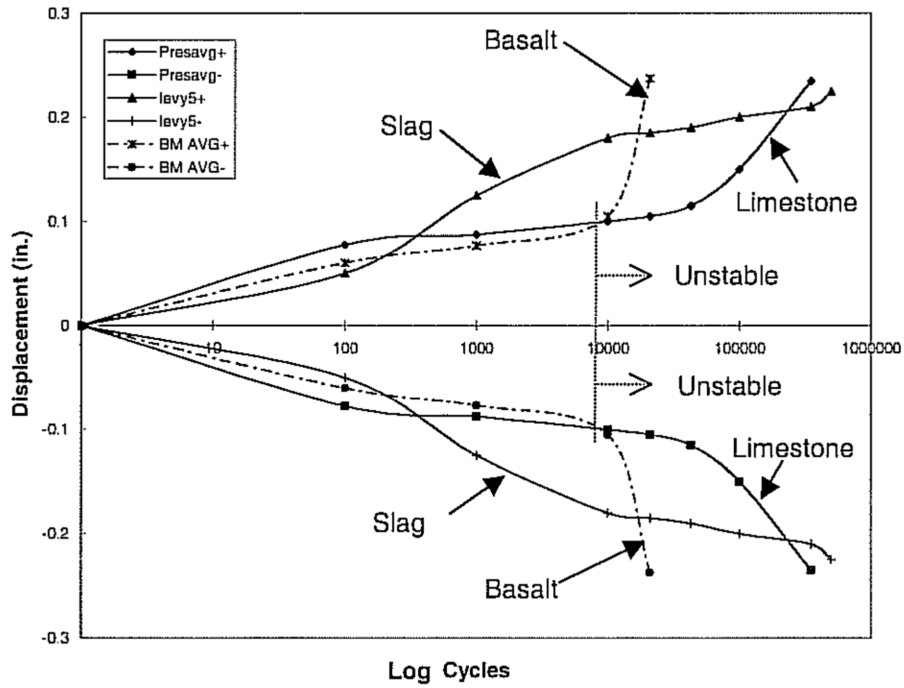

Figure 2.6 Aggregate interlock test results at 0.035 inch crack width.

(1) The aggregate interlock testing in this research was conducted under pure aggregate interlock since no base reaction was provided.

(2) It is believed that the aggregate interlock test system developed in this research, while experiencing some problems, is an effective system for testing aggregate interlock. Successful aggregate interlock testing can be accomplished provided the following changes are made.

a. First, the cavitation problem with the hydraulic return line must be eliminated. This can be accomplished by reducing the length of the supply and return hoses as well as slowing down the test frequency to one hertz (as opposed to two hertz). While changing the frequency will double the time for testing, it should also improve the test results.

b. Second, a different controller should be used to control the vertical (shear loading) actuator. A MTS TestStar controller (or equivalent) is recommended to accomplish this task.

c. Third, one concrete specimen per coarse aggregate type should be used to determine the system response and digital control settings. It should be noted, however, that this specimen once used for control setting would probably not be useful for further testing. The main reason for this is that significant degradation occurs in the early stages of loading. Consequently, it is likely that during the control setting that erratic behavior may occur, which will render the specimens as unusable.

(3) The aggregate interlock tests at a crack width of 0.024 inches indicated that this is an effective joint and crack width regardless of the aggregate type. However, it is speculated that stronger coarse aggregate concrete under very high vehicle loading may experience more degradation due to the gouging of the cement paste by the coarse aggregate. However, these higher deflections would be unrealistic given existing load limits on roads.

(4) It was apparent in the test results as well as in the literature that aggregate interlock stiffness varies significantly between aggregate concrete types at crack widths greater than 0.035 inches. Consequently, at large crack widths the coarse aggregate's strength and stiffness determines the degradation rate of the aggregate interlock, with the stronger aggregate providing better aggregate interlock over time. This was demonstrated in the Illinois research and in the two tests conducted at crack widths of 0.050 inches in this research.

(5) It appears that the coarse aggregate type also affects the morphology or texture of the concrete fracture surface. While the method used to quantify surface morphology was not as successful as anticipated, it did reveal to some extent that the stronger aggregate generated a rougher surface than the weaker aggregate.

(6) Tension fracturing of the concrete into specimen blocks was conducted at 18-hours. It was observed after fracture that coarse aggregate strength played a role in the overall tensile strength of the concrete at failure. For example, the Levy slag concrete always required higher pressure (tension load) to fracture the concrete compared to the other coarse aggregate concrete tested. It is speculated that the higher pressure, i.e., tension force required to fracture the concrete, was generated by the fracture surface going through the coarse aggregate as opposed to around the aggregate (pullout failure) resulting in smoother fractured surfaces. On the other hand, PCC with stronger coarse aggregate required lower pressure to fracture the PCC and produced rougher surfaces since less coarse aggregate fractured.

(7) The strength of the PCC specimens, as opposed to the strength of the coarse aggregate, does not appear to be a significant factor in aggregate interlock performance at large crack widths. This may be an important fact for future testing since it suggests that testing specimens at significantly different times in relation to their 28-day strength may not be important. That is, concrete may be tested anytime after a 28-day cure.

(8) The aggregate interlock test system was able to maintain an exceedingly constant crack width during testing. Since the horizontal actuator is used to accomplish this, it is relatively straightforward to adjust the crack width to any width desired. In addition, it would be possible to vary the crack width to simulate warm (small crack width) and cold weather (large crack widths) effects during a single test if desired.

(9) The aggregate interlock test set is relatively easy to use and can be set up in a short period of time. It is estimated that a test specimen (cured and ready for testing) can be set up in approximately two hours. As a comparison, in personal communications with the personnel at the University of Illinois it was stated that it took approximately two days to set up a test sample.

2 Recommendations for Further Research

The research investigated the novel use of a split Hopkinson pressure bar to investigate the dynamic properties of aggregate, mortar and concrete, as well as the development and use of an aggregate interlock test system. The research results strongly suggest that both dynamic testing and aggregate interlock research have significant potential. While the dynamic properties of concrete have been studied for military applications, there have not been studies for concrete pavements. The results of this research indicate that not only does dynamic testing describe the dynamic response of concrete, but it also may provide a means to quantify the properties of concrete, which may be related better to the performance of concrete than do traditional test such as the 28-day compression strength test. In addition, the aggregate interlock test system can provide a relatively efficient means to study the response of joints at various crack widths and in particular how the joint response varies with coarse aggregates strengths. Based on the results of this research, a number of recommendations are provided. These recommendations are divided into two areas; those involving dynamic testing and those involving the aggregate interlock testing.

2.1 Dynamic Testing Recommendations

The following research areas involving the dynamic response of aggregate, mortar and concrete are recommended for further study:

(1) The development of an aggregate database, which would include the dynamic to static ratio (D/S) and the strain rate parameter λ for aggregates used in Michigan. The database would consist of cross-section of aggregate such as metaphoric, igneous, sedimentary, including sandstone, limestone, dolomite, shale and industrial products such as blast furnace slag. Suggested specific research areas concerning the dynamic properties of aggregate are as follows:

- A detailed study of the dynamic properties of limestone and dolomite aggregate, since there was considerable difference between these two aggregate types. This would also include comparing the results of the dynamic parameters with the aggregate's microstructure. For example, the research showed an extremely high $\lambda = 25.5$ for the Port Inland limestone, near the level of igneous aggregate, while the Rockwood dolomite had a very low $\lambda = 4.5$ value. Yet, the Rockwood dolomite had a higher static strength than the Port Inland limestone. The microstructure analysis may provide insight into the reasons for this difference and to what extent grain size, fossil content and in the case of dolomites, whether the secondary replacement process has an effect on aggregate performance.

- Study the vertical variations in aggregate properties within quarries, e.g., various ledge formations in carbonate quarries.
- When freeze-thaw tests are conducted on concrete determine whether a correlation exists between the strain rate parameter λ and the freeze-thaw durability index.
- Investigate possible correlation of the dynamic parameters with aggregate wear, since aggregate wear is also a dynamic process.
- Further investigate the relationship between an aggregate's dynamic strength and its ability to function in aggregate interlock over the expected life of a pavement.
- Investigate why the dense portion of the air-cooled blast furnace slag had such good mechanical properties, as opposed to the water-quenched slag.

(2) The results of the dynamic parameters suggest that they could also be used to quantify the behavior of concrete in cases where traditional tests such as the 28-day strength, air content and slump may not. Therefore, the following recommendations concerning the concrete dynamic testing are suggested:

- It is recommended that research be conducted on the relationship between the D/S and λ values and the concrete microstructure. If a relationship can be established, then the dynamic testing might provide an inexpensive means to determine the long-term performance of the concrete without having to conduct more time-consuming microstructural characterization.
- A study of the effects of moisture on the D/S and λ parameters for concrete and the concrete's air-void system. Tests should also be conducted under fully saturated

conditions to study the effect of the loss of capillary stresses and its effect on the D/S and λ parameters, which may provide additional information on the nature of the air-void system.

- It is recommended that additional indirect tension testing be continued since the literature is indicating that a concrete might be better characterized by tensile strength as opposed to compressive strength. This might even be more important for the dynamic strength results.

2.2 Aggregate Interlock Recommendations

It is believed that the aggregate interlock test system developed in this research can be effectively utilized to conduct further aggregate interlock research. It is recommended that following research issues be considered:

- Correlation with the strain rate parameter λ: Additional aggregate interlock testing should be conducted to establish the relationship between λ and the rate of interface degradation at a constant crack width.
- Multiple tests at various constant crack widths: Develop a better understanding of the mechanism of shear transfer across various crack width settings. The crack width of 0.024 inches appears to be a very efficient crack width regardless of coarse aggregate type. However, aggregate interlock testing at larger crack widths would provide a measure of efficiency for aggregate interlock with different coarse aggregate types and crack widths.
- Changing crack widths during test: A specific joint/crack in the pavement does not maintain a constant width throughout its life. During the year the pavement goes through expansion and contraction cycles due to temperature change. Therefore, there are times that the pavement joint opening may be at 0.024 inches and other times, most likely in the winter months, that the crack width is much larger. With the capabilities of the dual actuators in the system, a scaled version of this cycle could be replicated. While the vertical actuator is administering the shear loading the horizontal actuator could change the crack width during the testing sequence.

For instance, if the frequency of the shear loading was kept at 1 Hz then the crack width could open and close over a range of 0.024-0.06 inches within a time period of 24 hours. This may better replicate the conditions in the field between summer and winter temperatures.

- Surface morphology: The surface morphology of the concrete interface should be measured using stereographic imagery, which would allow a better characterization of the concrete surfaces before and after testing. In addition, the effect of coarse aggregate type can also be examined as to how it affects the interface morphology after initial fracturing. This would also be very important in investigating the change in surface morphology with variations in crack width during loading. This information could be entered into a mathematical model to determine a relationship of surface texture to crack width. This is important to show that at larger crack widths, there is less surface area to generate the required load transfer. It could also be used to develop guidelines for crack width in the field.
- Aggregate Base: The effect of aggregate base and subgrade support for PCC pavements in aggregate interlock was not studied in this research. However, further research should be conducted to investigate the interaction of base material and the aggregate interlock load transfer mechanism. This has been anticipated in the design of the aggregate interlock system and can be accomplished by placing stiff springs between the fixed end holder and the steel frame base. The stiffness of the springs can be matched to various base stiffness or resilient modulus values. The system can then be modified to produce a load transfer mechanism that represents the base and sub-base material load transfer capabilities.

2.3 Automated Testing Recommendations

The automated specific gravity devices show excellent promise in quickly and accurately determining an aggregate's apparent specific gravity, bulk dry specific gravity and the maximum absorption. However, it is recommended that additional testing be considered to determine if the variation in test results with the blast furnace slag is due to the ASTM test method, which uses water to penetrate the aggregate, or with the helium pycnometer, which uses helium gas to penetrate the aggregate.

SECTION 3

Aggregate Selection and Characterization

1.1 Aggregate Selection

The aggregates used in this research were obtained from eight sources in Michigan, two in Ontario, Canada and one in Ohio. The location of each source is shown in Figure 1.1 while Table 1.1 lists the quarry, location and the MDOT aggregate source identification numbers. With the exception of the blast furnace slag, all of the aggregate samples were obtained from an active quarry soon after a production blast and prior to crushing and sizing. The blast furnace slag was obtained from steel mills in Ste. Sault Marie, Ontario and Detroit, Michigan.

