Use of Air-Cooled Blast Furnace Slag as Coarse Aggregate in Concrete Pavements

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16. Abstract

This report presents available information regarding the use of air-cooled blast furnace slag (ACBFS) as coarse aggregate in concrete pavements. The report discusses ACBFS aggregate production and properties, and the properties of concrete produced with ACBFS coarse aggregate. Both the physical and chemical properties of ACBFS are presented, as are the properties of the concrete produced with ACBFS coarse aggregate. Additionally, the field performance of concrete pavements containing ACBFS coarse aggregate and observed material related distresses in these pavements are presented, along with results from laboratory evaluations of concrete containing ACBFS coarse aggregate. Finally, the life-cycle and maintenance costs associated with concrete pavements incorporating ACBFS aggregate in the concrete are also discussed in the report.

While the available information included both domestic and international experience with the use of ACBFS aggregate in concrete, the predominance of information reviewed came primarily from States which have, or previously have had, supplies of ACBFS aggregate: Ohio, Michigan, Indiana, and New York.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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ACRONYMS AND ABBREVIATIONS

ACBFS	air-cooled blast furnace slag
ALDOT	Alabama Department of Transportation
ASR	alkali–silica reactivity
CEN	European Committee for Standardization
CTE	coefficient of thermal expansion
DI	distress index
EDF	expected durability factor
FDR	full-depth repair
FHWA	Federal Highway Administration
FTD	freeze–thaw dilation
IDOT	Illinois Department of Transportation
INDOT	Indiana Department of Transportation
JPCP	jointed plain concrete pavement
JRCP	jointed reinforced concrete pavement
KYTC	Kentucky Transportation Cabinet
LTE	load-transfer efficiency
MEPDG	Mechanistic-Empirical Pavement Design Guide
Mn/DOT	Minnesota Department of Transportation
MDOT	Michigan Department of Transportation
MRD	materials-related distress
MSHL	Michigan State Highway Laboratory
NSA	National Slag Association
NYSDOT	New York State Department of Transportation
OECD	Organization for Economic Cooperation and Development
ODOT	Ohio Department of Transportation
PennDOT	Pennsylvania Department of Transportation
SSD	saturated surface dry
TSA	thaumasite sulfate attack
VDOT	Virginia Department of Transportation
VSA	vacuum-saturated absorption
VSSG	vacuum saturated specific gravity
w/cm	water-to-cementitious materials ratio
WisDOT	Wisconsin Department of Transportation

CHAPTER 1. INTRODUCTION

AIR-COOLED BLAST FURNACE SLAG

Blast furnace slag is a nonmetallic material consisting of silicates and aluminosilicates of calcium and magnesium together with other compounds of sulfur, iron, manganese, and other trace elements. It is produced from a molten state simultaneously with pig iron in a blast furnace. The solidified product is further classified according to the process by which it was brought from the molten state. Air-cooled blast furnace slag (ACBFS) is produced through relatively slow solidification of molten blast furnace slag under atmospheric conditions, resulting in crystalline mineral formation. ACBFS is one of the most commonly utilized reclaimed construction materials, being used as coarse aggregate in portland cement concrete (PCC) (also referred to simply as concrete in this report), aggregate in hot-mix asphalt, road base material, and fill. According to the U.S. Geological Survey (USGS) *2009 Minerals Yearbook* (van Oss 2009), approximately 5.071 million tons (4.6 million metric tons) of ACBFS were used in the United States in 2009, having a value of about $33 million. This number is down from 7.606 million tons (6.9 million metric tons) of ACBFS produced in 2008, having a value of about $53 million, reflecting the dramatic downturn in the U.S. economy.

The USGS *2009 Minerals Yearbook* also reports that roughly 39 percent of ACBFS was used as road base or in road surface layers, 17 percent in asphalt concrete, 17 percent in ready-mixed concrete, 9 percent as fill, 4 percent in the manufacture of concrete products, and the rest was used in miscellaneous applications (e.g., railroad ballast, roofing). The selling price of ACBFS in 2009 ranged between $3.65 and $21.88 per ton ($3.31 and $19.84 per metric ton), with an average of $8.11 per ton ($7.35 per metric ton) (van Oss 2009). Although the total percentage of ACBFS used as aggregate in construction is relatively low (approximately 2 percent of the U.S. market), it is an important aggregate source in locations close to ACBFS processing plants or where plants historically existed and stockpiles of ACBFS remain. According to the National Slag Association (NSA 2011), ACBFS is produced in the following States: Alabama, Indiana, Kentucky, Maryland, Michigan, New York, Ohio, Pennsylvania, Utah, and West Virginia.

There are economic, environmental, and social benefits derived from the effective use of ACBFS rather than disposing of it as waste. As a result, the beneficial use of ACBFS has broad positive impacts on sustainability, a consideration of increasing importance. First, the use of ACBFS makes economic sense for the iron producers, who financially benefit from the sale of the material while avoiding disposal costs, and it is also an economic benefit to the direct user of ACBFS (e.g., ready-mix producer, contractor), who obtains a reclaimed aggregate that is relatively inexpensive compared to most naturally derived material. Use of ACBFS also economically benefits the public, who realize overall cost savings due to the reduced price of the material during initial construction.

However, the benefits go beyond simple economics, as the appropriate use of ACBFS also has wide-ranging environmental and societal benefits. For example, when ACBFS is used, less natural material needs to be mined, transported, and processed. This means less disruption to the land, less energy consumed, and less pollution and greenhouse gases generated from mining and transporting natural aggregate. Figure 1 illustrates the life cycle of typical aggregate, showing how reducing natural aggregate use can enhance sustainability by eliminating the extraction and transportation phases. A limited amount of energy is needed to process ACBFS, but the required energy is less than that used for extracting and processing aggregates from natural sources.

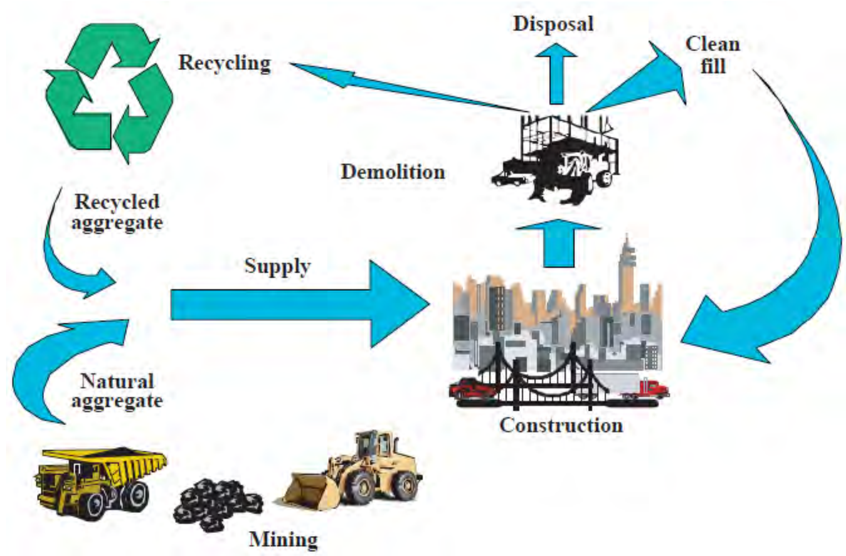

Figure 1. Materials flow cycle for aggregates.
(USGS 1998).

The increased use of ACBFS also means less waste is produced, resulting in less storage of materials in unsightly stockpiles and reduced disposal quantities in permanent landfills. All told, the appropriate and beneficial use of ACBFS can significantly enhance sustainability by effectively contributing to all aspects of the "triple-bottom line," resulting in economic, environmental, and social benefits as long as the performance of the concrete structure is not compromised through the use of ACBFS.

This last point is an essential part of enhancing the sustainability of any pavement through the use of a recycled or reclaimed material. Any short-term economic and environmental gain will be quickly overshadowed by the economic, environmental, and social costs associated with poor or reduced pavement performance. If a pavement does not meet its design expectations, the economic costs associated with earlier and more frequent maintenance and rehabilitation activities as well as their associated environmental and social costs (e.g., traffic congestion that generates considerable pollution and greenhouse gases as well as social disruption) can be extreme. To minimize risk of premature failure, the unique properties of a given material must be known and considered in the material processing, design, and construction phases. Ignoring these properties will have unexpected consequences that often result in poor performance, compromising the sustainability of the project. It is therefore essential that engineers and contractors who use ACBFS in concrete understand its unique properties to ensure that the desired performance of the pavement over its design life is achieved.

OBJECTIVES OF THE REPORT

This report is the product of a technical review of available information on the use and performance of ACBFS as coarse aggregate in paving concrete. The primary purpose of this review is to provide the Federal Highway Administration (FHWA) and other transportation agencies the most current available information on the use and application of ACBFS in concrete paving mixtures.

The primary impetus for this technical review were concerns expressed by the Michigan Department of Transportation (MDOT), which issued a moratorium on the use of ACBFS as a concrete coarse aggregate for most concrete pavements in Michigan (FHWA 2006). Initially, this moratorium was for concrete used on interstate highway pavements, but subsequent clarification of the moratorium has effectively extended it to include all freeways and other high-traffic concrete pavements in Michigan. Further, it includes pavements constructed using MDOT's Special Provision for High Performance Portland Cement Concrete Grade P1 (Modified), which requires that all aggregates "originate only from natural geological sources" (MDOT 2005). As stated in the memorandum, MDOT has over 70 years of experience using ACBFS in paving concrete, and "has noted serious concerns with the performance of many of the concrete pavements that utilized blast furnace slag as a coarse aggregate" (FHWA 2006). The memorandum also states that Michigan has an abundance of high-quality natural aggregates that do not exhibit the "materials variability, constructability, and ultimate performance issues" associated with pavements constructed with ACBFS.

Attached to the FHWA memorandum is a memorandum from Gloria Jeff, at the time the Director of MDOT (MDOT 2006), which included a report entitled, *A Summary of Historical Research of Blast-Furnace Slag Coarse Aggregate in Michigan Concrete Pavements* (Staton 2006). In the concluding remarks of this report, it is stated that "any use of manufactured aggregates in concrete pavement, including slag, should take place only after a thorough understanding of the material's engineering properties…and how those properties contribute to the pavement's expected performance." It is further stated that "the costs associated with unacceptable performance do not warrant the elevated risk when suitable alternatives have historically demonstrated their in-service performance" (Staton 2006).

This current research effort is, in part, an attempt to address MDOT's concerns cited above, drawing together the available information regarding the engineering properties of concrete made with ACBFS coarse aggregate and the performance of pavements made with that concrete. The intent is to summarize the existing level of understanding and to identify gaps in the current knowledge base. Inherent in this understanding is the consideration of risk posed through the use of ACBFS coarse aggregate as compared to the use of alternative, naturally derived materials. A number of recommendations are made to reduce this risk through adoption of best available technologies and execution of additional research to address gaps in current knowledge.

This report provides the technical background on the subject, including a discussion of the production of ACBFS, its properties, and application as an aggregate in concrete. It also identifies concerns that have arisen regarding the mechanical behavior and durability of concrete pavements made with ACBFS as a coarse aggregate. It consists of eight chapters, the first of which is this introduction. Chapter 2 discusses the production of ACBFS and its physical and chemical properties. Chapter 3 presents the use of ACBFS as an aggregate in concrete. Chapter 4 discusses concerns that have arisen regarding the performance of concrete pavements made with ACBFS in Michigan, and chapter 5 summarizes experiences from several other States and one Canadian province. Chapter 6 summarizes the results of a field and petrographic assessment of four pavement sections in Ohio and Indiana, and chapter 7 reviews the impact of the concrete's thermal properties with respect to pavement design features. The report concludes with chapter 8, which provides recommendations regarding the use of ACBFS to ensure adequate performance and discusses future research needs. Appendix A provides the quality control plan for use of coarse aggregate blast furnace slag on Interstate (I) 94, and appendix B provides comments from the blast furnace slag field surveys conducted in Indiana and Ohio and reported in chapter 6.

CHAPTER 2. ACBFS PRODUCTION AND PROPERTIES

PRODUCTION OF AIR-COOLED BLAST FURNACE SLAG

Slag is the byproduct of metallurgical operations, typically containing gangue from the metal ore, flux material, and unburned fuel constituents. Slag is often classified into nonferrous and ferrous slag, where nonferrous slag includes those derived from copper, lead-zinc, nickel, and phosphorus metallurgical operations, and ferrous slags are those derived from the production of iron and steel. This is illustrated in figure 2, where iron blast furnace slag is shown as a byproduct of the production of iron from iron ore (Hammerling 1999).

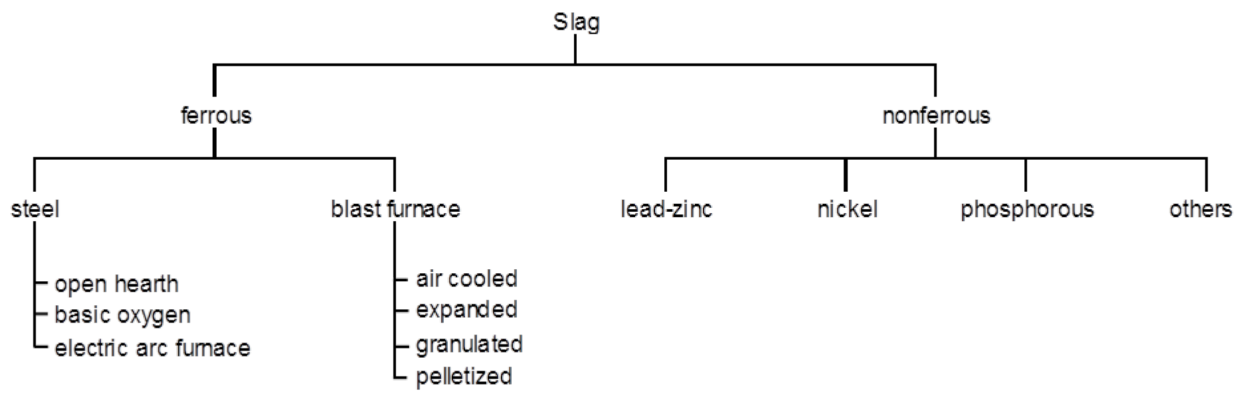

Figure 2. Types of slags classified based on origin.
(From Hammerling 1999, p. 4. © D. M. Hammerling 1999. Adapted with permission.)

As shown in figure 2, blast furnace slag is categorized based on how the molten slag is treated once it is removed from the furnace: air-cooled, expanded, granulated, or pelletized. Although some air-cooled slag may be sprayed with water to expedite processing, as performed at the Ford Rouge River Complex near Detroit, Michigan, and Essar Steel Algoma at Sault Ste. Marie, Ontario, it is still referred to as air-cooled.

In the production of pig iron, the vertical-shaft blast furnace is used to smelt iron from iron ore, which contains iron oxide and other minerals. The fuel is coke, which is subjected to a continuous blast of air, resulting in a high rate of combustion. The fuel and ore are supplied continuously through the top of the furnace, while the air is blown into the bottom of the furnace. The smelting process, in which the ore containing iron oxide is converted to metallic iron through a reduction process, occurs as the material moves downward. The end products are the molten metal (known as pig iron) and the slag, each of which is tapped from the bottom of the blast furnace. Figure 3 presents a schematic of an iron blast furnace, and figures 4 and 5 show the molten slag being tapped and redirected into ladles from one of the blast furnaces at the Ford Rouge River Complex.

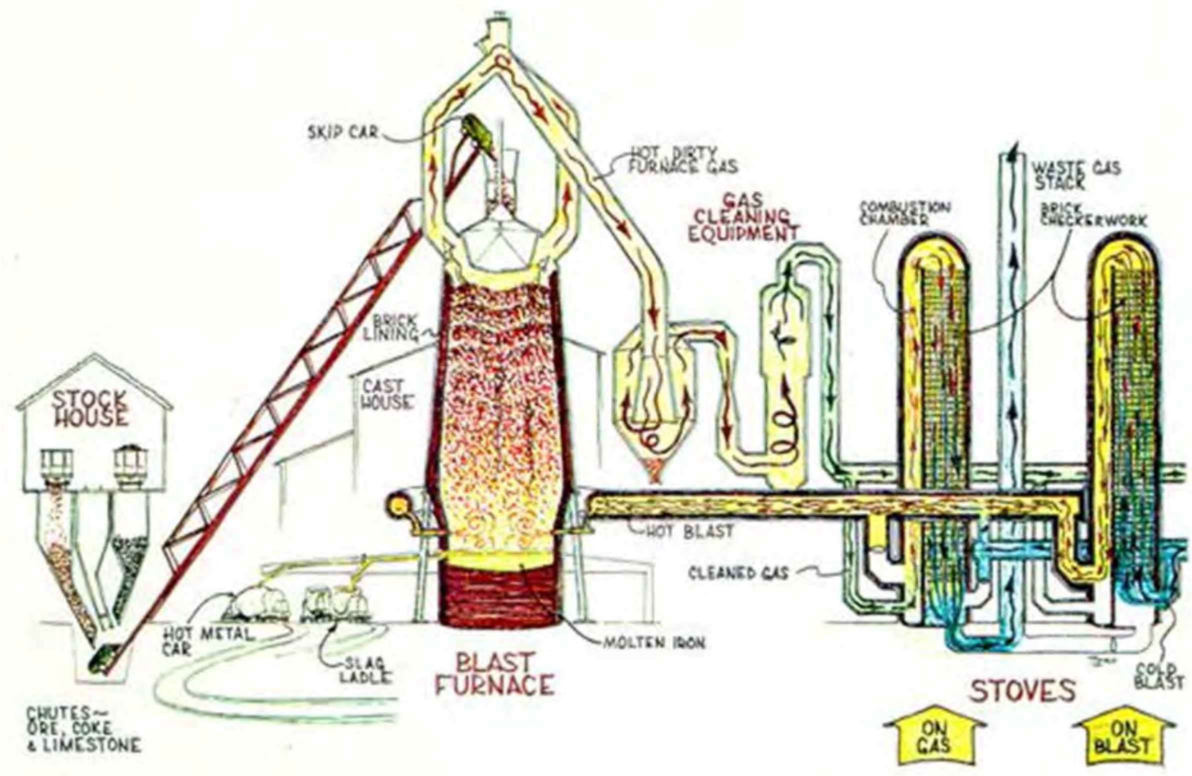

Figure 3. Schematic of iron blast furnace.
(Adapted from *The Iron Blast Furnace—Theory and Practice,* J. G. Peacey and W. G. Davenport, p. 2, figure 1.1. Adapted with permission.)

Figure 4. Molten blast furnace slab being tapped at the blast furnace at the Ford Rouge River Complex near Detroit, Michigan.

Figure 5. Tapped molten blast furnace slag being directed into ladles at the Ford Rouge River Complex near Detroit, Michigan.
(From Hammerling 1999, p. 12, figure 2.5. © D. M. Hammerling 1999. Reprinted with permission.)

CHEMICAL COMPOSITION OF AIR-COOLED BLAST FURNACE SLAG

As a product of calcinated fluxstone and the alumina and silica phases present in iron ore, the four major oxide phases present in ACBFS are CaO, SiO_2, Al_2O_3, and MgO. These oxides account for approximately 95 percent of the ACBFS composition, with the remaining 5 percent consisting of sulfur, manganese, iron, titanium, fluorine, sodium, and potassium, as shown in table 1 (Hammerling 1999). High magnesia content is generally attributed to the use of dolomite as a fluxing agent.

The oxide compositions are presented in table 1 with their respective weight percentages. Depending on the composition of the raw material, the fusion temperature, and the cooling rate, a variety of minerals can form. The glass content is mainly dependent on the cooling rate, with faster cooling resulting in the formation of more glass, whereas slower cooling allows more time for the formation of crystallized minerals. This is very important, as glassy phases are chemically more reactive, and is why rapidly quenched granulated slag can be ground and used as cement.

Table 2 presents common minerals found in ACBFS. The most prevalent mineral found in slag is melilite, which is a solid solution between the isomorphous minerals gehlenite ($2CaO \cdot MgO \cdot 2SiO_2$) and akermanite ($2CaO \cdot Al_2O_3 \cdot SiO_2$). Another common mineral that is potentially important to the performance of ACBFS as a coarse aggregate in concrete is calcium sulfide (CaS), commonly referred to as oldhamite (Peterson et al. 1999; Hammerling 1999; Hammerling et al. 2000; MPA 2011). The effect of calcium sulfide on concrete performance is discussed in the next chapter.

Table 1. Typical Composition of ACBFS
(From Hammerling 1999, p. 14, table 2.1. © D. M. Hammerling 1999.
Adapted with permission.)

Component	Percentage
Major Components	*95*
Lime (CaO)	30–40
Silica (SiO_2)	28–42
Alumina (Al_2O_3)	5–22
Magnesia (MgO)	5–15
Minor Components	*5*
Sulfur (CaS, other sulphides, sulfates)	1–2
Iron (FeO, Fe_2O_3)	0.3–1.7
Manganese (MnO)	0.2–1
Rare Components	
$Na_2O + K_2O$	0–1
TiO_2	0–1
V_2O_5	0–1
Cr_2O_3	0–1

Table 2. Typical Minerals Found in ACBFS
(From Hammerling 1999, p. 15, table 2.2. © D. M. Hammerling 1999. Adapted with permission.)

Mineral	Formula	Crystal System
Melilite	$(Ca,Na)_2(Mg,Al)(Si,Al)_2 0_7$	tetragonal
Wollastonite	$CaSiO_3$	triclinic
Oldhamite	CaS	cubic
Dicalcium silicate	$2CaO \cdot SiO_2$	
Rankinite	$3CaO \cdot 2SiO_2$	monoclinic
Merwinite	$3CaO \cdot MgO \cdot 2SiO_2$	monoclinic
Anorthite	$CaO \cdot Al_2O_3 \cdot 2SiO_2$	triclinic
Monticellite	$CaO \cdot MgO \cdot SiO_2$	orthorhombic
Spinel	$MgO \cdot Al_2O_3$	isotropic
Periclase	MgO	isotropic
Olivine	$(Fe,Mg)_2SiO_4$	orthorhombic
Glass	Variable	amorphous

PHYSICAL PROPERTIES OF AIR-COOLED BLAST FURNACE SLAG

The physical properties of ACBFS are largely controlled by how it cools and solidifies. Figures 6 and 7 illustrate the cooling process used in the production of ACBFS coarse aggregate at the Ford Rouge River Complex. The ladles containing molten blast furnace slag are moved from the blast furnace to a pit using specially modified front-end loaders (figure 6). After the molten slag is placed in the pit, jets of water are sprayed onto the surface to accelerate cooling and to facilitate expedited removal of the material so as not to inhibit the smelting process. The ACBFS mass in the pit has many cracks through which water seeps in, generating steam that forms vesicles, resulting in an expanded, very porous slag. The final product is then removed from the pit, as seen in the background of figure 7, transported to a crushing and screening facility, and then processed like conventional aggregate, except that magnetic separation is used to remove small pieces of pig iron. Once the material arrives at the processing facility, its temperature is checked, and if it is too hot, the material is further cooled with water.

The color of ACBFS coarse aggregate usually varies from light to dark gray, depending on chemical composition, although blue, green, and pink staining of smaller areas have been observed. As mentioned, the aggregate particles have a very rough texture due to the “vesicular” structure formed by gases entrapped in the ACBFS as it cools. This is particularly prevalent if water is used in the cooling process. It is generally thought that the pores present in the ACBFS particles are not interconnected, making the term “vesicular” more appropriate than “porous,” although some research suggests that considerable interconnectivity exists, especially under vacuum impregnation. This means that interior voids can be accessible to liquids and gases from the exterior of the aggregate particle (Hammerling 1999). This can be seen in figure 8, where the high interconnected porosity is readily evident as voids filled with vacuum-impregnated epoxy appear grey compared to the few empty voids that appear black.

Figure 6. Ladle containing molten blast furnace slag being dumped into a cooling pit at the Ford Rouge River Complex. Note water being sprayed onto the surface. (From Hammerling 1999, p. 14, figure 2.6. © D. M. Hammerling 1999. Reprinted with permission.)

Figure 7. Surface of slag pit at the Ford Rouge River Complex. Note the crushed ACBFS in background and water being sprayed in the foreground to cool the recently molten surface.

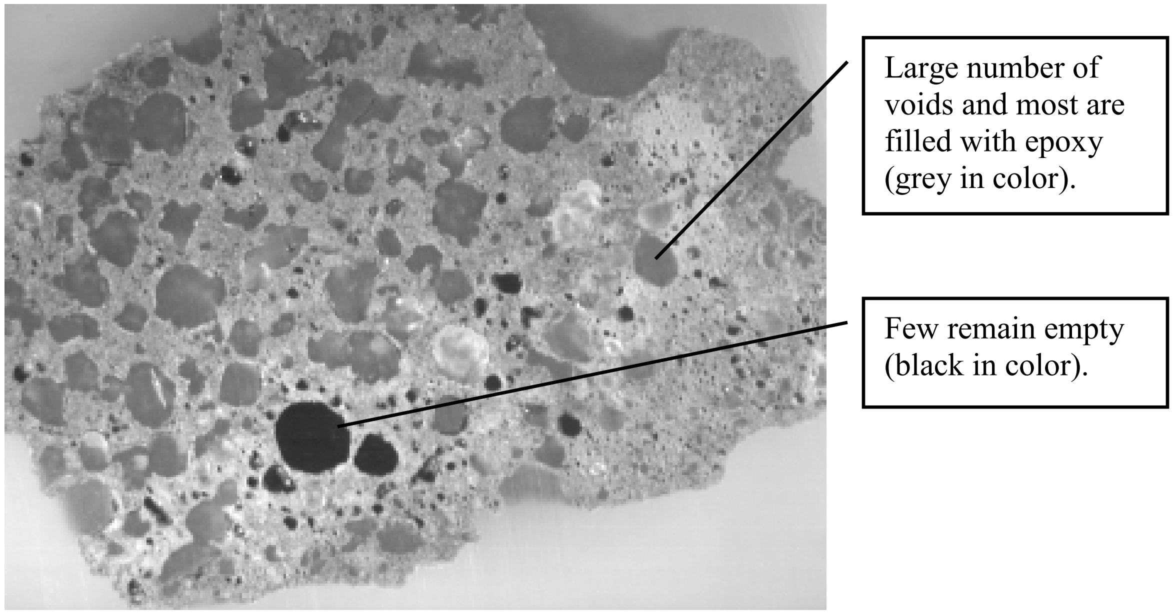

Figure 8. ACBFS coarse aggregate particle, vacuum-impregnated with epoxy and then sliced in half. Horizontal field of view is 3 cm.
(From Hammerling 1999, p. 22, figure 2.12. © D. M. Hammerling 1999. Adapted with permission.)

Crushed ACBFS used as coarse aggregate in concrete is angular and roughly cubical. Its texture ranges from rough and vesicular (porous) to glassy (smooth) with conchoidal fractures (Rao 2006). ACBFS aggregate typically has a high angle of friction, ranging between 40 and 45 degrees (Chesner, Collins, and Mackay 1998). The Los Angeles (L.A.) abrasion values for ACBFS typically range between 35 and 45 percent (Chesner, Collins, and Mackay 1998), although it may be as high as 55. This range of values is higher than that typical of natural aggregates. California bearing ratio values of ACBFS aggregate are typically greater than 100, owing to angular particle shape and rough texture (Chesner, Collins, and Mackay 1998). The hardness of ACBFS according to the Mohs scale ranges from 5 to 6 (Chesner, Collins, and Mackay 1998). These physical properties make crushed ACBFS aggregate an acceptable candidate for the replacement of natural aggregate for many applications.

The void structure of the ACBFS heavily influences the physical properties, including the bulk specific gravity and the absorption. Fine slag screenings are similar in density to natural sand, while the density of coarse aggregate particles are as much as 20 percent less than natural aggregates having the same gradation (Lewis 1982). This may provide a benefit when ACBFS is used as a coarse aggregate in concrete due to the reduced weight of the structural components. While the average bulk specific gravity of crushed limestone aggregate is about 2.65 (Somayaji 2001), the typical values for ACBFS are between 2.0 and 2.5, with the solids having an apparent specific gravity between 2.9 and 3.1 (Rao 2006). Although concerns have been voiced regarding the variability of the density of ACBFS, the Ohio Department of Transportation (ODOT) has observed that in recent years the variation in bulk specific gravity of ACBFS is no greater than that encountered for naturally derived materials (personal communication, J. Wigdahl, 2009).

The Edw. C. Levy Company documented the bulk specific gravity, saturated surface dry (SSD) specific gravity, apparent specific gravity, and absorption of ACBFS aggregate over time by performing tests over a 4-month period (personal communication, John Yzenas, Edw. C. Levy Company, 2010). The samples for these tests were obtained from what is now called the ArcelorMittal facility at Burns Harbor, Indiana. For each test date, an ACBFS aggregate sample was subjected to a washed gradation and the previously described tests were performed on material that was retained on eight sieves using the procedure described in ASTM C127. The results from the tests are shown in table 3.

The relationship between the particle size and the bulk specific gravity, SSD specific gravity, and absorption for all tested samples are shown in figures 9, 10, and 11, respectively. In these figures, the sieve size is shown on the x-axis. These figures illustrate the variability for each parameter for the material retained on each sieve.

The relationships between average absorption, average bulk specific gravity, and ACBFS particle size are shown in figure 12. The average values shown in this figure were computed by averaging the results obtained for multiple test dates that are shown in table 3. The bulk specific gravity increases as the particle size decreases because internal voids are exposed as the aggregate particle is reduced in size. The absorption of the particles is nearly constant for particle sizes ranging from 1 in. to 0.0937 in. (25.4 mm to 2.38 mm; No. 8 sieve), but shows a rapid decrease for particle sizes passing the No. 8 sieve. These results indicate variability of the overall specific gravity of ACBFS aggregate will be heavily influenced by a change in gradation, therefore emphasizing the need to maintain tight control of gradation during concrete production.

The typical compacted unit weight of ACBFS ranges between 70 and 85 lb/ft^3 (1,121 and 1,362 kg/m^3) as measured by ASTM C33. The compacted unit weight of lightweight aggregates typically range between 55 and 70 lb/ft^3 (881 and 1,121 kg/m^3), while that of normal weight aggregates typically range between 75 and 110 lb/ft^3 (1,201 and 1,762 kg/m^3). The Illinois Department of Transportation (IDOT), Pennsylvania Department of Transportation (PennDOT), and Kentucky Transportation Cabinet (KYTC) require that ACBFS have a compacted weight of not less than 70 lb/ft^3 (IDOT 2007; KYTC 2008; PennDOT 2000).

Table 3. Results From Tests Performed on ACBFS by Levy Company
(© Edw. C. Levy Company 2010. Reprinted with permission.)

Sieve Size	Test Date	Bulk Specific Gravity	SSD Bulk Specific Gravity	Apparent Specific Gravity	Absorption (%)
No. 30 (0.0234 inch)	10/28	2.756	2.832	2.983	2.763
	09/24	2.745	2.809	2/931	2.311
	09/16	2.725	2.793	2.926	2.521
	10/08	2.667	2.734	2.857	2.499
	Average	**2.723**	**2.792**	**2.924**	**2.523**
	Std. Dev.	**0.040**	**0.042**	**0.052**	**0.185**
No. 16 (0.0469 inch)	09/16	2.619	2.684	2.801	2.480
	09/24	2.644	2.719	2.858	2.835
	10/08	2.607	2.666	2.772	2.288
	10/28	2.707	2.783	2.929	2.798
	Average	**2.644**	**2.713**	**2.840**	**2.600**
	Std. Dev.	**0.045**	**0.051**	**0.069**	**0.262**
No. 8 (0.0937 inch)	08/12	2.491	2.586	2.750	3.777
	08/31	2.464	2.558	2.719	3.804
	09/08	2.489	2.580	2.738	3.659
	08/21	2.518	2.598	2.738	3.194
	07/23	2.492	2.576	2.720	3.371
	11/23	2.511	2.576	2.686	2.601
	09/15	2.465	2.557	2.713	3.687
	Average	**2.490**	**2.576**	**2.723**	**3.442**
	Std. Dev.	**0.020**	**0.015**	**0.021**	**0.433**
No. 4 (0.187 inch)	09/15	2.379	2.464	2.600	3.579
	11/23	2.370	2.450	2.577	3.395
	09/08	2.375	2.464	2.607	3.744
	08/21	2.417	2.493	2.615	3.129
	08/11	2.390	2.473	2.607	3.489
	08/31	2.382	2.467	2.604	3.568
	08/03	2.401	2.454	2.534	3.182
	Average	**2.388**	**2.466**	**2.592**	**3.441**
	Std. Dev.	**0.017**	**0.014**	**0.028**	**0.222**
3/8” (0.375 inch)	09/08	2.312	2.393	2.517	3.521
	08/11	2.330	2.408	2.528	3.362
	07/23	2.322	2.408	2.540	3.699
	08/31	2.301	2.386	2.515	3.690
	08/21	2.330	2.410	2.534	3.459
	09/15	2.313	2.394	2.516	3.479
	08/03	2.278	2.356	2.471	3.438
	Average	**2.312**	**2.394**	**2.517**	**3.521**
	Std. Dev.	**0.018**	**0.019**	**0.022**	**0.128**
½” (0.5 inch)	08/21	2.275	2.352	2.466	3.406
	08/11	2.253	2.338	2.462	3.767
	09/15	2.260	2.337	2.448	3.410
	08/31	2.273	2.356	2.479	3.658
	09/08	2.289	2.373	2.498	3.661
	11/23	2.284	2.352	2.453	3.050
	Average	**2.272**	**2.351**	**2.468**	**3.492**
	Std. Dev.	**0.014**	**0.013**	**0.168**	**0.261**
¾” (0.75 inch)	11/23	2.199	2.275	2.380	3.469
	08/21	2.254	2.327	2.432	3.246
	08/11	2.244	2.317	2.447	3.698
	08/31	2.242	2.323	2.439	3.606
	09/15	2.224	2.301	2.410	3.476
	Average	**2.233**	**2.309**	**2.422**	**3.499**
	Std. Dev.	**0.022**	**0.021**	**0.027**	**0.170**
1”	08/11	2.207	2.291	2.411	3.826
	08/21	2.176	2.257	2.368	3.729
	09/08	2.184	2.271	2.391	3.952
	11/23	2.194	2.262	2.354	3.102
	Average	**2.190**	**2.270**	**2.381**	**3.652**
	Std. Dev.	**0.013**	**0.015**	**0.025**	**0.378**

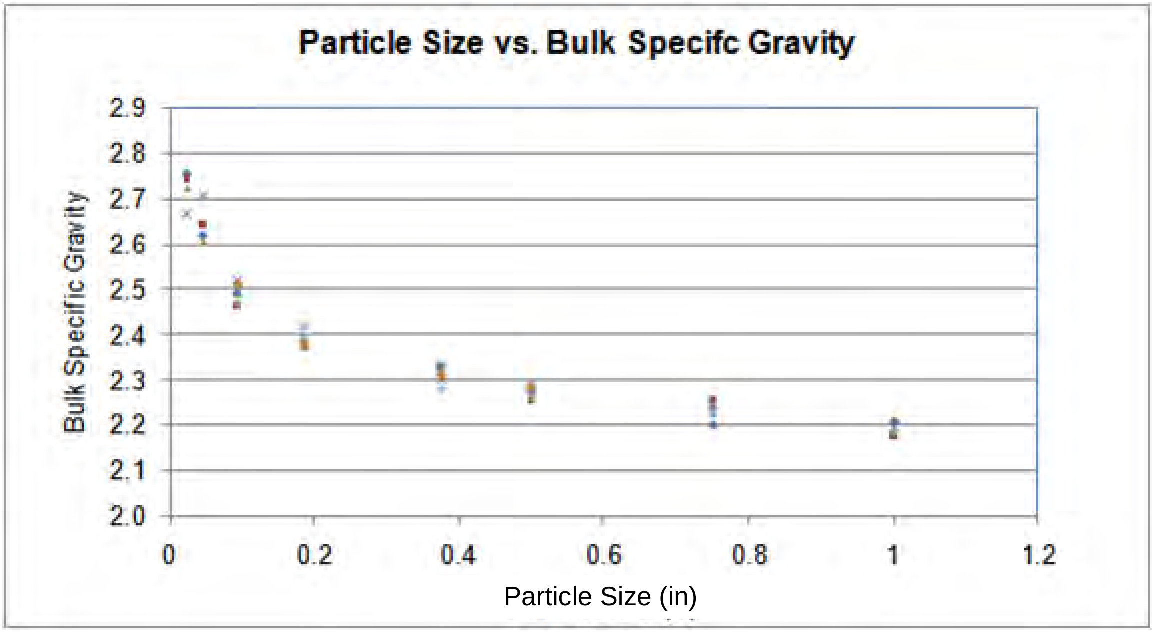

Figure 9. Relationship between bulk specific gravity and ACBFS particle size.
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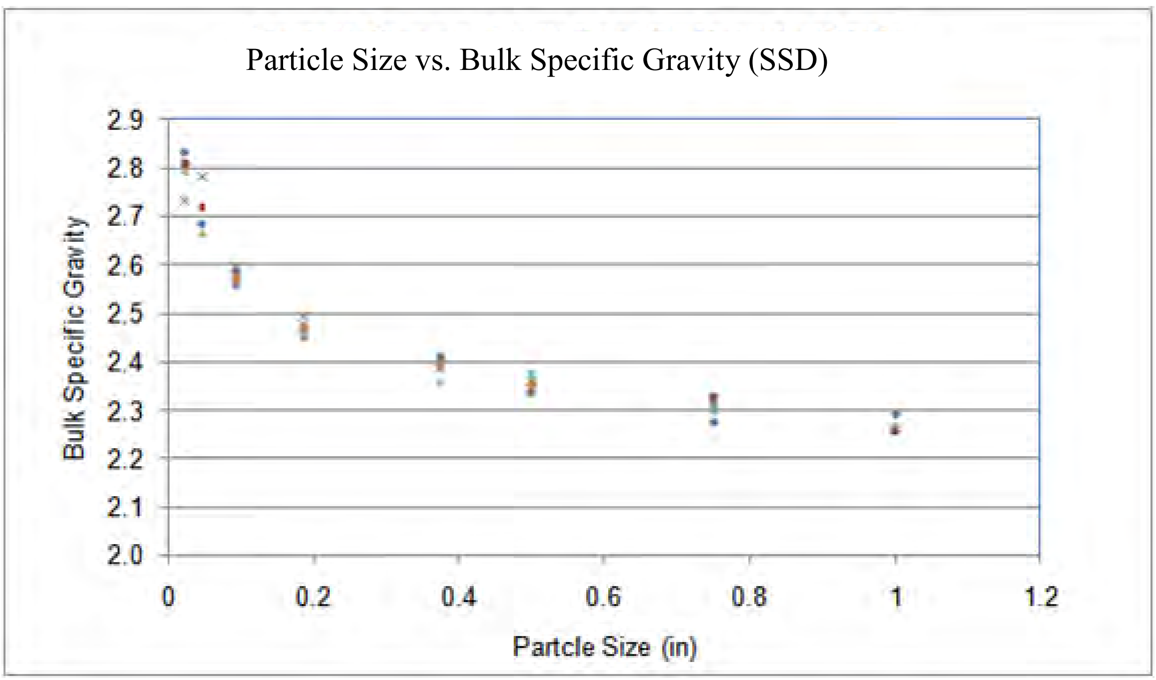

Figure 10. Relationship between bulk specific gravity (SSD) and ACBFS particle size.
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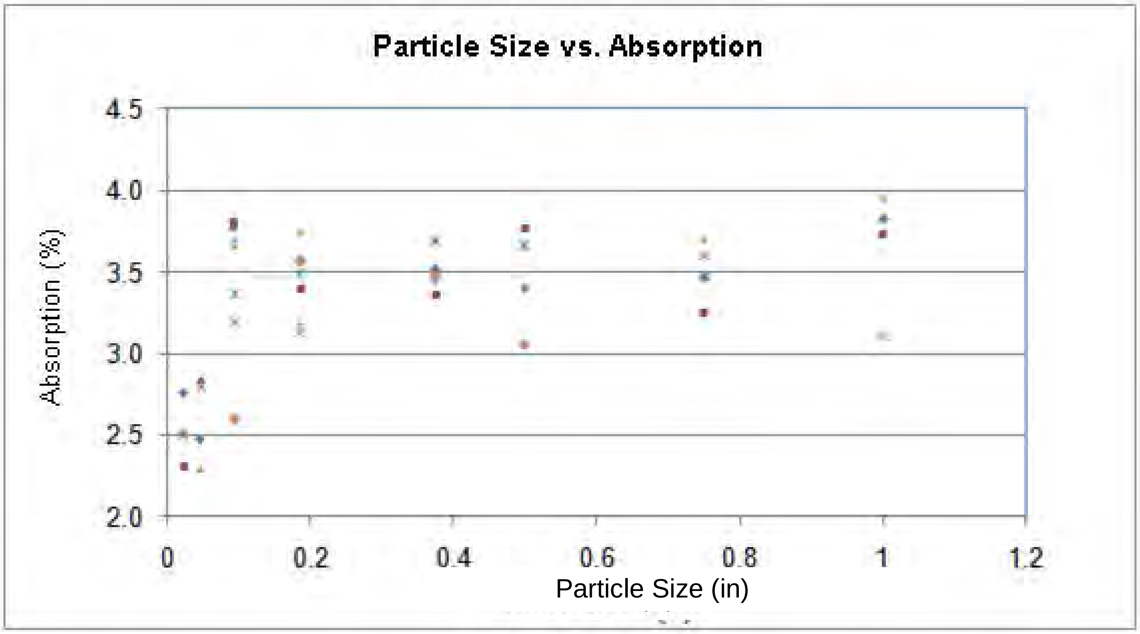

Figure 11. Relationship between absorption and ACBFS particle size.
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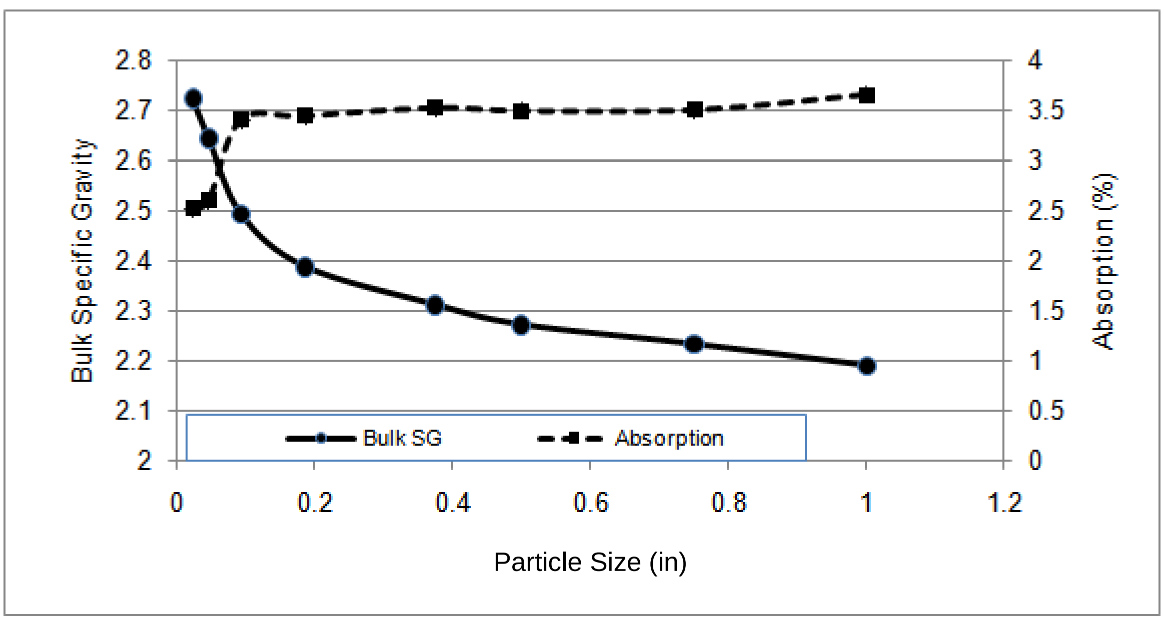

Figure 12. Relationship between absorption, bulk specific gravity, and ACBFS particle size.
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The average absorption for ACBFS is relatively high when compared to that of most natural aggregates, varying from 1 to 8 percent. The intrinsic vesicular/porous nature of the larger ACBFS particles is responsible for the high absorption, as are other factors including greater surface area and a stronger resistance to removal of water held in shallow surface voids. ACBFS also experiences constant and continuous uptake of water over an extended period of time when subjected to 100 percent relative humidity.

The relationship of the observed void structure to absorption has been investigated by a number of researchers. Several State DOTs place limits on the absorption of aggregates to help prevent damage to the concrete due to freezing and thawing, but ACBFS may behave differently than aggregates derived from natural sources with respect to freezing and thawing, as discussed in the next chapter. The Minnesota Department of Transportation (Mn/DOT), for example, limits absorption of PCC aggregates to 1.7 percent (Williamson 2005). In addition to concerns regarding freeze-thaw durability, the high absorption of ACBFS can be a problem if batched dry of SSD in the production of concrete, as the aggregate will absorb water from the mix, causing stiffening and shrinkage-related cracking, a construction concern discussed later in this document.

Staton and Anderson (2009) reported the results of a study performed by MDOT to investigate moisture-conditioning methods used to test the freeze-thaw properties of coarse aggregates that are used in concrete. The intent of the absorption study was to (1) determine the rate of absorption of aggregates used in the aggregate test road (constructed on southbound US-23 just north of the Ohio border), (2) verify that aggregates exposed to continuous immersion in water achieved vacuum-saturation levels of absorption over time, and (3) confirm that aggregate is not damaged as a result of vacuum saturation (i.e., the ultimate absorption potential for the aggregate samples used to fabricate laboratory freeze-thaw test specimens is not altered).

The first task of this study investigated the laboratory absorption characteristics of the five types of coarse aggregate (including ACBFS) used in MDOT’s aggregate test road. The aggregate samples were oven-dried and submerged in water. Five samples were tested for absorption at the following time periods: 24 hours, 7 days, 30 days, 90 days, 180 days, 1 year, and 3 years. Samples of the aggregate were also tested after vacuum saturation in accordance with Michigan Test Method 113 (MTM 113 2007) to determine the absorption level. The results from the study are shown in table 4. As expected, the ACBFS aggregate had the highest absorption level. Table 5 shows the absorption values for the coarse aggregates at 24 hours, 1 year, and 3 years as a percentage of the vacuum-saturated absorption level.

This study was performed to determine if vacuum saturation can be used as an acceptable surrogate for estimating the long-term absorption that would be expected to occur under field conditions. After 24 hours of soaking, the coarse aggregates from natural sources (i.e., gravels and limestone) achieved saturation levels between 62 and 78 percent of the vacuum-saturated value, while ACBFS achieved only 33 percent saturation. Aggregate from natural sources achieved close to a 90 percent vacuum saturation level in 30 days, whereas ACBFS achieved a similar saturation level after 1 year of submersion. At 1 year, the four natural aggregates had become fully saturated, achieving saturation levels of 98 to 99 percent of the vacuum-saturation levels, whereas the ACBFS took 3 years of soaking to reach a similar level of saturation.

Table 4. Absorption Values at 24 Hours, 30 Days, 1 Year, and 3 Years
(From Staton and Anderson 2009. © Michigan Department of Transportation.
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Coarse Aggregate		Absorption (%) Soaking Period				
Type	**Source**	**Vacuum Saturated**	**24 hours**	**30 Days**	**1 Year**	**3 Years**
Natural gravel	Bundy Hill	1.16	0.91	1.03	1.14	1.17
Limestone	Holloway/Rockwood Stone	3.39	2.38	3.19	3.35	3.33
Natural gravel	American Aggregate	1.51	1.18	1.37	1.48	1.52
ACBFS	Levy Plant # 4	7.13	2.34	3.70	6.43	6.98
Limestone	France Stone	3.87	2.42	3.39	3.82	3.92

Table 5. Absorption Values as a Percentage of Vacuum-Saturated Values
(From Staton and Anderson 2009. © Michigan Department of Transportation.
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Coarse Aggregate		Absorption as a Percent of Vacuum-Saturated Value Soaking Period			
Type	**Source**	**24 hours**	**30 Days**	**1 Year**	**3 Years**
Natural Gravel	Bundy Hill	78	89	98	101
Limestone	Holloway/Rockwood Stone	70	94	99	98
Natural Gravel	American Aggregate	78	91	98	101
ACBFS	Levy Plant # 4	33	52	90	98
Limestone	France Stone	62	88	99	101

The results of this study are quite relevant as MDOT uses vacuum saturation to condition aggregates for laboratory freeze-thaw testing to achieve a level of saturation that is representative of that expected to occur in the field at the bottom of the slab and in the vicinity of joints. This study indicates that the pore structure of ACBFS is uniquely different from that present in aggregates derived from natural sources, not only resulting in higher overall levels of absorption, but also reaching various levels of saturation more slowly during continuous immersion.

The third goal of the MDOT study was to statistically demonstrate that the vacuum-saturation method does not alter the pore characteristics of typical gravel, carbonate, or ACBFS coarse aggregates. This was accomplished by analyzing the difference in percent absorption as calculated by the 24-hour soaking method both before and after the samples underwent vacuum saturation; no statistically significant differences were observed.

It is noted that these results are from a laboratory study in which the aggregate underwent continuous immersion for up to 3 years. It is not unreasonable to assume that the relatively large pores in ACBFS aggregates would be less likely to saturate when entirely encapsulated in concrete than the smaller pores in naturally derived aggregates simply based on surface tension, which dictates that smaller pores will be saturated first. Still, there is the strong potential for the coarse aggregate to become critically saturated at exposed locations such as joints and cracks. Thus, the degree to which laboratory vacuum saturation of aggregates simulates actual levels of saturation of aggregates in the field has yet to be determined.

SUMMARY

ACBFS has unique chemical and physical properties that influence its behavior as an aggregate in concrete. Several of the key chemical properties are discussed in the next chapter (particularly the presence and dissolution of calcium sulfide), but the physical property of greatest concern is the high level of porosity compared to that present in naturally derived aggregates, which contributes to high absorption capacities. This is important during construction, as the moisture condition of the aggregate will impact workability and early-age, shrinkage-related cracking if the aggregate is not kept sufficiently moist prior to batching. It may also have long-term ramifications on inservice durability, depending on the level of saturation those aggregates are subjected to either at the bottom of the slabs or in the vicinity of joints and cracks. The next chapter discusses the behavior of concrete made with ACBFS.

CHAPTER 3. ACBFS AS COARSE AGGREGATE IN CONCRETE

Aggregates typically occupy 70 to 80 percent of concrete by volume (Mindess, Young, and Darwin 2003) and are often viewed as inert filler that results in good physical properties at low cost. Naturally derived aggregates consist of natural sand and gravel, crushed rock, or mixtures of the two. It is reported that the total crushed stone consumption in the United States in 2010 was about 1.27 billion tons (1.15 billion metric tons) valued at $11 billion, which is a dramatic decrease from the peak usage in 2006 of approximately 2.0 billion tons (1.8 billion metric tons) (USGS 2011). Roughly 40 percent of crushed stone was being used as coarse or fine aggregate in the transportation industry in 2003 (USGS 2005). Further, there were 838 million tons (760 million metric tons) of sand and gravel produced for the construction industry in 2010 at a value of $5.9 million, down from peak production of 1.4 billion tons (1.3 billion metric tons) in 2006 (USGS 2011).

Sources of high-quality aggregate near urban areas that are suitable for use in concrete are becoming increasingly more difficult to obtain as existing pits and quarries become exhausted (USGS 2011). Part of the problem is that easily obtained sources of high-quality aggregates have already been exploited and difficulties in obtaining permits and local opposition often delay or block the opening of new gravel pits and quarries in urban and suburban locations. As such, the necessity of using reclaimed (e.g., ACBFS) and recycled (e.g., recycled concrete aggregate) materials continues to grow.

ACBFS coarse aggregate has a long history of use in concrete; in fact, Lewis (1982) reports that ACBFS aggregate has been used in concrete since about 1880. State highway agencies located in industrial States with ready access to ACBFS have made use of it in concrete pavements for many decades, with recorded use going back to at least the 1930s. However, for ACBFS to be used successfully in concrete, its unique properties must be taken into consideration so that a readily constructible, structurally reliable, and highly durable concrete is produced. This is influenced by both the chemical and physical properties of the ACBFS and how they interact within the concrete environment, as discussed in the next sections.

INFLUENCE OF CHEMICAL PROPERTIES OF AIR-COOLED BLAST FURNACE SLAG ON CONCRETE

Iron and Dicalcium Silicate Unsoundness

It has long been known that the chemical properties of ACBFS must be taken into consideration when considering it for use in concrete. Two primary concerns are iron unsoundness and dicalcium silicate unsoundness (OECD 1997; Williamson 2005). Iron unsoundness is considered to be very rare, arising only if partially reduced iron oxides in the slag oxidize, with the resulting expansive reaction causing the ACBFS particles to disintegrate. Testing to detect iron unsoundness is conducted by immersing pieces of slag in water for a period of 14 days and observing whether any of the particles crack or disintegrate.

Dicalcium silicate unsoundness is caused by an increase in volume due to a phase inversion from beta-form to gamma-form during cooling, which will damage the ACBFS aggregate particles in a process commonly referred to as falling. An older American Concrete Institute report (ACI 1930) states that at one time it was thought advisable to specify that the slag lie in the pits for a defined period of aging to let it "fall," but tests and observations have shown this to be unnecessary, and now ACBFS that is less than 2 weeks old is used in concrete. Juckes (2002) discusses the beta-form of dicalcium silicate transitioning to the gamma-form, stating that it can undergo the expansive phase inversion on cooling through a temperature of about 750 ^{o}F to 930 ^{o}F (400 °C to 500 °C) with a disruptive effect on the aggregate structure. This inversion can only occur during the cooling process, and thus has effectively been completed within a few days as the slag reaches ambient temperatures.

Some countries have standards to guard against the marketing of ACBFS weakened by this mechanism; for example, British Standard BS EN1744: Part 1 (BS 1998) contains such a test. In the past, there were concerns, especially in England, that if the beta-form of dicalcium silicate did not invert during cooling it might do so destructively at ambient temperatures at a later date (known as late falling slag). There is no documented evidence that such an inversion at ambient temperatures has occurred, or that the performance of ACBFS aggregate in concrete has been negatively affected by this inversion (Juckes 2002).

In an older Australian publication, the Works Research Department (WRD 1975) advocated that managing stockpiles and weathering of crushed ACBFS is the only positive measure to ensure that the hardened concrete will be free of pop-outs resulting from the presence of free lime in the ACBFS and at the same time allowing the beta-form of dicalcium silicate to change to the stable gamma-form. WRD suggested that at least 2 (and preferably 3) months of weathering should be provided before ACBFS is released for sale. It was also recommended to moisten the stockpiled slag. An Australian Standard Specification (AS 1974) from the same time period suggests that the occurrence of pop-outs can be minimized if the crushed aggregate is weathered in the moist condition for a period of 3 months.

More recently, Australian Standard AS 2758.1 indicates that weathering of ACBFS aggregate is not needed as the prolonged weathering of ACBFS aggregate was specified to eliminate pop-outs that occurred due to incomplete assimilation of the calcined limestone in the blast furnace (AS 1998). The new standard indicates that modern blast furnace practice in Australia results in all flux materials, including limestone, being fully assimilated into the molten slag. When this condition is achieved, no unassimilated particles of calcined limestone are present in the solidified slag, and hence pop-outs that used to result from the hydration of these particles cannot occur. Further, the Australian standard AS 2758.1 indicates no evidence has been found, either in Australia or overseas, of delayed inversion of beta dicalcium silicate in modern ACBFS or of deterioration of concrete due to the presence of beta dicalcium silicate (AS 1998).

Calcium Sulfide

Concerns have also been expressed regarding the presence of calcium sulfide (also called oldhamite) in ACBFS. Detailed work conducted in evaluating this mineral phase is presented by Hammerling (1999) and Peterson et al. (1999). Calcium sulfide is present in ACBFS as a result of the sulfur from the coke fuel reacting with calcium from the dolomite or limestone used as a flux. It is most often observed in dendrites within the melilite, as shown in figure 13. The top micrograph shows a cross section through dendritic calcium sulfide, whereas the bottom micrograph shows dendrites in a plane approximately parallel to the thin section surface.

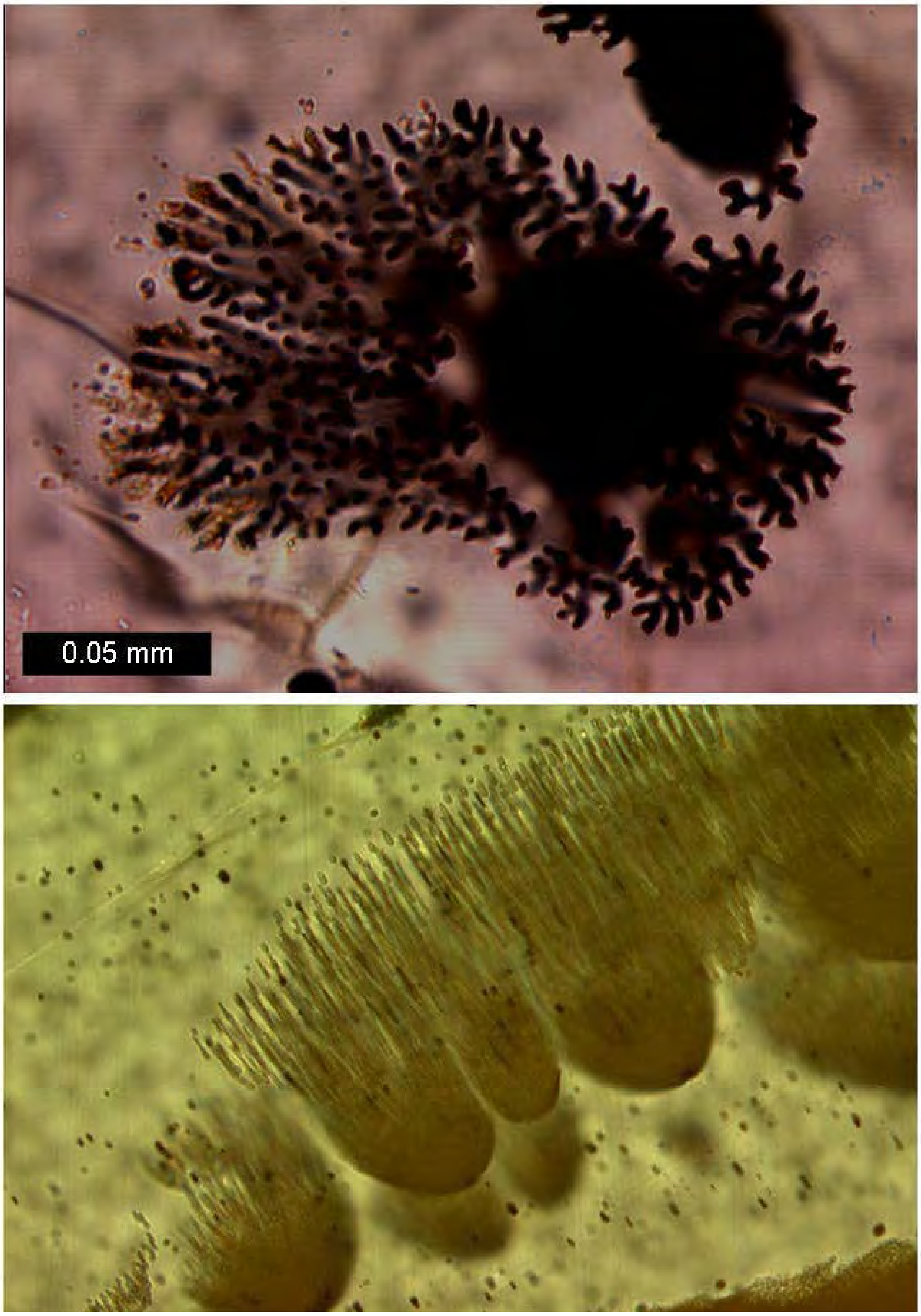

Figure 13. Micrographs of oldhamite in dendritic form.
(From D. M. Hammerling 1999, p. 37, figure 4.1. © Karl Peterson 1999.
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Sulfides and sulfates typically make up 1 to 2 percent of ACBFS. The predominant occurrence of sulfide is in the form of calcium sulfide, with smaller amounts of iron and manganese sulfides. The sulfide compounds found in ACBFS are slightly soluble in water, but in solution are always associated with a highly alkaline environment (NSA undated, a). Hammerling (1999) found that calcium sulfide was more soluble than previously indicated, and demonstrated through thermodynamic calculations that the solubility of calcium sulfide increased with rising pH; it was also shown that within typical concrete conditions (pH > 13), calcium sulfide would be expected to be highly soluble and will not be present as an equilibrium phase. The use of high-alkali cement and high-alkali fly ash would thus increase the solubility of this phase. Once dissolution occurs, the preferred species would be ettringite, a calcium sulfoaluminate mineral that is a common cement hydration phase.

Supporting the thermodynamic calculations is an extensive array of physical data including petrographic optical and scanning electron micrographs. Figure 14 shows an example of an ACBFS particle in contact with the hydrated cement paste. The top two images are reflected-light images of an ACBFS aggregate that contain dendritic inclusions of calcium sulfide within the cement matrix. The middle image presents a closeup view of the contact zone between the ACBFS aggregate and the cement matrix. The dissolution of calcium sulfide is observed as dark spots in the ACBFS aggregate close to the cement matrix, which are now voids. The initial growth of ettringite, exhibiting the typical needle-like structure, is seen in the surrounding air voids. The lower image is a backscatter electron image of the contact zone between the cement matrix and an ACBFS aggregate. Empty inclusions, once occupied by calcium sulfide, are located near the cement matrix as well as adjacent to the cracks in the ACBFS aggregate.

This pattern has been observed in numerous field studies (Van Dam et al. 2001; Van Dam et al. 2002; Van Dam et al. 2003; Delem et al. 2004; Grove, Bektas, and Gieselman 2006; Lankard 2010b). Figure 15 shows the extensive infilling of the air-void system in concrete containing ACBFS coarse aggregate from US-23 south of Flint, Michigan. In a separate study, it was reported that "moderate to extensive secondary ettringite deposition" was observed in the air-void system and that "the slag coarse aggregate may be a source of sulfur that is contributing to the development of significant ettringite deposition in the air-void system" (Grove, Bektas, and Gieselman 2006). Lankard (2010b) also observed that the presence of ACBFS aggregates led to extensive infilling of the air-void systems with secondary ettringite in a number of concrete pavement sections in Ohio. It is noted that secondary ettringite infilling of air voids is a normal feature of distressed concrete, but that the level of infilling observed in concrete made with ACBFS in many of these studies was beyond that normally observed in concrete made with naturally derived aggregates.