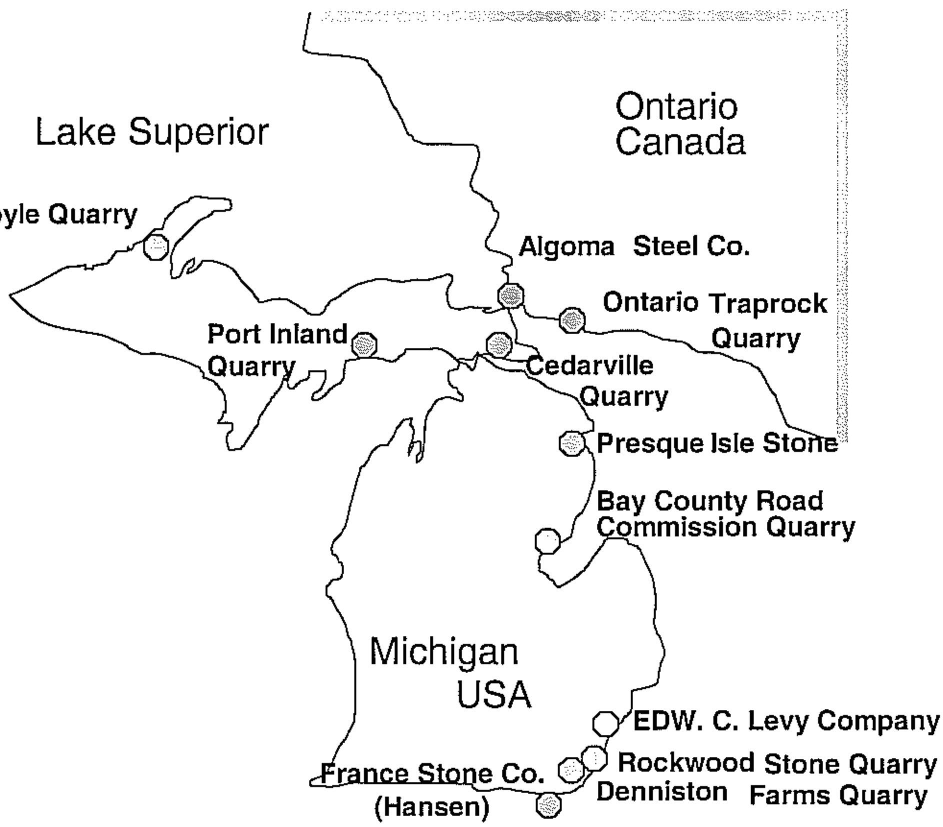

Figure 1.1 Aggregate sources locations.

Table 1.1 Aggregate Source Information

No.	Quarry	Aggregate Type	Location	MDOT Pit ID No.
1	Algoma	Air Cooled	Ste. Sault Marie, Ontario, Canada	95-006
2	Algoma	Water Quenched	Ste. Sault Marie, Ontario, Canada	95-006
3	Levy	Water Quenched	Detroit, MI	82-019
4	Presque Isle	Limestone	Rogers City, MI	71-047
5	Bay County	Limestone	Au Gres, MI	06-008
6	Port Inland	Limestone	Manistique, MI	75-005
7	Cedarville	Dolomite	Cedarville, MI	49-065
8	Denniston Farms	Dolomite	Monroe, MI	58-009
9	Rockwood	Dolomite	Newport, MI	58-008
10	France Stone (Hanson)	Dolomite	Sylvania, OH	93-003
11	Moyle	Basalt	Houghton, MI	31-076
12	Ontario Traprock	Diabase	Bruce Mines, Ontario, Canada	95-010

1.2 Aggregate Geology

1.2.1 Slag

Slag is a manufactured aggregate produced as a by-product in the production of pig iron and contains mainly silicates and calcium. During the making of pig iron the lighter slag material floats to the top of the blast furnace while the heavier pig iron sinks to the bottom where access doors are located to extract both materials. Consequently, the pig iron is released first followed by the slag. Generally, the pig iron is placed in containers and transported to a steel making facility while the slag is either sold or stockpiled, depending on its commercial value. Prior to slag being used commercially, it was simply disposed of in large slag piles generally located near the steel mills. The disposal was typically conducted using large buckets where the molten slag was placed and transported to the disposal area and dumped forming large slag piles. Significant tonnage of slag still exists at some blast furnace facilities. Later, when slag became a commercial product the slag was allowed to flow into trenches adjacent to the blast furnace building where it formed into a layer approximately a few feet thick. As the molten slag flows into the trench, water is sprayed onto the slag to induce thermal cracking in the slag allowing the slag to cool faster. However, the thermal mass of water is insufficient to appreciably cool the slag and so the slag is considered "air-cooled". After approximately 24 hours the slag is broken up by a front-end loader, transported to a crusher area, crushed and placed in stockpiles. However, to differentiate between the two types of slag the older technique of disposing of the slag in dumps will be referred to as "air-cooled" slag while the later method of placing the molten slag in trenches while spraying water on the slag will be referred to as "water-quenched" slag.

Thin-sections of the aggregates used in this research are provided in Appendix 3A, where each thin-section is provided with both a positive and negative image. However, only one thin-section was made from each aggregate and therefore they may not be representative of all of the aggregates tested in this research.

The air-cooled slag is shown in Figure A.1 while the water-quenched slag is shown in Figure A.2. Both the rate of cooling and method of deposition contribute to the crystalline nature of the slag. Comparing Figures A.1 for air-cooled slag and Figure A.2 for the water-quenched slag it can be seen that air-cooled slag has significantly smaller, splinter-like crystals while the water-quenched slag has a larger crystalline texture. Since the cooling rates are more than likely relatively similar, one possible explanation for the difference in the crystalline texture of the slag may be the method of deposition. As previously mentioned the air-cooled slag was deposited in large disposal areas where it was typically transported to the top of a pile and discharged over the slope. The molten slag would then flow down slopes of the piles forming along the slope of the pile. The water-quenched slag on the other hand was placed in shallow trenches where it spread out in the trench in a relatively uniform manner. Stratification of the slag with the heavier and darker mineral sinking towards the bottom, while the lighter minerals moves towards the top is clearly event in the slag. For the air-cooled slag it may be possible that the mixing action of the steep slope may allow better association of elements to form more nucleation sites for crystal growth in addition to generating the more splinter-like crystals. However, less crystal nucleation sites may develop in the slag deposited in trenches and as a consequence fewer but larger crystals form in the water-quenched slag. Nonetheless, there are a number of other possibilities that may contribute to the formation of the crystalline nature of the slag. In the study of material fracture, however, one of the primary controls of the fracture process is the crystalline nature of the material. Consequently, understanding the processes that contribute to the formation of an aggregate may also provide a better understanding of it strength and other performance related issues.

1.2.2 Carbonates

Figure 1.2 illustrates the stratigraphic sequence of the natural aggregates studied in this research. The carbonate aggregates were all mined in the Michigan Basin and are of Paleozoic age, ranging from 300 to 425 million years old. Figure 1.3 illustrates the unique geology of the Michigan Basin presented by Dorr and Eschman (1970) with the the seven carbonate aggregate source locations superimposed on the figure. One source is of Mississippian age (310 to 345 million years old), three sources are Devonian age (345 to 405 million years old), while three sources are of the older Silurian age (405 to 425 million years old). In general, the Michigan Basin consists of sedimentary rocks ranging from sandstones, shales, limestones, dolomites and evaporites (salts). Economically, the carbonates and salt deposits are the dominant materials mined in the Michigan Basin. The carbonates form as a chemical precipitate from an ancient ocean in which most of the sedmintary rocks in the basin were formed. Carbonate rocks can form either with a clastic texture, i.e., a collection of carbonate fragments, or in a crystalline form directly from a precipitate. The clastic textured carbonates go through a lithification process by which the unconsolidated materals are converted into rock. Lithification generally results from the pressure overlying rock and geothermal heat flow. In general, the greater the lithification (measured in both time, pressure and temperature) the more consolidated the rock is and theoretically stronger. However, studies conducted to relate strength, for example, to the degree of consolidation have not been successful at proving this point. If this were the case, the older more deeply buried carbonates would possess greater strength than the younger shallower carbonates. A complicating factor in this generalization is the texture of the carbonate. It may be reasonable to assume that the strength of clastically formed carbonates may be more a function of the lithification process than the crystalline formed carbonates. However, other contributing factors may affect carbonate strength, such as the amount of noncarbonate materials, mircostructure defects, pore structure, fossil content, or depositional environment. Thin sections of the carbonates are presented in Appendix A in Figures A.3 through A.9.

The limestones aggregates are composed primarily of calcium carbonate ($CaCo^3$) and have a fairly consistent specific gravity of 2.68. The three limestones studied were from the Presque Isle, Bay County and Port Inland quarries. It can be seen in Figures A.3 through A.5 that the texture of the three limestones vary from a larger crystalline texture in the Presque Isle to a smaller crystalline texture with fossils for the Bay County and Port Inland limestone. Note that the scale for the Presque Isle limestone is 500 μm while the other two thin sections are at a 250 μm scale, thus showning a fairly large crystalline texture size for the Presque Isle limestone.

The four dolomites tested were from the Cedarville, Denniston, Rockwood, and France Stone quarries. Dolomite is a carbonate composed primarily of calcium and magnesium, $CaMg(CO_3)^2$. Due to the presence of magnesium the specific gravity of dolomite is higher generally around 2.85. However, there is generally greater variability in specific gravity for dolomites than limestones due to the varying percent of magnesium. Thin sections of the dolomites can be seen in Figures A.6 through A.9 in Appendix 3A. The France Stone and Cedarville dolomites appear to have a clastic structure while the Dennison and Rockwood appear to have a crystalline texture. However, additional testing and analysis would have to be performed to verify these observationsconcerning whether the carboantes are crystalline or clastic. The grain size of the Cedarville dolomite is noteably larger than the France stone while the texture of the Dennison and Rockwood dolomites appear to be similar with the exception that there appears to be more larger size material, possibly silica grains in the Rockwood dolomite. It can also be seen that the crystalline texture of the two dolomites are larger than the crystalline structure of the limestone and that there appears to be less quartz grains in the limestone as opposed to the dolomite.

1.2.3 Igneous

The two igneous sources used in this research are Precambrian in age (over 600 millions old). Two mafic igneous rocks, both of Keweenawan age, were tested: a Nippissing Diabase from Bruce Mines, Ontario and a Portage Lake Volcanic basalt from Moyle Quarry, Houghton, Michigan. The Nippissing diabase is also known as a traprock since it was formed at depth in a trap environment. The trap environment allowed a slower cooling of the rock and thus a longer time for crystals to form during the cooling process. The Portage Lake volcanic basalt formational environment was that of a flood basalt, i.e., the basalt flowed onto the surface of the earth cooling in place at atmospheric conditions. Due to this type of formation and quick cooling the basalt has a less crystalline structure than the diabase. In addition, due to the more rapid cooling, significant gas bubbles, which are known as amyadoles, are trapped in the basalt. In most cases the gas bubbles have been filled with quartz, epidote, or native copper. Figure A.10 and A.11 in Appendix A illustrate thin sections of the Moyle and Ontario Traprock aggregates, respectively. The crystalline structure of the igneous rocks can easily be seen in the thin sections in addition, to the larger crystal size of the diabase.

1.3 Aggregate Characterization

1.3.1 Density

Measurement of the aggregate's apparent and bulk density was conducted using an automated Micromeritic's 1330 Accupyc helium pycnometer, which is shown in Figure 1.4, and a Micromeritic's 1360 Geopyc envelope density analyzer, which is shown in Figure 1.5. From data collected with these instruments the effective porosity, absorption, and bulk density SSD were then estimated. The automated testing methods are described in a paper by Vitton, Lehman and Van Dam, (1998), which was published in an ASTM special technical publication. The paper describes the use of the instruments and the calculation of the density parameters and density measurements for three Michigan aggregates.

In this research the apparent specific gravity, G_{ab}, and the bulk specific gravity, G_B, were measured while the bulk specific gravity, $G_{b,SSD}$ and the porosity, n, were calculated. The apparent specific gravity, G_{ab}, is equivalent to the specific gravity of soils, G_S, since both methods measure only the solid portion and not the volume of voids when calculating the volume of the material. The bulk specific gravity, G_B, however, includes the volume of voids when estimating the volume of the aggregate. This volume is sometimes referred to as the envelope volume. The difference between the two bulk specific gravity definitions is that the bulk specific gravity measurement is conducted when the voids are dry (oven-dried) and the bulk specific gravity (SSD) is conducted when the voids are water saturated but have dry surfaces, thus the designation saturated-surface dry or SSD. The AASHTO procedures to determine specific gravity depend on the ability of water to absorb into the soil and aggregate saturating the material. Current AASHTO test methods require that the soil or aggregate be saturated for 24 hours to satisfy most of the absorption potential of the material. However, it is recognized that for some materials not all of the effective pore space may be saturated after 24 hours. Helium gas, on the other hand, can more easily and effectively absorb into a material's effective pore space. This, coupled with recent advances in instrumentation and sensors, has led to the development of a helium pycnometer. This device uses the ideal gas law to determine the volume of a material based on pressure measurements of helium gas. By knowing the dry mass of a soil or aggregate its specific gravity, G_S, can be determined.