British Standards (1998) confirm that the dissolution of calcium sulfide can occur and that it may increase the opportunity for the calcium sulfide to hydrate. Hydration of sulfide phases in ACBFS can contribute to volume instability when those phases are present in substantial amounts (British Standards 1998). The hydration process undergoes chemical progression displayed by the equation below (NSA 2003):

$$CaS + H_2O + CO_2 \textit{ yields } H_2S + CaSO_4 + CaCO_2 + S \qquad \text{Eq. 1}$$

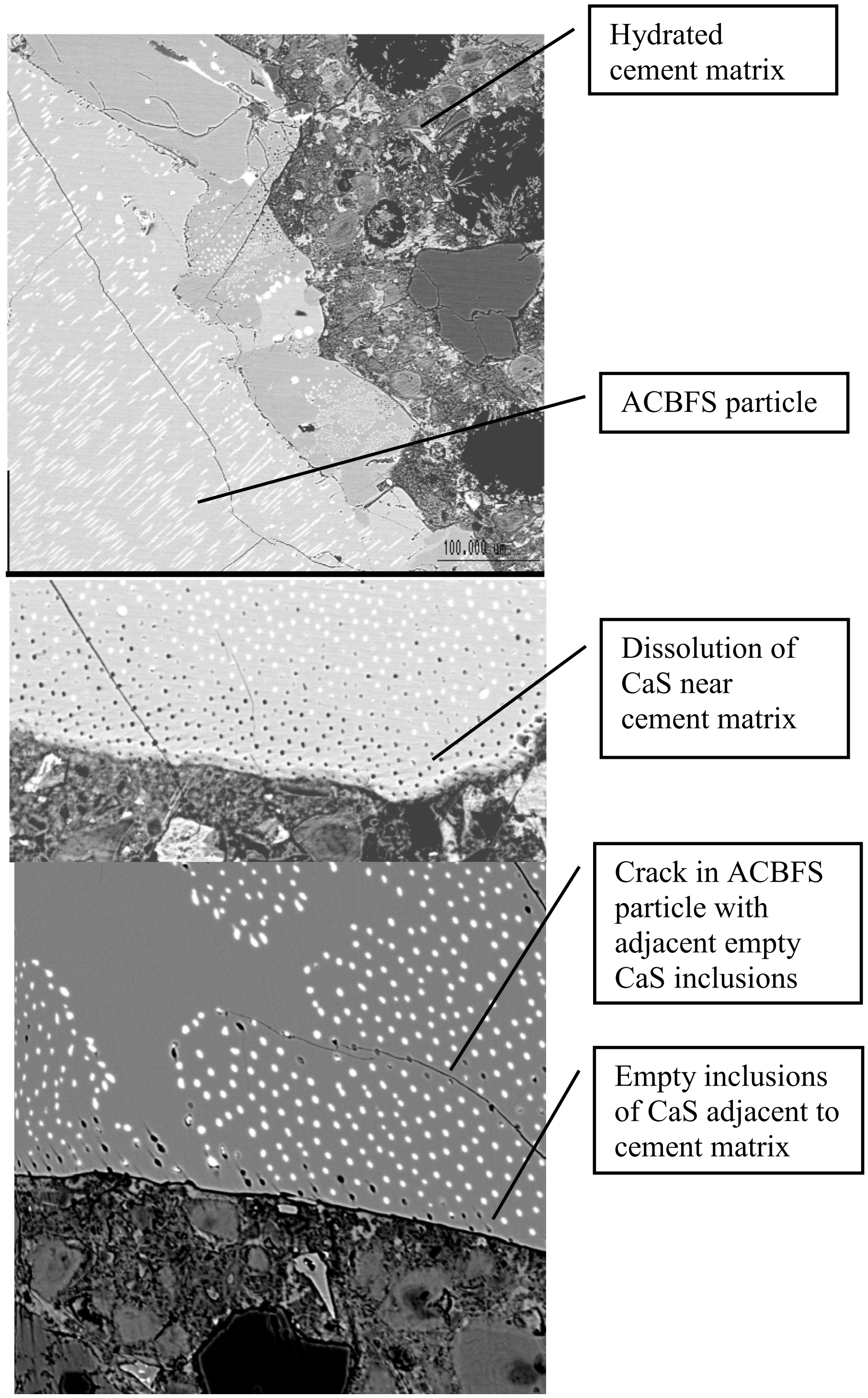

Figure 14. Reflected-light images and a backscatter electron image.
(From D. M. Hammerling 1999, p. 42, figures 5.1 and 5.2. © Karl Peterson 1999.
Adapted with permission.)

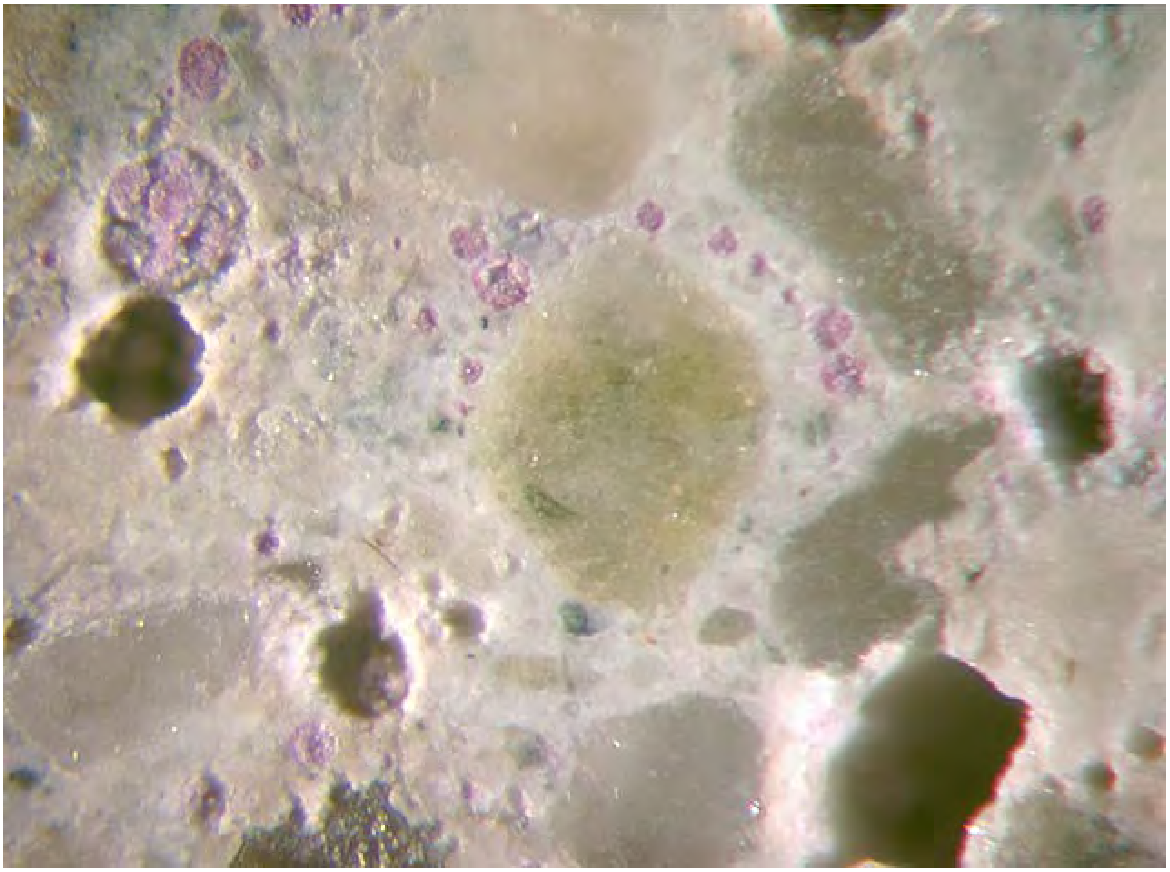

Figure 15. Stereomicroscope image of polished slab surface. Air voids are filled with a sulfate mineral, most likely ettringite, stained pink by potassium permanganate, magnified 83x. (From T. J. Van Dam, K. R. Peterson, L. L. Sutter, and N. Buch 2002. © Michigan Department of Transportation. Reprinted with permission.)

Skalny, Marchand, and Odler (2002) formulated that in concrete exposed to water that contains calcium sulfate ($CaSO_4$), the existing monosulfate ($4CaO, Al_2O_3 \cdot SO_3 \cdot 12H_2O$) phase converts to ettringite ($6CaO \cdot Al_2O_3 \cdot SO_3 \cdot 32H_2O$) in a surface-close region whose thickness increases with time according to the following equation:

$$4CaO \cdot Al_2O_3 \cdot SO_3 \cdot 12H_2O + 2Ca^{2+} + SO_4^{2-} + 24H_2O \textit{ yields } 6CaO \cdot Al_2O_3 \cdot SO_3 \cdot 32H_2O \qquad \text{EQ. 2}$$

As all of the calcium ions needed for the monosulfate to ettringite conversion are supplied by calcium sulfate, no additional calcium ions need originate from the cement hydration products (i.e., calcium hydroxide or calcium silicate hydrate [C-S-H]). Thus, unlike sulfate attack that occurs with alkali sulfates, no decalcification of the C-S-H phase takes place in calcium sulfate attack, and the integrity of the hydrated cement phases remain preserved.

Under these conditions, the initial manifestation of the interaction between hydrated portland cement paste and calcium sulfate solution is an increase in strength, as the pores of the paste become filled with newly formed ettringite. As the ettringite forms only in available space, no significant stresses are generated. But it is uncertain what might occur once the available pore space is filled or if evaporation leads to crystallization of secondary minerals within the pore space.

Hammerling (1999) discussed how later ettringite formation is often associated with the deterioration of concrete structures. It was observed through optical microscopy that ettringite and gypsum (a form of hydrated calcium sulfate, $CaSO_4 \cdot 2H_2O$) were found together in the vesicles of weathered ACBFS collected near Sault Ste. Marie, Ontario. Gypsum was the main weathering product observed, with ettringite seen in small quantities often interconnected with gypsum. In some cases the ettringite was observed growing in small veins within larger gypsum crystals. The presence of gypsum and ettringite in the weathered ACBFS demonstrates that calcium sulfide is indeed soluble under the right conditions. Small amounts of SiO_2 were also found in the ettringite, possibly in a solid solution with thaumasite ($Ca_6[Si(OH)_6](SO_4)_2(CO_3)_2 \cdot 24H_2O$). This is consistent with observations reported by Crammond (2002), who described the presence of thaumasite in old sulfide-bearing blast furnace slag.

The solubility of calcium sulfide has been calculated. Figure 16 demonstrates how calcium sulfide's solubility increases as pH and pe increase. The pe is an indicator of the tendency of a solution to either gain or lose electrons. A positive pe indicates that the solution is oxidizing, meaning that it can be considered to have a deficiency in aqueous electrons, promoting the yielding of electrons to the solutions by ions being oxidized. A negative pe indicates that the solution is reducing and can be considered to have an excess of aqueous electrons, promoting the acceptance of electrons by ions being reduced.

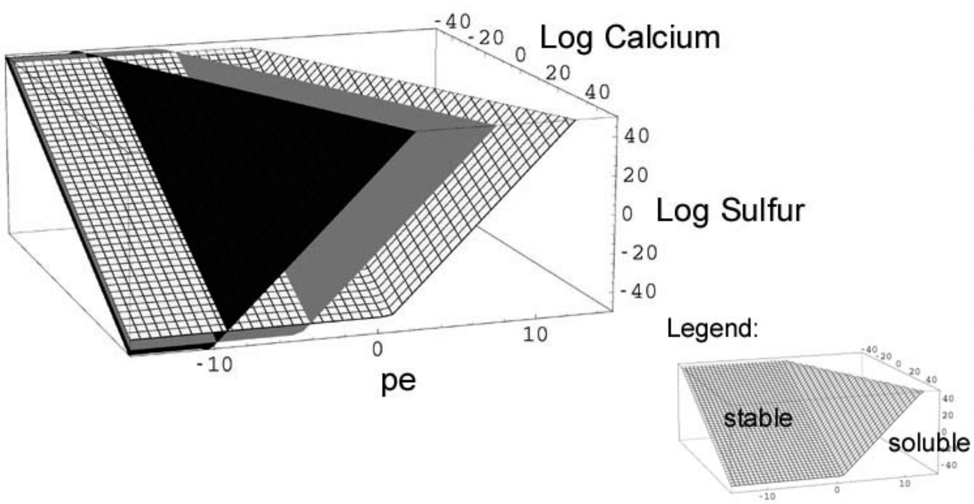

Figure 16. Phase diagram of the solubility of calcium sulfide at three different pH values. The white surface represents pH = 3, the gray one pH = 8, and the black one pH=13. A trend of increasing solubility with increasing pH values is observed.
(From Hammerling 1999. © Dorit Hammerling 1999. Adapted with permission.)

In the presence of water, calcium sulfide is only stable in a reducing, low pH environment, which allows the sulfur and calcium concentrations to be very high without ettringite formation. As pH increases, ettringite will form at lower concentrations of sulfur and calcium.

Having established that the dissolution of calcium sulfide occurs, it is still uncertain how this may be potentially harmful to concrete, noting that the reduction of pore volume actually increases strength as discussed earlier. Two possible scenarios have been considered. The first is that sufficient infilling of the air-void system with secondary ettringite may hamper its ability to protect the concrete against freeze-thaw damage. It is debated whether an air-void system can be sufficiently filled with secondary ettringite to make it ineffective, with good arguments supporting both sides. Recent work conducted at Purdue University on concrete made with both limestone and ACBFS aggregates has drawn a link between secondary ettringite infilling and freeze-thaw damage (Olek et al. 2009). In the petrographic work reported by Grove, Bektas, and Gieselman (2006), it is also suggested that the infilling of the air-void system is at least partially responsible for the observed damage. Similarly, Lankard (2010b) found "abnormally large amounts of ettringite" infilling air voids in some of the concrete he evaluated that contained ACBFS coarse aggregate and concluded that in one case, the infilling was sufficient to compromise freeze-thaw durability. Research published by Stark and Bollmann (1999) and Ouyang and Lane (1999) also supports this conclusion. On the other hand, some researchers have stated that it is impossible to sufficiently fill an air-void system with secondary ettringite to the point where it becomes compromised (Detwiler and Powers-Couche 1999). Thus, the research findings on this topic are mixed, but it appears possible that even if an adequate air-void system is initially created in the concrete, it can be compromised through secondary ettringite infilling, particularly if the air void system is marginal to begin with.

A second potential distress mechanism is that the dissolution of calcium sulfide might lead to a form of internal sulfate attack, in which sulfate ions in solution negatively interact with hardened cement paste constituents, either through the formation of expansive minerals such as ettringite or through thaumasite formation. Lankard (2010b) specifically considers the expansive formation of ettringite within the cement paste that results when sulfur supplied by the ACBFS aggregate interacts with aluminate phases (monosulfate) as one of the primary distress mechanisms in the Ohio pavements that were studied.

The latter mechanism, thaumasite formation, is gaining particular interest of late, primarily due to the addition of limestone ($CaCO_3$) to portland cement, which provides the carbonate needed for thaumasite formation. A possible relationship between the calcium sulfide present in ACBFS and thaumasite-formation sulfur attack (TSA) has been made by some (Thaumasite Expert Group 1999), although reasonable doubt exists whether this is really an issue in field-placed concrete containing ACBFS. Nevertheless, the presence of thaumasite has been reported in weathered ACBFS in several instances (Hammerling et al. 2000; Crammond 2002). The proposed relationship is based on work conducted by various researchers who found that the combination of relatively high sulfate levels and alkaline conditions bring about the formation of thaumasite (Gaze and Crammond 2000; Crammond 2003; Hill et al. 2003; Zhou et al. 2006). Both these conditions exist in concrete made with high-alkali cement and ACBFS, and the potential for such distress increases as the limestone content of cement increases. TSA is accompanied by a reduction in the binding ability of the cement in the hardened concrete, causing loss of strength. The expansive disruption that is normally associated with sulfate attack sometimes accompanies the formation of thaumasite, but is not a characteristic feature (Crammond 2002).

To address the potentially adverse effects of calcium sulfide present in ACBFS, organizations have implemented limits on sulfur content of concrete aggregates. The British Standards Institute has adopted a total sulfur limit specification of 2 percent by weight, influenced by a series of exposure tests that evaluated the expansion of concrete made with ACBFS aggregates with varying sulfur contents as displayed in figure 17 (Hammerling 1999, based on Parker 1950). Parker concluded that during exposure to weathering, the sulfate content increased. This indicates oxidation of the sulfides. He also concluded that crystalline sulfide is more susceptible to oxidation. Lea also observed unsoundness in concrete samples containing calcium sulfide in its crystalline form but not in its dendritic form (Lea 1970).

The Organization for Economic Co-operation and Development (OECD) published a report that identified sulfur and sulfate content of ACBFS as an important factor that affects performance of concrete (OECD 1997). The report states that ACBFS must have a total sulfur content less than 2 percent and a sulfate content less than 0.7 percent in order for ACBFS to be used as a concrete aggregate. If the ACBFS is used in the unbound state, it may have a soluble sulfate content of more than 2g/liter (OECD 1997). The Japanese Standards have similar limits, allowing maximum total sulfur content of 2.0 percent and maximum percent of acid-soluble sulfates of 0.5 percent (JIS 2003).

The European Committee for Standardization (CEN) Standard for Aggregates in Concrete, EN 12620, specifies an upper total sulfur limit of 2 percent for ACBFS aggregates and also specifies the level of acid-soluble sulfate (CEN EN12620 2002). This standard is designed to guard against the inclusion of either sulfide or sulfate-bearing impurities within aggregate sources for use in concrete/concrete products either above or below ground (Thaumasite Expert Group 1999). The standard recommends that the ACBFS aggregate be tested a minimum of twice per year for sulfur-containing compounds, whereas other aggregates are recommended for testing once a year. Justification for the increased frequency for testing ACBFS is not provided, but is assumed that it is based on the variability of the material's properties. The standard also states that a substantial proportion of the sulfate in crystalline ACBFS is encapsulated in the slag grains and therefore plays no part in the hydration reactions of cement. This standard does not acknowledge an effect from leaching of sulfate over time or when the ACBFS aggregate is cut or cracked as will occur on jointed concrete pavements. They further reason that a higher proportion of sulfate is tolerable in ACBFS. Under certain circumstances other sulfur compounds present in the aggregates can oxidize in the concrete to produce sulfates. It is stated that these can give rise to expansive disruption in concrete but that this is less likely in ACBFS produced in modern production units (CEN EN12620 2002).

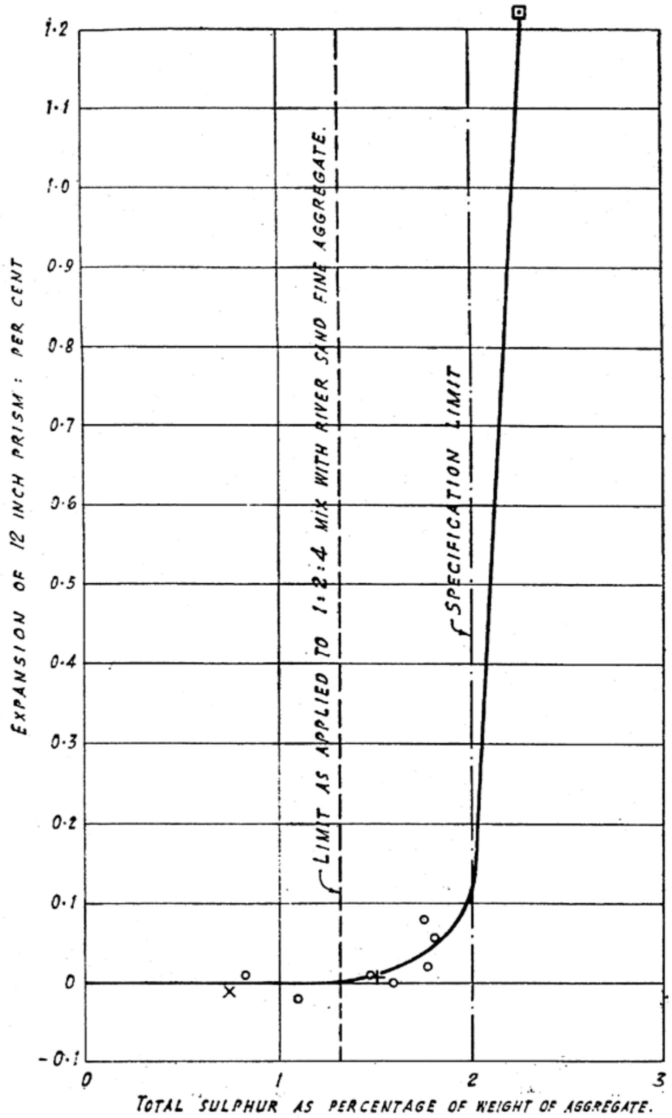

Figure 17. Expansion of concrete specimens made with slags of different sulfur content; results from experiments by Parker. (T. W. Parker 1950, as cited by Hammerling 1999.)

Summary of Chemical Properties

In summary, potential materials-related distress (MRD) mechanisms have been identified that may link chemical properties of ACBFS in concrete to durability issues. It is recognized that ACBFS has been used as a coarse aggregate in concrete pavements for decades with little reported occurrence of MRD being directly linked to the ACBFS. The dissolution of calcium sulfide is considered to pose the most serious risk, with calcium sulfide becoming more soluble in a high-alkali environment. Once dissolution occurs, ettringite formation will result, possibly contributing to the occurrence of paste freeze-thaw attack or an internal sulfate attack or both. European standards address this by putting limits on the total sulfur content (2 percent) of ACBFS to be used in concrete.

INFLUENCE OF PHYSICAL PROPERTIES OF AIR-COOLED BLAST FURNACE SLAG ON CONCRETE

A number of physical properties of ACBFS differ significantly from those of naturally derived aggregates and thus influence the fresh and hardened properties of concrete as discussed below.

Influence on Fresh Concrete Properties

An important consideration in working with ACBFS as a coarse aggregate is its particle-to-particle variability. Individual ACBFS aggregate particles vary widely, from being highly porous and light to being extremely dense. The crushing and blending operations typically ensure that the variation of large batches of aggregate is relatively small (personal communication, J. Wigdahl, 2009), but the variability of individual particles must be considered when testing small amounts of aggregate or concrete, and thus statistically representative samples must be collected.

As described earlier, a dominant characteristic of ACBFS is the rough, porous surface texture of the aggregate particles that results in increased surface area. One result of this is that additional mortar (cementitious materials, water, and sand) is often needed when proportioning concrete containing ACBFS to overcome the high angularity/surface area of the particles and maintain workability. The use of water-reducing admixtures can assist in this effort to some degree, but it has been observed that it is almost always necessary to have some additional mortar to "coat" the ACBFS surfaces to create a workable mixture.

As discussed, the particle roughness/surface porosity also results in a high water-absorption capacity for ACBFS aggregate, as high as 7 to 8 percent. It is known that if ACBFS aggregate is batched dry during concrete production, stiffening will result and it may lead to early-age cracking as water is absorbed from the paste by the aggregate. ACI Standard 221 recommends saturation of porous aggregate (those with absorptions in excess of 2.5 percent) and proper wetting of stockpiles to minimize such absorption to avoid early-age shrinkage (ACI 2006). This is absolutely essential if ACBFS aggregate is to be used in paving concrete, as these mixtures are necessarily stiff (with typical slumps between 0.5 and 1.5 in. (13 and 38 mm)) and absorption of even a small amount of mix water by the aggregate will result in an unworkable mixture. The ACBFS aggregate moisture content must be closely monitored during batching to maintain the desired water-to-cement ratio. The practical consequence of this is that during construction, ACBFS stockpiles must be kept wet. This requires the contractor to maintain an extra level of stockpile management that is not necessarily required when naturally derived aggregates are used.

Grove, Bektas, and Gieselman (2006) cited a discussion at the Michigan Concrete Paving Association Annual Workshop that indicated that the dry aggregate problem had been addressed in more recent construction projects through diligent wetting of the ACBFS stockpiles. They went on to propose a theory that saturated, highly absorptive aggregate can provide a significant benefit to concrete during the curing process as this abundance of water within the aggregate pores acts as an internal source of cure water, promoting cement hydration beyond the normal curing period. This will ultimately increase strength and reduce permeability of the concrete. It is noted that work on internal curing is based almost exclusively on the use of porous, light-weight, fine aggregate and not coarse aggregate, so this advantage is only speculative at this time.

In the ACI publication *Guide for Structural-Lightweight Aggregate Concrete* (ACI 2003), structural lightweight aggregate is defined as fine aggregate having a bulk density less than 70 lb/ft^3 (1,121 kg/m^3) and coarse aggregate having a bulk density of less than 55 lb/ft^3 (881 kg/m^3). There is some similarity between lightweight aggregates and ACBFS aggregate because both have a vesicular nature; however, ACBFS is not considered a lightweight aggregate because of its higher density. Yet elements of this document are of relevance when evaluating ACBFS as a coarse aggregate. For one, ACI 213R emphasizes the importance of prewetting the aggregate to minimize absorption of mix water into the aggregate during concrete production and placement. Also, ACI 213R indicates that dry aggregates may take several days to achieve an adequate prewetted state once subjected to sprinkling or soaking methods. Furthermore, ACI 213R discusses the benefits of internal curing that occur when lightweight aggregates are used in concrete because the water present in the aggregate is not a part of the mixing water, yet it becomes available for hydration once the concrete has set. This occurs as the pore system in the hydrating cement paste becomes more refined with time, drawing the water present within the larger pores of the lightweight aggregate out under capillary action. Thus, the curing period is extended, improving the quality of the hydrated paste and leading to increased strength and reduced permeability. ACI 213R summarizes a series of publications that indicate that the permeability of concrete containing lightweight aggregate is generally equal to or significantly lower than that for normal weight concrete specimens.

One final construction concern is that it is recommended that the pressure method (AASHTO T 152) should not be used to monitor total air content during construction, as it will give erroneous results. Instead the air content of freshly mixed concrete should be measured by the volumetric method (AASHTO T 196) if ACBFS is used in the concrete.

Influence on Hardened Concrete

Strength and Stiffness

A number of studies have evaluated the strength and stiffness properties of concrete made with ACBFS as a coarse aggregate. One of the first is described in a report prepared in 1933 by the Michigan State Highway Laboratory (MSHL) that summarized the results of a study performed to compare the flexural and compressive strength of concrete made with ACBFS and gravel aggregate (MSHL 1933). The ACBFS aggregate was obtained from two sources: the Detroit Slag and Dock Company (Detroit, Michigan) and the France Stone Company (Toledo, Ohio). The gravel was obtained from Killin's Gravel Company in Ann Arbor, Michigan. Concrete mixes were prepared using each aggregate type with the gradation of the coarse aggregate being the same for all three mixes. Table 6 shows the properties of the mixes. The report did not indicate if the concrete had any air entrainment, but given the time period, it is likely that it was not entrained.

Table 6. Properties of the PCC Mixes in the Michigan State Highway Laboratory Study (MSHL 1933)

Parameter	**Mix 1**	**Mix 2**	**Mix 3**
Coarse aggregate	Slag	Slag	Gravel
Source of coarse aggregate	France	Detroit	Killin's
Specific gravity of coarse aggregate	2.55	2.47	2.65
Weight of aggregate (loose) per cubic foot	81	81	99
Cement (lb) per cubic yard	564	564	564
Sand (lb) per cubic yard	1,578	1,530	1,284
Coarse aggregate (lb) per cubic yard	1,620	1,620	2,034
Water (lb) per cubic yard	263	259	242
Slump (in.)	1.75	1.09	1.36

Twenty test beams and 20 cylinders were prepared from each mix. To obtain similarly workable mixes (this was before water-reducers were available), more sand and water were required with the ACBFS aggregate than with the gravel coarse aggregate. This increased the water-to-cementitious materials ratio (*w/cm*) for Mix 1 and Mix 2 to 0.466 and 0.459 compared to 0.429 for Mix 3. The samples were cured in a moist curing room where the average temperature during the curing period was 81 °F (27 °C). Half of the samples were tested at 7 days, and the other half were tested at 28 days. The average compressive strength values and the flexural strength values are shown in tables 7 and 8, respectively.

Table 7. Compressive Strength Test Results (MSHL 1933)

Mix No.	**Coarse Aggregate**	**Aggregate Source**	**Average Compressive Strength (lbf/in^2)**	
			7-Day	**28-Day**
1	ACBFS	France Stone Co.	4,092	5,449
2	ACBFS	Detroit Slag and Dock Co.	4,380	5,571
3	Gravel	Killin's	4,403	5,841

Table 8. Flexural Strength Test Results (MSHL 1933)

Mix No.	**Coarse Aggregate**	**Aggregate Source**	**Average Flexural Strength (lbf/in^2)**	
			7-Day	**28-Day**
1	ACBFS	France Stone Co.	539	642
2	ACBFS	Detroit Slag and Dock Co.	616	684
3	Gravel	Killin's	613	757

The report does not indicate if the flexural strength tests were performed using center-point or third-point loading. Currently, MDOT performs flexural strength tests using the center-point method.

At 7 days, Mix 2 (Detroit Slag) had compressive and flexural strength results similar to those of Mix 3 (Killin's Gravel). Mix 1 (France Slag) had compressive and flexural strength test results considerably lower than those of Mix 3 (Killin's Gravel). The 28-day compressive strengths for both ACBFS aggregate mixes were slightly less than that of the gravel aggregate mix. The 28-day flexural strengths for the samples with ACBFS aggregate were considerably less than those of the samples with gravel aggregate. Mix 1 and Mix 2 had 28-day flexural strengths lower than that of Mix 3 by 15 and 10 percent, respectively.

This study concluded that concrete mixtures with ACBFS aggregate have lower strengths than concrete containing gravel aggregate, indicating that lower strength was expected for the ACBFS aggregate mixes because of the higher sand and the higher *w/cm* that was used in the concrete mix. As discussed, these modifications were necessary to create similarly workable mixes, a situation that could be addressed today through the use of water reducing admixtures.

In another, larger study, Oehler and Finney (1953) reported flexural strength data for several projects constructed in 1953 in Wayne County, Michigan, that used dolomite, Green Oaks Gravel, and Ford ACBFS as the coarse aggregate. The concrete used in these projects had 5 to 7 percent air content. The projects that used ACBFS and gravel aggregate used 6.25 sacks of cement per cubic yard (4.78 sacks per cubic meter), while the projects that used dolomite used 6.05 sacks per cubic yard (4.62 sacks per cubic meter). The average flexural strengths at 7 and 28 days from samples cast for these projects are shown in table 9. The tested specimens were prepared and cured under field conditions.

Table 9. Wayne County Flexural Strength Test Results
(Oehler and Finney 1953)

Aggregate	**No. of Projects**	**No. of Samples**	**Average Flexural Strength (lbf/in^2)**	
			7-Day	**28-Day**
Dolomite	4	60	648	766
Green Oaks Gravel	2	54	559	708
Ford ACBFS	1	190	561	648

The table shows the number of projects where each type of aggregate was used and the total number of samples that were tested. The authors reported that Green Oaks Gravel was not considered to be one of the better gravels for concrete work. The flexural strength of concrete containing ACBFS aggregate had the lowest flexural strength at 28 days. The report does not indicate if the flexural strength tests were performed using center-point or third-point loading. The *w/cm* for the various mixtures is not reported.

Oehler and Finney (1953) also reported the results of a laboratory study performed by the Michigan State Highway Department's testing laboratory in Ann Arbor that compared flexural strength of concrete made with Ford ACBFS and limestone aggregate. Four concrete batches were used for each aggregate type, and two beams were cast from each batch. Table 10 shows the average values of the test results from this study. It was reported that the department's specification required a flexural strength value of 550 (3.79 MPa) and 650 lbf/in^2 (4.48 MPa) at 7 and 28 days, respectively. The report does not indicate whether the flexural strength tests were performed using center-point or third-point loading. The flexural strength of the beams made from ACBFS aggregate was approximately 14 percent less than those of the beams made from limestone aggregate for both 7 and 28 days. The report does not provide the *w/cm* of the concrete mixes, but does report the slump value of each mixture with the average slump of the four concrete mixes containing ACBFS aggregate being similar to the average slump of the concrete mixes containing limestone aggregate.

Table 10. Michigan State Highway Department Flexural Strength Comparative Study of ACBFS Aggregates (Oehler and Finney 1953)

Aggregate	Cement Content (sacks/cyd)	Air Content	Average Flexural Strength (lbf/in^2)	
			7-day	28-day
Limestone	5.57	5.2	706	785
Ford ACBFS	5.54	5.3	609	680

Oehler and Finney (1953) also report on a study using ACBFS aggregate that investigated the flexural strength of concrete made with varying cement contents and slumps. The results from this study are shown in table 11. The samples with higher slumps had lower flexural strengths at 28 days. The slump of the samples varied, indicating that it is likely that the *w/cm* was not held constant from mix to mix, making direct strength comparisons based on aggregate type questionable.

Table 11. Michigan State Highway Department Flexural Strength Study of ACBFS Aggregates With Varying Cement Content (Oehler and Finney 1953)

Sample	Slump (in.)	Cement Content (Sacks/cyd)	Air Content (%)	Flexural Strength (lbf/in^2)	
				7-day	28-day
1	1 to 2	5	4 to 5	532	641
2	1 to 2	5.5	4 to 5	595	662
3	3 to 4	5.45	4 to 5	550	629
4	3 to 4	5.9	4 to 5	553	634

In a study completed by Construction Technology Laboratories (CTL) for the Edw. C. Levy Company (CTL 1991), two similar concrete mixtures were prepared, one with a limestone coarse aggregate (*w/cm* = 0.434) and another with an ACBFS coarse aggregate (*w/cm* = 0.450). The strength and stiffness data for the two concretes are presented in table 12.