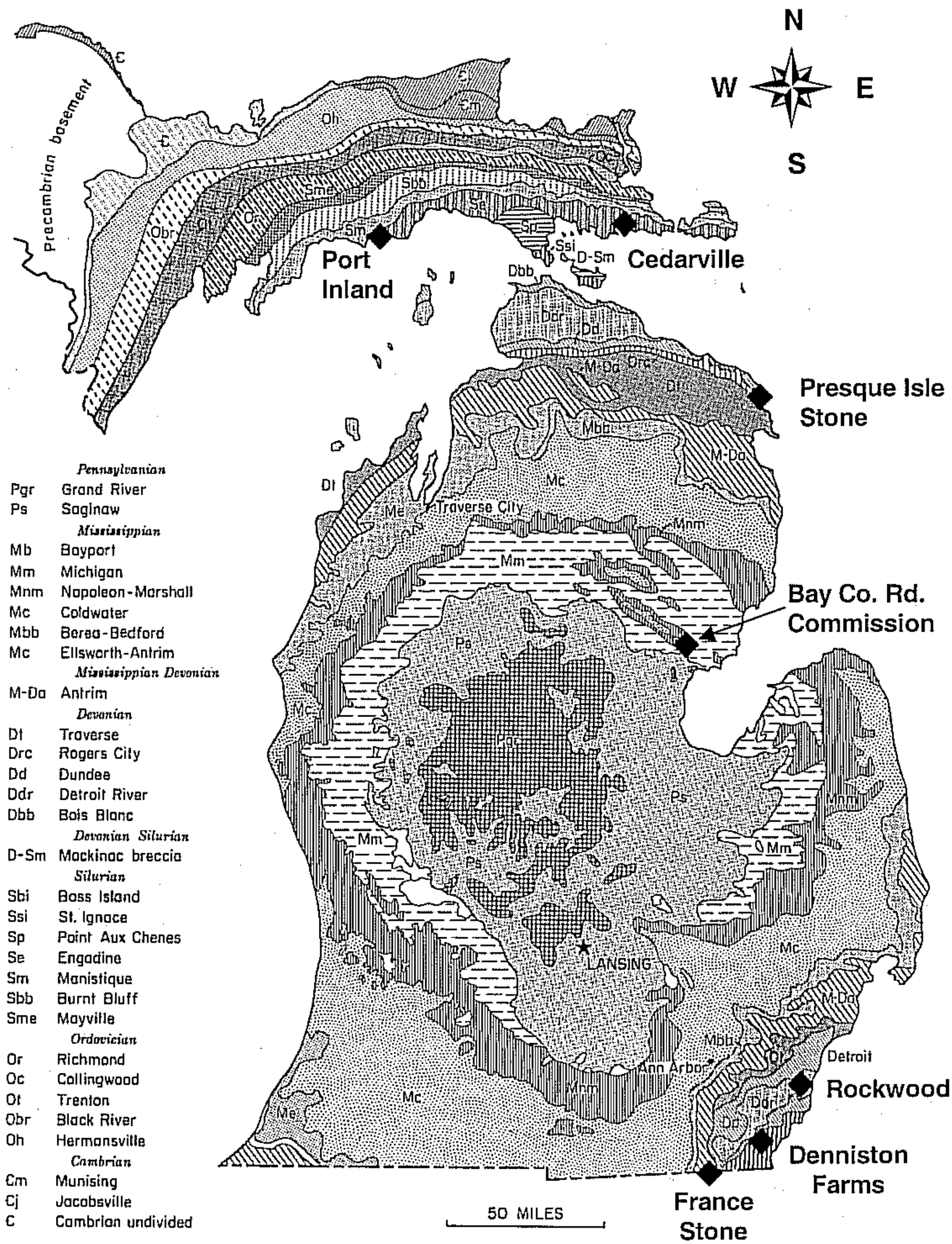

Figure 1.2 Geology of the Michigan Basin (Dorr and Eschman, 1970).

Era	Geologic Period		Formation	Geologic Time Million Years
Cenozoic			Sand and Gravel Deposits	63
Mesozoic				230
Paleozoic	Permian			280
Paleozoic	Carbon-iferous	Pennsylvanian		310
Paleozoic	Carbon-iferous	Mississippian	Bayport Limestone (Bay Co.)	345
Paleozoic	Devonian		Lucas Dolomite (Denniston) Lucas Dolomite (Rockwood) Rogers City Limestone (Presque Isle)	405
Paleozoic	Silurian		Raisin River Dolomite (France Stone) Engadine Dolomite (Cedarville) Fiborn Limestone (Port Inland)	425
Paleozoic	Ordovician			500
Paleozoic	Cambrian			600
Precambrian	Proterozoic	Keweenawan	Portage Lake Volcanics (Moyle) Nippissin Diabase (Ontario Traprock)	1,600
Precambrian	Proterozoic	Huronian		

Figure 1.3 Geologic formation and age of aggregates.

Figure 1.4 Micromeritics 1330 helium pycnometer.

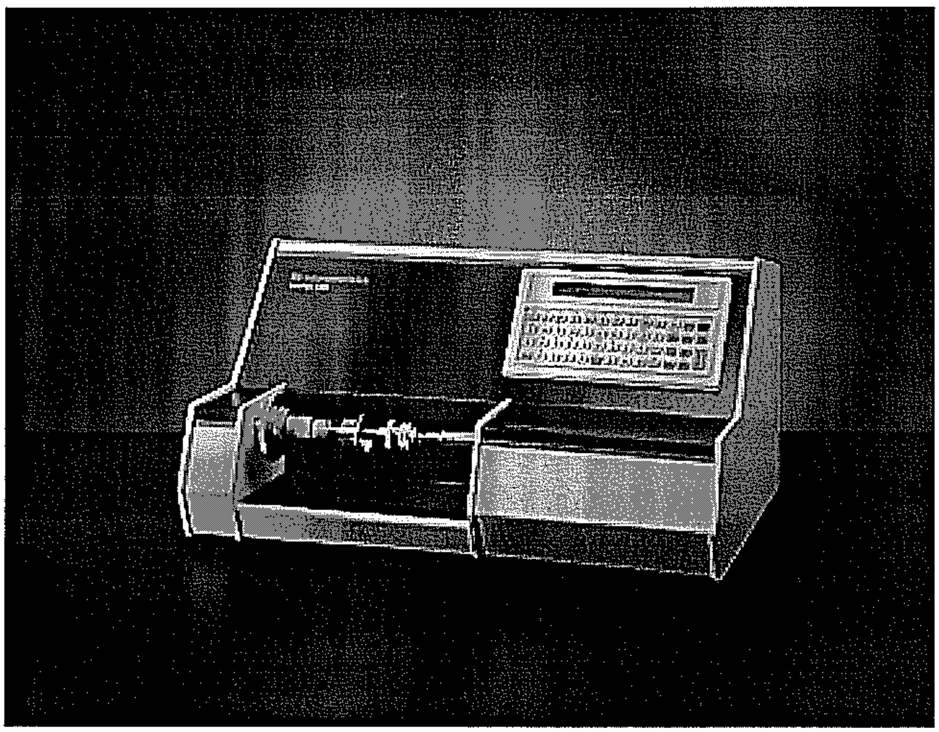

Figure 1.5 Micromertic's 1360 bulk density analyzer.

The bulk specific gravity of a material, G_B, can be determined using the Micromeritic's 1360 automated envelope density analyzer. This device determines the bulk volume or envelope volume of a sample by measuring the volume of a fine-grained material in a cylinder and then again measuring the volume of the fine-grained material plus the sample. By determining the difference in volume between the two measurements, the bulk volume of the sample can be calculated and the bulk specific gravity determined. If the apparent specific gravity, G_S, and bulk specific gravity, G_B, of the soil or aggregate is known, the effective porosity of a material, n, can also be determined by the following relationship:

$$n = \left[1 - \frac{G_B}{G_S}\right] x \;\; 100 \qquad \textbf{1.1}$$

Assuming that the bulk specific gravity, G_B, measurement is accurate, then the measured effective porosity, n, will be a function of the effectiveness of the fluid or gas used to permeate into the pore structure of the aggregate. When saturating the aggregate with water, the increase in the weight of the aggregate due to water in the pores, but not including water adhering to the outside surface, is referred to as the aggregate's absorption, A. While the ASTM standard is based on 24-hour saturation, it is possible that not all of the effective porosity, n, will be filled with water. However, if it is assumed that the aggregate is completely saturated (100% of the effective pores are filled with water), the aggregate's maximum absorption can be determined as a function of the bulk specific gravity and the effective porosity as follows:

$$A = \frac{n}{G_B} \qquad \textbf{1.2}$$

By knowing the effective porosity of a material an estimate of the bulk specific gravity (SSD), $G_{B,SSD}$, can also be made assuming that the material's effective pore structure is completely filled with water (specific gravity = G_{water} = G_w). This relationship is as follows:

$$G_{B,SSD} = G_B + nG_w \qquad \textbf{1.3}$$

The results of the density testing conducted in this research are presented in Table 1.2.

Table 1.2 Aggregate Type and Specific Gravity

No.	Source (MDOT ID)	Material Type	Orientation to Bedding	G_{ab}	G_B	$G_{B,SSD}$	Porosity (%)
1.0	Algoma Steel	Air-Cooled Blast	Porous Region	2.973	2.09	2.41	30
1.2	(95-006)	Furnace Slag	Dense Region	2.888	2.40	2.57	17
2	Algoma Steel (95-006)	Water Quenched Blast Furnace Slag	Random	2.942	2.43	2.61	17
3	Levy Co. (82-019)	Water Quenched Blast Furnace Slag	Random	2.985	2.42	2.61	19
4	Presque Isle Stone (71-047)	Limestone	Random	2.687	2.51	2.58	6
5	Bay County (06-008)	Limestone	Perpendicular	2.697	2.63	2.68	2
6	Port Inland (75-005)	Limestone	Random	2.69	2.68	2.68	<1
7	Cedarville (49-065)	Dolomite	Random	2.770	2.71	2.75	2
8	Denniston (58-009)	Dolomite	Perpendicular	2.828	2.48	2.65	12
9	Rockwood (58-008)	Dolomite	Parallel	2.836	2.49	2.63	12
			Perpendicular	2.834	2.60	2.70	8
10	France Stone (Hansen) (93-003)	Dolomite	Perpendicular	2.818	2.78	2.82	1
11	Moyle (31-076)	Flood Basalt	Random	2.938	2.89	2.91	1.6
12	Ontario Traprock (95-010)	Diabase	Random	2.931	2.91	2.92	<1

1.3.2 Drill Quality Index Test (DQI)

The initial dynamic and static strength testing for this research required that the aggregate be cored using a 3/8 inch diamond core bit. A six-inch direct drive drill was obtained to core the block aggregate samples. In addition, a precision cut-off saw was used to cut the core to length as well as produce parallel ends. During coring, the core bit was drilled approximately six inches into the aggregate block. With some aggregate types that are highly intact most, if not all of a six-inch core were obtained. However, it was found that some aggregate types tended to break up into a number of pieces while being cored. To quantify this core breakage a drill quality index (DQI) was developed based on a rock quality designation (RQD) used in the mining industry (Deere and Deere, 1988). This DQI index is based on the following formula

$$DCI = \frac{\frac{L_C}{L_D} + \frac{D}{3}}{N \times D} \qquad \mathbf{1.4}$$

where:
L_C = Length of core obtained from the core barrel
L_D = Length of block cored
D = Diameter of core
N = Number of pieces core broke into

A numerical rating system for the DQI was then developed and is presented in Table 1.2.

Table 1.2 Drill Quality Index (DQI) rating system

DQI	Rating	Designation
1.5 or higher	Highly Intact	1
1.00 - 1.50	Intact	2
0.70 – 1.00	Moderately Intact	3
0.70 – 0.44	Moderately Broken	4
0.44 – 0.33	Broken	5
0.33 – 0	Highly Broken	6

The DQI is highly dependent on both the microstructure of the aggregate as well as the macrostructure. The macrostructure, such as large fractures and fissures, will result in breakage of the core. Consequently, the DQI will include both effects, with highly fractured aggregates having high DQI values, while relatively intact aggregate blocks will have lower DQI numbers. Therefore, the DQI is a very relative measure of an aggregate intactness and will vary from block to block. A record was kept of all core lengths and their calculated DQI's. Table 1.3 provides the DQI values. Representative samples of the drilled aggregate core are provided in Appendix 3B.

Table 1.3 Drill Quality Index (DQI) values.

Quarry	Pit ID	DQI Block 1, 2, 3	DQI Block Average	Average DQI Rating
1 Algoma Steel (air-cooled)	95-006	1[1], 4[2], 1[1]	2	Intact
2 Algoma (water-quenched)	95-006	4, 2	3	Moderately Intact
3 Levy Steel (water-quenched)	82-022	5, 4, NR[3], 4	5	Broken
4 Presque Isle Stone	71-047	1	1	Highly Intact
5 Bay County	06-008	4, 1, 4	3	Moderately Intact
6 Port Inland	75-005	2	2	Intact
7 Cedarville	49-065	1, 1, 1	1	Highly Intact
8 Denniston Farms	58-009	2, 4, 1, 1	2	Intact
9 Rockwood	58-008	1	1	Highly Intact
10 France Stone	93-003	1, 1, 3	2	Intact
11 Moyle	31-045	1	1	Highly Intact
12 Ontario Traprock	95-010	1	1	Highly Intact

[1] Dense dark region (specimen 1.2)

[2] Porous light colored region (specimen 1.0)

[3] NR: No core recovered, i.e., coring did not provide length long enough for testing. A value of 6 is given a NR core.