Table 12. Strength and Stiffness Data for Concrete Made With Limestone and ACBFS Aggregate
(From CTL 1991. © Edw. C. Levy Co. 1991. Reprinted with permission.)

Mixture	**Compressive Strength (lbf/in^2)**		**Flexural Strength (lbf/in^2)**		**90-day Modulus of Elasticity (lbf/in^2)**
	28-day	**90-day**	**28-day**	**90-day**	
Limestone	6,330	7,370	500	580	$6.18x10^6$
ACBFS	6,110	6,840	490	490	$4.32x10^6$

It can be seen that although the 90-day compressive strength of the two concrete mixes are close to each other, the 90-day flexural strength and stiffness are significantly lower for the concrete made with ACBFS, possibly reflecting the use of a slightly higher *w/cm*.

In a recent study to determine the coefficient of thermal expansion (CTE) of typical concrete paving mixtures used in Michigan, Buch and Jahangirnejad (2008) fabricated samples in the laboratory with seven different coarse aggregate sources. As a part of this study, three replicate concrete samples of each coarse aggregate type were tested at 3, 7, 14, 28, 90, 180, and 365 days to determine compressive strength, flexural strength, split tensile strength, and the elastic modulus. The tests for compressive strength, flexural strength, split tensile strength, and elastic modulus were performed in accordance with ASTM test methods C39, C78 (third-point loading), C496, and C469, respectively. In this study, the *w/cm* of the various concrete mixes was held close to each other.

Table 13 shows the average 7-day and 28-day test results for compressive strength and flexural strength for the samples made from the seven aggregate sources. Table 14 shows the average 28-day values for compressive strength, split tensile strength, flexural strength, and elastic modulus.

Table 13. The 7-Day and 28-Day Average Compressive and Flexural Strengths
(From Buch and Jahangirnejad 2008; © Michigan Department of Transportation 2008. Reprinted with permission.)

Mix ID	**Aggregate Type**	**Compressive Strength (lbf/in^2)**		**Flexural Strength (lbf/in^2)**	
		7-Day	**28-Day**	**7-Day**	**28-Day**
CTE 1	Limestone	4,619	5,129	683	836
CTE 2	Gravel	3,790	4,965	651	692
CTE 3	Dolomitic Limestone	3,336	3,967	639	645
CTE 4	ACBFS	4,416	5,169	687	831
CTE 5	Dolomite	3,271	4,035	619	731
CTE 6	Gabbro (Trap Rock)	3,902	5,125	633	731
CTE 7	Dolomite	4,431	5,825	680	820

Table 14. The 28-Day Average Strength Properties
(From Buch and Jahangirnejad 2008. © Michigan Department of Transportation 2008.
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Mix ID	Coarse Aggregate	Compressive Strength (lbf/in^2)	Split Tensile Strength (lbf/in^2)	Flexural Strength (lbf/in^2)	Elastic Modulus (million lbf/in^2)
CTE 1	Limestone	5,129	516	836	4.50
CTE 2	Gravel	4,965	502	692	4.89
CTE 3	Dolomitic Limestone	3,967	489	645	4.57
CTE 4	ACBFS	5,169	507	831	4.66
CTE 5	Dolomite	4,035	511	731	4.65
CTE 6	Gabbro (Trap Rock)	5,125	500	731	5.39
CTE 7	Dolomite	5,825	561	820	4.48

The following observations were noted for the concrete containing ACBFS based on the 28-day test results: (1) it had the second-highest compressive strength; (2) it had the fourth-highest split tensile strength; (3) it had the second-highest flexural strength; and (4) it had the fourth-highest elastic modulus. These results indicate that the *w/cm* is of primary importance with regard to the strength of hardened concrete, a result consistent with previous research. The importance of aggregate type is secondary, with concrete made with ACBFS having very favorable strength properties.

The question has been frequently raised as to whether or not glassy particles in ACBFS are detrimental to concrete strength. In considering the results given in table 15, it should be remembered that ACBFS particles vary widely in other characteristics as well as in surface texture (Allen 1948). It can be seen from table 15 that when the *w/cm* is held constant, the amount of glassy particles (ranging from 0 to 30 percent) does not appear to have an impact on either the compressive strength or the flexural strength of concrete.

Table 15. Effect of Glassy Particles in ACBFS on Strength of Concrete
(Allen 1948)

Glassy Particles (%)	Cement Factor	Water-Cement Ratio (gal per sack)	Slump (in.)	Compressive Strength, 28-day (lbf/in^2)	Flexural Strength, 28-day (lbf/in^2)
0	6.3	5.67	4.5	4,354	665
15	6.3	5.67	4.5	4,179	698
30	6.3	5.67	6	4,390	675

The Alabama Department of Transportation (ALDOT) requires that ACBFS furnished for use in wearing surface layers in asphalt concrete shall be restricted in its glassy particles content to the values shown in table 16 (ALDOT 2008). The procedure for determining the glassy particles is described in ALDOT test method 321, *Test for Glassy Particles in Crushed Slag* (ALDOT 321 1994). In this test, the ACBFS sample is dried and reduced to testing size in accordance with AASHTO T 248, and sieved in accordance with AASHTO T 27. Glassy particles are visually identified as particles that have a glossy, slick, nonporous glassy finish on any face. The percentage of glassy particles is computed as the glassy particles retained on the No. 4 sieve on the basis of the total sample weight.

Table 16. Glassy Particle Content Specifications
(From ALDOT 2008. © Alabama Department of Transportation 2008.
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Type of Wearing Surface	Glassy Particle Content
Surface treatments	10% Maximum
Open-graded polymer-modified	10% Maximum
Dense-graded polymer-modified	25% Maximum

The Virginia Department of Transportation (VDOT) Road and Bridge Specifications (VDOT 2007) specify that ACBFS shall be relatively free from foreign minerals and glassy or spongy pieces, having a limit of 4 percent for glassy particles for ACBFS to be used in concrete. Penn-DOT specifies that pieces of slag containing more than 50 percent glass are considered glassy particles (PennDOT 2000). Waste glass is also considered glassy particles. According to the PennDOT specification, the maximum allowable glassy particles in ACBFS is 4 percent when used for concrete and 10 percent for other uses; however, coarse aggregate containing glassy particles consisting of waste glass is not permitted for use in concrete or bituminous wearing courses.

Coefficient of Thermal Expansion of Concrete

A number of studies have measured the CTE of concrete made with ACBFS. The reported CTE of ACBFS is between 5.9 and 6.0 x 10^{-6} in/in/°F (3.2 and 3.3 x 10-6 mm/mm/°C) (USDIBM 1949). Table 17 shows the typical CTE ranges for various coarse aggregates and concrete made with these aggregates (Hall and Tayabji 2011). The CTE of concrete made with ACBFS falls within the typical range of CTE values for concrete made with dolomite and sandstone. Concrete containing quartz sands, gravels, and quartzite exhibit higher CTE values than concrete containing ACBFS coarse aggregate. However, concrete containing the following aggregates exhibit lower CTE values than concrete containing ACBFS aggregate: limestone, granite and gneiss, syenites, diorites, andesite, basalt, gabbros, and diabase.

In one study, two specimens from a test site on eastbound I-96 in Michigan that were tested using the FHWA CTE test procedure had CTE values of 5.3 and 5.6 × 10^{-6} in/in/°F (2.9 and 3.1 × 10^{-6} mm/mm/°C), which are considered to be at the higher end of the typical range for aggregates (Smiley 2007). Buch and Jahangirnejad (2008) performed a laboratory study to determine the CTE of typical concrete paving mixtures used in Michigan made with coarse aggregates from eight different sources: limestone (two sources), dolomite (three sources), ACBFS (one source), gravel (one source), and gabbro/trap rock (one source). The 28-day CTE values determined in this study for the eight aggregate types for the second and third test cycles, as well as the average CTE of these two cycles, are shown in table 18.

Table 17. Coefficient of Thermal Expansion (CTE) of Concrete by Aggregate Type
(LTPP Standard Date Release 25.0)
(From Hall and Tayabji 2011, p. 2)

Primary Aggregate Class	**Average CTE (/°F x 10-6)**	**Standard Deviation (s) (/°F x 10-6)**	**Average CTE (/°C x10-6)**	**Standard Deviation (s) (/°C x 10-6)**	**Sample Count[1]**
Andesite	4.32	0.42	7.78	0.75	52
Basalt	4.33	0.43	7.80	0.77	141
Chert	6.01	0.42	10.83	0.75	106
Diabase	4.64	0.52	8.35	0.94	91
Dolomite	4.95	0.40	8.92	0.73	433
Gabbro	4.44	0.42	8.00	0.75	8
Gneiss	4.87	0.08	8.77	0.15	3
Granite	4.72	0.40	8.50	0.71	331
Limestone	4.34	0.52	7.80	0.94	813
Quartzite	5.19	0.50	9.34	0.90	131
Rhyolite	3.84	0.82	6.91	1.47	7
Sandstone	5.32	0.52	9.58	0.94	84
Schist	4.43	0.39	7.98	0.70	30
Siltstone	5.02	0.31	9.03	0.56	21
				Total Sample Count	2,251

1. A total of 2,991 CTE values are available in the Long-Term Pavement Performance (LTPP) Standard Data Release 25.0 (January 2011); 628 CTE values were not used due to aggregate class not defined or only one sample available for the primary aggregate type, and 112 CTE outlier values were also not included in the table.

Table 18. CTE Values for Concrete Made With Different Coarse Aggregates
(From Buch and Jahangirnejad 2008. © Michigan Department of Transportation 2008. Reprinted with permission.)

Mix ID	Aggregate Type	Pit No.	28-Day CTE (μ€/°F)		
			Cycle #2	Cycle #3	Average
CTE 1	Limestone	71-47	4.54	4.55	4.55
CTE 2	Gravel	19-56	5.84	5.84	5.84
CTE 3	Dolomitic Limestone	75-05	4.51	4.47	4.49
CTE 4	ACBFS	82-19	5.69	5.73	5.71
CTE 5	Dolomite	49-65	5.91	5.91	5.91
CTE 6	Gabbro (Trap Rock)	95-10	5.40	5.38	5.39
CTE 7	Dolomite	58-11	5.93	5.77	5.85
CTE 8	Dolomite	91-06	5.90	5.87	5.89

The CTE values for concrete containing limestone, dolomitic limestone, and gabbro were lower than the CTE of concrete containing ACBFS aggregate, while concrete made with gravel or dolomite had higher CTE values than concrete made with ACBFS aggregate. These results are consistent with the values recommended in the MEPDG (AASHTO 2008).

Freeze-Thaw Durability

Freeze-Thaw Durability and Aggregate Porosity

Several publications have discussed the importance of pore characteristics of the coarse aggregate on the durability of concrete. Most researchers agree that the abundance, size, shape, and continuity of the pores determine the amount of water the aggregate can absorb, its absorption rate, its ease of draining (water-retention properties), its internal surface area, and portion of its bulk volume that is occupied by solid matter (Lewis and Dolch 1955). The size of the pores may be of greatest importance, but a critical size is difficult to specify. Some correlations between freeze-thaw durability and porosity have been found involving a critical pore diameter in the range of 0.16 to 0.20 mils (4 to 5 microns), which defines the transition between "micropores" and "macropores." Micropores are commonly thought to be harmful to concrete durability, while macropores typically are not harmful to concrete. In an attempt to clarify the debate, Walker and Hsieh (1968) ran a series of laboratory tests and concluded that 0.3 mils (8 microns) is the appropriate critical diameter.

In deriving this conclusion, Walker and Hsieh (1968) found that ACBFS did not follow the typical relationship between pore characteristics and durability. ACBFS continued to perform reasonably well after undergoing 100 freeze-thaw cycles in relationship to absorption, pore index, and total porosity. As seen in figure 18, aggregate H (ACBFS) possessed high absorption, high pore index, and high total porosity while still contributing to a durable concrete product. This is not typical of the empirical relationship developed for naturally derived aggregates in which concrete experienced lower durability as absorption and pore index increased. Total porosity did not show a correlation for any of the aggregates tested.

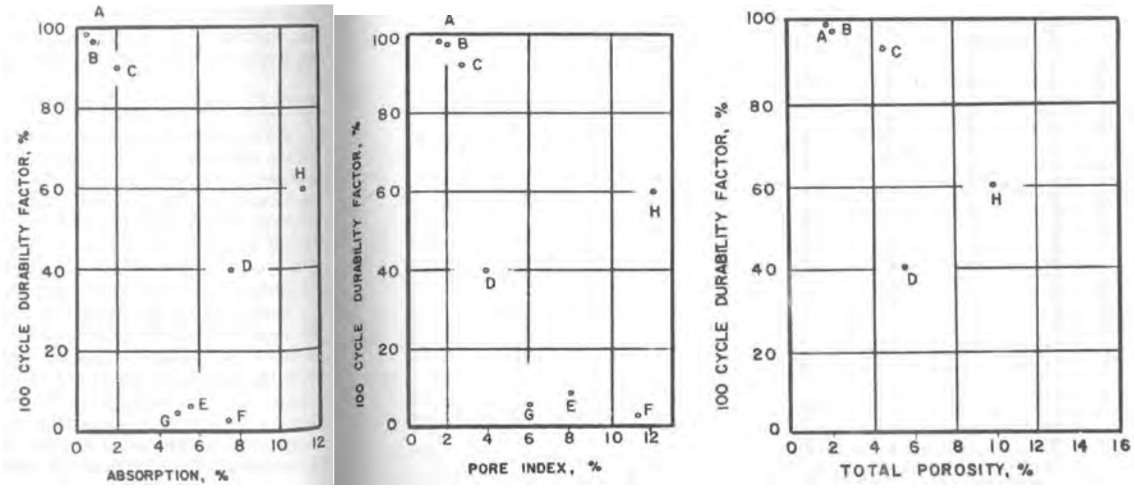

Figure 18. ACBFS coarse aggregate's (H) relationship between pore characteristics and durability as subject to freeze-thaw testing.
(From R. Walker and T. Hsieh. Relationship Between Aggregate Pore Characteristics and Durability of Concrete Exposed to Freezing and Thawing. In *Highway Research Record 226,* Figure 1, p. 44, and Figures 2 and 3, p. 45. Copyright, National Academy of Sciences, Washington, DC, 1968. Reproduced with permission of the Transportation Research Board.)

Walker and Hsieh (1968) reasoned that due to the vesicular nature of ACBFS, the pores are less easily filled with water, and thus have a decreased tendency to cause damage, and that the many large pores do not fill with water as they probably are able to drain by gravity.

In another study on natural aggregates, Kaneuji, Winslow, and Dolch (1980) also determined a relationship between freeze-thaw durability of coarse aggregate and its pore structure. They used mercury intrusion porosimetry to determine both the pore volume and the size of pores of several natural aggregates samples obtained from different sources in Indiana. These aggregates were subjected to vacuum saturation and then used to make air-entrained concrete. The concrete was cast into prism samples, cured in a fog room for 2.5 months, and then subjected to freeze-thaw testing in accordance with ASTM C666. A durability factor for each aggregate was determined, and these durability factors were then normalized to a scale from 0 to 100, with the best aggregate assigned a durability factor of 100 to obtain an adjusted durability factor. The durability data and the pore size distributions showed some obvious correlations.

Figure 19 shows the pore size distribution of three aggregates that all have approximately the same total pore volume but different pore sizes. The adjusted durability factor is shown within parentheses next to each curve. As seen in this figure, for a constant total pore volume, a smaller pore size yields a lower durability factor. Figure 20 shows the pore size distribution of three aggregates that have similar predominant pore sizes, but differing total pore volumes, indicating that for a given pore size, a larger total pore volume results in a lower durability. These results demonstrate that both the total pore volume and the pore size of an aggregate influence the freeze-thaw durability.

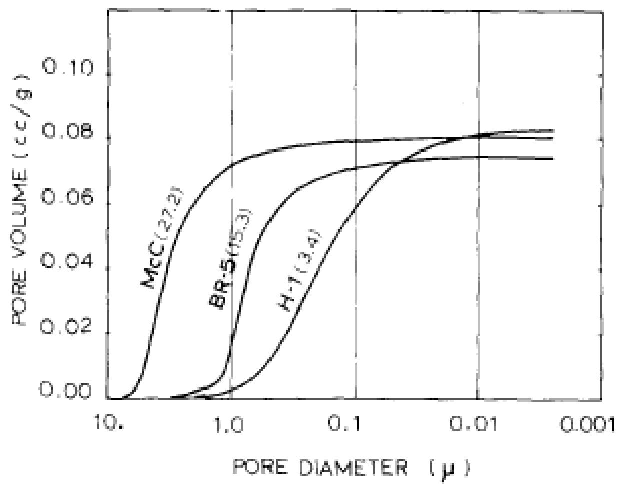

Figure 19. Pore size distribution of aggregates with similar total pore volumes. (Reprinted from *Cement and Concrete Research, 10:*3, pp. 433–41, M. Kaneuji, D. N. Winslow, and W. L. Dolch, The Relationship Between an Aggregate's Pore Size Distribution and Its Freeze Thaw Durability in Concrete, Copyright 1980, with permission from Elsevier.)

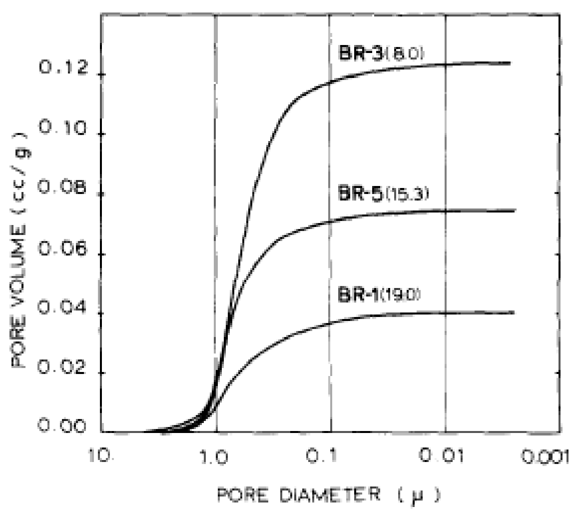

Figure 20. Pore size distribution of aggregates with similar predominant pore sizes. (Reprinted from *Cement and Concrete Research, 10:*3, pp. 433–41, M. Kaneuji, D. N. Winslow, and W. L. Dolch, The Relationship Between an Aggregate's Pore Size Distribution and Its Freeze Thaw Durability in Concrete, Copyright 1980, with permission from Elsevier.)

As a means of distinguishing between durable and nondurable aggregates, Kaneuji, Winslow, and Dolch (1980) also presented the concept of relating an expected durability factor (EDF) with the median pore size and total pore volume as shown in figure 21. Aggregates that have a pore volume and a median pore diameter such that they fall below the EDF curve should yield satisfactory freeze-thaw performance, while those that fall above this curve are expected to exhibit unsatisfactory performance. This concept differs from the commonly used absorption test, which simply sets a maximum specified pore volume level in which any aggregate that falls below the limit would be acceptable regardless of the pore size that contributes to the total pore volume.

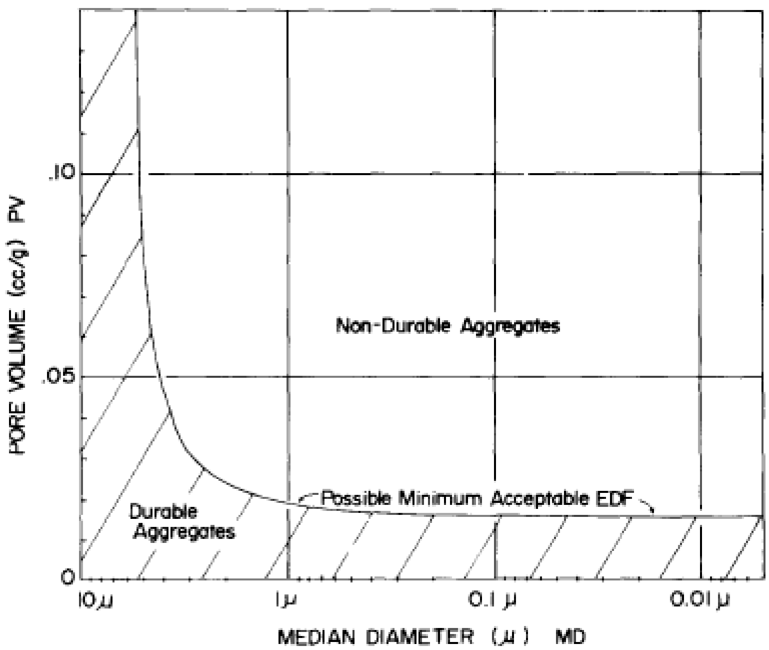

Figure 21. A constant EDF used as a criterion for separating aggregates.
(Reprinted from *Cement and Concrete Research, 10:*3, pp. 433–41, M. Kaneuji, D. N. Winslow, and W. L. Dolch, The Relationship Between an Aggregate's Pore Size Distribution and Its Freeze Thaw Durability in Concrete, Copyright 1980, with permission from Elsevier.)

Rhodes and Mielenz (1946) point out that the absorption test of bulk aggregates does not discriminate between absorption by a few highly porous particles or absorption by many moderately porous particles. Therefore, absorptions of lithologically complex aggregates, such as ACBFS, do not show quantitative relationships to soundness. This is because water moving by capillarity will not enter aggregates containing only large voids, even if these voids are interconnected and penetrable. If the voids are smaller than those in hydrated cement paste, the water will be preferentially drawn from the paste into the aggregate. If the voids are larger, they will likely remain empty even as the hydrated cement paste becomes saturated. Sweet (1948) summarized several laboratory tests and came to similar conclusions, postulating that the significance of the void size in aggregates is probably related to their water-retention capacity and capillary action as influenced by their degree of saturation. Lewis and Dolch (1955) also suggested that very large pores are not subject to significant capillary action, which affects the rate of absorption and increases the rate that water can escape from the aggregate.

Verbeck and Landgren (1960) found that some aggregates, such as ACBFS, require a long time to become critically and damagingly saturated. This is because of their porosity and pore size, which help to prolong the time required to attain critical saturation and hence successfully pass through the freezing season to the next drying and recuperating season. In tests such as the sodium or magnesium sulfate soundness tests, the mechanism of disruption is different from that resulting from the freezing of water. Such tests are based on coarse and difficult to interpret empirical correlations with concrete performance. Verbeck and Landgren (1960) further indicated that unconfined freeze-thaw tests incorporate unrealistic conditions when the aggregates are presoaked in water or subjected to a vacuum saturation procedure prior to freezing. They indicated that these tests do not take into account the time period required for aggregate to become critically saturated when in field concrete. They also state that the outcome of tests of this type depends upon several arbitrarily selected test conditions, and may yield results having little relationship to actual concrete performance.

Michigan Department of Transportation Freeze-Thaw Testing

MDOT tests the freeze-thaw durability of coarse aggregates used for concrete pavement construction using the procedures described in MDOT Test Methods MTM 113 (MTM 2007), MTM 114 (MTM 2007), and MTM 115 (MTM 2007). MTM 113 covers the moisture conditioning that is performed on coarse aggregates prior to their being used to prepare concrete. The moisture conditioning for all aggregate types (except ACBFS) requires vacuum saturation, in which aggregates are subjected to a vacuum for 1 hour while submerged in water. After 1 hour, the vacuum is released and the aggregates are then soaked for 23 hours. The moisture conditioning for ACBFS aggregate does not include vacuum saturation, instead consisting of only a 24-hour soak.

MTM 114 covers the procedure for making the concrete beams using the moisture-conditioned aggregate whereas MTM 115 covers the procedure for performing the freeze-thaw tests on the prepared beams. The beams are subjected to 300 freeze-thaw cycles or until the length of the beam reaches a 0.1 percent total dilation, whichever occurs first. For beams reaching 0.1 percent total dilation before 300 cycles, the number of cycles at that point is used to compute the dilation per 100 cycles. For tests conducted for 300 cycles, the percent change in length at 300 cycles is linearly interpolated to obtain the percent dilation for 100 cycles. This computed value is referred to as the freeze-thaw dilation (FTD) value of the coarse aggregate. MDOT requires coarse aggregates used for concrete pavements with low-volume traffic and other low-priority concrete structures such as curb and gutter to have a FTD value of less than 0.067 percent dilation per 100 cycles, with coarse aggregates used in freeways and major arterials (average directional commercial daily traffic greater than 5,000) required to have a FTD value of less than 0.040 percent dilation per 100 cycles (MDOT 2003).

ACBFS was first tested for freeze-thaw durability by MDOT in 1972, at which point it became evident that some ACBFS performed poorly when tested after being vacuum-saturated. A decision was made in 1976 to exempt ACBFS from vacuum saturation. The justification for this decision stemmed from the assumption that ACBFS used in concrete was fresh from the blast furnace and was not saturated, whereas crushed stone and gravel were assumed to be saturated by ground water for thousands of years (Vogler 1992).

Subsequent studies indicated that ACBFS used in concrete had moisture contents significantly less than vacuum-saturation, usually closer to the 24-hour-soak moisture content (40 to 60 percent of vacuum-saturation). As discussed, a recent MDOT laboratory study (Staton and Anderson 2009) showed that after a 24-hour soak, ACBFS aggregate had only achieved 33 percent of the vacuum-saturated absorption level, whereas natural aggregates were between 62 and 78 percent of the vacuum-saturated absorption level. However, this study did not investigate the actual moisture condition ultimately attained by ACBFS aggregate at the bottom of a concrete pavement or near joint locations under actual field conditions. As a result, since 1976, ACBFS coarse aggregate tested by MDOT has been conditioned prior to testing using a 24-hour soak and no vacuum saturation. All subsequent freeze-thaw tests on concrete made with ACBFS have resulted in FTD values of 0.002 percent dilation per 100 cycles or less.

Questions remain as to whether the 1976 decision to exempt ACBFS from vacuum saturation is technically correct. It is clear that the 24-hour soak only achieves roughly 33 percent of the vacuum-saturated level for ACBFS, whereas the level of saturation for naturally derived aggregates is between 62 and 78 percent (see table 5, page 17). As such, MDOT personnel have expressed the view that the omission of vacuum saturation means that ACBFS coarse aggregate is effectively being exempted from freeze-thaw testing in accordance with MDOT procedures, as it is not conditioned in strict accordance with the MTM 113.

It has been suggested that perhaps vacuum saturation itself was altering the aggregate pore structure of the aggregate, allowing water into locations where it would not go under normal conditions. This hypothesis is tentatively supported by Subhash, Vitton, and Chengyi (2008), who conducted a study on various aggregates to determine their strain rate behavior. Core samples (with a diameter of 0.37 in. [9.4 mm]) of aggregate were tested under dry and wet conditions in that study. Specimens were submerged in water prior to testing to simulate the wet condition as vacuum saturation was rejected because “it was found that with the slag specimens vacuum saturation caused damage to some of the aggregates.” Yet Staton and Anderson (2009) found in their study that the vacuum saturation method for moisture conditioning coarse aggregates does not alter the pore characteristics of the typical gravel, carbonate, or ACBFS coarse aggregates historically used in MDOT concrete pavements.

Muethel (2007) presented FTD values for six types of coarse aggregate where the FTD values were determined using the procedures outlined in MTM test methods 113, 114, and 115. An ACBFS coarse aggregate was included in this study and subjected to vacuum saturation, as were all the other aggregates tested. As shown in table 19, the FTD value of ACBFS was 0.1 percent dilation per 100 cycles, which is higher than the value allowed by MDOT for coarse aggregates used in concrete pavements. These results indicate that ACBFS aggregate will not satisfy the FTD value required by MDOT for coarse aggregates used in concrete if the ACBFS aggregate is subjected to the same vacuum-saturation moisture conditioning as are naturally derived aggregates.

Table 19. FTD Values for Various Coarse Aggregates
(From Muethel 2007. © Michigan Department of Transportation. Reprinted with permission.)

Aggregate Type	Aggregate Source	FTD Value (% dilation per 100 cycles)
Expanded shale	Carolina Solite	0.525
ACBFS	Levy (Dix)	0.100
Recycled concrete	I-94 PCC	0.067
Heterogeneous gravel	Round Lake	0.160
Heterogeneous quarried stone	Rockwood	0.044
Homogeneous quarried stone	Cedarville	0.003

The percentage of expansion measured for each aggregate is shown in figure 22. As seen in this figure, the specimens with expanded shale, ACBFS, and recycled concrete show a very high amount of expansion, indicating that vacuum saturation of any porous, highly absorptive aggregate will result in it failing the freeze-thaw testing protocol. The MDOT requirement of a maximum allowable FTD value of 0.067 percent dilation per 100 cycles was based on field observations that pavements with approximately 0.060 percent dilation per 100 cycles experienced 100 percent joint deterioration at 20 years, whereas pavements with less than 0.010 percent dilation per 100 cycles had nearly zero percent joint deterioration at 20 years. The 0.067 percent dilation per 100 cycles value was also based on "common MDOT supplier" protocol, where some suppliers were providing aggregates that had a FTD value of approximately 0.067 percent dilation per 100 cycles.

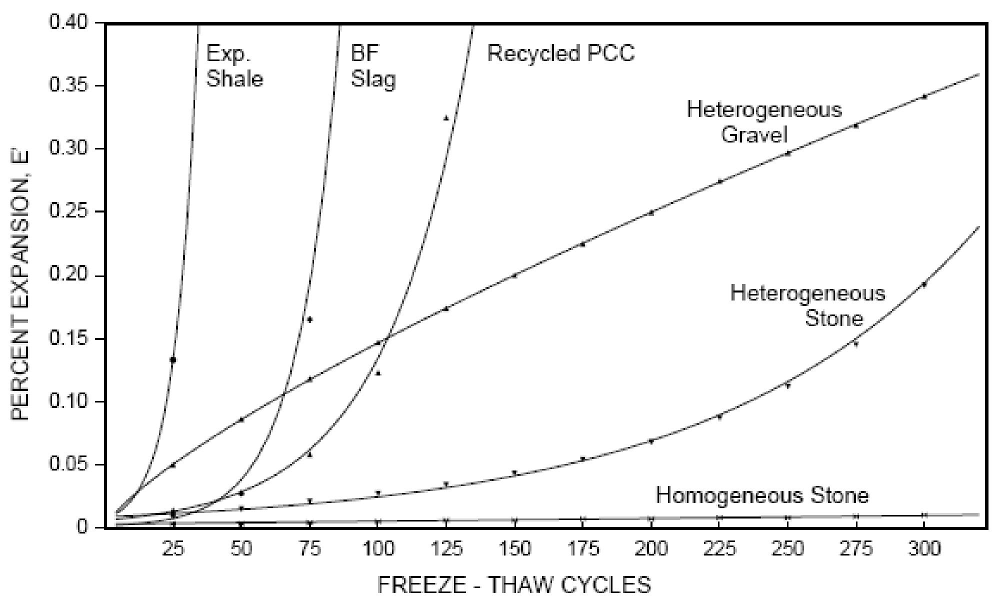

Figure 22. Freeze-thaw curves for various coarse aggregates.
(Muethel 2007; © Michigan Department of Transportation. Reprinted with permission.)

Summary of Freeze-Thaw Durability

It is commonly reported in the literature that ACBFS has excellent freeze-thaw durability, primarily due to the unique nature of its porosity, which prevents the particles from saturating in concrete due to capillary action. MDOT does not agree with this assessment, having put in place stringent freeze-thaw requirements that include vacuum-saturation of all coarse aggregates as part of their conditioning prior to making concrete specimens for freeze-thaw testing. This approach is taken to address the serious problems MDOT has faced with regard to poor coarse aggregate freeze-thaw durability in the past. Yet aggregate freeze-thaw distress was not observed in recent studies of MRD in Michigan that included the evaluation of a number of concrete pavements made with ACBFS coarse aggregate (Van Dam et al. 2002; Sutter, Van Dam, and Peterson 2009).

Polishing/Wear Resistance

Controversy exists over whether ACBFS should be subject to specifications for polishing and wear resistance using common testing methods (e.g., L.A. abrasion). For example, as early as 1930, the American Concrete Institute indicated that sharp edges of ACBFS aggregates are rapidly broken off during abrasion testing and appear as “loss by abrasion,” even though such breakage represents a large part of the breakage that occurs throughout the duration of the tests (ACI 1930). It has been shown this actually happens when ACBFS is tested, where the sharp edges were first worn off in the Deval machine, and then the material was subjected to the Standard Deval test (ACI 1930). Standard Deval test results showed that ACBFS aggregate that averaged 13.2 percent loss in the natural state averaged only 8.2 percent loss with the sharp edges removed. As a result, ACI (1930) indicates that the Standard Deval abrasion test should be omitted from specifications for ACBFS aggregates.

The National Slag Association (NSA) publication *Material of Choice* (NSA undated, b) states that the wear has been higher for ACBFS than for natural aggregates as tested by the L.A. Abrasion Machine. It is stated that there is no correlation between the L.A. abrasion loss for ACBFS in laboratory tests and degradation of skid resistance in field applications. The higher L.A. abrasion loss for ACBFS is caused from the chipping away of the rough edges and crushing of vesicular particles. This higher loss, however, does not mean that ACBFS is softer than natural aggregates. The hardness of ACBFS as measured by the Mohs scale is between 5 and 6, which compares favorably with the hardness reported for such materials as durable igneous rocks. For this reason, ASTM has deleted L.A. abrasion loss requirements for ACBFS in its specifications (see ASTM D692, D1139, and so on) and most State highway agencies in States where ACBFS is available do not require this test for ACBFS. L.A. abrasion limits for ACBFS, if included in specifications, should be somewhat higher than that for natural aggregates, allowing a maximum of approximately 50 percent loss. MDOT and ALDOT both still specify the L.A. abrasion test for ACBFS used in concrete (MDOT 2003; ALDOT 2008), limiting the maximum abrasion loss to 40 and 55 percent, respectively.

Summary of Physical Properties

In summary, the literature suggests that the most important physical attributes of ACBFS coarse aggregate used in concrete relate to its vesicular nature, which results in a relatively low specific gravity, high adsorptivity, high angularity, and increased L.A. abrasion loss, and may impact freeze-thaw durability. Absorption is of particular concern, since it has been shown that if ACBFS coarse aggregate is batched dry, it will result in poor workability and increased shrinkage potential in the concrete. Concrete made with ACBFS coarse aggregate possesses acceptable strength, stiffness, CTE, and wear resistance to be used in concrete pavements. Whether freeze-thaw resistance of concrete made with ACBFS coarse aggregate is compromised remains unresolved. The most common view is that the unique nature of the pore structure within the ACBFS aggregate should result in good performance, as the aggregate will not saturate under field conditions. Yet MDOT is convinced through laboratory and field investigations that freeze-thaw durability of paving concrete made with ACBFS is compromised.

HIPERPAV® SOFTWARE

HIPERPAV® (HIgh PERformance Concrete PAVing) is software that was developed under the sponsorship of FHWA to analyze the behavior of jointed plain concrete and continuously reinforced concrete pavements during the first 72 hours after placement (Ruiz et al. 2005). This software predicts the development of strength and stresses in the concrete using a broad set of inputs reflecting the pavement design, type of materials used in the concrete, mix design information, construction information (e.g., temperature of concrete, curing method), concrete properties, environmental conditions, and so on. Based on the predicted strength and stresses, the software predicts the risk of random pavement cracking. The user can modify the various input parameters, including mixture design parameters and environmental conditions, to evaluate the effect of strength and stress development on the potential for cracking to occur once the pavement has been placed. If a risk of cracking is predicted, HIPERPAV can be used to evaluate strategies for preventing cracking (e.g., placing cotton mats on the pavement to keep the pavement warm). The most recent version of this software is HIPERPAV® III.

The heat transport prediction model in HIPERPAV is used to predict the temperature development in the concrete. Heat transfer depends on the material's thermal conductivity, density, and specific heat (Ruiz et al. 2003). The thermal conductivity of the concrete is an important parameter, as it affects the rate of penetration of heat into the concrete and thereby controls the temperature gradients and thermal stresses (Mindess, Young, and Darwin 2003). The thermal conductivity varies with the type of aggregate used in the concrete (Scanlon and McDonald 1994). HIPERPAV III considers the following types of coarse aggregates: basalt, granite/gneiss, limestone, sandstone, and siliceous gravel. HIPERPAV III does not include the option of considering ACBFS coarse aggregate, although the Edw. C. Levy Company recently contracted for the development of a version of HIPERPAV® that considered ACBFS aggregate. Ruiz et al. (2003) presented the details of the study to incorporate ACBFS as an aggregate type in HIPERPAV, which included a field investigation to validate the temperature prediction models for ACBFS aggregate. The validation was carried out on two jointed reinforced concrete pavements (JRCP) constructed on I-75 in Detroit. The temperature model was able to predict the field measured temperatures in the concrete slab reasonably well, with the average coefficient of determination between the predicted and field values being 0.80.

Recent information suggests that the FHWA is working on an updated version of HIPERPAV that will incorporate ACBFS considerations (personal communication with Richard Lehman, June 5, 2011).

INTERNATIONAL STANDARDS FOR USE OF AIR-COOLED BLAST FURNACE SLAG IN CONCRETE

There is currently no ASTM International standard that specifically addresses the use of ACBFS as an aggregate in concrete. Several countries have previously had separate standards for ACBFS aggregate, but more recently have integrated the information on ACBFS into their standards for concrete aggregates, as summarized in the following sections. The international standards use SI units, and the information related to these standards presented in this section is therefore presented in SI units. The standards discussed address requirements for ACBFS aggregate use in concrete, and limited special mention is made in any of these standards regarding the use of concrete for paving applications.

Japanese Standards

The Japanese Industrial Standard JIS A 5011-1977, *Air-Cooled Iron Blast Furnace Slag Aggregate for Concrete* (JIS 1977), which has not been withdrawn, specified the following properties for ACBFS coarse aggregate to be used in concrete:

- Two classes of aggregate, A and B, are defined in the standard. The following properties are described for the two classes: (a) specific gravity in dry conditions—2.2 minimum for Class A and 2.4 minimum for Class B; (b) water absorption—5 percent maximum for Class A and 4 percent maximum for Class B; (c) mass per unit volume of aggregate—1.25 kg/l minimum for Class A and 1.35 kg/l minimum for Class B. This standard does not indicate situations where Class A and Class B aggregates are to be used, but it is evident that Class B is of a higher grade than Class A.
- The maximum limits for the following four chemical components are specified: Calcium Oxide (CaO)—45 percent maximum; Sulfur (as S)—2 percent maximum; Sulfur Trioxide (as SO_3)—0.5 percent maximum; Iron (as FeO)—3 percent maximum. The standard describes the analytical procedure for determining these components.
- The standard describes the immersion test that is to be performed on the aggregates. The immersion test is performed by immersing 30 pieces of aggregate that are retained on the 10-mm sieve in water for 2 days. At the end of the test, the aggregates should be free from cracks, decomposition, muddiness, and dusting.
- The standard describes the procedure for performing an irradiation test by ultraviolet light on the aggregates. In this test, 10 aggregate particles are crushed with a hammer and chisel to expose fresh fracture faces and subjected to an ultraviolet light having a wavelength of 360 nm. The aggregates are regarded as being acceptable if the aggregates do not radiate or shine evenly in violet shades.
- The standard describes gradation limits for the following five aggregate classes: (a) nominal aggregate size from 40 to 5 mm, (b) nominal aggregate size from 40 to 20 mm, (c) nominal aggregate size from 25 to 5 mm, (d) nominal aggregate size from 20 to 5 mm, and (e) nominal maximum aggregate size from 15 to 5 mm.

A newer Japanese Standard, *JIS A 5011-1:2003—Slag Aggregate for Concrete—Part I: Blast Furnace Slag Aggregate,* was published in 2003 (JIS 2003). This is a detailed document describing the use of both ACBFS as coarse aggregate and rapidly-cooled blast furnace slag as fine aggregate. With regard only to ACBFS, the following items are presented:

- The chemical composition and physical and chemical properties of ACBFS coarse aggregate are shown in table 20. Each criterion is linked to a test method presented in the document.
- There are two classes of ACBFS, Class L and Class N, of which Class N is of higher quality. These are separated by oven-dried density, absorption, and bulk density. Japan's *Standard Specifications for Concrete Structures—2007* (JSCE 2010) states that generally Class N ACBFS is used, with Class L aggregate "used only in concrete whose freeze-thaw resistance need not be so high and characteristic compressive strength is less that 21 N/mm^2."
- Maximum total sulfur content is set at 2.0 percent and maximum percent of acid-soluble sulfates is 0.5 percent (see also ASTM C114).
- Various tests are conducted to ensure stability of the ACBFS and uniformity. Detailed reporting requirements are provided in the standard. Japan's *Standard Specifications for Concrete Structures—2007* (JSCE 2010) recommends that testing should be conducted prior to construction and at least monthly during construction.

Table 20. Chemical Composition and Physical and Chemical Properties of ACBFS (JIS A 5011-1:2003)

<table>
<tr><th colspan="3" rowspan="2">Item</th><th colspan="2">Blast Furnace Slag Coarse Aggregate</th></tr>
<tr><th>Class L</th><th>Class N</th></tr>
<tr><td rowspan="4">Chemical composition</td><td>Calcium oxide (as CaO)</td><td>%</td><td colspan="2">45.0 max.</td></tr>
<tr><td>Total sulfur (as S)</td><td>%</td><td colspan="2">2.0 max.</td></tr>
<tr><td>Sulfur trioxide (as SO_3)</td><td>%</td><td colspan="2">0.5 max.</td></tr>
<tr><td>Total iron (as FeO)</td><td>%</td><td colspan="2">3.0 max.</td></tr>
<tr><td colspan="2">Density in oven dry condition</td><td>g/cm^3</td><td>2.2 min.</td><td>2.4 min.</td></tr>
<tr><td colspan="2">Water absorption</td><td>%</td><td>6.0 max.</td><td>4.0 max.</td></tr>
<tr><td colspan="2">Bulk density</td><td>kg/L</td><td>1.25 min.</td><td>1.35 min.</td></tr>
<tr><td colspan="3">Immersion in water</td><td colspan="2">Shall be no phenomena such as cracks, decomposition, muddiness, powdering.</td></tr>
<tr><td colspan="3">Irradiation by ultraviolet light (360.0 nm)</td><td colspan="2">Shall be no light emission or uniform purple luminance.</td></tr>
</table>

Japan's *Standard Specifications for Concrete Structures—2007* (JSCE 2010) recommends that if ACBFS coarse aggregate is used, "watering facilities such as sprinklers should be provided in addition to drainage facilities." This point is emphasized in an earlier document that specifically states that ACBFS coarse aggregates will be stored in facilities equipped with suitable sprinkler and drainage systems to uniformly maintain the desired moisture content (JSCE 1993).

With regard to L.A. abrasion loss, it is stated in the *Guidelines for Construction Using Blast-Furnace Slag Aggregate Concrete* (JSCE 1993) that a 35 percent abrasion loss is acceptable when ACBFS concrete is used for pavement applications.

British/European Standards

The British Standard BS 1047-1983, *Air-Cooled Blast Furnace Slag Aggregate for Use in Construction* (BS 1047 1983), which was withdrawn in 2004, specified detailed requirements for ACBFS coarse aggregate when used in concrete. The following is a summary of the requirements for ACBFS aggregate used in concrete (BS 1047 1983):

- Iron pellets should not be present in a sufficient quantity to cause surface spalling or staining.
- The bulk density of the aggregate should not be lower than 1,100 kg/m^3 (when determined according to BS 812). The sample used for determining the bulk density should pass the 14-mm sieve and be retained on the 10-mm sieve.
- The stability of the aggregate against iron unsoundness must be established by testing the aggregate. The test is performed by immersing not less than 12 pieces of aggregate that pass the 40-mm sieve but are retained on the 20-mm sieve in distilled or deionized water for 14 days. Aggregates that develop no cracking or disintegration are regarded as being free from iron unsoundness. The standard describes the detailed procedure for performing this test.
- The aggregate should be tested for "falling" or dicalcium silicate unsoundness. The standard describes the procedure for performing this test. In the test procedure, the aggregate is analyzed chemically to determine the percent content by mass of CaO, MgO, SiO_2, Al_2O_3 and S (total). To pass this test, the percentage of CaO must meet the following criteria:

  $CaO \leq 0.9SiO_2 + 0.6Al_2O_3 + 1.75S$ *based on mass percentages* EQ. 3

  The standard indicates that ACBFS aggregate that fails this criterion is not necessarily unsound, stating the decision as to whether the aggregate is unsound should be made based on the results from the microscope test. The procedure for performing the microscope test is described in the standard. In this test, suitable samples are prepared and polished and then etched in a magnesium sulfate solution and examined under a metallurgical microscope.
- The total sulfur content in the aggregate should not be greater than 2 percent, and the percentage of acid-soluble sulfates expressed as SO_3 should not be greater than 0.7 percent.
- The percentage absorption of the aggregate should not exceed 10 percent (when tested according to BS 812).

- The limits of the Flakiness Index of the aggregate for various grades of concrete are presented in the standard. The Flakiness Index is found by expressing the weight of the flakey aggregate as a percentage of the aggregate tested. "Flakey" is the term applied to aggregate that are flat and thin with respect to their length or width.
- The limiting content of 10 percent fines value (in kN) determined in accordance with BS 812 for three types of concrete (i.e., heavy duty floor finishes, pavement wearing surfaces, and other) is presented in this standard. The 10 percent fines value is the force at which 10 percent of fines are produced.
- The standard indicates the grading requirements for the following four aggregate classes: (a) aggregate size from 40 to 5 mm, (b) aggregate size from 20 to 5 mm, (c) aggregate size from 14 to 5 mm, and (d) single sized aggregate—20 mm.

A new European standard, *CEN EN 12620—Aggregates for Concrete*, has now replaced the withdrawn BS 1047 (CEN 2002). This standard is much simplified from the British standard, integrating guidance on the use of ACBFS in concrete with that of naturally derived aggregates. Thus, in most cases, ACBFS aggregates need to meet the same requirements as all other aggregates for a given application. Specific areas where ACBFS is mentioned include the following:

- Clause 6.3—Allows ACBFS aggregates to have higher acid-soluble sulfate (up to 1 percent) content and total sulfate content (up to 2 percent). The reason a higher proportion of sulfates is allowable in ACBFS is discussed in Annex G.2 of the document, which states that a substantial proportion of the sulfate is encapsulated in crystalline phases in the slag grains and therefore is unavailable to participate in hydration reactions. The test methods to determine acid-soluble sulfate and total sulfur content are referenced in the document. According to Table H.3 in the document, these should be tested twice per year.
- Clause 6.4.2—ACBFS shall be free from dicalcium silicate disintegration and iron disintegration. The test methods to determine these characteristics are referenced in the document. According to Table H.3 in the document, these should be tested twice per year.
- Annex F.2.3—Specifically states that although ACBFS often has absorption in excess of 2 percent, it is still known to have adequate freeze-thaw resistance.
- Annex G.6—States that although some constituents of ACBFS can adversely affect its volume stability when used in concrete, ACBFS from modern production units is less likely to be unsound in this way.

Australian Standard

The Australian Standard, *AS 2758.1-1998—Aggregate and Rock for Engineering Purposes—Part 1: Concrete Aggregates*, addresses requirements for aggregate that is used for concrete and describes specific requirements that apply to ACBFS aggregate. The following are the specific requirements related to ACBFS aggregate used in concrete (AS 1998):

- Water Absorption: Water absorption of vesicular aggregates can exceed 2 percent significantly without affecting many of the properties of concrete made using such aggregates. To minimize any effect of absorption variations, vesicular aggregates should be pre-wetted before the mixing process.
- L.A. Abrasion Test Values: The standard describes the maximum L.A. abrasion test values for two rock-type classes (i.e., coarse-grained rocks, all other rocks) for different concrete exposure conditions. However, the standard notes that for some aggregates, other values could be adopted based on satisfactory local experience of materials and performance (e.g., vesicular aggregates).
- Iron Unsoundness: The standard indicates that when chemical analysis of the ACBFS aggregate shows that the ferrous oxide content equals or exceeds 3 percent and the total sulfur content equals or exceeds 1 percent, the aggregate should be tested for iron unsoundness. This test is performed in accordance with AS 1141.37, and if the iron unsoundness exceeds 1 percent, the aggregate should not be used in concrete. The standard indicates iron unsoundness is highly likely when ACBFS aggregate contains more than the above limits for ferrous oxide and sulfur, but notes that iron unsoundness has not been recorded in Australian iron blast furnace slag.
- Falling or Dusting Unsoundness: The standard states that fresh slag shall only be used as an aggregate in concrete if it has been allowed to cool to below 50°C. It is stated that "during the cooling of some blast furnace slag the inversion at around 490°C of any beta dicalcium silicate in the slag to the gamma form, may result in disruption of the slag mass. This disruption leads to what is known as falling or dusting unsoundness. Any beta dicalcium silicate that is retained in the cooled slag is considered to be kinetically stable and will not invert to cause disruption of the slag. No evidence has been found either in Australia or overseas of delayed inversion of beta dicalcium silicate in iron blast furnace slag, or of deterioration of concrete due to the presence of beta dicalcium silicate."
- Stockpiling of ACBFS Aggregate: The standard indicates crushed ACBFS aggregate should be stockpiled in moist condition at or near the SSD condition before use, with the moisture condition being maintained by sprinkling with water.
- Free Lime: The standard indicates that before using ACBFS aggregate from a new source or when significant changes in furnace chemistry have occurred in an existing source that may result in the presence of free lime, the potential for formation of pop-outs in concrete should be evaluated. This is performed by petrographic examination or quantitative x-ray diffractometry on a representative sample to determine the free-lime content of the ACBFS aggregate using the procedures in AS 1141.3.1. The standard indicates that if the level of particles containing free lime exceeds 1 in 20, then the stockpile of the ACBFS aggregate from which the sample was obtained should be kept at or near SSD conditions until further testing shows that the level has fallen below 1 in 20.

Summary of International Standards

The most striking feature in the international standards reviewed is the direct acknowledgment that the physical and chemical properties of ACBFS are important and thus should be tested. In particular, it is recognized:

- The vesicular nature of ACBFS will result in higher absorption and lower bulk specific gravity, but this is not expected to compromise strength or durability of the concrete as long as minimum requirements are met. The European standard specifically states that although ACBFS often has absorption in excess of 2 percent, it has good freeze-thaw durability.
- Although the L.A. abrasion loss for ACBFS will often be higher than that for wear-resistant natural aggregates, as long as experience dictates, wear-resistant concrete can be made with ACBFS. The Japanese standards specifically identify an L.A. abrasion loss of 35 percent as being acceptable for ACBFS used for pavements.
- Modern ACBFS has little potential for iron unsoundness, but chemical testing should be conducted to ensure that it will not be a problem.
- If allowed to cool below 50^{o}C, modern ACBFS has little potential for dicalcium (falling) unsoundness. Both the European and Japanese standards recommend chemical testing at periodic intervals (every 6 months in Europe and every month in Japan).
- The presence of free lime (CaO) is rare in modern ACBFS but should be routinely tested.
- Although the sulfate and sulfur content in ACBFS can be higher than would be allowed in natural aggregate, limits are still placed upon both, usually permitting an acid-soluble sulfate content of up to 0.5 to 1.0 percent and total sulfur content not to exceed 2 percent. The European standards require that testing be done twice a year, whereas the Japanese standard requires testing every month.
- ACBFS should be wetted and kept moist through processing and concrete batching. Concrete plants must be equipped with sprinklers and drainage facilities.

CHAPTER 4. USE OF ACBFS IN PAVING CONCRETE IN MICHIGAN

As indicated previously, ACBFS has been the focus of a number of laboratory investigations and field demonstration projects, but it also has a long history of use in production concrete paving projects. In the laboratory, the physical properties of paving concrete made with ACBFS coarse aggregate compare favorably to concrete made with natural aggregates, with the potential exception being freeze-thaw durability as noted earlier. At the same time, economic savings may be realized due to its lower unit weight, which reduces trucking costs. It also lowers the weight for a given volume of concrete compared to more dense natural aggregates. And, due to the vesicular nature of ACBFS producing a rough, porous surface, its bond with cement mortar is good, but workability requirements may involve slightly more mortar (cementitious material, sand, and water) during proportioning. Finally, concrete made with ACBFS has been shown to exhibit good skid resistance (Emery 1982).

Although ACBFS has been used in paving concrete for decades, concerns continue to exist regarding the performance of concrete pavements incorporating ACBFS. Michigan has been the largest single user of ACBFS in concrete pavement, but has also expressed the greatest concerns regarding its use. The vast majority of ACBFS currently used in Michigan comes from the Ford Rouge River Complex, located in Detroit, Michigan. The blast furnaces producing this slag were originally constructed in the 1920s. Specifications for the use of ACBFS coarse aggregate in concrete first appeared in the Michigan standard specifications in 1934, and most of the Detroit freeway system, as well as many interstate and primary highway pavements and structures, were constructed with concrete containing ACBFS coarse aggregate (Staton 2006). In addition, many of the local roads in Detroit and surrounding communities also used ACBFS coarse aggregate (Grove, Bektas, and Gieselman 2006), as did some of the pavement construction at the Detroit Metropolitan Airport (personal communication, T. Van Dam, 2008).

MDOT has experienced mixed results with the performance of concrete pavements containing ACBFS aggregate, with the service life ranging from less than 10 years to a little over 30 years before requiring major rehabilitation (Staton 2006; Sutter, Van Dam, and Peterson 2009). There have been many instances in which ACBFS has been linked to premature pavement failures, although the role of ACBFS as the sole contributor to premature pavement distress has not been established (Staton 2006). However, concerns regarding the use of ACBFS aggregate in concrete have been raised as early as the 1940s in Michigan. In 2002, MDOT assessed documented experience and past research findings to determine the risk and benefits associated with the use of ACBFS as a coarse aggregate in concrete pavements (Staton 2006). It was concluded that using ACBFS instead of readily available, high-quality, naturally derived aggregates introduced an unnecessary level of risk in obtaining a long-life concrete pavement. In response, in 2006, the FHWA Division Office imposed a moratorium on the use of ACBFS aggregate in concrete in federally funded projects (FHWA 2006).

This chapter presents a review of the performance of concrete pavements in Michigan constructed with ACBFS aggregate, from its earliest use in the 1940s through more recent evaluations.

FIELD STUDIES OF CONCRETE PAVEMENTS MADE WITH AIR-COOLED BLAST FURNACE SLAG

Issues regarding the uniformity of fresh concrete made with ACBFS aggregates first appeared in documents from the 1940s with McLaughlin (1945) stating:

> It was determined that in the laboratory, concrete of satisfactory workability and strength may be designed using slag as coarse aggregate. Attention, however, is called to the fact that considerable difficulty was encountered in controlling the consistency of the various mixes. A sharp breaking point was found from the dry to the wet mixes where the addition of very small amounts of water increases the slump far more than it would when stone or gravel is used. This might cause considerable trouble in the field, especially when transit mixed concrete is permitted. Approval of slag as coarse aggregate for concrete structures should be withheld until information has been obtained about its behavior under construction conditions.

In work published 2 years later, Finney (1947) stated:

> In view of the performance record of the slag projects on the Willow Run and Detroit Industrial Expressway Systems and on account of the difficulties encountered in obtaining uniform quality materials and in controlling slag concrete mixtures in order to realize a continuous uniform-concrete product, it is recommended that slag, as an aggregate for concrete pavement slabs should not be considered on future construction projects or at least until such time that more definite information can be obtained on the behavior of slag concrete in pavements.

Likely related to the uniformity of the fresh concrete, but also reflecting the unique mechanical properties of hardened concrete, were concerns with slab cracking. Oehler and Finney (1953) recommended, based on observations of high amounts of cracking encountered on several projects, that when ACBFS aggregate is used for concrete pavements, the pavement thickness should be increased over that required for natural aggregate by a value not less than 1 in. (25 mm) to compensate for the apparent loss of flexural strength of concrete containing ACBFS aggregate. They also recommended the cement content of concrete with ACBFS aggregate be increased from 5.5 sacks (517 lb) to a minimum of 6 sacks (564 lb) per cubic yard (from 307 to 335 kg/m^3) to compensate for the loss in mortar richness. They indicated that experience has shown that it is difficult to control the consistency of ACBFS aggregate concrete mixtures because the specific gravity, absorption, and unit weight of ACBFS vary greatly. They also indicated that the properties of ACBFS particles—such as brittleness and softness—may cause concrete containing ACBFS aggregates to fail more rapidly in fatigue than concrete made with other aggregate types.

The following are some specific examples where MDOT has documented the poor uniformity and cracking tendency of concrete made with ACBFS, which has resulted in poor performance.

Willow Run and Detroit Industrial Expressway Systems

Oehler and Finney (1953) reported the results of a crack survey performed in 1953 on the Willow Run Expressway (US-12 between I-94 and Willow Run Airport) and Detroit Industrial Expressway (I-94). This survey was performed on concrete pavements constructed with limestone, gravel, and ACBFS aggregate. The combined lengths of pavements evaluated for both these roadways were: 11.4 mi (18.4 km) of ACBFS aggregate, 0.9 mi (14.5 km) of limestone aggregate, and 24.8 mi (39.9 km) of gravel aggregate. The projects had been constructed between 1942 and 1944. The crack survey consisted of counting the number of cracks per slab and measuring the lineal feet of cracking.

The concrete slab thicknesses of the evaluated pavements were 9 and 10 in. (230 and 250 mm), with all of the pavements containing ACBFS being 9 in. thick. The lengths of the 9-in. and 10-in. pavements with the gravel coarse aggregate were 17.6 and 7.2 mi (28.3 and 11.6 km), respectively. The contraction joint spacing of the pavements varied from 20 to 25 ft (from 6.1 to 7.6 m), and expansion joints were located every 120 ft (36.6 m). Most of the gravel aggregate used in these projects was obtained from American Aggregates Corporation at Green Oaks. All of the ACBFS aggregate was obtained from the Great Lakes Steel Company at Zug Island. Steel reinforcement, load transfer devices at transverse joints, and ties between passing and driving lanes were not used in these pavements because steel was restricted to the war effort (Finney 1947). Proper compaction of the subbase was also not performed, with the compaction being provided by construction traffic (Finney 1947).

Table 21 shows the percent slabs cracked in the passing and the driving lane for pavements 9-in. (230 mm) thick. The results from a previous crack survey that was performed in 1946 and the percentage difference in cracked slabs between the 1946 and 1953 surveys are also shown in this table. Table 22 shows the average length of cracking per lane-mile for the traffic and the passing lane for 9-in.-thick slabs.

Table 21. Percent Slabs Cracked for Pavements 9 in. Thick
(Oehler and Finney 1953)

Aggregate Type	Percent Slabs Cracked (%)					
	Traffic Lane			Passing Lane		
	1946 Survey	1953 Survey	% Difference	1946 Survey	1953 Survey	% Difference
Gravel	22	35	13	6	13	7
Limestone	41	52	11	8	7	-1
ACBFS	89	95	6	21	62	41

Table 22. Average Lineal Feet of Cracking per Lane-Mile for Pavements 9-in. (230 mm) Thick (Oehler and Finney 1953)

Aggregate Type	Average Lineal Feet of Cracking per Lane-Mile	
	Traffic Lane	Passing Lane
Gravel	1,152	446
Limestone	1,633	213
ACBFS	3,564	2,042

As shown in table 21, the 1946 survey indicated that 89 percent of the slabs in the traffic lane containing ACBFS aggregate had cracked compared to 22 percent for gravel aggregate. This crack survey was performed approximately 2 years after construction. By 1953, the percentage of slabs cracked in the traffic lane for pavements constructed with ACBFS aggregate increased to 95 percent. In the traffic lane, for pavements constructed with gravel aggregate, the percentage of slabs cracked increased from 22 to 35 percent from 1946 to 1953. In the passing lane, when the 1946 and 1953 results were compared, the percentage of slabs cracked increased from 21 to 62 percent for pavements with ACBFS aggregate, and from 6 to 13 percent for pavements with gravel aggregate. As shown in table 22, the length of cracking in pavements with ACBFS aggregate was 3.1 and 4.6 times greater than the cracking in pavements with gravel aggregate in the traffic lane and passing lane, respectively.

Dix Highway—From US-24 to Ecorse

Oehler and Finney (1953) also reported the results of a crack survey performed on a concrete pavement on Dix Highway between US-24 and Ecorse. It consisted of a concrete pavement containing gravel aggregates constructed in 1928 and a concrete pavement with ACBFS aggregate constructed in 1935. The ACBFS aggregate pavement was constructed as a widening of the existing roadway.

Besides the type of coarse aggregate used, the major difference in the two projects was that the gravel aggregate section had expansion joints at a 100-ft (30.5 m) spacing with no intermediate joints, while the project with ACBFS aggregate had expansion joints at 50-ft (15.2 m) intervals with control joints at 25 ft (7.6 m) midway between expansion joints, thus creating 25-ft long slabs. The expansion joints in the ACBFS project coincided with those in the gravel projects, with another expansion joint being present in the ACBFS project midway between the expansion joints in the gravel project. The pavement thickness for both projects was the same.

The results of the crack survey, which was performed in 1953, are shown in table 23. The pavement with ACBFS aggregate was observed to have approximately three times as many transverse cracks as the pavement constructed with gravel aggregate. This difference is significant, as the slab lengths for the ACBFS sections are considerably shorter. In addition, roughometer measurements showed that the pavement with ACBFS aggregate was approximately 36 percent rougher than that of the pavement constructed with gravel aggregate.

Table 23. Results From the 1953 Dix Highway Crack Survey
(Oehler and Finney 1953)

Aggregate Type	Total Miles	No. of Slabs	No. of Cracked Slabs	Slabs Cracked (%)	No. of Transverse Cracks	No. of Longitudinal Cracks
ACBFS	4.65	925	771	84%	1068	34
Gravel	4.65	268	151	56%	296.5	50

Gratiot Avenue—8 Mile Road to 13 Mile Road

Finney (1961) reported the results from a series of crack surveys that were performed in 1948, 1953, 1956, and 1960 on Gratiot Avenue from 8 Mile Road to 13 Mile Road. This segment of concrete pavement was constructed in 1947 with ACBFS aggregate. The pavement was reinforced and consisted of a slab 10-in. (250 mm) thick with 100-ft (30.5 m) joint spacing. The project length was 4.22 mi (6.79 km). The results from the crack surveys are shown in table 24. The percent of cracked slabs increased from 25.6 to 84.5 percent between 1948 and 1953, with the cracking being uniformly distributed throughout the project length. In 1960, 97.4 percent of the slabs were cracked, with most slabs exhibiting multiple cracks.

Table 24. Results From Crack Surveys—Gratiot Avenue
(Finney 1961)

Parameter	Year			
	1948	1953	1956	1960
Percent slabs cracked (%)	25.6	84.5	95.4	97.4
Average number of cracks per slab	0.46	2.43	3.13	4.05
Lineal feet of cracking per lane-mile	294	1537	1979	2548

US-23 Aggregate Test Road

In an attempt to resolve continued issues regarding performance concrete pavements made with various coarse aggregates, MDOT constructed an aggregate test road on southbound US-23 north of the Ohio-Michigan border. The project was paved in September 1992, consisting of JRCP 10.5-in. (265 mm) thick with 27-ft (8.2 m) joint spacing. Table 25 shows the notation assigned to each test section, the type of coarse aggregate used, the FTD value of the aggregate, and the length of each test section. (Note: The FTD value of ACBFS is obtained using a 24-hour soak, while the other aggregates are subjected to vacuum saturation.) All sections were constructed on an open-graded, asphalt-treated base to provide uniform support and to provide an avenue for good drainage. There is a transition zone between the test sections that is approximately 0.1 mi (0.16 km) in length.

Table 25. Details of Test Sections in the Aggregate Test Road

Section	Coarse Aggregate	FTD Value	Length of Section (ft)
A	Crushed Dolomite	0.006	5,500
B	ACBFS	0.001*	5,000
C	Natural Gravel	0.022	6,500
D	Crushed Dolomite	0.031	6,500
E	Natural Gravel	0.075	5,000

*ACBFS was not vacuum-saturated prior to testing.

Table 26 shows the different types of subbases used for each section, the drainability of the subbase, and the length within each test section where each subbase type was used.