APPENDIX 3A

AGGREGATE THIN SECTIONS

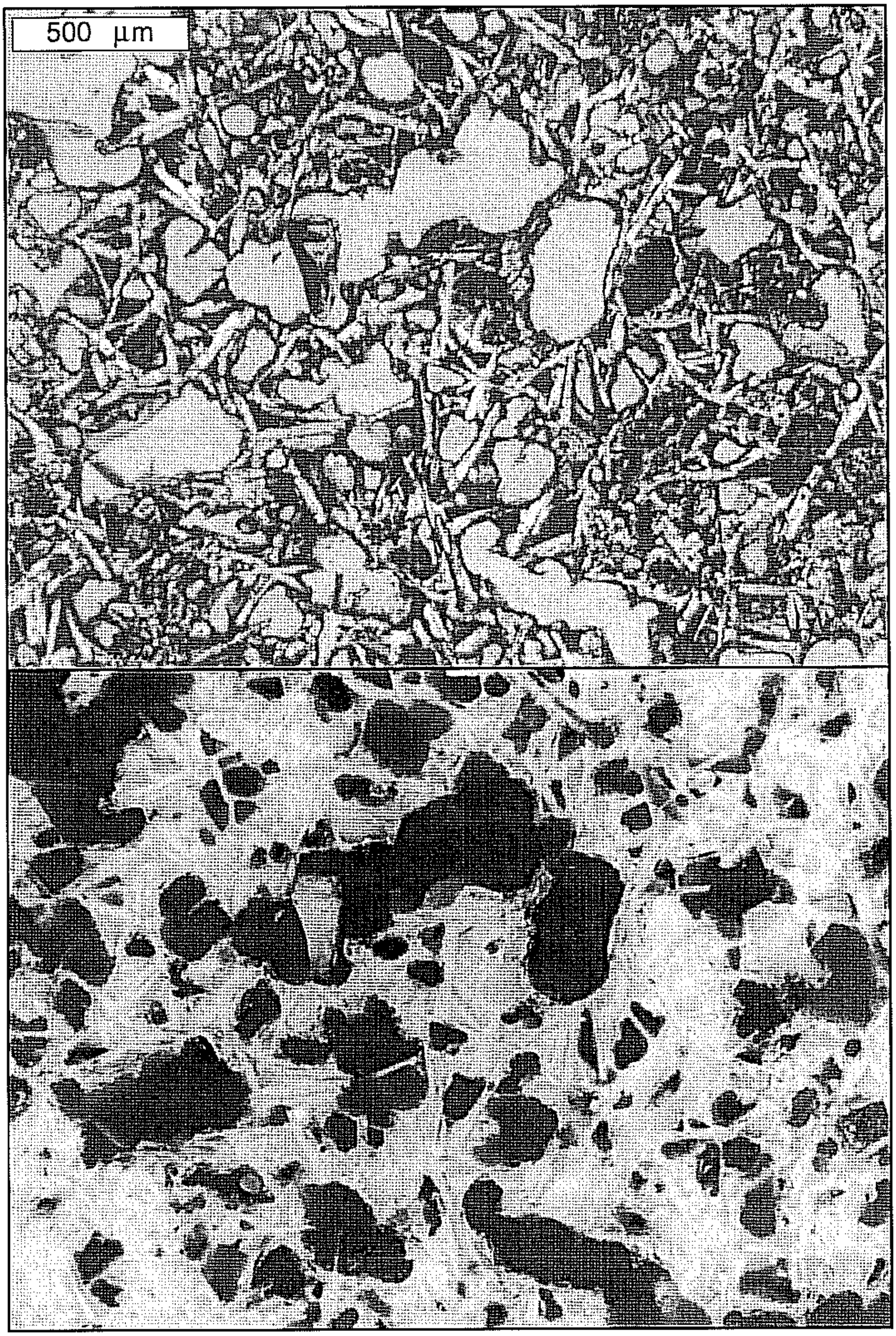

Figure A.1 Thin section of Algoma air-cooled slag (96-006).

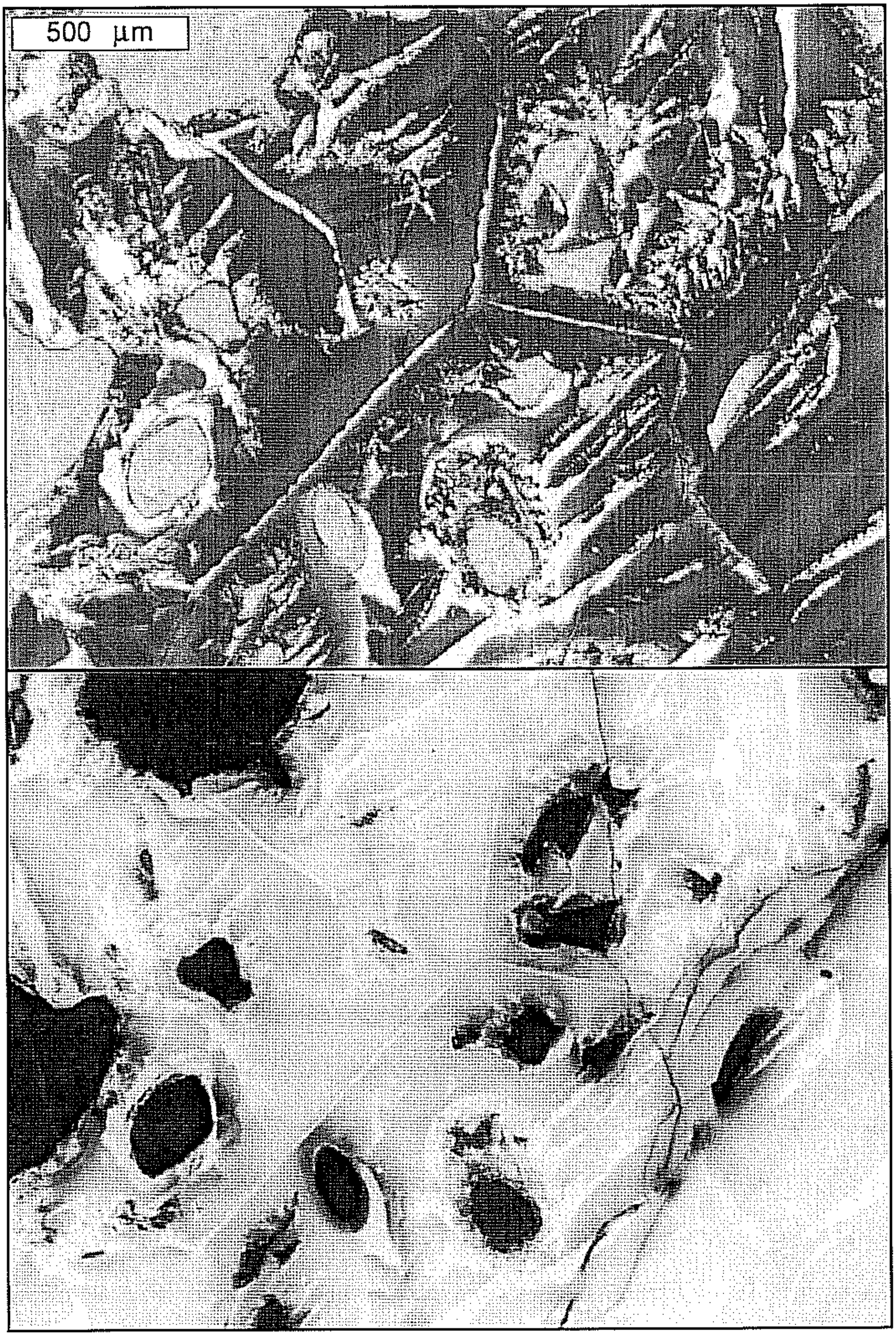

Figure A.2 Thin section of water-quenched Levy slag (82-019).

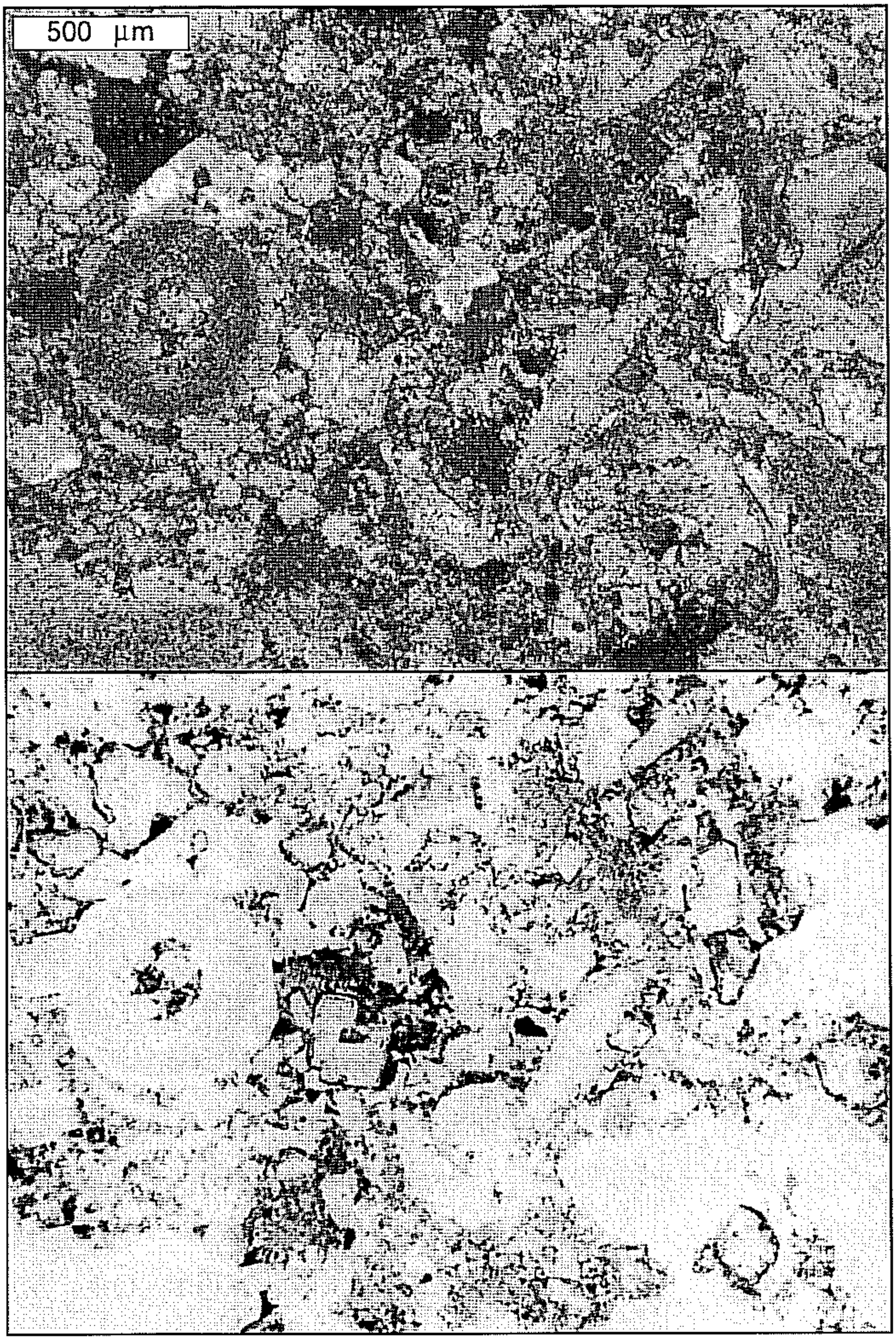

Figure A.3 Thin section of Presque Isle limestone (71-047).

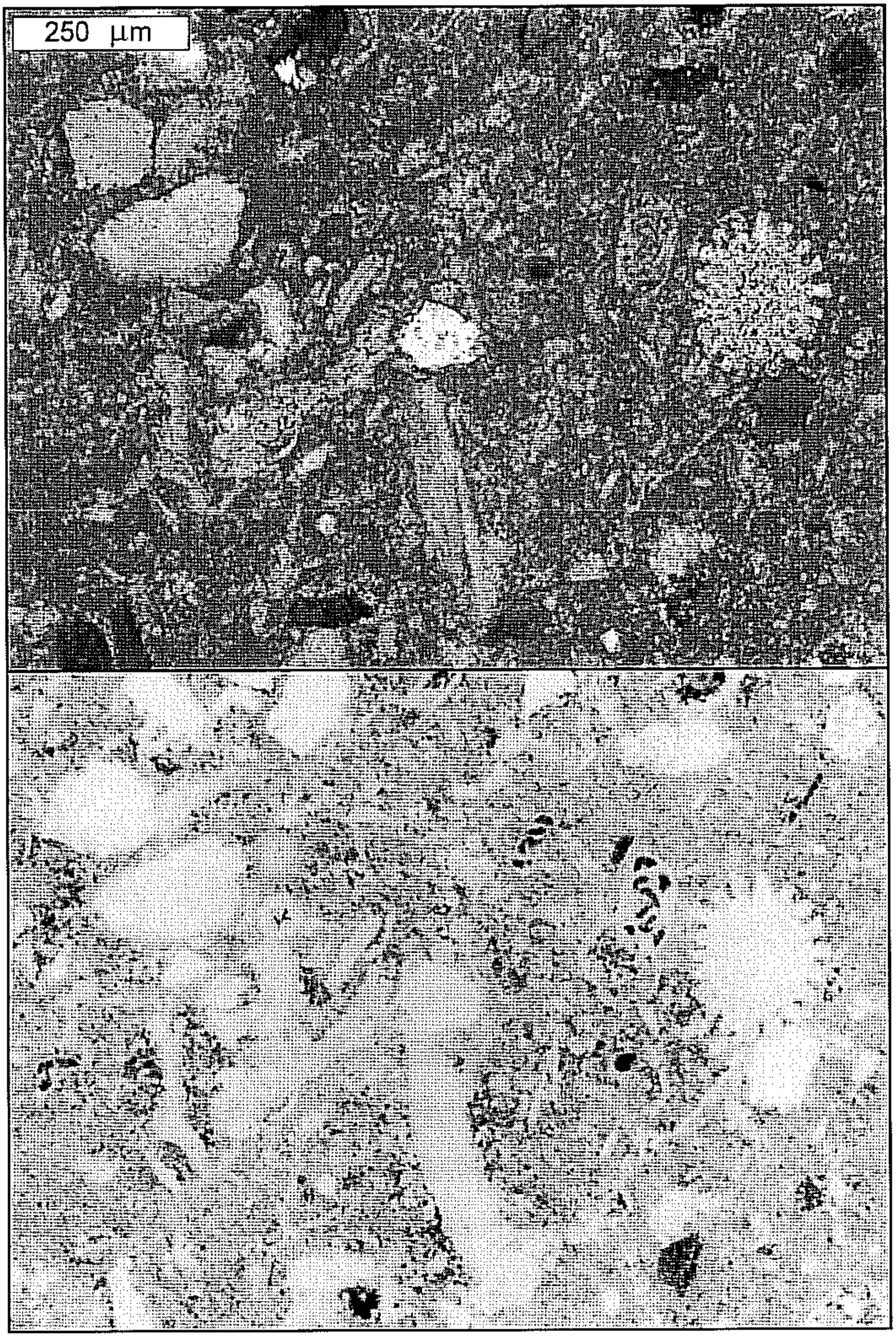

Figure A.4 Thin section of Bay County limestone (06-008).