Transverse crack surveys were performed on the test sections in 1998, 1999, 2000, 2001, and 2003 (MDOT undated). The crack surveys were carried out on the outside truck lane. Tables 27 through 31 show the results from the surveys for sections A, B, C, D, and E, respectively. Each table shows the percentage of slabs cracked categorized according to the number of cracks observed in each panel (either 1, 2, or 3) as well as the total percentage of cracked slabs (i.e., sum of percentage of slabs with 1, 2, and 3 cracks). All these cracks were full depth and full width.

Table 26. Subbase Types Used in the Aggregate Test Road

Section	Type Coarse Aggregate	Length of Section (ft)	Subbase Type	Drainability (ft/day)	Length (ft)
A	Crushed dolomite	5,500	Ohio #8	198	2,100
			Existing subbase	Impermeable	3,400
B	ACBFS	5,000	Existing subbase	Impermeable	2,500
			Ohio #9	288	2,100
			Ohio #8	198	400
C	Natural gravel	6,500	Ohio #8	261	3,000
			Existing subbase	Impermeable	3,500
D	Crushed dolomite	6,500	Ohio #8	230	3,500
			Existing subbase	Impermeable	3,000
E	Natural gravel	5,000	Special	Not available	2,500
			Existing subbase	Impermeable	2,500

Table 27. Results From Crack Surveys of Section A: Crushed Dolomite

Survey Year	Crushed Dolomite: Percent Slabs Cracked by Subbase Type							
	Special: Ohio #8				Existing Subbase			
	1 Crack	2 Cracks	3 Cracks	Total	1 Crack	2 Cracks	3 Cracks	Total
1998	3	-	-	3	0	-	-	0
1999	3	-	-	3	0	-	-	0
2000	3	-	-	3	2	-	-	2
2001	3	-	-	3	2	-	-	2
2003	4	-	-	4	2	-	-	2

Table 28. Results From Crack Surveys of Section B: ACBFS

Survey Year	ACBFS: Percent Slabs Cracked by Subbase Type											
	Existing Subbase				Special: Ohio #9				Special: Ohio #8			
	No. of Cracks				No. of Cracks				No. of Cracks			
	1	2	3	Total	1	2	3	Total	1	2	3	Tota l
1998	78	13	1	92	78	19	-	97	74	13	-	87
1999	79	13	1	93	78	21	-	99	74	13	-	87
2000	75	17	1	93	77	22	-	99	74	13	-	87
2001	76	18	1	95	77	23	-	100	74	13	-	87
2003	75	19	1	95	74	26	-	100	74	13	-	87

Table 29. Results From Crack Surveys of Section C: Gravel

Survey Year	Section C: Natural Gravel (Percent Slabs Cracked) by Subbase Type							
	Special: Ohio #8				Existing Subbase			
	1 Crack	2 Cracks	3 Cracks	Total	1 Crack	2 Cracks	3 Cracks	Total
1998	16	-	-	16	11	-	-	11
1999	21	-	-	21	13	-	-	13
2000	22	-	-	22	14	-	-	14
2001	27	-	-	27	16	-	-	16
2003	32	-	-	32	18	-	-	18

Table 30. Results From Crack Surveys of Section D: Crushed Dolomite

Section D: Crushed Dolomite (Percent Slabs Cracked) by Subbase Type								
Survey Year	Special: Ohio #8				Existing Subbase			
	1 Crack	2 Cracks	3 Cracks	Total	1 Crack	2 Cracks	3 Cracks	Total
1998	7	-	-	7	6	-	-	6
1999	8	-	-	8	6	-	-	6
2000	7	1	-	8	10	-	-	10
2001	6	2	-	8	11	-	-	11
2003	7	2	-	9	11	-	-	11

Table 31. Results From Crack Surveys of Section E: Gravel

Section E: Natural Gravel (Percent Slabs Cracked) by Subbase Type								
Survey Year	Special				Existing Subbase			
	1 Crack	2 Cracks	3 Cracks	Total	1 Crack	2 Crack	3 Cracks	Total
1998	1	-	-	1	4	4	-	8
1999	1	-	-	1	4	4	-	8
2000	1	-	-	1	7	7	-	14
2001	1	-	-	1	8	8	-	16
2003	1	-	-	1	8	8	-	16

The percentages of slabs cracked at the five test sections in 2003 are shown in table 32. Section B with ACBFS had the highest percentage of cracked slabs, ranging from 87 to 100 percent for the three subbase types. After the 2003 survey, no joint or crack faulting was noted, thus confirming the benefits of the premium stabilized base. Also, the stabilization of the base was effective towards insulating against poorer quality subbase. Additionally, there were no indications on any of the sections that the concrete was being affected by MRD such as alkali–silica reactivity (ASR) or freeze-thaw damage.

It was observed that the midpanel cracking of close to 95 percent of the JRCP slabs occurred in the ACBFS section within 6 years of construction, whereas less than 16 percent slab cracking had occurred on the next-poorest-performing section, which contained naturally derived gravel aggregate. Further, the cracks in the ACBFS section ran transversely across the pavement in a straight line following the transverse tining, almost as if they had been cut. This was not the case with transverse cracks that occurred in the concrete containing naturally derived aggregate.

Table 32. Percent Slabs Cracked in 2003 at Test Sections

Section	Coarse Aggregate	Subbase		Percent Slabs Cracked (%)			
		Type	Drainability (ft/day)	1 Crack	2 Cracks	3 Cracks	Total
A	Crushed dolomite	Ohio #8	198	4			4
		Existing	Impermeable	2			2
B	ACBFS	Existing	Impermeable	75	19	1	95
		Ohio #9	288	74	26		100
		Ohio #8	198	74	13		87
C	Natural gravel	Ohio #8	261	32			32
		Existing	Impermeable	18			18
D	Crushed dolomite	Ohio #8	230	7	2		9
		Existing	Impermeable	11			11
E	Natural gravel	Special	Impermeable	1			1
		Existing	Impermeable	8			8

In detailed surveys conducted in 2006 and 2008, it was noted that although some new cracking continued to develop, the more troubling observation was that many of the cracks observed in the 2003 survey in Section B (the ACBFS section) had begun to deteriorate, some of them severely. Deterioration including numerous cracks that had faulted, spalled to widths as wide as 4 in. (100 mm), developed corner breaks, some of which "punched through" into the base, and multiple patches. Detailed surveying with photographs taken for comparison revealed that the severity of distress accelerated from 2006 to 2008.

The higher levels of cracking noted in Section B were in sharp contrast to the cracking observed in the other sections, as noted below:

- Section A: Very few cracks, with one new crack observed. All cracks remained tight, hairline cracks with little distress, and no appreciable change in condition was observed between 2006 and 2008.
- Section C: No new cracking. Most cracking remained tight, but minor spalling was observed at most cracks. Very little change observed in severity between 2006 and 2008.
- Section D: No new cracking. The few cracks that were observed remained very tight with no to minor spalling, and very little change had occurred from 2006 to 2008.
- Section E: Very little cracking. One crack had moderate spalling and one centerline "punch out" had formed but it was not related to transverse cracking.

Michigan State University Study

A study was performed by Michigan State University to investigate factors affecting the shear capacity of transverse cracks in jointed concrete pavements (Frabizzio and Buch 1999; Buch, Frabizzio, and Hiller 2000). Forty-nine test sites were established on inservice pavements that exhibited transverse cracking, with the length of the test sites ranging from 82 to 213 ft (from 24.9 to 74.9 m). All but two of the pavements included in this study were JRCP. The joint spacing of the JRCP pavements ranged from 27 to 71 ft (from 8.2 to 21.6 m). The types of coarse aggregate used in the concrete were carbonate (17 sites), natural gravel (8 sites), recycled concrete (14 sites), and ACBFS (10 sites).

A comparison of the average number of transverse cracks per slab for pavements with 41-ft (12.5 m) joint spacing was made to evaluate the difference in performance for different coarse aggregate types. The average age of pavements containing gravel, carbonate, and ACBFS coarse aggregates were 12, 19.6, and 11 years, respectively. The average number of transverse cracks per slab for gravel, carbonate, and ACBFS coarse aggregate was 1.2, 2.5, and 6, respectively. These results indicate that pavements containing ACBFS coarse aggregate had a higher number of transverse cracks per slab than pavements with either gravel or carbonate coarse aggregate. During the field surveys it was observed that the sections containing ACBFS aggregate also had a higher number of shrinkage cracks that were partial width across the slab than sections with carbonate or gravel aggregates. These shrinkage cracks were not considered in computing the average cracks per slab. A similar comparison for other slab lengths could not be performed because of insufficient sample size. It was reported that the higher number of cracks in pavements with ACBFS aggregate was probably due to the higher susceptibility of the concrete containing this aggregate to early-age shrinkage cracking, because ACBFS aggregates can absorb water from the concrete if batched drier than SSD conditions.

A limited laboratory study was also performed in this project to evaluate crack deterioration by preparing concrete slab specimens in the laboratory and subjecting them to repeated loading cycles. Concrete slab specimens that were prepared using carbonate or gravel coarse aggregate showed better ability to maintain aggregate interlock across the crack face, effectively transferring load, than the specimens prepared with ACBFS coarse aggregate.

Evaluation of the crack faces of cores obtained over cracks in the field as well as the crack faces of the laboratory specimens showed that concrete with carbonate or ACBFS aggregate have a smoother texture on the crack face, while the concrete made with gravel aggregate had a rough texture. For the smoother-textured surfaces, the crack propagated through the aggregate, while in the rough-textured surfaces the crack propagated around the aggregate.

Frabizzio and Buch (1999) recommended that “natural aggregates, such as gravel and carbonates, should be used when possible in construction of new concrete pavements due to their superior performance in this study over manufactured aggregates such as slag and recycled concrete.”

I-75 Goddard to Sibley Road

Robords (2010a) reported the results from a survey performed on a segment of I-75 from Goddard Road to Sibley Road, 4.7 mi (7.6 km) long to determine the number of full-depth repairs (FDRs) performed on the pavement. This roadway has three lanes in both the north and southbound directions. The lanes were constructed first with the left and the center lane being paved simultaneously. The right lane was paved next with the shoulder being paved last.

The coarse aggregate used in the concrete for the northbound pavement was ACBFS. Limestone was used as the coarse aggregate in the concrete in the right lane for the entire project in the southbound direction. In the southbound left and center lane, limestone was used as the coarse aggregate in the concrete from Goddard to Eureka (2.4 mi (3.9 km)) and ACBFS was used as the coarse aggregate in the concrete from Eureka to Sibley (2.3 mi (3.7 km)). The construction date, the joint spacing, and the date of the FDR survey were not reported.

Table 33 shows the number of FDRs and repair density (number of repairs per mile) for each lane in both directions, with the FDRs being separated in the southbound direction for the pavement segments with the different coarse aggregate types. As seen in table 33, the pavement concrete made with limestone aggregate had very few FDRs compared to sections constructed with ACBFS aggregate. The northbound right lane that had ACBFS aggregate exhibited 124 FDRs (26.38 repairs per mile (16.36/km)), while the southbound right lane that was constructed with limestone aggregate exhibited 6 FDRs (1.28 repairs per mile (0.79/km)). In the southbound center lane, the pavement from Goddard to Eureka (2.4 mi (3.9 km)) constructed with limestone aggregate exhibited two FDRs (0.83 repairs per mile (0.52/km)), while the pavement from Eureka to Sibley (2.3 mi (3.7 km)) that had ACBFS aggregate exhibited 29 FDRs (12.61 repairs per mile (7.83/km)). These data suggest that not only is cracking more prevalent, the need to repair deteriorating cracks is much higher when ACBFS is used as a concrete coarse aggregate.

Table 33. Number and Density of Full-Depth Repairs (FDRs) on I-75 From Goddard to Sibley (Robords 2010a)

	Left Lane			Center Lane			Right Lane		
	Coarse	FDRs		Coarse	FDRs		Coarse	FDRs	
Location	Aggregate	No.	Density	Aggregate	No.	Density	Aggregate	No.	Density
NB 1-75 Sibley to Goddard	ACBFS	20	4.25	ACBFS	35	7.45	ACBFS	124	26.38
SB 1-75 Goddard to Eureka	Limestone	1	0.42	Limestone	2	0.83	Limestone	1	0.42
SB 1-75 Eureka to Sibley	ACBFS	0	0.0	ACBFS	29	12.61	Limestone	5	2.17

Density = no. of repairs per mile

Evaluations of Jointed Plain Concrete Pavement Performance

Smiley (2007) performed an evaluation on 15 jointed plain concrete pavement (JPCP) projects that were constructed from 1995 through 2000. The projects were located along Michigan's major freeways: I-94, I-96, and I-75. Five projects showed signs of early midpanel cracking, which is considered very detrimental to JPCP performance. Poor construction practices were linked to three of the five failures, with the lack of stockpile moisture management being a contributing factor in one of the projects. Of these projects, I-96 and I-94 contained concrete with ACBFS as the coarse aggregate. The causes of the midslab cracking for these two projects appear to be similar, with midslab cracking being attributed to four major factors. The causes are low load transfer efficiencies (LTEs), loss of support beneath the slab, high truck loading, and crack sensitivity and propagation. Additional causes are presented in table 34.

Table 34. Causes of Cracking and Suitability of Preventative Maintenance
(From Smiley 2007. © Michigan Department of Transportation. Reprinted with permission.)

Cause of Cracking	Suspected	Confirmed	Correct with Previous Maintenance? Yes	Correct with Previous Maintenance? No
Design				
Missing isolation joints		X		X
Slab joint location		X		X
Joint spacing		X		X
Construction				
Cross section not per plan		X		X
Frozen base/subbase		X		X
OGDC segregated or not compact	X			X
Subgrade disturbed – not repaired		X		X
Poor concrete consolidation	X			X
Late relief sawing at joints	X			X
Hot weather construction		X		X
Early loading by construction vehicles	X			X
Material				
High concrete CTE	X			X
Low concrete toughness	X			X
Aggregate interlock lacking < LTE		X		X
Concrete shrinkage—wider relief crack		X		X
Non-Category				
Environmental impacts	X			X
Heavy truck loading	X			X

OGDC = open-graded drainage course; CTE = coefficient of thermal expansion; LTE = load-transfer efficiency

In some circumstances, JPCP appears to be susceptible to top-down transverse cracking from high combined stresses resulting from a loss of slab support as the slab upwardly curls at joints and corners due to a temperature gradient while simultaneously being loaded at opposite joints from a multi-axle truck. Smiley (2007) explained that the I-94 and I-96 projects have the highest deflections with a significant loss of support at the transverse joints. Deflection values are highest in the morning, when a negative temperature differential exists from top to bottom, resulting in upward curling while at the same time opening the joints and reducing LTE. “Slab rocking” in the morning, which signifies that high slab deflections are occurring, was very evident when the researcher made site investigations. The loss of support depends upon the actual temperature differential. Without adequate support for the slab, the dowels at the joints quickly experience high dowel bearing stress that can lead to socketing of the dowel and reduced LTEs.

Probably the most dramatic finding was the disparity in LTE when measured on both sides of the joint, sometimes with a difference of nearly 50 percent. The most probable explanation for this difference is the rapid slippage that occurs as the approach side of the joint shears (depresses) against the leave side of the crack. As the coarse aggregate particles break down (loss of interlock), LTE decreases, which forces the dowels alone to carry the load. LTE values were measured as low as 20 percent.

The width of the relief crack after thermal contraction will directly determine the endurance level of the initial aggregate interlock capacity. Excessive mass shrinkage, which most often occurs when construction is done during hot summer weather (as was the case for I-96 and I-94), will result in larger than normal relief cracks. This situation tends to amplify the effect on aggregate interlock because a wider relief crack at the sawed joints results in larger joint deflections from decreased aggregate interlock caused by more differential slippage across the crack interface. If the ACBFS used on I-96 and I-94 was batched drier than SSD, this would contribute to additional shrinkage, further increasing joint openings and thus exacerbating the problem.

In the most recent study, eastbound I-94 had 74 percent of the slabs cracked full width. For westbound I-94, 36 percent of the slabs were cracked full width. It was noted that there was no moisture control of the ACBFS coarse aggregate stockpile during the construction of the eastbound lanes, and thus it is likely the ACBFS was batched drier than SSD. As a result, it was observed that the concrete mix was frequently stiff and difficult to place. The stockpile was occasionally wetted with sprinklers during the paving of the westbound lanes the following year, and the paving quality improved.

MAINTENANCE COST COMPARISON

Krom (2010) performed a maintenance cost comparison between concrete pavements containing ACBFS coarse aggregate and those containing coarse aggregate from natural sources (i.e., natural gravel and crushed quarried stone) using the maintenance cost data in the MDOT database. Pavements built between 1977 and 2002 were considered for this analysis, and the only criterion for selecting pavements to be included in this analysis was the type of coarse aggregate, with no distinction made between other factors such as joint spacing or base type. Data from 148 projects were analyzed, with 107 projects being concrete pavements with natural aggregates and the remaining 41 being concrete pavements made with ACBFS aggregate. Maintenance projects that were let through December 2008 were considered for this analysis.

The maintenance summary for projects that had natural aggregate is as follows (Krom 2010):

- 56.5 projects (52.8 percent) had maintenance. Of these that had received maintenance, 1.5 projects (1.4 percent) were reconstructed (0.5 of a project refers to one bound, or a portion of a bound, on a project).
- 3 projects (2.8 percent) received an inlay (remove and replace right lane) as their first maintenance activity.

- 4.5 projects (4.2 percent) were reconstructed before any maintenance was performed.
- 2 projects (1.9 percent) were turned into a composite pavement before any maintenance was performed.
- 41 projects (38.3 percent) had not received any maintenance (these were built between 1986 and 2002).

The maintenance summary for projects that had ACBFS aggregate is as follows (Krom 2010):

- 25.5 projects (62.2 percent) had maintenance.
- 1 project (2.5 percent) received an inlay (remove and replace right lane) as its first maintenance activity.
- 3 projects (7.3 percent) were reconstructed before any maintenance was performed.
- 3 projects (7.3 percent) were turned into a composite pavement before any maintenance was performed.
- 8.5 projects (20.7 percent) had not received any maintenance (these were built between 1994 and 1999).

The maintenance costs in MDOT's pavement management system were used to compute historical average per-lane-mile maintenance costs (Krom 2010). The costs were obtained for the two aggregate classes, and then averaged for each maintenance cycle. These costs were then converted into 2008 dollars using the producer price index for highway and street construction. The computed maintenance costs are shown in table 35. These maintenance costs do not include costs associated with reconstruction, inlays, or placing an overlay.

Table 35. Average Cost per Maintenance Cycle
(From Krom 2010. © Michigan Department of Transportation. Reprinted with permission.)

Coarse Aggregate	Average Per-Lane-Mile Maintenance Costs in 2008 Dollars (No. of Projects With Maintenance Performed)						
	Cycle 1	**Cycle 2**	**Cycle 3**	**Cycle 4**	**Cycle 5**	**Cycle 6**	**Sum of All Cycles**
Natural	\$40,690 (56.5)	\$36,340 (26.5)	\$52,093 (9.5)	\$36,634 (1.5)	\$73,619 (0.5)	n/a 0	\$239,375
ACBFS	\$62,946 (25.5)	\$47,034 (13)	\$84,289 (6.25)	\$115,591 (4)	\$103,653 (1.25)	\$51,503 (1)	\$465,015

Note: Numbers of projects with maintenance performed are shown within parentheses.

Krom (2010) also reported the average maintenance cost (table 36), which was calculated by first summing the per-lane maintenance cost for each project for all cycles, and then computing an average cost. As seen in this table, the average per-lane-mile repair cost for pavements containing ACBFS aggregate was significantly higher than for those that contained natural aggregate.

Table 36. Average Maintenance Expenditures
(From Krom 2010. © Michigan Department of Transportation. Reprinted with permission.)

Coarse Aggregate	Average Maintenance Expenditure (Per-Lane-Mile Cost in 2008 Dollars)
Natural	$71,666
ACBFS	$136,473

In another study, Robords (2010a) evaluated the maintenance costs of concrete pavements constructed with different aggregate types. In this study, a search was performed to locate either adjoining sections of concrete pavements in which only the coarse aggregate type varied or to identify paving projects that used more than one aggregate type. The two adjacent pavement segments had to satisfy the following criteria: have the same design structure, have the same joint spacing, and be subjected to the same traffic level. Once the pavement segments meeting the criteria were identified, a survey of the cost of repairs per lane-mile for both pavement segments was performed.

Table 37 presents the results with the costs adjusted to 2009 dollars using the consumer price index. The repair cost shown in table 37 includes only the cost of full-depth concrete repairs in the right lane; it does not include costs associated with partial-depth repairs, asphalt patching, or joint sealing. Table 37 shows that pavements containing ACBFS aggregate had significantly higher repair costs than pavements containing natural aggregates.

Table 37. Cost of Repairs per Lane-Mile
(From Robords 2010a. © Michigan Department of Transportation. Reprinted with permission.)

Project Location	Year Constructed	Survey Year	Base Type	Cost of Repairs per Lane-Mile by Aggregate Type		
				ACBFS	Gravel	Limestone
I-94, Masonic Blvd. to St. Clair Hwy.	1963–1964	1989	Dense-graded	$84,437	$11,339	$0
I-75, Mt. Morris Rd. to Bridgeport	1958 Gravel 1961 Limestone	1989	Dense-graded	Not used	$87,349	$0
M-59, Dequindre Rd. west 2.94 mi.	1972	2003	Dense-graded	$152,915	$53,195	Not used
I-69 EB, US-127 to 1.4 mi. East of M-52	1991 ACBFS 1990 Gravel 1988 Limestone	2003	Open-graded	$114,208	$8,116	$17,065
I-69 WB, 1.4 mi. East of M-52 to US-127	1991 ACBFS[1] 1990 Gravel 1988 Limestone	2003	Open-graded	$124,808	$21,992	$12,217
I-75 Goddard Rd. to Sibley Rd.	Southbound 1991	2010	ATB*	Not used	Not used	$2,257
	Northbound 1991	2010	ATB*	$44,745	Not used	Not used

1 Right lane of ACBFS section replaced in 2007
* ATB = asphalt-treated base

PERFORMANCE OF CONCRETE PAVEMENTS ON STABILIZED BASES

MDOT uses the Distress Index (DI) to denote the amount of distress present on a pavement. The DI is zero for a pavement that has no distress, while a pavement with a DI of 50 is considered to have zero remaining life. The DI is computed based on the type, extent, and severity of various distresses that are present. MDOT captures video images of the pavement surface of the outside lane at highway speeds on a 2-year cycle using a van equipped with cameras, and the captured images are reviewed later to document the distresses and compute the DI of the pavement. MDOT uses these time-sequence DI values to evaluate the pavement performance over time.

Robords (2010b) studied the change in DI over time for concrete pavements constructed on stabilized bases. An average DI was computed for each pavement segment, where a pavement segment consists of a project in one direction of travel. As most projects are constructed in both travel directions, one project will often yield two segments.

The DI-versus-age curves for concrete pavements containing quarried aggregate, gravel aggregate, and ACBFS aggregate are shown in figures 23, 24, and 25, respectively (Robords 2010b). Robords (2010b) reports that minor variations in DI value (less than 5) can occur from year to year because of how distresses are identified from the videotapes. For example, if slight moisture is present during the time when the pavement is videotaped, more cracks are visible compared to the situation when no moisture is present. These figures show that pavement segments containing ACBFS aggregate show a higher rate of increase in DI when compared to those with either quarried or gravel aggregates. Three pavement segments out of the total of eight pavement segments shown in figure 25 show a steep increase in DI. These three pavement segments have subsequently undergone rehabilitation, which occurred before 12 years of service.

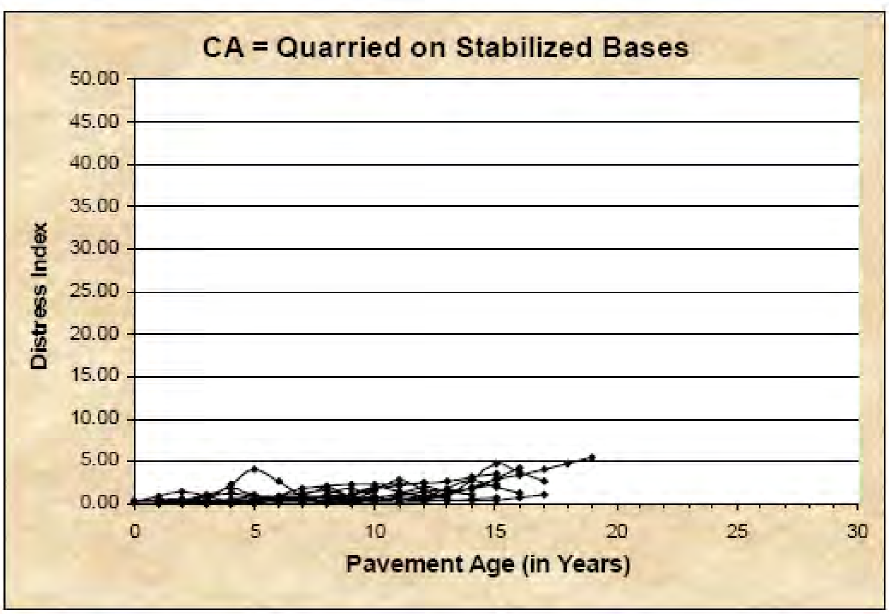

Figure 23. DI versus pavement age for pavements with quarried coarse aggregates. (Robords 2010b; © Michigan Department of Transportation 2010. Reprinted with permission.)

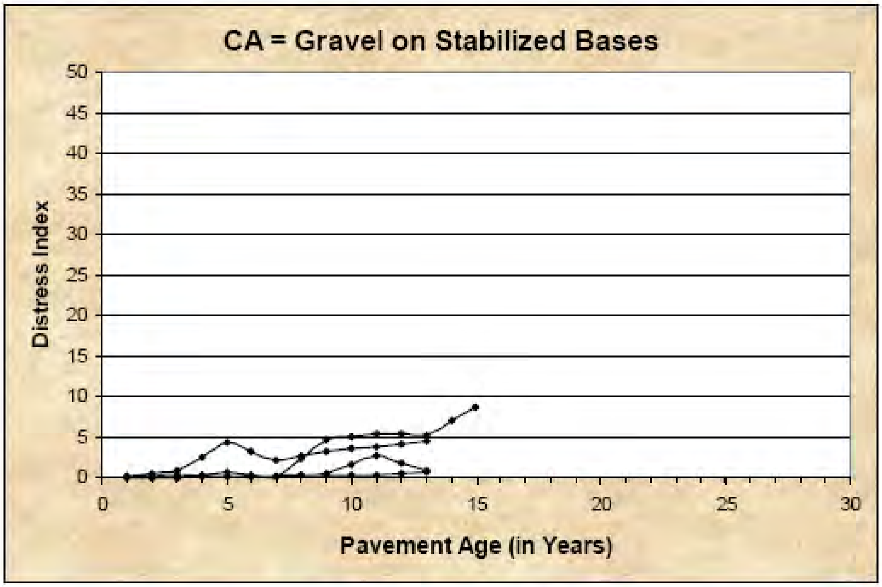

Figure 24. DI versus pavement age for pavements with gravel aggregates.
(Robords 2010b; © Michigan Department of Transportation 2010. Reprinted with permission.)

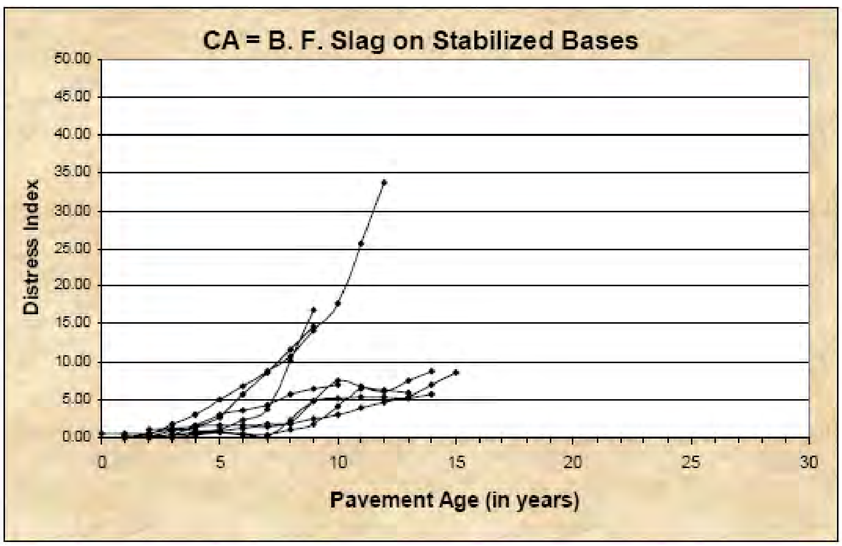

Figure 25. DI versus pavement age for pavements with ACBFS aggregates.
(Robords 2010b; © Michigan Department of Transportation 2010. Reprinted with permission.)

Figure 26 shows the mean DI curves for the three coarse aggregate types. As the pavement segments have been constructed at various times, it should be noted that the number of segments used for computing the mean DI curve decreases with increasing time. The mean DI curves show that the pavements with ACBFS aggregate (shown by the dashed line) have the highest increase in DI, and the pavements with quarry aggregate have the lowest increase in DI. The curve corresponding to ACBFS aggregate diverges from the other two curves at 5 years. The curve corresponding to gravel aggregate diverges from that corresponding to quarried aggregates after 8 years, and remains nearly constant up to 13 years. It should be noted the sample size used in this analysis, especially for the segments with gravel aggregates, is small, and that the statistical validity of the differences was not evaluated.

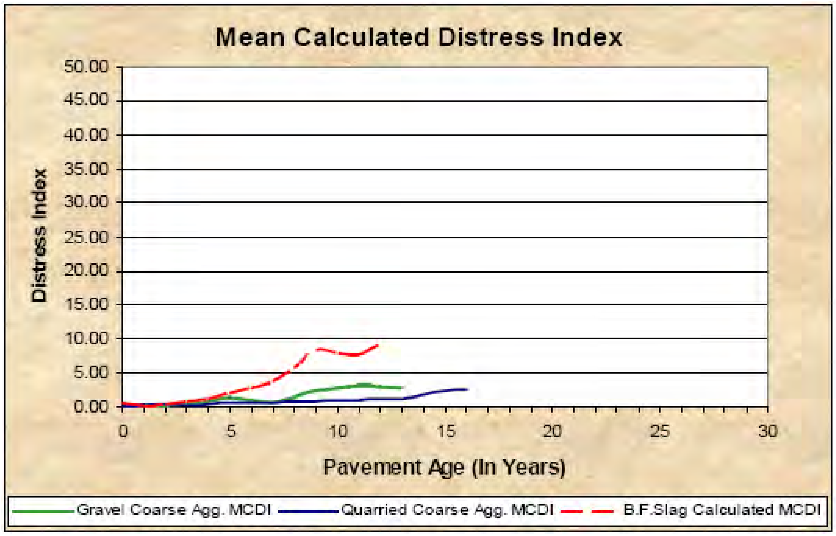

Figure 26. Mean DI curves for the three aggregate types.
(From Robords 2010b. © Michigan Department of Transportation 2010.
Reprinted with permission.)

AIR-COOLED BLAST FURNACE SLAG AND MATERIALS-RELATED DISTRESS

Although most early research on the performance of ACBFS as a coarse aggregate in concrete focused exclusively on the mechanical properties of the concrete and the resulting structural performance of pavements, work initiated in the mid-1990s began to look closely at the potential for MRD in Michigan's concrete pavements including those made with ACBFS coarse aggregate. The initial Phase I work conducted for MDOT was by Michigan Tech University in collaboration with Michigan State University. A large number of concrete pavements located around the State were evaluated representing various aggregate types, to determine the causes of observed deterioration (Hiller, Buch, and Van Dam 1998). The two primary distress mechanisms identified were paste freeze-thaw deterioration and ASR in the natural-sand fine aggregate. It was noted that although the ACBFS coarse aggregate was not directly implicated in the occurrence of the observed distresses, it was present in all of the cases where ASR was observed. However, no direct relationship between ACBFS and ASR was established.

The results of the Phase I study led to a more detailed analysis of the factors contributing to the occurrence of MRD in concrete pavements in Michigan (Hammerling 1999; Hammerling et al. 2000; Van Dam et al. 2001; Van Dam et al. 2002; Van Dam et al. 2003; Sutter, Van Dam, and Peterson 2009), as well as multiple other smaller studies evaluating the performance of concrete bridge barriers (Delem et al. 2004), bridge decks, local roads, and airports. Additional work was conducted by Grove, Bektas, and Gieselman (2006), the results of which were consistent with many of the observations made by others.

Based on this work, the following factors were identified as contributing to the MRD observed in concrete pavements and bridge barriers made with ACBFS coarse aggregate in Michigan:

- The fine aggregates located through much of southeast Michigan (as well as pockets located throughout the Lower Peninsula) have reactive chert and siltstone constituents, making them susceptible to ASR.

- The cement used in many instances had a high alkalinity (Na_2O_{eq} in excess of 1.0 percent), and high cement factors were often used.

- A Class C fly ash was often used in concrete that suffered ASR. It was often added at a 15 percent replacement for cement, which is probably at or near the pessimum limit, meaning it probably made ASR worse. The Class C fly ashes available in southeast Michigan had high total alkali contents (some had Na_2O_{eq} in excess of 7.0 percent), based on oxide analysis.

- The crystalline calcium sulfide (oldhamite) phase in the ACBFS aggregate in contact with hydrated cement paste was observed to have gone through dissolution, resulting in the formation of extensive secondary ettringite in the entrained air void.

Michigan Technological University and Michigan State University Study

Van Dam et al. (2002; 2003) investigated the occurrence of MRD on 14 concrete pavement projects in Michigan. The work consisted of visual distress surveys, nondestructive deflection testing, strength and permeability testing, coring, microstructural characterization of the concrete using stereo and petrographic microscopy, and chemical methods to identify the cause of MRD. The coarse aggregate types included in the study were natural gravel (eight projects), crushed limestone (two projects), and ACBFS (four projects). The results from this study relevant to ACBFS are presented below.

Petrographic analysis of concrete samples indicated that the original air content of concrete made with ACBFS aggregate ranged from 9.3 to 10.9 percent, which was higher than the specified air content of 5.5 to 8.0 percent. This higher air content did not seem to have a large effect on strength, but did illustrate difficulties in controlling the air content of concrete that contains ACBFS. It was also indicated that the coarseness of the ACBFS particles may have led to more vigorous agitation of the fresh paste during mixing, resulting in an increase in entrained air content.

In addition, evidence suggested that an unknown relationship exists between the occurrence of deleterious ASR and the presence of ACBFS coarse aggregate. The premise for this was the observation that the same chert constituents that were highly reactive and deleterious (meaning that concrete degradation was directly attributable to the ASR) in concrete containing ACBFS aggregate were observed to be nondeleterious in concrete made with naturally derived coarse aggregate. However, one concrete pavement with ACBFS aggregate that contained Class F fly ash did not show signs of ASR even though the concrete contained known reactive chert constituents in the fine aggregate. This suggests that Class F fly ash may be advantageous in addressing ASR in pavements containing ACBFS aggregate, although additional work is needed to substantiate that observation.

The authors also noted a deterioration mechanism that was only seen in concrete pavements constructed with ACBFS aggregate and led to a complete breakdown of the concrete matrix close to the joints. Based on petrographic analysis and thermodynamic calculations described earlier, it was hypothesized that under certain circumstances, especially in the presence of certain Class C fly ashes, a type of internal sulfate attack may be responsible for this observed deterioration that led to the formation of high amounts of secondary ettringite readily observed infilling the air-void system.

An additional phase of this work involved the petrographic analysis of cores from six concrete pavements, four containing ACBFS and two made with carbonate coarse aggregate (Sutter, Van Dam, and Peterson 2009). The pavements varied in age, being constructed from 1978 to 2000, and in condition, with some being considered in excellent condition and others having high amounts of MRD. The following general conclusions were drawn from this study:

- Deleterious ASR in the fine aggregate was responsible for the high level of distress in the newer I-696 sections (constructed in 1995) but was not observed in the older sections (constructed in 1978), even though they both were constructed with ACBFS coarse aggregate and similar fine aggregate. The air-void system varied from adequate to poor with little infilling except in isolated cases.
- Examination of cores from M-5 (constructed with ACBFS) found that the sections that included fly ash had a lower occurrence of distress, yet it is uncertain what role that played in the development of distress, which was predominately deterioration along the longitudinal joint. In general, the air-void system was adequate and relatively free of secondary ettringite infilling. No reason is given for the occurrence of distress.
- M-59, constructed in 1997/1998 with ACBFS aggregate, was significantly affected by ASR in the fine aggregate to varying degrees. For the most part, the air-void system was adequate to marginal with some minor infilling. The ASR appears to be the primary cause of distress.
- Cores obtained from M-14, constructed in 2000 with a carbonate aggregate, all appeared to be in good condition even though there was longitudinal spalling with staining present. Although the fine aggregate appeared to be reactive, there were no signs of deleterious expansion or cracking. Although the total air content met specification, the air-void system was found to be inadequate to protect the concrete against freeze-thaw damage having spacing factors well in excess of 0.008 in. (0.200 microns). Little to no infilling was observed.
- US-23, SHRP Section 19, was constructed in 1993 with limestone aggregate. It has a high level of joint spalling. Although the fine aggregate indicates reactivity, few to no signs of deleterious ASR were observed. As was true with the previous section, although the total air content met specification, the air-void system was found to be inadequate to protect the concrete against freeze-thaw damage, having spacing factors well in excess of 0.008 in. (0.200 microns). No infilling was observed.
- US-23, Section B of the MDOT Aggregate Test Site, was constructed in 1992 with ACBFS aggregate. As discussed previously, transverse cracks in this section have deteriorated in recent years suffering significant amounts of spalling. Although the fine

aggregate is reactive, few signs of deleterious ASR were observed except in isolated instances. The concrete mixture included fly ash. The air-void system parameters were adequate with little infilling. The spalling is not associated with MRD.

The general conclusion of this study is that the predominant distress affecting the pavements under consideration is ASR, which occurred exclusively in concrete made with ACBFS. Further, when fly ash was used, it appears to have had a beneficial effect in mitigating the ASR. The two carbonate aggregate sections were both deteriorating due to poor air-void systems that were insufficient to protect the concrete against freeze-thaw deterioration.

Due in part to these results, a small additional study was carried out to investigate whether the ACBFS coarse aggregate could influence ASR in a reactive fine aggregate (Hiller et al. 2011). Five coarse aggregates were tested: natural gravel, limestone, recycled concrete aggregate (RCA), fresh ACBFS, and weathered ACBFS (which was treated with dilute hydrochloric acid to leach exposed calcium sulfide). ASTM C1260 was used to determine ASR potential for mortar made solely with the crushed coarse aggregate per the requirements of the standardized test, and then blends of aggregate made with 1.0 percent and 2.5 percent chert (of similar origin to that found in southern Michigan) added to the coarse aggregate. The 14-day average expansion results for the coarse aggregates with 0, 1.0, and 2.5 percent chert additions are presented in table 38. Also shown in table 38 is the change in 14-day average expansion (as a percent of the expansion of the control) for each aggregate type due to the addition of 1 and 2.5 percent chert. These results are plotted in figures 27 and 28.

Table 38. ASTM C1260 14-Day Average Expansion Results and Percent Change in Expansion Due to Chert Addition Compared to Control
(From Hiller et al. 2011. ©Michigan Department of Transportation 2011. Reprinted with permission.)

Coarse Aggregate Type	**Expansion 0% Chert**	**1% Chert Addition**		**2.5% Chert Addition**	
		Expansion	**% Change**	**Expansion**	**% Change**
Natural gravel	0.127%	0.158%	24.4%	0.28%	120.5%
Limestone	0.078%	0.1%	28.2%	0.21%	169.2%
RCA	0.135%	0.13%	-3.7%	0.245%	81.5%
Fresh ACBFS	0.065%	0.198%	204.6%	0.28%	330.8%
Weathered ACBFS	0.062%	0.112%	80.6%	0.255%	311.3%

RCA = recycled asphalt

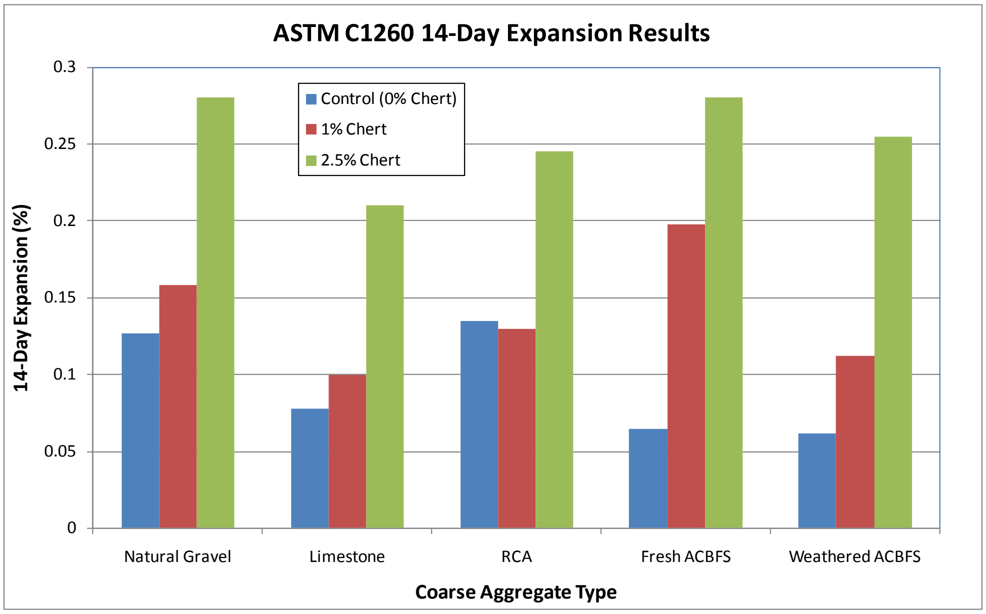

Figure 27. ASTM C1260 14-day average expansion for control (0 percent chert),
1 percent, and 2.5 percent chert addition.
(Hiller et al. 2011; ©Michigan Department of Transportation 2011. Reprinted with permission.)

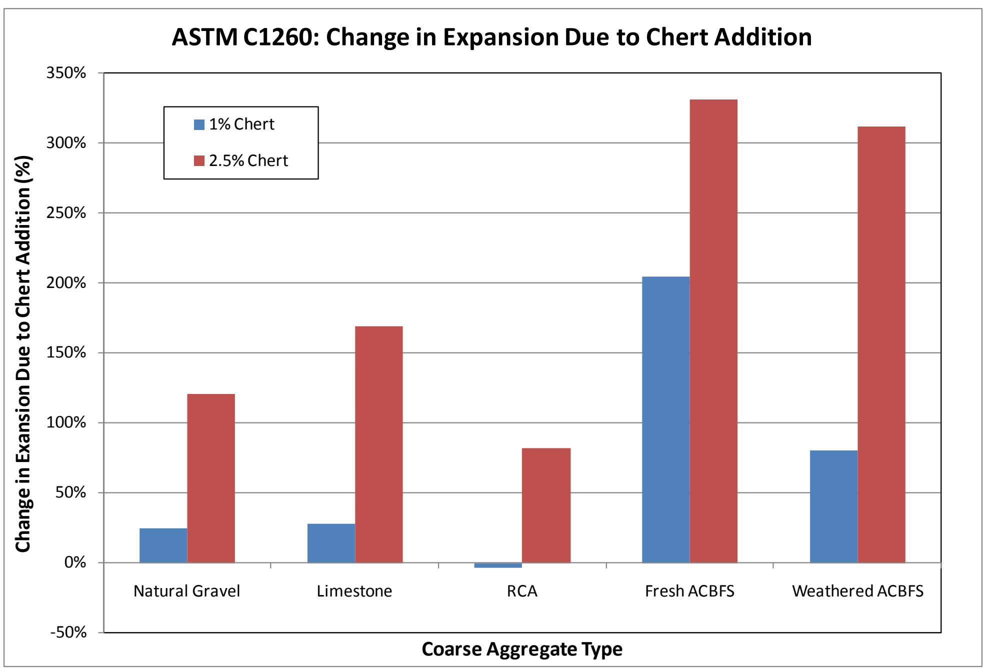

Figure 28. Percent change in ASTM C1260 average 14-day expansion resulting
from the addition of 1 percent and 2.5 percent chert.
(Hiller et al. 2011; ©Michigan Department of Transportation 2011. Reprinted with permission.)

It is observed that when tested in accordance with ASTM C1260, the natural gravel and RCA would be considered potentially reactive, having 14-day average expansions between 0.10 and 0.15 percent, whereas the limestone, fresh ACBFS, and weathered ACBFS would be considered innocuous, having expansions below 0.08 percent. In fact, the two ACBFS aggregates had the lowest rate of expansion at just over 0.06 percent expansion.

What is striking is how the addition of chert, even at 1.0 percent, significantly increased the expansion of the mortar bars made with ACBFS, especially those made with fresh ACBFS in which calcium sulfide was newly exposed as the coarse aggregates were crushed to size for the mortar in accordance with the ASTM C1260 standard. The fresh ACBFS specimens went from having nearly the lowest 14-day expansion at 0.065 percent to the highest at 0.198 percent, an increase of over 200 percent. In contrast, the two natural coarse aggregate sources saw an increase of 24.4 and 28.2 percent for the gravel and limestone, respectively. At the addition rate of 2.5 percent, all of the 14-day average expansions increased significantly, exceeding 0.20 percent, indicating that the higher percentage of chert is dominating the behavior; however, the fresh ACBFS specimens still had the highest 14-day average expansion at 0.28 percent, an increase of over 300 percent from the control.

It is speculated that the presence of calcium cations (Ca^{2+}) freed through the dissolution of calcium sulfide are rapidly being incorporated into the ASR gel, thickening it and resulting in the generation of high levels of expansion according to the mechanism presented by Ichikawa and Miura (2007). Additional work is required to determine the exact mechanism, but the result strongly suggests that the dissolution of calcium sulfide may contribute to aggressive deleterious ASR if susceptible fine aggregate is used and no mitigation strategy is employed to address it.

Deterioration of Concrete Bridge Barriers

Delem et al. (2004) assessed the possible causes for the premature deterioration of 16 concrete bridge barriers. Two of the sites, sites F and H, contained ACBFS as coarse aggregate. Signs of ASR were present (figure 29) in the deteriorated barriers, as were cracked fine chert particles with reaction rims and cracked fine black siltstones not directly linked to ASR. The concrete evaluated from these sections had high spacing factors near or exceeding the 0.008 in. (200 micrometers) limit in ASTM C457 and air contents well above the specified limit. The high spacing factors were attributed to an extensive amount of infilling of air voids with secondary ettringite.

Figure 30 shows the stereo-optical micrographs of the air-void structure in concrete from sites C (natural aggregate) and F (ACBFS aggregate). The white “dots” in the concrete from site F are ettringite-filled air voids. Similar infilling is not observed in site C. Stereo and petrographic optical examination found that ASR was observed in the fine aggregate fraction of these two ACBFS sites. Similar fine chert particles and siltstones were found in concrete without ACBFS that were not deleteriously reactive and the damage at those sites was attributed to freeze-thaw deterioration. Delem et al. (2004) concluded that the frost-susceptibility of the siltstones and the presence of secondary ettringite in the air voids might have exacerbated the distress caused by ASR in the concrete made with ACBFS coarse aggregate.

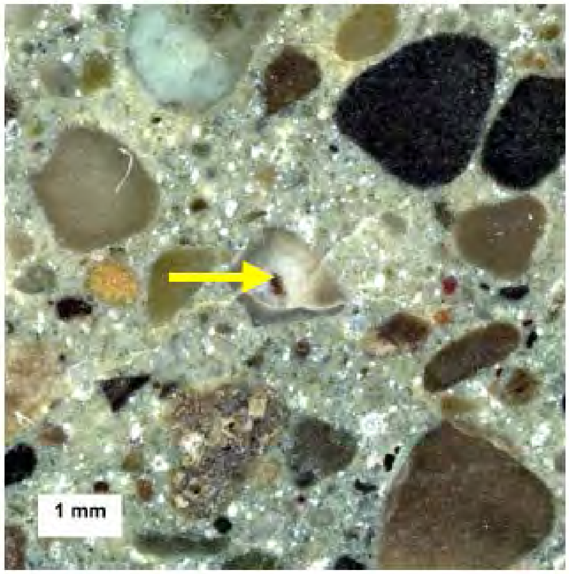

Figure 29. Alkali-silica reactive chert (note arrow) in concrete from Site F.
(From Delem et al. 2004. Evaluation of Premature Deterioration of Concrete Bridge Barriers by Petrographic Examination. In *Transportation Research Record No. 1893*, pp. 11–17. Copyright, National Academy of Sciences, Washington, DC, 2004. Reproduced with permission of the Transportation Research Board.)

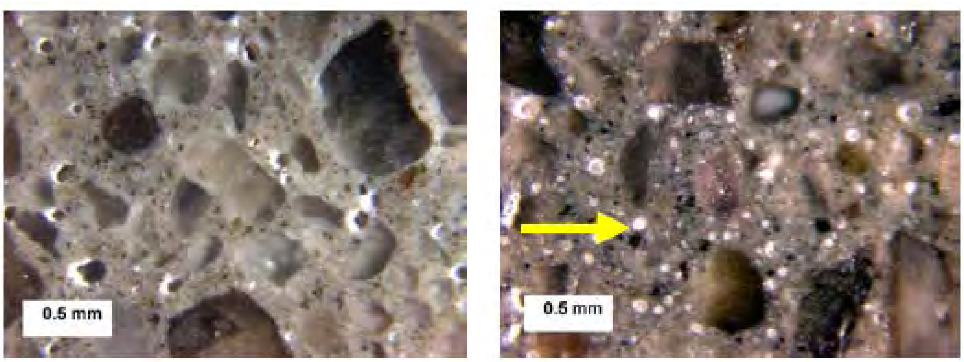

Figure 30. Stereo optical micrographs showing air-void systems in core C2 (left) and F1 (right). Arrow shows ettringite filled air void in F1.
(From Delem et al. 2004. Evaluation of Premature Deterioration of Concrete Bridge Barriers by Petrographic Examination. In *Transportation Research Record No. 1893*, pp. 11-17. Copyright, National Academy of Sciences, Washington, DC, 2004. Reproduced with permission of the Transportation Research Board.)

Southeast Michigan Local Road Study

Grove, Bektas, and Gieselman (2006) evaluated local concrete roads in southeastern Michigan to identify the factors contributing to early distress occurring in pavements built after 1991. The types of distress that had occurred included joint spalls/deterioration, midpanel cracks, and punchouts. Of particular concern were the joint spalling and other forms of deterioration that had occurred at a number of locations.

Twelve projects in southeastern Michigan were selected for this study and surveyed in November 2005. Most of them showed early pavement deterioration, predominantly at the joints. The median year of construction of these projects was 1995, with the construction year of the projects ranging from 1992 to 1999, except for one that was constructed in 1984. Nine of the projects were JPCP, and the other three were JRCP. Fly ash was used in only one of the projects. Of the 12 projects, 8 contained ACBFS as the coarse aggregate in the concrete, while carbonates were the coarse aggregate in the other 4 projects. Twenty-three cores were obtained from these projects and subjected to petrographic analysis.

Problems were noted regardless of whether the concrete was made with ACBFS or carbonate coarse aggregate. The two pavements in the best condition, as determined from a visual survey, contained carbonate coarse aggregate. However, the other two projects with carbonate aggregate were rated as being in poor condition. Although ACBFS aggregate was used in a number of poorly performing pavements, some projects where ACBFS was used as the coarse aggregate have performed well. The two projects observed to have concrete in the best condition, as determined from petrographic analysis, contained ACBFS aggregate. Therefore, it was clear that the cause of the distress observed on the pavements was not necessarily a coarse aggregate issue.

With regard to concrete made with ACBFS, the petrographic analysis found drying around most of the ACBFS aggregate particles in almost every core, indicating that the aggregate was dry and in an absorptive state when batched during construction. Grove, Bektas, and Gieselman (2006) reported this condition causes a lack of water in the area of the aggregate, thereby depriving the cement of water needed for hydration, which in turn has an effect on the strength and porosity of concrete. It was also indicated that dry aggregate can cause coalescing of air bubbles. The petrographic analysis also found metal inclusions in the ACBFS aggregate in six projects, although there was no evidence to suggest that these metal inclusions contributed significantly to concrete deterioration. The petrographic analysis showed secondary ettringite deposits infilling air voids to some degree in all cores. In the petrographic report prepared by the American Petrographic Services, Inc. (Lankard 2010b), it is stated that "the slag aggregate may be a source of sulfur that is contributing to the development of significant ettringite deposition in the air void system." The petrographic report also stated that "moderate to extensive secondary ettringite deposition, that has lined and filled entrained voids, has compromised the effectiveness of the air void system, and therefore, the durability of the concrete."

The petrographic analysis indicated that all of the 23 cores contained evidence of some degree of ASR. It was noted that the fine aggregate had a significant amount of chert, which as previously discussed, is very reactive. The petrographic analysis indicated that in 39 percent of the cores, the air-void spacing factor was not adequate to protect the concrete from freeze-thaw damage. Inadequate mixing, dry aggregate, or an admixture problem could have resulted in an inadequate spacing factor. Although the remaining cores had an adequate spacing factor, many of the cores had other issues related to air voids such as uneven distribution, clumping, and coalescing. It was reported that only 22 percent of the cores had an adequate and well-developed air-void system. It was also observed that projects with air contents above 6 percent, as determined from petrographic analysis of cores, had an acceptable spacing factor.

In the end, Grove, Bektas, and Gieselman (2006) concluded that the distress observed was caused by several deterioration mechanisms; hence, it was difficult to determine the cause that initiated the distress and identify other resultant distresses. It was reported that the major material issue was ASR due to the sand used in the concrete, while the major construction issue related to the distress was problems with the air-void system in the concrete such as an inadequate spacing factor and uneven distribution of air voids.

LABORATORY STUDIES OF ACBFS PERFORMANCE IN CONCRETE PAVEMENTS

Michigan State Study

In a laboratory study performed at Michigan State University, Bruinsma et al. (1995) investigated the relative effects of several pavement materials and design features on the rapid deterioration of transverse cracks in JRCP pavements. The effect of the following factors on the performance of transverse cracks was evaluated in this study: aggregate type (gravel, limestone, ACBFS, RCA), aggregate gradation (MDOT 6A, MDOT 17A, and blends of aggregates), foundation support, reinforcement type (smooth wire mesh and deformed wire mesh), reinforcement amount (0.16 and 0.23 percent), and slab tension (to simulate foundation friction effects).

Reinforced concrete slabs that were 10 ft (3.0 m) long, 4.5 ft (1.4 m) wide, and 10 in. (250 mm) thick were cast in the laboratory for this study. A transverse crack was induced in the slab using a metal joint insert that was 1 in. (25 mm) deep and 0.25 in. (6 mm) thick in the bottom of the slab at its center, and then jacking one end of the slab while the other end was fixed in place. This crack was induced about 18 hours after each slab was cast. The slabs were tested after 28 days of curing, being placed on a test stand and subjected to dynamic loading simulating the passage of a 9,000-lb (4,082 kg) wheel at 55 mi/h (89 km/h) over the crack. Deflections were recorded on both sides of the crack. The load applications continued until the reinforcing steel ruptured, which was evidenced by a rapid increase in the crack width. The load transfer at the crack was computed by dividing the deflection of the unloaded side by the deflection of the loaded side, and then expressing this value as a percentage. The load transfer versus the number of load cycles was plotted for each test, with the number of load cycles corresponding to the point on the load transfer history curve where a 45-degree line could be drawn tangent to the curve being defined as failure.

An endurance index was developed in this study to evaluate the load transfer performance of a slab. The endurance index was expressed as the percentage of the area under the curve of load transfer versus the logarithm of the number of load applications, compared to the area bounded by load transfer limits of 0 to 100 percent and logarithmic limits of 0 and 8 (i.e., 0 to 100 million load applications) as shown in figure 31. For this example, the endurance index is 22.3 percent. Several forms of the endurance index were examined in this study, and the version of the endurance index that was finally adopted used a load cycle limit of 10 million with the load application scale being linear instead of logarithmic.

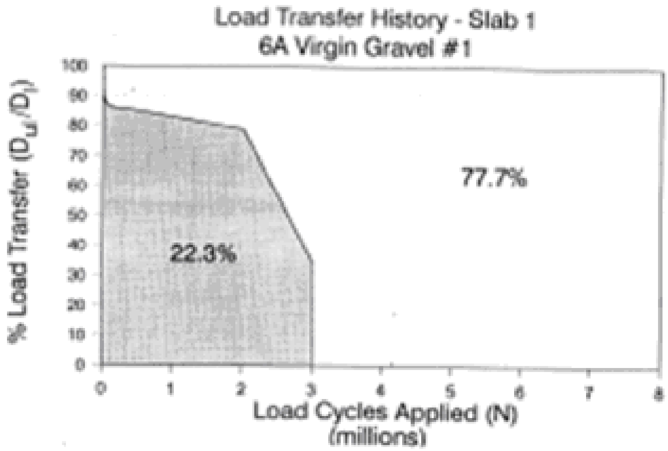

Figure 31. Illustration of endurance index.
(Bruinsma et al. 1995; © Michigan Department of Transportation 1995. Reprinted with permission.)

The following conclusions with regard to the use of ACBFS were drawn based on this study (Bruinsma et al. 1995):

- Better crack deterioration performance was observed in concrete specimens made with naturally derived aggregates (gravel and limestone) than in specimens made with ACBFS aggregate when all other factors were held constant. The ACBFS aggregate appears to have fractured at the time when the cracking occurred, whereas the limestone and gravel aggregates pulled out of the mortar. The crack face for the former was relatively smooth compared to the latter, resulting in lower load transfer values for the specimens with ACBFS aggregate.

- Reducing the tensile force induced in the slab was very effective in reducing the rate of deterioration of transverse cracks. Reducing the tensile force in the slab allows for better aggregate interlock at the cracks. The tensile force in slabs can be reduced by using shorter slabs or by reducing friction between the slab and foundation.

- Transverse crack deterioration was correlated to concrete strength. When ACBFS aggregate is used in concrete, mix designs should be developed that result in concrete strengths that are comparable to mixes made with virgin aggregates. In addition, structural designs should be such that stresses induced in the slab are appropriate when compared to the strength of the concrete.

- The following actions can be taken to reduce the transverse crack deterioration in JRCP: (1) providing stiff foundation support, (2) using deformed steel instead of plain reinforcements or using a larger quantity of steel, (3) using a short joint spacing to reduce tensile stresses induced in the slab, and (4) using a subbase material that minimizes the friction between the slab and the subbase.

Bruinsma et al. (1995) recommended that the structural designs for pavements made with ACBFS should minimize reliance on aggregate interlock for providing load transfer at either cracks or joints, and that appropriate structural designs for pavements with ACBFS aggregate might include JPCP having a maximum 16.4-ft (5.0 m) joint spacing, with the joints being doweled. The authors concluded ACBFS could be used for concrete pavements if appropriate mix designs were developed and with modifications to the structural design procedures.

University of Michigan Study

The University of Michigan performed a laboratory study evaluating JPCP whose objectives were to: (1) quantify the effect of coarse aggregate type, maximum particle size, and concrete age on the concretes resistance to cracking; and (2) quantify the relationship of coarse aggregate type and size on LTE of fully cracked JPCP slabs (Jensen and Hansen 2000; Hansen and Jensen 2001).

Beam tests were used to perform the research to address the first objective. Concrete beams 4 x 8 x 38 in. (100 mm by 200 mm by 1 m) were cast in the laboratory using five types of coarse aggregate. The coarse aggregate types were gravel from two sources, limestone, dolomitic limestone, and ACBFS. The nominal maximum aggregate size of all coarse aggregates was 1 in. (25 mm), and the aggregate conformed to MDOT 6AA specification. A notch was saw-cut into the beams at mid-span before testing. The complete load-deformation response of the notched beam subjected to center-point dynamic loading was obtained using an MTS servo hydraulic testing machine. Tests were performed on the beams at 7, 28, and 91 days of age. The load-deflection curves were used to determine the fracture toughness (a measure of resistance to crack initiation) and the fracture energy (a measure of the resistance to crack propagation) of the concrete made with the various types of coarse aggregate.

Concrete properties, such as compressive strength, elastic modulus, and split tensile strength, were similar for all five concrete mixes. Results from this study showed that concrete containing gravel had the highest fracture toughness, while concrete containing limestone and ACBFS had the lowest values. The fracture toughness of concrete with dolomitic limestone fell between those of these other two groups. The fracture toughness increased with age for all five aggregates. Similarly, the gravel aggregate samples had the highest fracture energy (values around (125 lbf/ft (170 N/m)), while the samples with limestone and ACBFS had the lowest values (around 59–88 lbf/ft (80–120 N/m)). The fracture energy of the sample with dolomitic limestone fell between those of these other two groups, with values around 103 lbf/ft (140 N/m). The study concluded that transverse cracks (full width and depth) might be delayed if aggregate types were selected that resulted in higher fracture toughness in concrete.

An examination of the fractured beams showed that for the gravel samples, a large percentage of aggregates remained intact after cracking for all three test ages. For the other three aggregate types, the 7-day testing showed partial cracking of coarse aggregate, but the fractured aggregates approached 100 percent for the 28- and 91-day tests.

Early-age resistance of concrete to crack initiation, determined from fracture toughness, is improved for concretes that develop higher early strength, as this property is based on the peak load. Concretes containing ACBFS and crushed carbonate coarse aggregate fall into this category. However, after cracking has initiated, concrete containing weaker coarse aggregate exhibit more brittle fracture behavior (relatively low characteristic strength) than concrete containing glacial gravel. It was also observed that the specific fracture energy is less for concrete made with ACBFS and that the resistance to cracking does not change significantly as the strength increases. An increase in the peak load is observed, but the post-peak behavior is only slightly affected, causing almost no change in the specific fracture energy. Further, the authors found that the concrete containing ACBFS loses the ability to transfer stresses across the crack at smaller deflections than the concrete with glacial gravel.

The second objective of the project was addressed by testing concrete slabs that were cast in the laboratory. Concrete slabs that were 9.8 in. (250 mm) thick, 11.8 ft (3.6 m) long, and 5.9 ft (1.8 m) wide were fabricated in the laboratory for this study. The slabs were constructed on a open-graded base course 4 in. (100 mm) thick that was underlain by a sand subbase 15.7 in. (400 mm) thick. The sand subbase was placed on a silty sand subgrade. Gravel, limestone, and ACBFS coarse aggregate with a nominal maximum aggregate size of 1 in. (25 mm) that met the MDOT specification 6AA were used in the slabs. A notch was sawed at the midpanel, and a crack at the midpanel was induced in the slabs using an actuator when the slab had achieved 70 percent of the 28-day split tensile strength. Midpanel cyclic load tests were conducted on one side of the crack using a load level of 9000 lb (4,082 kg), and the deflections were measured on the loaded and the unloaded slab. The evaluation was carried out for seven or eight crack opening widths ranging from 0.012 to 0.1 in. (from 0.302 to 2.54 mm). For each crack width, 300,000 load cycles were applied.

The authors reported that the deflection-based load transfer was adequate (i.e., 60 percent or higher) and remained constant for load repetitions up to 300,000 for crack widths up to 0.035 in. (0.889 mm) for all three aggregate types. However, for crack widths greater than 0.035 in., only the concrete made with gravel aggregate maintained a LTE above 60 percent, which is considered a critical level for high and medium traffic. Evaluation of the crack faces showed that the concrete slabs with limestone and ACBFS showed smooth and straight-line crack paths with a high percentage of fractured aggregates. This crack path characteristic generates low specific fracture energy values and brittle behavior. This type of cracking (caused by mechanical overloading and hot weather construction) is undesirable if it occurs in a JPCP, as it is expected to dramatically reduce LTE and would be a contributing factor for development of premature distress. The concrete slabs with the gravel aggregate had protruding aggregates across the crack face, thereby providing high load transfer.

A similar study was conducted using a blended aggregate having a nominal maximum aggregate size of 2 in. (50 mm) for coarse aggregate types of gravel, limestone, and ACBFS. Increasing the aggregate size for the gravel aggregate had a positive impact on LTE for crack widths greater than 0.035 in. (0.889 mm), with the LTE being about 80 percent for crack widths of 0.1 in. (2.54 mm), compared to 60 percent for maximum aggregate size of 1 in. (25 mm). However, for limestone and ACBFS aggregate, the aggregate with a 1-in. top size performed better than the aggregate with a 2-in. top size. The authors indicated that this may be due to the high percentage of cracked aggregates and the loss of aggregate interlock points.