Figure A.5 Thin section of Port Inland limestone (75-005).

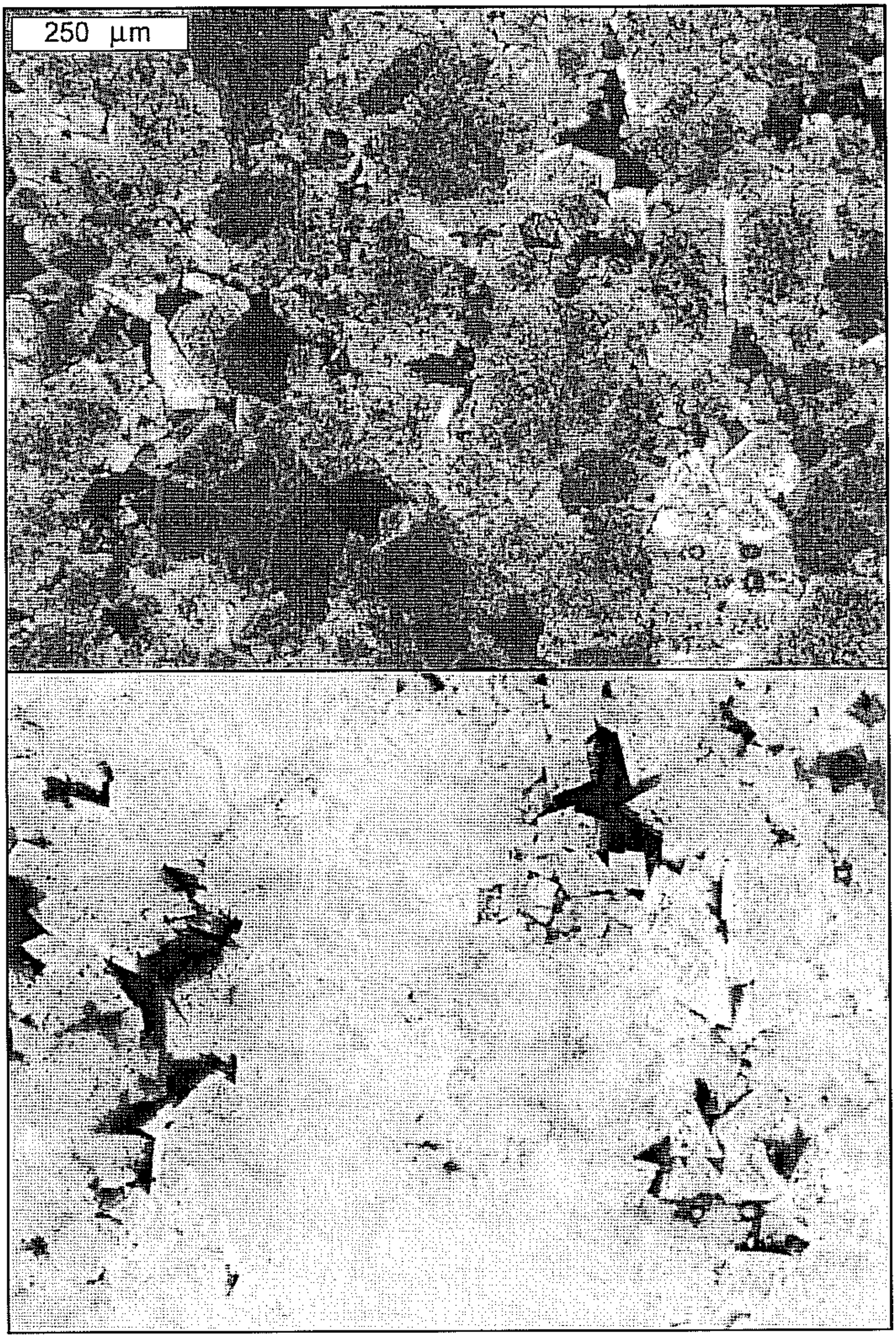

Figure A.6 Thin section of Cedarville dolomite (49-065).

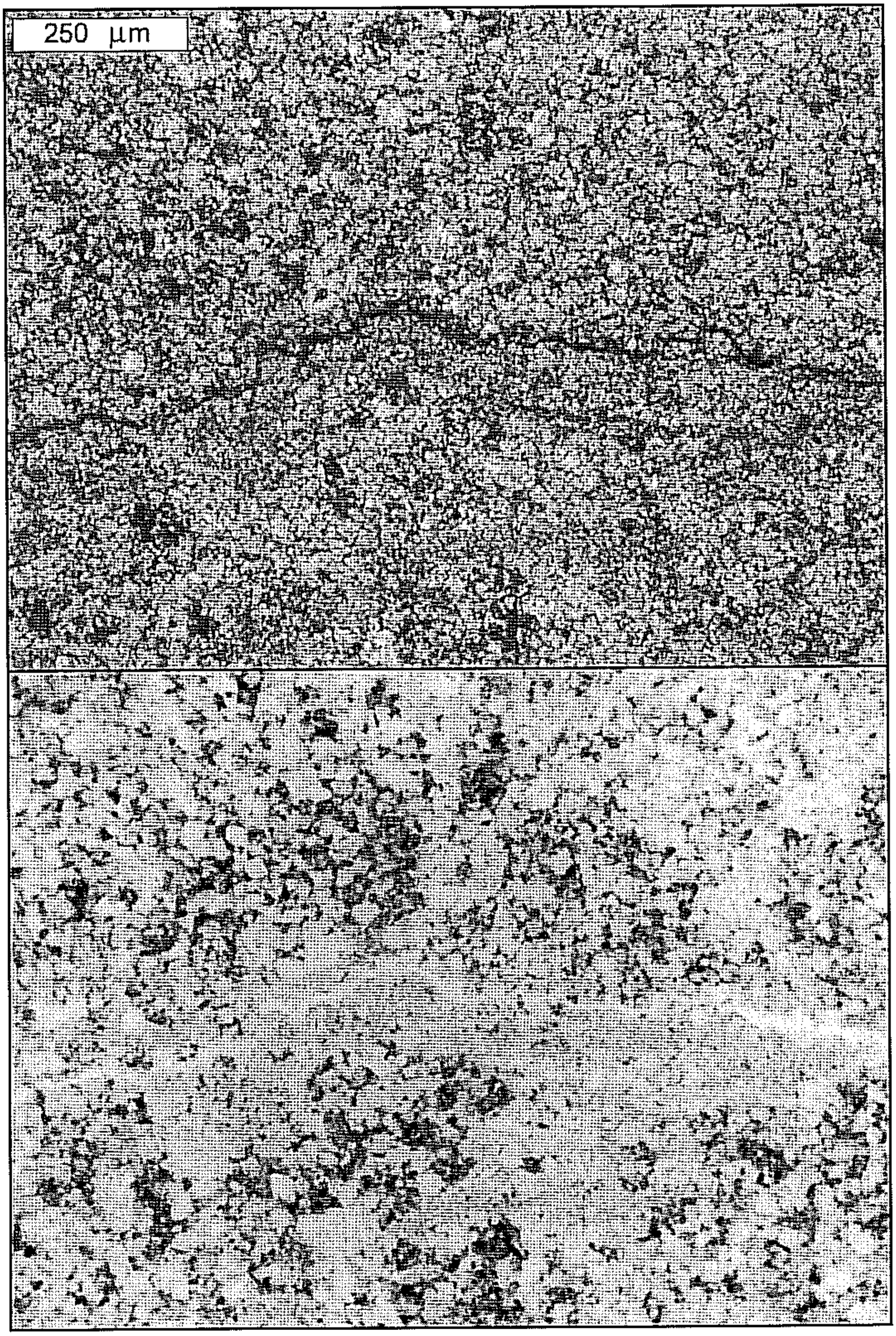

Figure A.7 Thin section of Dennison Farms dolomite (58-009).

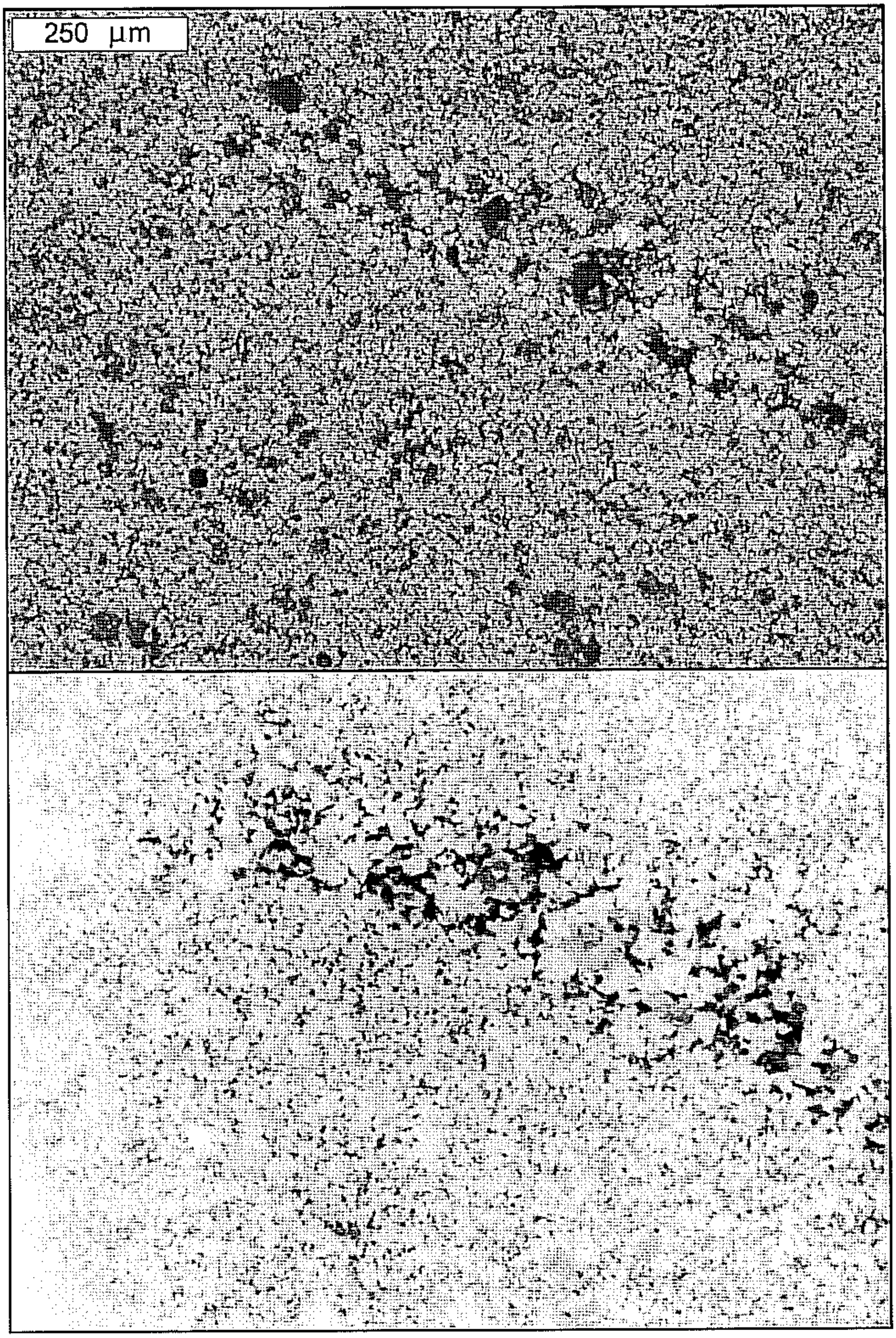

Figure A.8 Thin section of Rockwood dolomite (58-008).

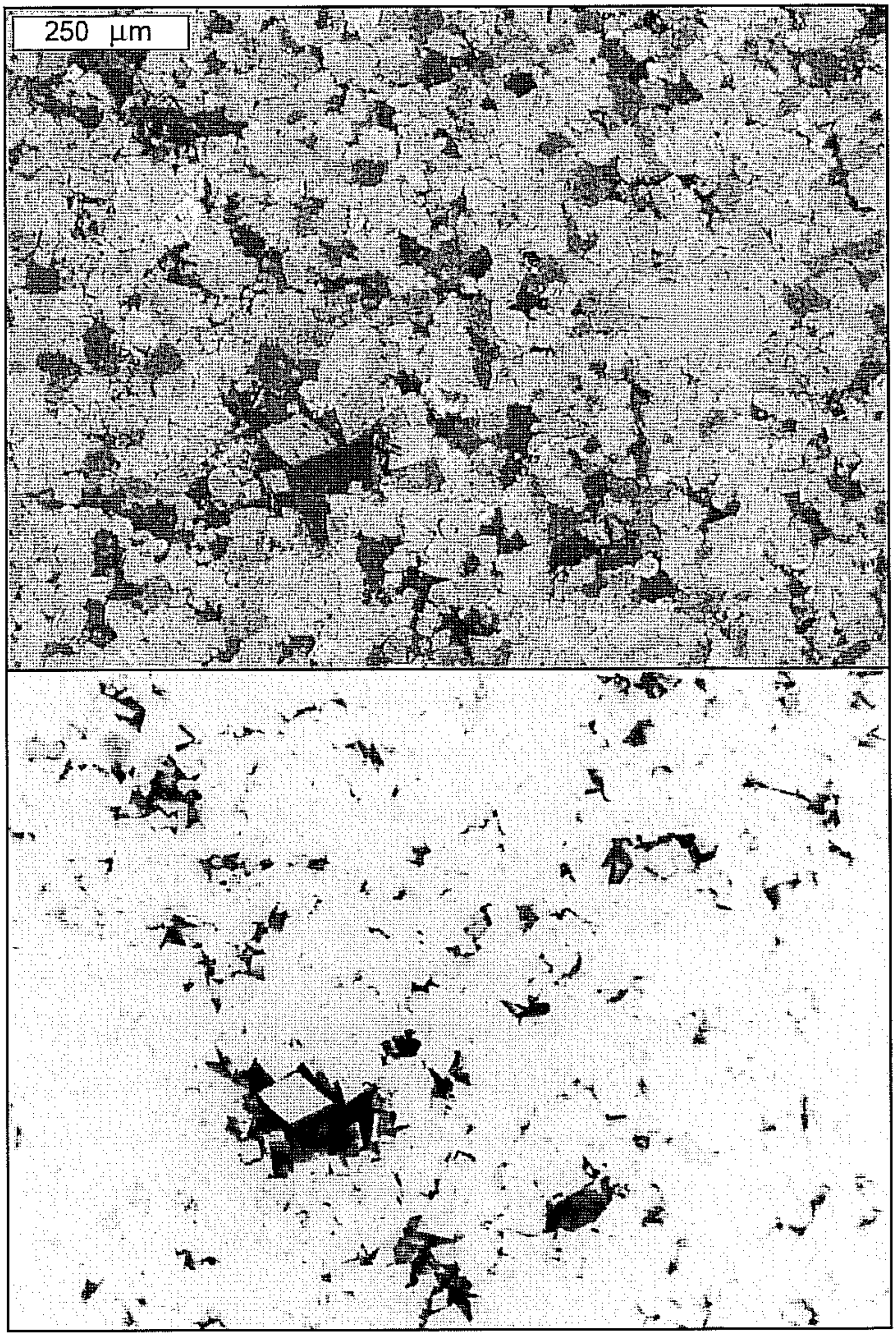

Figure A.9 Thin section of France Stone dolomite (93-003).

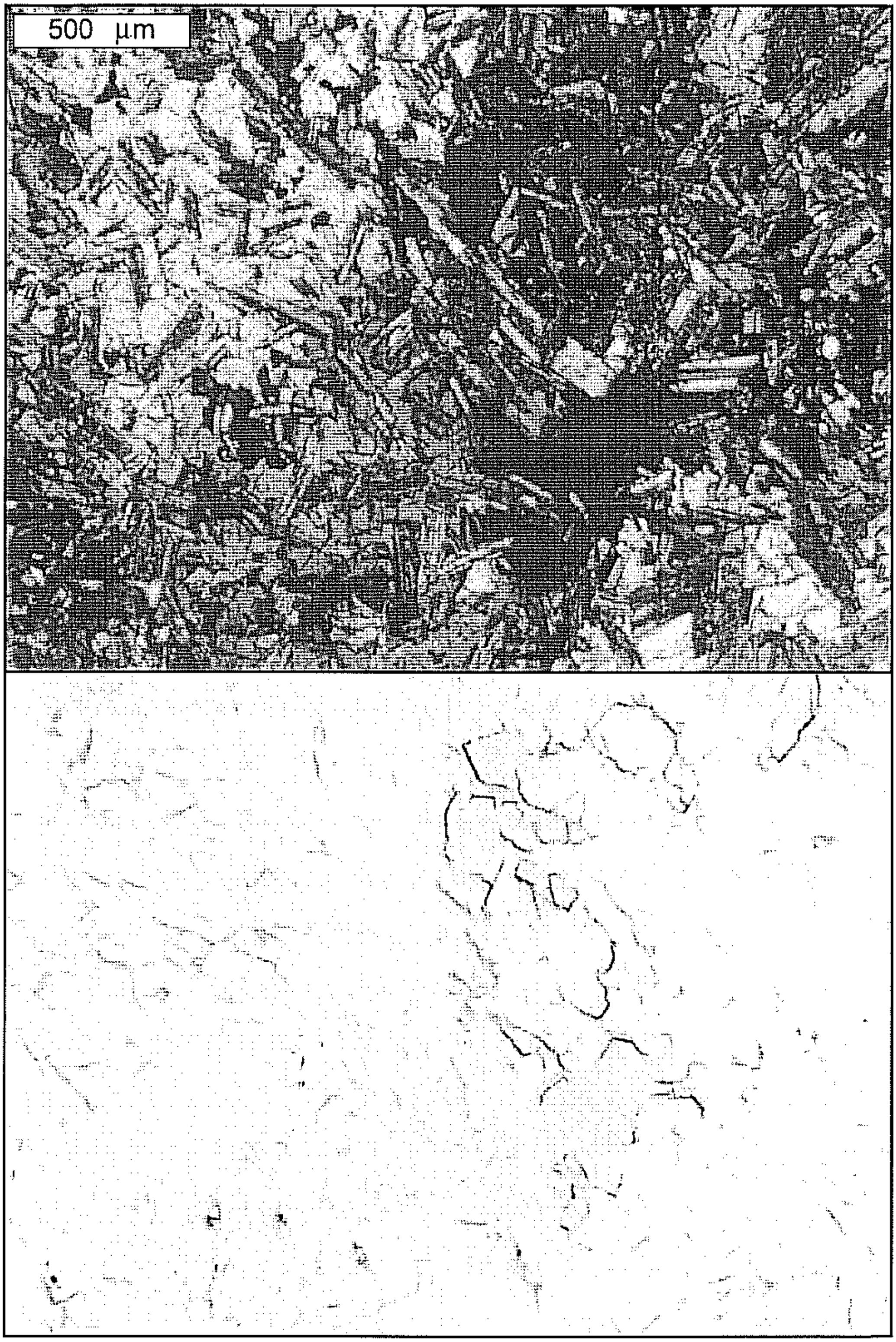

Figure A.10 Thin section of Moyle basalt (31-076).

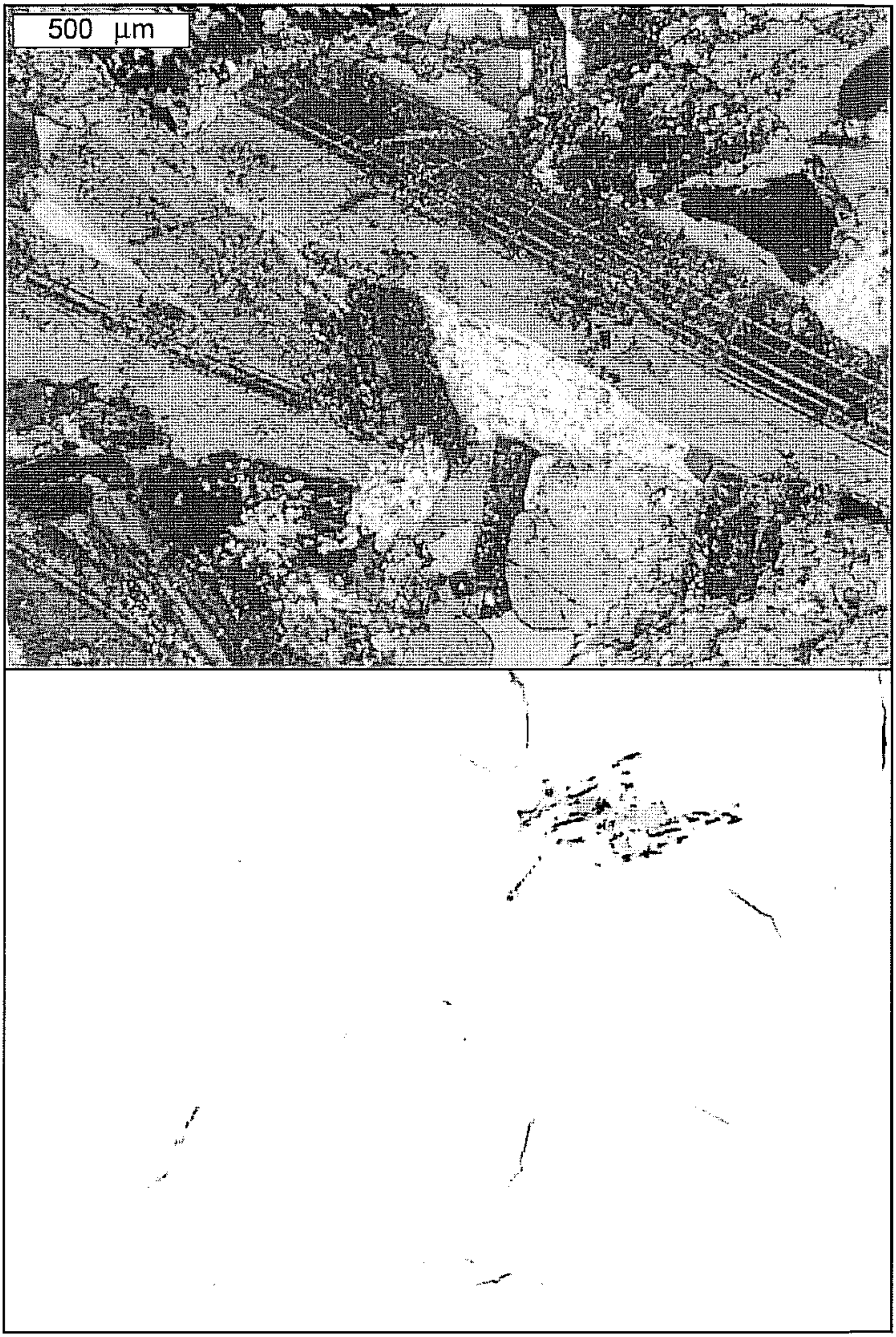

Figure A.11 Thin section of Ontario Trap Rock Diabase (95-010).

APPENDIX 3B

AGGREGATE DQI CORE

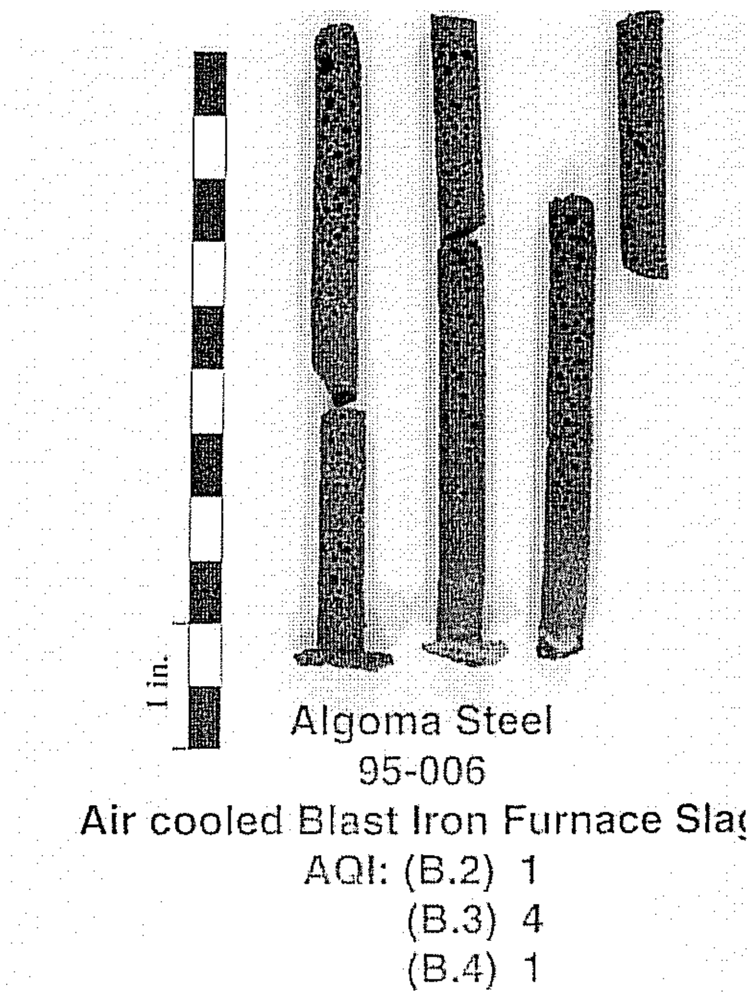

Figure B.1 Algoma air cooled Slag drill core.

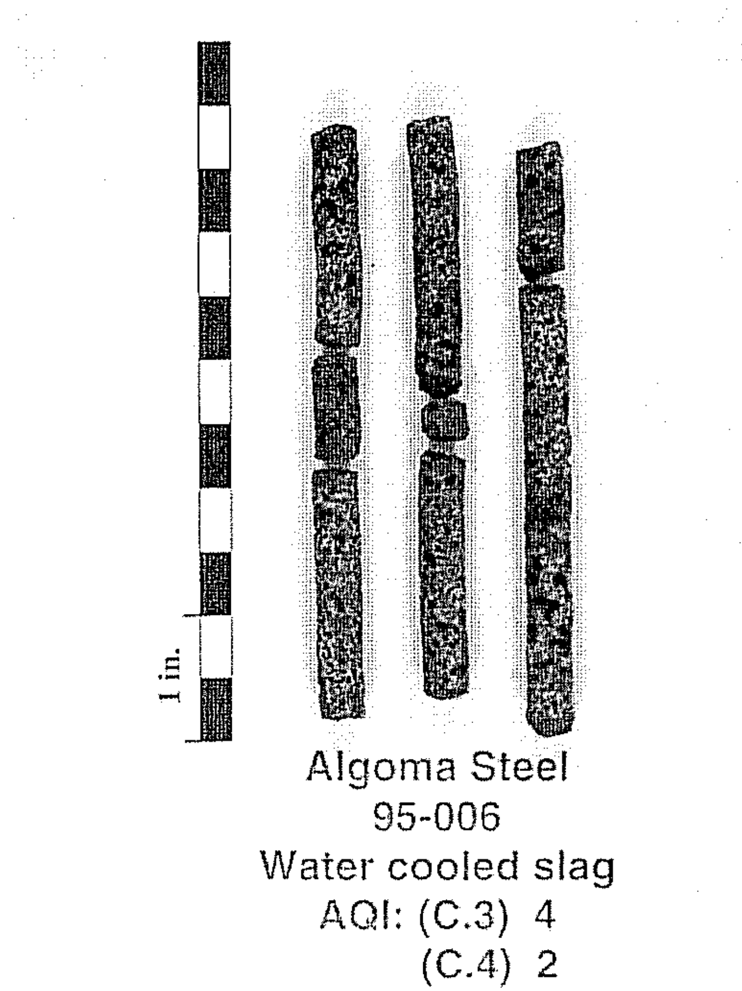

Figure B.2 Algoma slag water cooled drill core

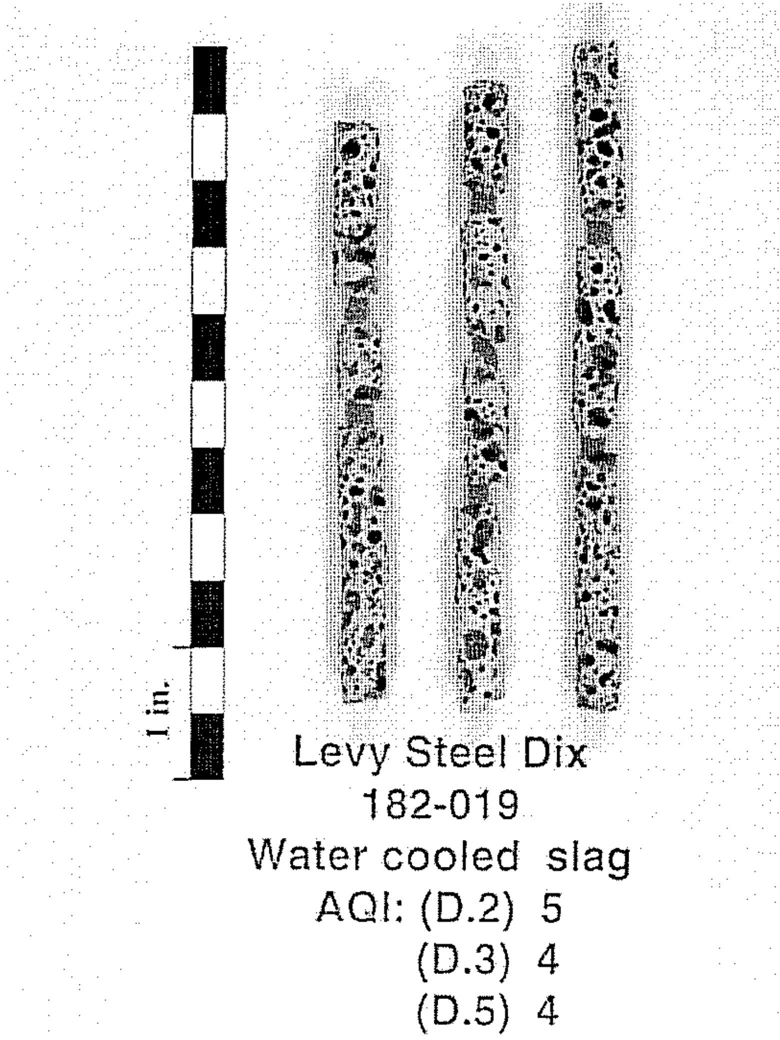

Figure B.3 Levy water cooled Slag drill core.

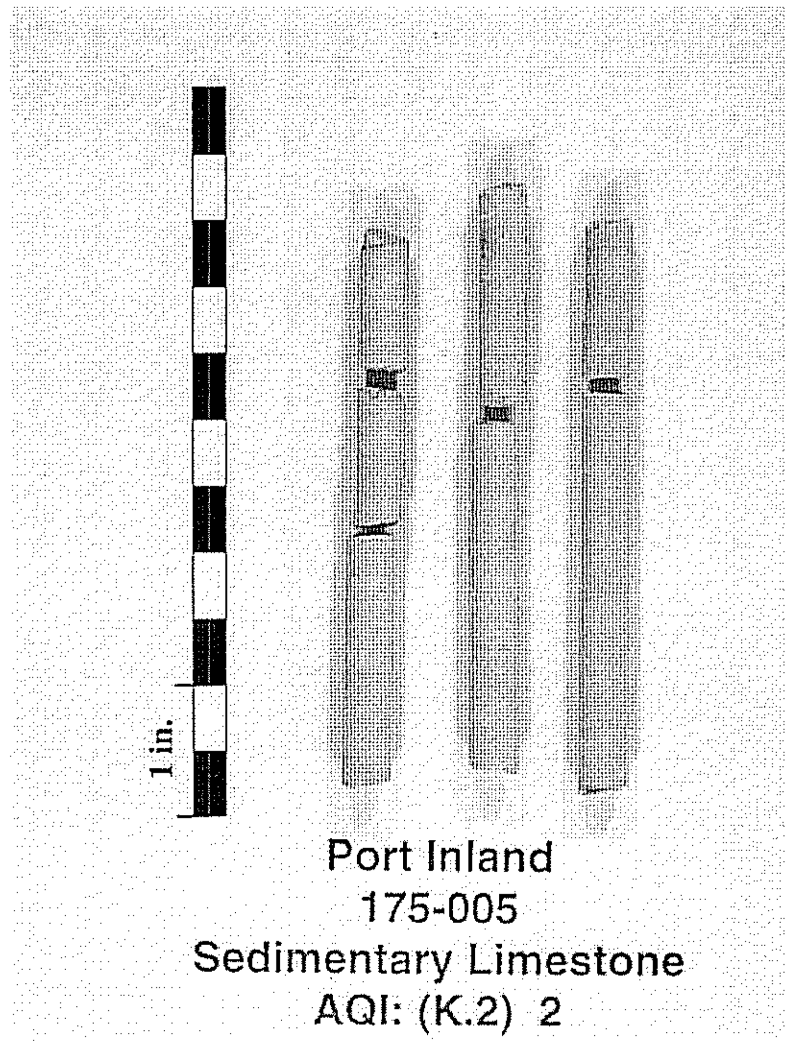

Figure B.4 Port Inland drill core.

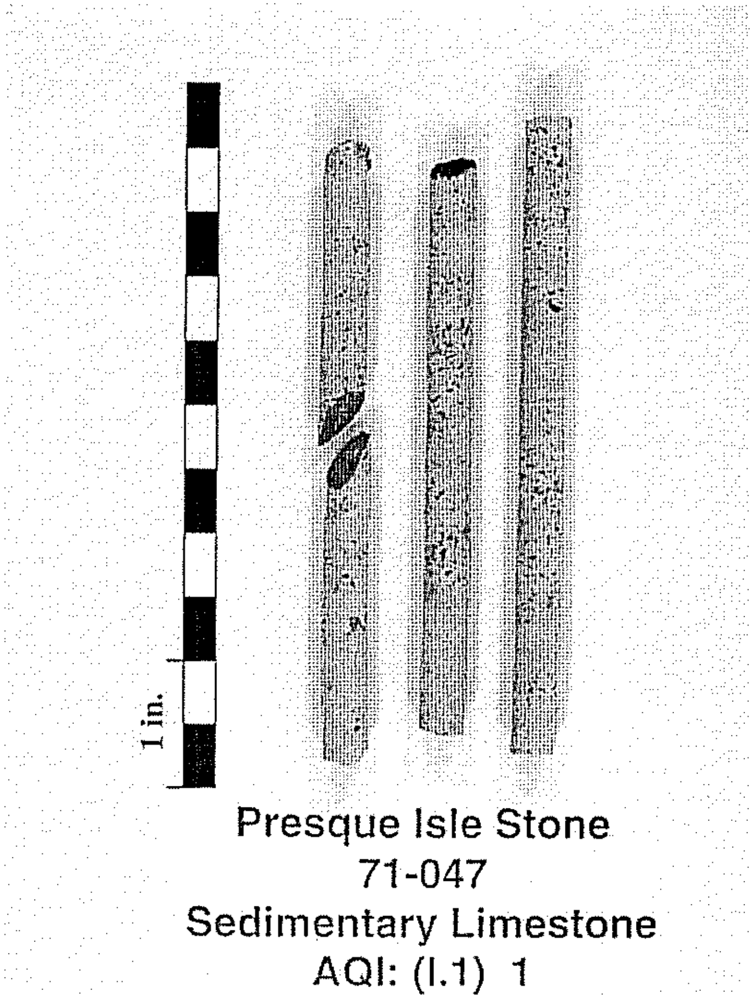

Figure B.5 Presque Isle drill core.

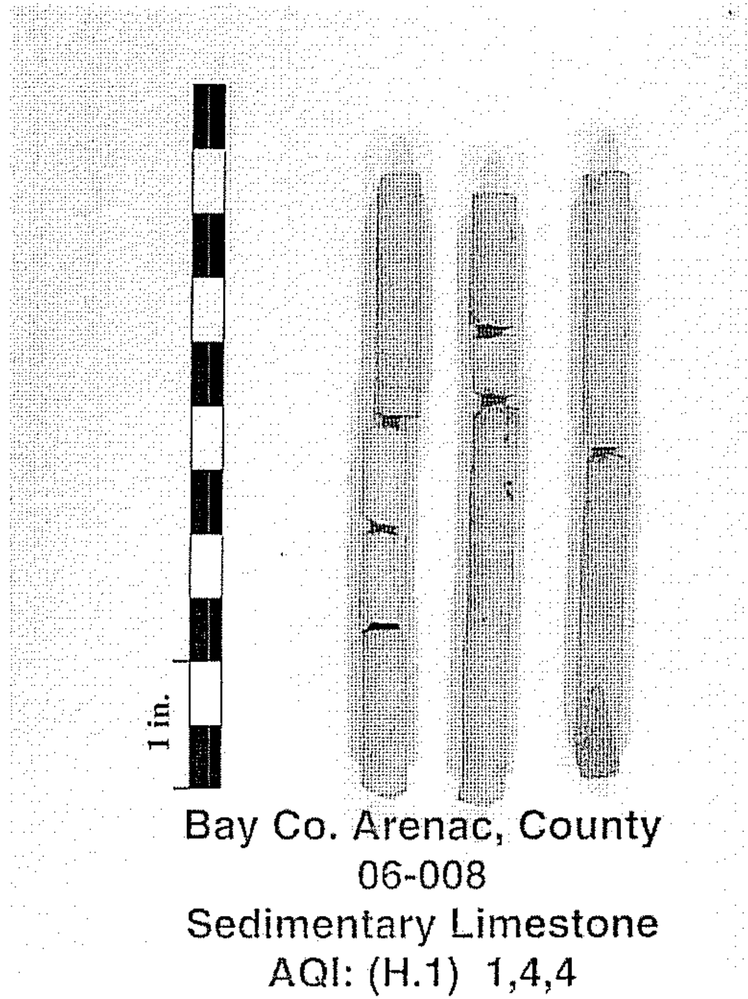

Figure B.6 Bay County drill core.

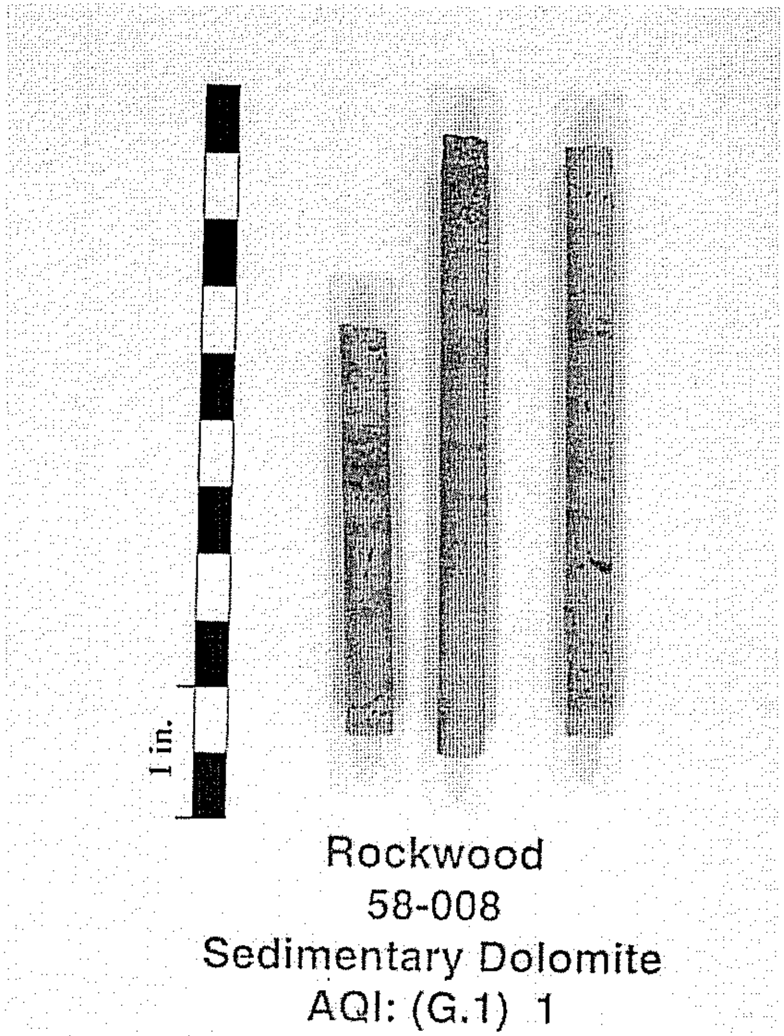

Figure B.7 Rockwood drill core.

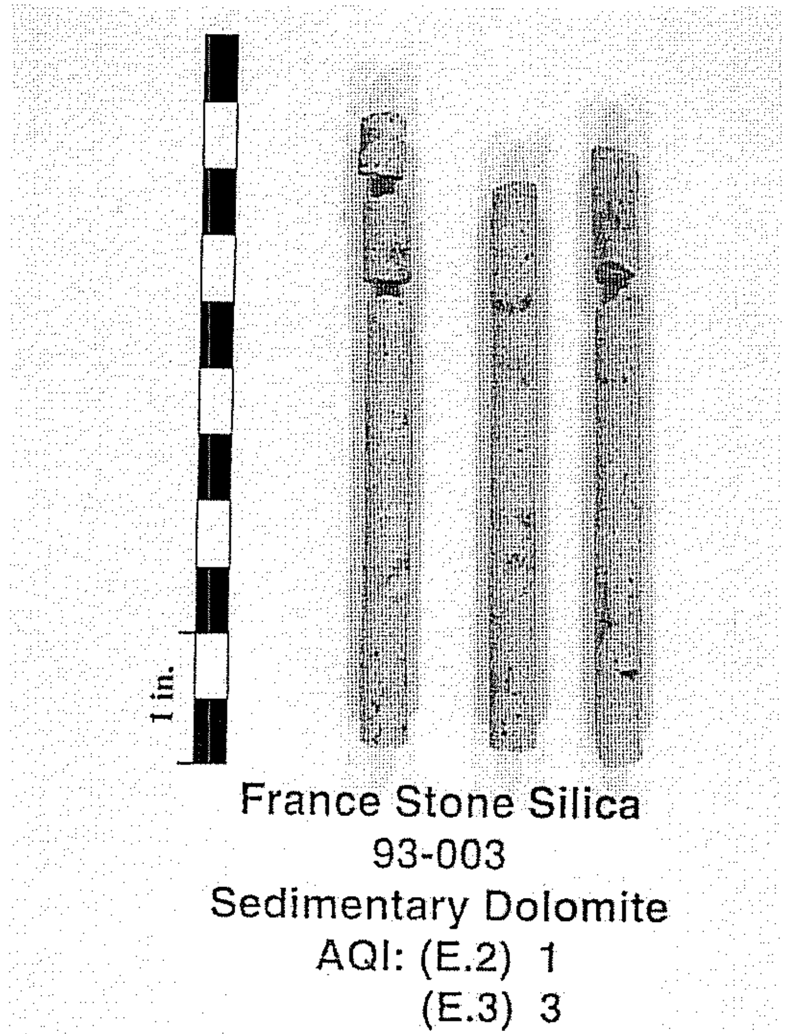

Figure B.8 France Stone drill core.

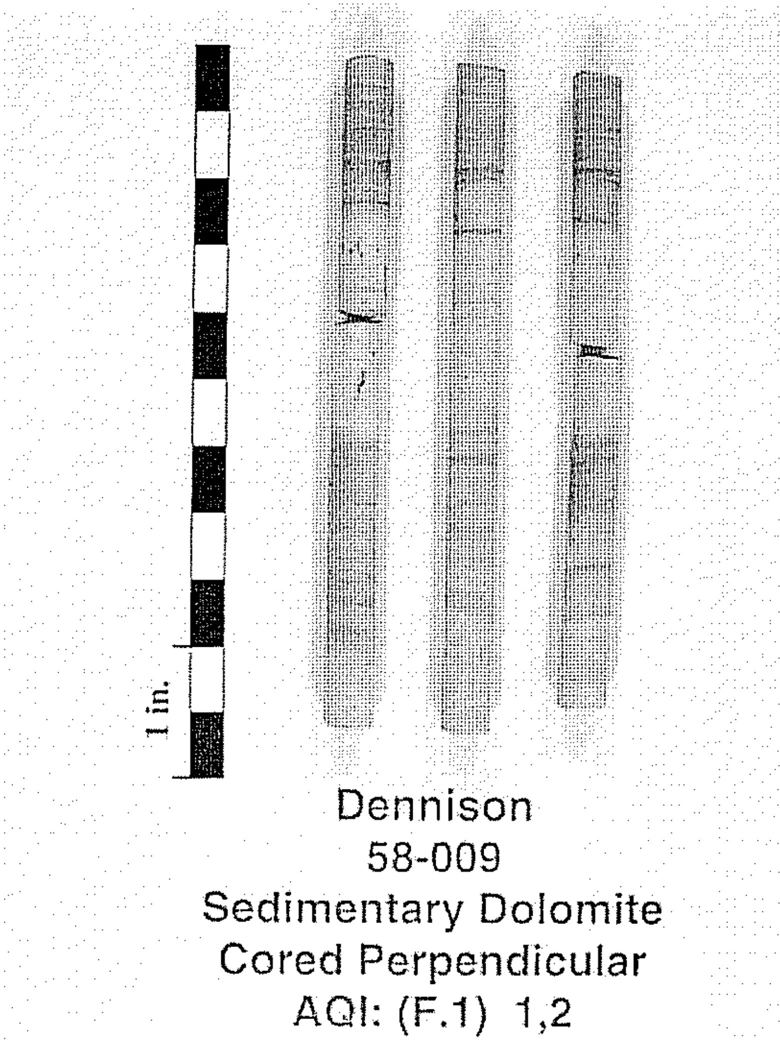

Figure B.9 Denniston Farms drill core.

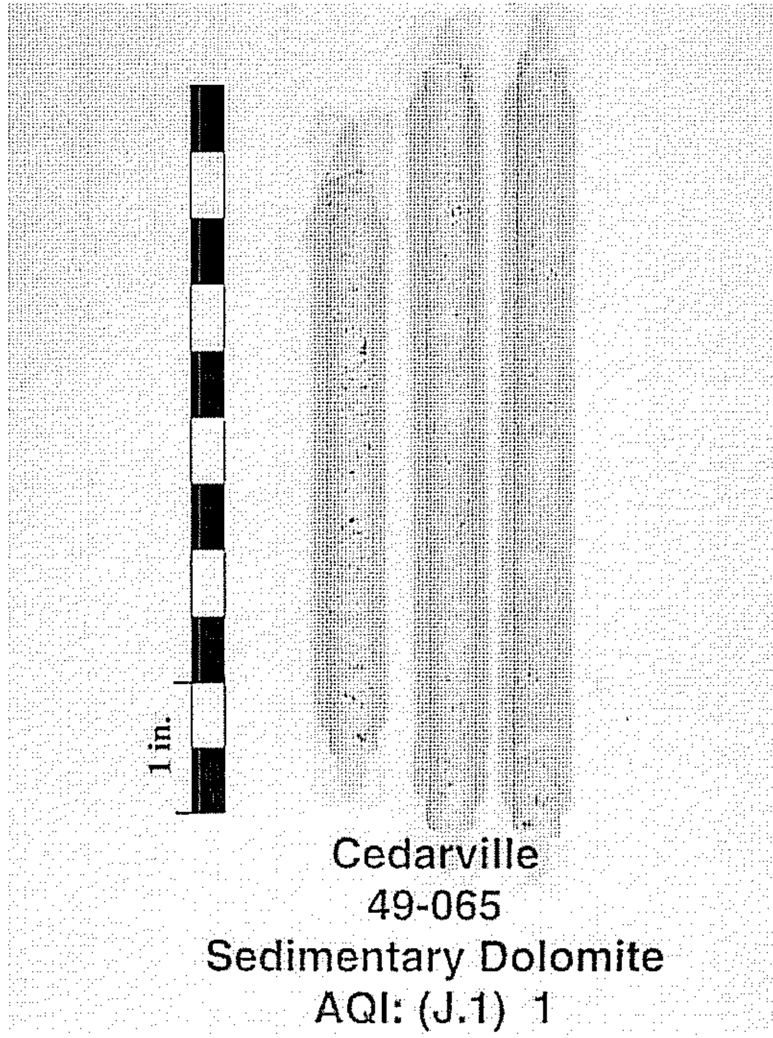

Figure B.10 Cedarville drill core.

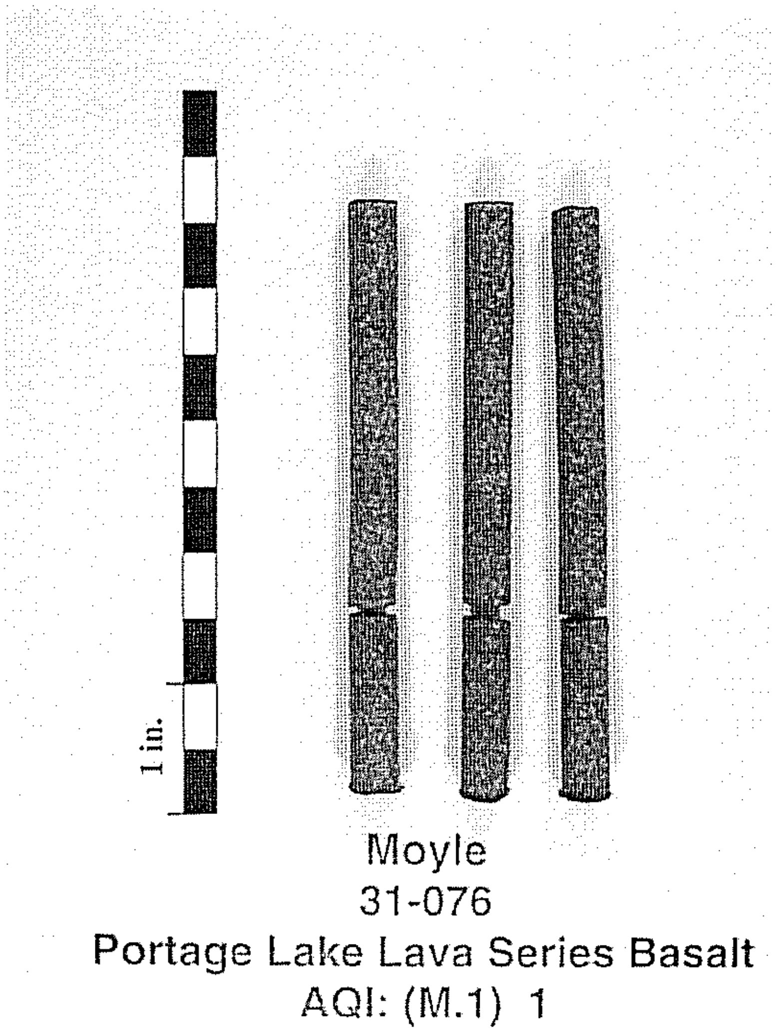

Figure B.11 Moyle drill core.

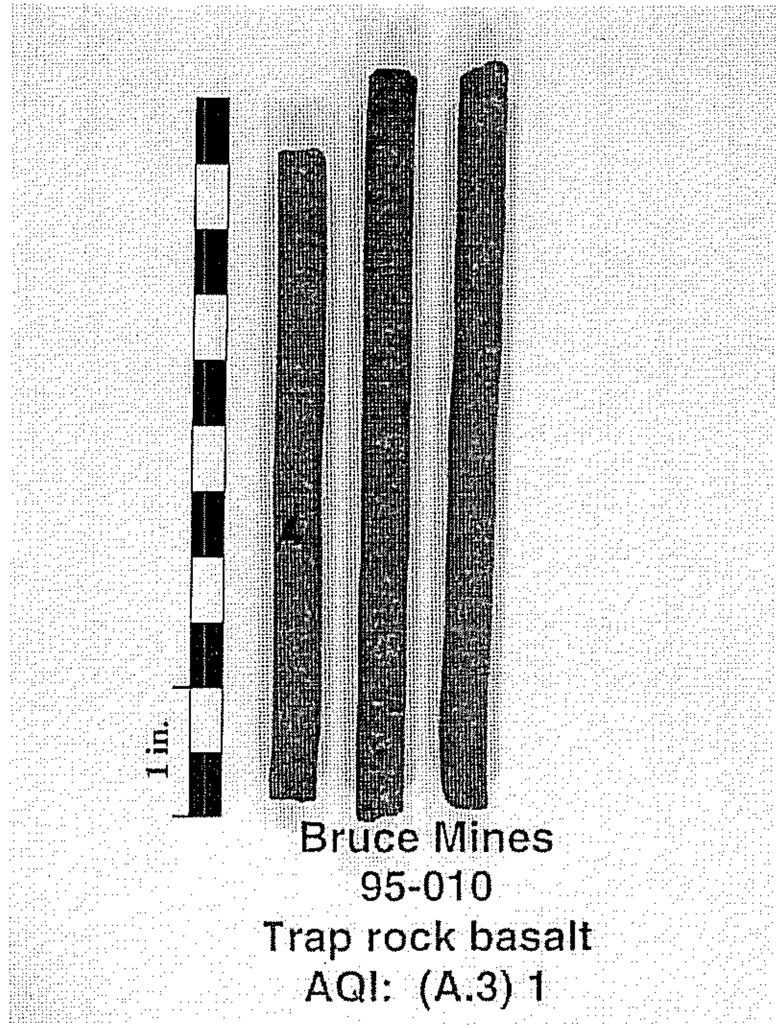

Figure B.12 Bruce Mines drill core.

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