The overall conclusion from this study was that concrete containing ACBFS is more susceptible to early loss of LTE because of greater crack widths. The project found that the LTE for ACBFS concrete drops dramatically when the crack width exceeds 0.035 in. (0.889 mm).

Michigan Tech Study

In a study conducted at Michigan Tech University, the static and dynamic strengths of various coarse aggregates, and of concrete made with them, were assessed in an attempt to link fundamental material strength properties to pavement performance (Vitton, Subhash, and Dewey 2002; Subhash, Vitton, and Chengyi 2008). Cores (0.38 in. diameter) of aggregate material were used for testing, being obtained from samples weighing between 115 and 450 lb (52 and 204 kg). These samples were obtained from quarries after a production blast prior to crushing, while ACBFS samples were obtained from steel mills. The results from these tests are shown in figure 32.

The ACBFS samples 1 and 2 were obtained from Algoma (Sault Ste. Marie, Ontario), whereas sample 3 was obtained from the Levy plant in Detroit. Sample 1 was only subjected to air cooling, while samples 2 and 3 were sprayed with water during production. The samples from Algoma showed ACBFS with two different structures, one which was relatively dense and the other having significant porosity. During the cooling process, the surface that is exposed to the atmosphere cools rapidly, while the material at the bottom is insulated to some degree and cools more slowly. This difference in cooling rates will result in different mechanical properties of the material (Subhash, Vitton, and Chengyi 2008).

In figure 32, the result shown for sample 1 as 1.2 was obtained from a dense portion of ACBFS, while the other result was obtained for a sample that had more pores. As shown in figure 32, ACBFS aggregate was found to have the lowest static strength, being rated as Category E: Very Low Strength according to the Deere and Miller Rock Classification System. However, the dense portion of the ACBFS from Algoma (sample 1.2) was rated as Category C: Medium Strength for static strength, being very close in strength to dolomites. Under dynamic testing, the strength of almost all aggregates increased by one category.

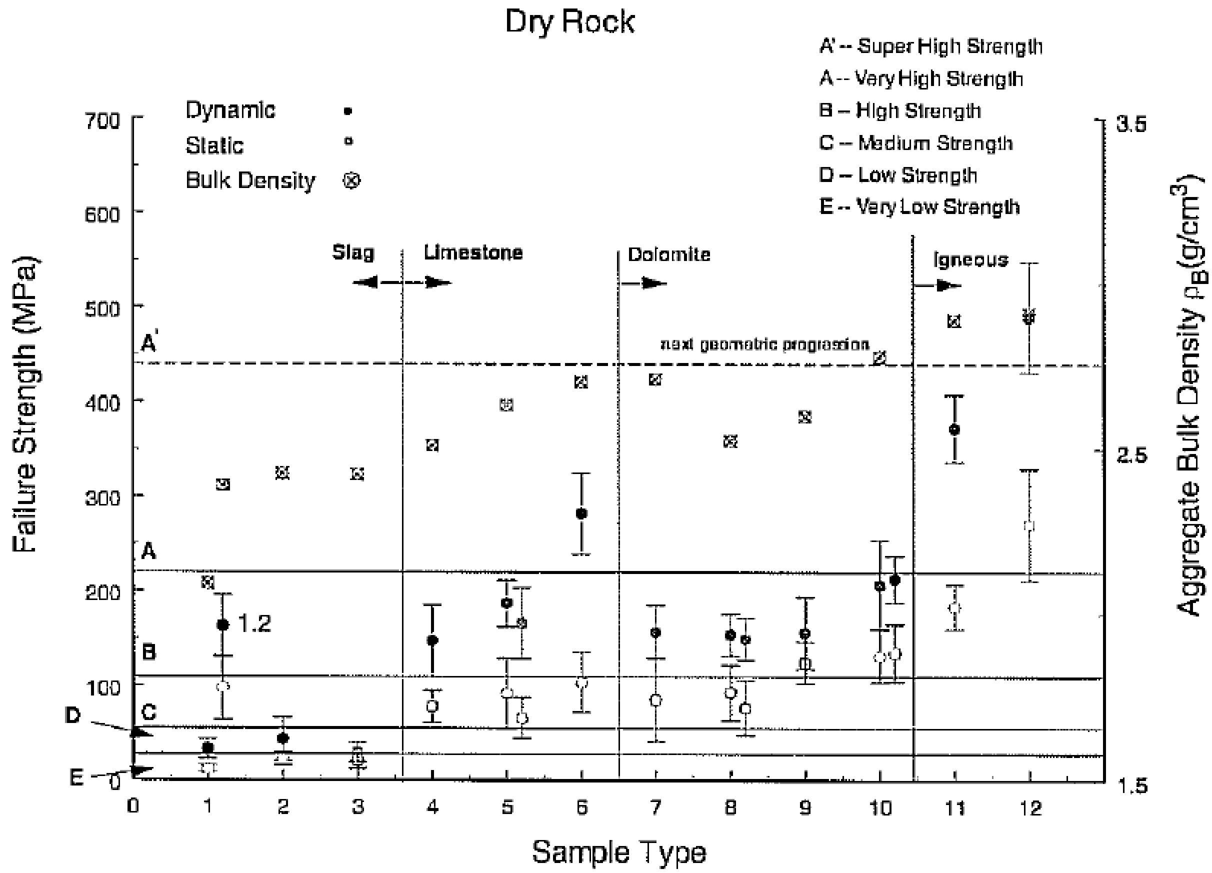

Figure 32. The Deere and Miller Strength Classification System and results of static and dynamic testing of various aggregates in dry condition. (From Vitton, Subhash, and Dewey 2002; © Michigan Department of Transportation 2002. Reprinted with permission.)

The ability of an aggregate to resist dynamic loading compared to static loading can be expressed by the strain rate sensitivity. This parameter shows the increase in compressive strength for a given change in the applied strain rate. The effectiveness of an aggregate to resist dynamic loads can be assessed by this parameter (Subhash, Vitton, and Chengyi 2008). The strain rate sensitivity is calculated by obtaining the difference between the dynamic and static fracture strengths, and then dividing this quantity by the logarithm of the ratio between the dynamic and static strains. The strain rates are computed by dividing the strain by the time to fracture. The air-cooled Algoma ACBFS porous section and the water-quenched ACBFS from Algoma and Levy had the lowest strain rate sensitivity values of all the tested samples. The air-cooled Algoma dense section had a strain rate sensitivity that was more than three times the strain sensitivity of the porous section from the same source (Subhash, Vitton, and Chengyi 2008). It is noted that static behavior is dominated by larger macrodefects (such as shrinkage cracking and larger voids), whereas dynamic behavior is affected by the microstructural inhomogenieties (such as micropores, microcracks, and impurities that exist along grain boundaries) (Vitton, Subhash, and Dewey 2002). Overall, the lowest compressive strength and strain rate sensitivity were obtained by ACBFS aggregate, and the cause for these low values is attributed to the high porosity of these aggregates (Subhash, Vitton, and Chengyi 2008). Limestones and dolomites had intermediate values with the highest compressive strength and strain rate sensitivity being obtained by mafic igneous aggregates (Subhash, Vitton, and Chengyi 2008).

The dynamic strength of concrete made with ACBFS was found to have characteristics similar to those of concrete made with naturally derived aggregate, leading the researchers to conclude that the behavior of the mortar dominated the dynamic response of the concrete.

The study also investigated aggregate interlock established at crack interfaces (Vitton, Subhash, and Dewey 2002). Citing research conducted at the University of Illinois that identified coarse aggregate type as dramatically affecting aggregate interlock (with higher strength aggregate providing better performance), the researchers designed an experiment to measure this for concrete made with ACBFS as well as naturally derived aggregates. This phase of the research experienced some technical difficulties due to the test setup and equipment failure, yet enough information was gathered to conclude that at narrow crack widths (up to 0.024 in. (0.610 mm)), the coarse aggregate type has little influence on aggregate interlock. For crack widths greater than 0.035 in. (0.889 mm), coarse aggregate having higher strength and stiffness will have a lower rate of aggregate interlock degradation. This conclusion is largely based on the work conducted at the University of Illinois, which was validated by the limited testing that was performed at Michigan Tech. It was also shown that stronger aggregate will result in a concrete fracture surface at joints that is rougher than that created by weaker aggregates, thus enhancing aggregate interlock.

Another observation from this study was that concrete specimens made with ACBFS required higher tension forces to fracture them at 18 hours than similar concrete specimens made with naturally derived aggregate, and that the fracture always occurred through the ACBFS aggregate. This was not the case with high-strength, naturally derived aggregates, in which the fracture often occurred around aggregates at their interface with the paste. This distinction resulted in the differences in LTE previously discussed, but might also have practical impacts that have not been discussed.

RECORDED GOOD PERFORMANCE OF ACBFS PAVEMENTS IN MICHIGAN

Although there have been a number of documented issues with the performance of concrete pavements containing ACBFS aggregate, several examples of ACBFS concrete pavements that have performed well were identified in the literature. The first is a section of I-696 westbound near the I-94 interchange in Roseville, Michigan, as documented by Sutter, Van Dam, and Peterson (2009). This section was constructed with ACBFS coarse aggregate in 1978, a JRCP 9 in. (225 mm) thick that remains in service with little sign of materials-related deterioration. As reported, this section was contrasted with an adjacent heavily distressed concrete section constructed in 1995 that also used ACBFS as a coarse aggregate. Although the fine aggregate used in both projects was similar, deleterious ASR was only observed in the 1995 section. No information was presented on the cement used, so it is not possible to determine what impact cement alkalinity may have played. No fly ash was observed in either section. It should be noted, however, that the focus of this investigation was on the occurrence of MRD and not specifically on the performance of ACBFS pavements. Furthermore, only current pavement conditions were examined without consideration of maintenance or rehabilitation expenditures.

Concrete containing ACBFS aggregate was also used successfully at the Detroit Metropolitan Airport on RW 3R/21L, which remained in service for 30 years before being replaced in 2006/2007 with a new concrete runway that also contains ACBFS. The original runway was constructed in 1973/1974 and was showing signs of MRD but still serving traffic when it was replaced. The major surface distress manifestation was map cracking that subsequent petrographic analysis determined was related to ASR in the fine aggregate. Even with the ASR, the pavement remained in service for 33 years, well exceeding its 20-year design life.

A concrete test section that used ACBFS as coarse aggregate was constructed on I-94 near Detroit Metropolitan Airport. This test section was constructed on both eastbound and westbound directions, with the limits of the section being from Oakwood Drive to Outer Drive. Construction was performed in 2005 by Ajax Paving Industries under an MDOT special provision using a quality control plan (included in appendix A) that was developed by the aggregate supplier, the Edw. C. Levy Co. A 10-year study was to be performed on this test section, performing visual reviews of the project every 6 months for 3 years, and visual reviews performed annually thereafter. Levy has been performing visual inspections of the site since construction, most recently in December 2009. MDOT recently observed severe spalling that has occurred in some of the shoulder areas and is most severe in sections constructed with ACBFS. Investigations are underway to determine the cause of the spalling, but preliminary petrographic analysis is assigning mechanical causation, as there are no signs of MRD (Anzalone 2011).

SUMMARY OF PERFORMANCE OF ACBFS PAVEMENTS IN MICHIGAN

The studies reviewed here indicate that the use of ACBFS as coarse aggregate in paving concrete requires special consideration in the design and construction process. From a design perspective, it has been shown that long-jointed JRCP constructed with ACBFS coarse aggregate have a tendency to crack more frequently than comparable pavements constructed with natural aggregate and that the cracks have a tendency to rapidly lose LTE, resulting in spalling. Thus, the use of JPCP with short joint spacing, dowel bars for transferring load across joints, and the use of a stiff stable base are all expected to contribute to improved performance of ACBFS pavements, but these are among some of the design changes that MDOT has adopted over the last decade. In fact, MDOT has had a long-established practice of using dowel bars in transverse joints, which significantly reduces the role of aggregate interlock in contributing to load transfer. Some concerns may still exist, however, regarding the possibility of dowel “socketing” since the dowels have to carry nearly the entire applied load if aggregate interlock is lost, especially if the joints open wider than 0.035 in. (0.889 mm) (Hansen and Jensen 2001; Vitton, Subhash, and Dewey 2002).

The largest construction issue raised is that of batching ACBFS aggregates drier than SSD, as this can lead to significant rapid absorption of concrete mixing water, resulting in reduced workability, shrinkage cracking, and possibly localized drying of the paste in the immediate vicinity of the ACBFS aggregate. Many of the problems cited regarding excessive shrinkage cracking of ACBFS pavements are thought to be linked, to some degree, to poor stockpile management resulting in concrete being batched with dry aggregates. Less absorptive, naturally derived aggregates maintained in less than SSD condition are less susceptible to erratic mixture issues, and thus specific attention needs to be paid to aggregate moisture content when ACBFS is used.

Data from the MDOT's pavement management system have been collected and compared to assess the cost of maintaining concrete pavements with and without ACBFS in otherwise similar designs. Data from 148 projects were analyzed (107 with natural aggregate and 41 with ACBFS aggregate), representing maintenance projects let through December 2008. The results indicate that average maintenance expenditure for ACBFS concrete pavements is nearly twice that of pavements with natural aggregate.

The final observations made in Michigan concern the potential contribution of ACBFS to MRD. Various investigators have found deleterious ASR occurring in the fine aggregate in the presence of ACBFS aggregate, while concrete containing naturally derived aggregates having the same fine aggregate have shown little or no signs of deleterious ASR. Further, the dissolution of calcium sulfide in the ACBFS is a commonly observed phenomenon, and once the sulfide oxidizes, it will provide an internal source of calcium cations and sulfate. The dissolution of calcium sulfide can result in infilling of air voids with secondary ettringite, which may compromise the freeze-thaw resistance of the concrete. In-depth studies on the potential impact of calcium sulfide dissolution on MRD have not been conducted at this juncture.

CHAPTER 5. ADDITIONAL USE OF ACBFS IN PAVING CONCRETE

INDIANA

The Indiana DOT (INDOT) has been using ACBFS in paving concrete for several decades, likely starting in the early 1980s (Nantung 2010). Most projects featuring the use of ACBFS are located in northwestern Indiana, as this is close to the blast furnaces located in Gary, Indiana, and Chicago, Illinois.

INDOT implemented the MEPDG design method in 2009, having worked on it since 2004. They obtained initial CTE information from the Long-Term Pavement Performance (LTPP) Program test results of Indiana ACBFS aggregate test sections. They also have an ongoing program of determining the CTE of concrete made with all aggregate sources used in Indiana, which is being conducted on a district by district. While the CTE of concrete made with ACBFS aggregate differs from that of most natural aggregates, it is not significantly different. As the CTE of concrete containing ACBFS is generally a little higher than for that made with most naturally derived aggregates, the required pavement thickness is slightly higher based on the MEPDG for a given joint spacing.

INDOT has also worked with the concrete paving industry on important design and performance issues such as reliability and the use of tied concrete shoulders. For projects with more than 30 million design equivalent single-axle loads (ESALs), the shoulder thickness is required to match that of the mainline pavement. For projects with less than 30 million ESALs, the shoulders can be flexible material, with 4 in. (200 mm) of surface and aggregate base thickness adjusted to match the mainline pavement. Concrete strength for pavements containing ACBFS aggregates are about the same as for pavements with other types of aggregate. INDOT requires a flexural strength of 550 lbf/in^2 (3.79 MPa) for opening to traffic, which is higher than required by most agencies.

INDOT has a strong quality control/quality assurance (QC/QA) program. For the construction of ACBFS aggregate pavements, INDOT relies heavily on the contractor's QC program. INDOT typically performs QA testing, using the contractor's equipment, but recently the agency is using consultants to assist in the QA role. INDOT requires that ACBFS aggregate be in SSD condition at batching after 15 hours of soaking.

Overall, INDOT is pleased with the performance of ACBFS as a coarse aggregate in paving concrete (Nantung 2010). ACBFS was used as the coarse aggregate for a large design-build project constructed in 2001 on I-65 just south of where it intersects I-80/I-94. For the most part, this project is performing very well after nearly 10 years in service, although there are isolated joint spalls along some longitudinal joints. The total cementitious materials content was approximately 570 lb/yd^3 (338 kg/m^3), which included 110 lb/yd^3 (65 kg/m^3) fly ash (20 percent replacement). The average *w/cm* was 0.39.

ACBFS coarse aggregate was also used in 2004 in another design-build project to construct a section of I-80/I-94 just east of the intersection with SR-912. This is one of the most heavily trafficked pavements in Indiana, and the pavement has been performing well with no observed distress.

Nantung (2010) also discussed a number of projects where ACBFS was used on State routes (SR-331, SR-19, SR-933, and SR-912) near South Bend and Calumet, Indiana; the oldest of these was constructed in 1983, with the others being constructed in 1994/1995 and from 2003 to 2005. The SR-331–Capital Avenue project was constructed in 2004 using a cement content that varied from 440 to 657 lb/yd^3 (261 to 390 kg/m^3) and containing 120 lb/yd^3 (71 kg/m^3) of slag cement. Most of the pavement is in good condition, although there is some spalling along longitudinal joints and minor deterioration at the intersection of longitudinal and transverse joints in some locations. The following year, another section of SR-331 was constructed immediately south of the previous project. That project, which remains in very good condition, used a lower cement content of 480 to 564 lb/yd^3 (285 to 335 kg/m^3) with 0 to 110 lb/yd^3 (0 to 65 kg/m^3) fly ash added. A section of SR-19 was constructed during the same timeframe, over 2003/2004. The cement content was 480 to 501 lb/yd^3 (285 to 297 kg/m^3) with 89 to 110 lb/yd^3 (53 to 65 kg/m^3) of fly ash added.

Two older pavement sections were described by Nantung (2010). The ACBFS section on SR-933 near Roseland was constructed in 1994/1995. This project suffered extensive deterioration of both the longitudinal and transverse joints and has been studied by a number of investigators (Liu 2005; Olek and Arribas 2006). The conclusion of these petrographic examinations is that the air-void system is marginal and that a significant number of air voids are filled with secondary ettringite. Further, in the work conducted by Liu (2005), the ettringite was discovered to not be isolated to the air voids, but instead was formed on aggregate surfaces and in other open spaces throughout the depth and mass of the concrete, whether deteriorated or sound. As stated by Liu (2005), “normally some ettringite is found in concrete in service, but the amount found here was more than normal.”

The oldest ACBFS pavement discussed by Nantung (2010) was SR-912 near Calumet, Indiana, in 1983. This JRCP has numerous cracked slabs and some spalling but is still in service. Signs of MRD are not readily apparent.

Nantung (2010) concluded his presentation by stating that newer concrete pavements constructed with ACBFS are performing well in a very harsh environment, whereas the older pavements have suffered some durability and/or performance issues. To ensure good performance, strict guidelines in handling ACBFS aggregates must be adopted and followed, including keeping aggregate stockpiles wet during batching operations.

NEW YORK

Amsler, Chamberlin, and Jaqueway (1975) reported on a study performed in New York to investigate cracking of concrete pavements containing ACBFS. A large amount of transverse cracking early in the life of a new concrete pavement containing ACBFS aggregate near Buffalo resulted in the inspection of 42 pavement segments in New York State Department of Transportation (NYSDOT) Region 5. These pavements represented 33.7 centerline miles

(54.2 km) of pavements containing ACBFS and 106.1 centerline miles (170.8 km) of pavements containing aggregates from local natural sources. The concrete slabs in these JRCPs were 60.8 ft (18.5 m) long. The survey found the average frequency of transverse cracks in pavements containing ACBFS aggregate to be seven times greater than that in similar pavements containing local natural aggregates. No significant differences were noted in the location of cracks within the slabs for slabs with and without ACBFS. Variables analyzed in this study include (1) pavement thickness; (2) slab length; (3) type of load transfer device; (4) weight, style, and location of steel-mesh reinforcement; (5) cement brand; (6) fine aggregate source; (7) coarse aggregate source; and (8) concrete mix design. No relationship was observed between these variables and the observed cracking other than the use of ACBFS coarse aggregate. Ride quality measurements were made on a sample of these pavement segments using a PCA Roadmeter and a BPR-type roughometer, and no significant difference in ride quality was observed between pavements with and without ACBFS aggregate. Skid-resistance measurements indicated that, as a group, pavements with ACBFS have a significantly higher level of skid resistance than pavements without ACBFS.

As a result of these findings, and based upon the fear that the greater amount of cracking would result in a shorter pavement life and high maintenance costs, New York barred the use of ACBFS aggregate in paving concrete (NSA 1974). It was also reported that the transverse cracks in concrete pavements with ACBFS aggregate appear at an early age, running directly across the paving lanes, and that a high incidence of early-age cracking in ACBFS pavements has been reported previously in other areas. At the time, it was believed that these cracks had not been detrimental to pavement performance, and that the use of dry ACBFS aggregate, high paving temperatures, poor curing, and absorptive subgrades were factors that influenced the cracking (NSA 1974).

Laboratory tests were undertaken to determine if differences in physical properties of the concrete might explain the differences in cracking tendencies. Limestone and ACBFS aggregates used in the Buffalo area were used to prepare concrete for this study. Laboratory tests were performed to evaluate the following parameters: (1) temperature change due to cement hydration, (2) compressive strength (1, 7, and 28 days), (3) splitting tensile strength (1, 7, and 28 days), (4) modulus of elasticity (1, 7, and 28 days), (5) CTE, (6) drying shrinkage (measured over a 7- to 8-week period), and (7) oven-dry weight and absorption tests (after 28 days of standard curing). Moisture conditioning of the aggregate used for preparing the concrete consisted of immersing the aggregate in water for 24 hours followed by overnight drainage in a moist room. The *w/cm* of the two mixes was the same. The two concrete mixes containing limestone aggregate and ACBFS aggregate were comparable with respect to slump, air content, and yield within the normal laboratory variations for duplicate mixes.

Compressive strength tests and splitting tensile strength tests were performed at 1, 7, and 28 days, with three cylinders being tested at each age. Modulus of elasticity tests were also performed at these three ages on two samples. CTE testing was performed after 28 days of moist curing in both oven-dry and saturated (inundated in water) conditions. The temperature ranged from 32 °F to 140 °F (from 0 °C to 60 °C) for the inundated specimens and from 45 °F to 140 °F (from 7 °C to 60 °C) for the dried specimens tested in air. Duplicate specimens were used for drying shrinkage tests. The oven-dry weight and absorption tests were run on two samples.

The following results were obtained from the laboratory tests (NSA 1974):

- Both concretes reached their maximum temperature gain at 24 hours, and by 48 hours, the temperature had dropped down to the ambient temperature. The concrete containing ACBFS aggregate had a somewhat greater temperature increase, which may be due to the lower thermal conductivity and, consequently, a lower rate of heat loss.

- Concrete containing ACBFS aggregate had a higher compressive strength than the concrete containing limestone aggregate for all three test ages. The average 1-, 7-, and 28-day compressive strengths for the samples with ACBFS were 1,840, 3,840, and 4,600 lbf/in^2 (12.69, 26.48, and 31.72 MPa), respectively. The average 1-, 7-, and 28-day compressive strengths for the samples with limestone were 1,470, 2,900, and 3,970 lbf/in^2 (10.14, 19.99, and 27.37 MPa), respectively.

- Concrete containing ACBFS aggregate had a higher split tensile strength than the concrete containing limestone aggregate for all three test ages. The average 1-, 7-, and 28-day splitting tensile strengths for the samples with ACBFS were 240, 410, and 465 lbf/in^2 (1.7, 2.8, and 3.2 MPa), respectively. The average 1-, 7-, and 28-day splitting tensile strengths for the samples with limestone were 220, 325, and 405 lbf/in^2 (1.5, 2.2, and 2.8 MPa), respectively.

- Both mixes had similar modulus of elasticity at 1 day, but concrete with ACBFS aggregate had a slightly lower modulus of elasticity than the mix with limestone aggregate for the other two tests (at 7 days, 3.13 million lbf/in^2 (21.58 GPa) versus 3.57 million lbf/in^2 (24.61 GPa); and at 28 days, 3.33 million lbf/in^2 (22.96 GPa) versus 3.83 million lbf/in^2 (26.41 GPa)).

- The CTE of concrete made with ACBFS aggregate was higher than the CTE of concrete made with limestone aggregate: wet CTE 4.89 X 10^{-6} in/^{o}F versus 4.25 X 10^{-6} in/^{o}F (wet CTE 8.80 X 10^{-6} in/^{o}C versus 7.65 X 10^{-6} in/^{o}C) and dry CTE of 4.90 X 10^{-6} in/^{o}F versus 3.19 X 10^{-6} in/^{o}F (8.82 X 10^{-6} in/^{o}C versus 5.74 X 10^{-6} in/^{o}C).

- Length change from drying was essentially the same during the first stages of drying for both concrete mixes. However, the concrete with ACBFS aggregate showed a more rapid rate of shrinkage later and had a total shrinkage 13 percent greater than the concrete with limestone aggregate (at about 49 days).

- Concrete with ACBFS aggregate had a greater absorption than concrete with limestone aggregate (7.46 versus 5.62 percent by weight).

It was concluded in this study that concrete made with ACBFS was of high quality and had superior strength than concrete containing limestone aggregates. For the two aggregates that were used in this study, the following characteristics were identified that might make concrete containing ACBFS aggregate more prone to cracking when compared to concrete containing limestone aggregates: (1) possible higher hydration temperatures and higher early strength gain, which can result in a higher temperature drop, inducing high tensile stress when cooling to ambient temperature, (2) higher rate of strength development that increases the susceptibility to shrinkage cracking by decreasing creep deformations, (3) higher CTE values that result in higher stresses when the temperature decreases, and (4) potentially higher volume change when drying that would decrease the chance of lowering stresses due to creep or plastic flow.

Based on these factors, it was concluded that concrete pavement with ACBFS coarse aggregate can potentially be subjected to high early-age stresses, and hence cracking, when compared to concrete with limestone aggregate. A combination of tensile stresses produced by drying and thermal effects can be close to the tensile strength of the slab. In such a situation, factors such as higher concrete placement temperature, unusually cool nights, loss of water to a dry subgrade, delay in application of the curing compound, or a rougher subgrade that creates increased restraint against slab movement can make the difference between little or no cracking and extensive cracking (NSA 1974).

A recent communication with NYSDOT indicated that there was no evidence that the use of ACBFS aggregate in concrete was discontinued as reported by NSA (1974) or as a result of the study by Amsler, Chamberlin, and Jaqueway (1975). NYSDOT indicated that the only source of ACBFS aggregate in New York is Buffalo Slag in Woodhaven, and since 1975, production of ACBFS at this facility has become insignificant. Today this source is no longer included in NYSDOT's approved list of coarse and fine aggregate sources. NYSDOT also indicated that since the 1974 report, ACBFS aggregate has not emerged as a factor in premature failure or diminished service life of concrete pavements.

OHIO

ODOT funded a study conducted at Ohio University to examine the performance of asphalt and concrete pavements that have shown acceptable or exceptional service in order to gain insight into the factors contributing to good performance. This study was conducted in three phases with Phase A studying asphalt pavements, Phase B studying concrete pavements, and Phase C studying pavements constructed in Cuyahoga County using ACBFS coarse aggregate. Results from Phase B (Lankard 2010a) and Phase C (Lankard 2010b) are presented in this section. The main aim of these studies was to identify factors that contributed to the satisfactory performance of the pavements, with the focus being on the material properties of concrete.

Ohio Phase B Results

Twenty pavement sections were selected for the concrete pavement study, with half of the sections falling into the "Exceptional" category and the other half falling into the "Average" category with respect to performance (Lankard 2010a). The coarse aggregate types used in the concrete in the 20 sections were ACBFS (3 sections), gravel (6 sections), and limestone (11 sections). In the study, two cores were obtained from each section, one through the joints and the other at midpanel. For the sections that had midpanel cracks, the midpanel core was obtained through the crack. A petrographic examination of the cores was performed. Additional cores were obtained to determine the compressive strength, splitting tensile strength, and elastic modulus.

The authors concluded that the following factors contributed to satisfactory concrete pavement performance:

- A good-quality cementitious phase that was mostly attributed to a satisfactory *w/cm*, which ranged from 0.42 to 0.48 for the 20 sections.
- A good-quality coarse aggregate that has resisted freeze-thaw damage.
- A good-quality fine aggregate (natural sand) that had hard quartz particles as the dominant mineral phase and was shown to be chemically resistant.
- An air-void system that had provided adequate protection (concrete in all sections were air-entrained). It was noted that although some of the air-void system parameters in concrete in some sections did not meet minimum standards, the pavements have performed satisfactorily.

Table 39 describes the three sections in the Ohio Phase B study that had ACBFS as the coarse aggregate. Table 40 shows some additional information about the sections, including whether a blend of cement and fly ash was used, w*/cm* of the concrete, cement paste content, and the maximum size of the coarse aggregate.

Table 39. Sections With ACBFS as the Coarse Aggregate
(Lankard 2010a)

Test Section	Route	Year of Construction	Mile Marker	Direction	Condition
JEF-22	US-22	1990	15	Eastbound	Average
JEF-7	SR-7	1990	19	Southbound	Excellent
CUV-176	SR-176	1994	10	Southbound	Average

Table 40. Properties of Cement and Concrete for the ACBFS Sections
(Lankard 2010a)

Test Section	Cement/Fly Ash	Water-to-Cement Ratio	Cement Paste Content (%)	Maximum Aggregate Size
JEF-22	Portland cement only	0.42	27.9	1 in.
JEF-7	Portland cement only	0.46	26.4	¾ in.
CUV-176	Portland cement and fly ash	0.44	31	1 in.

Table 41 provides information regarding the bond between the aggregate and the cement and other aggregate properties that were determined from the petrographic examination of the cores. Table 42 presents information regarding the air-void system parameters determined from the petrographic examination. The cores from section CUV-176 had very low air contents, and the air voids were infilled with secondary deposits.

Table 41. Results From Petrographic Examination of Cores (Lankard 2010a)

Test Section	Quality of Bond Between Cement Paste and Aggregate	Evidence of Destructive Cement Aggregate Reaction	Evidence of Freeze-Thaw Damage in Aggregate	Overall Condition of Coarse Aggregate Particles
JEF-22	Good(a)	No(b)	1–5%	Good(e)
JEF-7	Good(a)	Very lmited(c)	1–5%	Good(e)
CUV-176	Good(a)	No*(d)	None	Excellent(f)

(a) Great majority (≥ 95%) of the bonds are tight and uninterrupted. Few (≤ 5%) or no instances of elevated water-to-cement ratio in the paste in contact with the aggregate particles.

(b) No positive identification of ASR gel. No ASR-type cracking (or cracks of any type) in rimmed aggregate particles.

(c) A few instances of positive identification of ASR gel and cracking in rimmed coarse aggregate particles that extends into the adjacent cement paste.

(d) A few (one to three) instances of "positive" identification of ASR gel. Cracks in a few rimmed coarse aggregate particles that extend into the adjacent cement paste.

(e) No cracking related to service conditions in 95% of the particles. A few instances (≤5 %) where the origin of the cracks in the particles is in question.

(f) No cracking in any of the particles related to service conditions.

Table 42. Air-Void System Parameters for ACBFS Sections (Lankard 2010a)

Test Section	Specific Surface Area (in^2/in^3)	Air-Void Spacing Factor (in.)	Total Air-Void Content (%)
JEF-22	658	0.007	5.16
JEF-7	611	0.007	6.10
CUV-176	708	0.011	1.93

Ohio Phase C Results

At the time Phase B was being concluded, field inspections identified a number of unusually distressed concrete pavements in Cuyahoga County that had been made using ACBFS aggregate. Three of these pavement sites (four sections), summarized in tables 43 and 44, were thus added to the Phase C study to determine the cause of the observed distress, which included joint deterioration and map cracking. Two cores were analyzed from each of the four sections: one core extracted in the vicinity of a joint (referred to as "distressed") and another in the interior portion of the slab (referred to as "no distress"), although as noted, even some cores from "no distress" locations were suffering distress (Lankard 2010b).

Table 43. Sections With ACBFS as the Coarse Aggregate (Lankard 2010b)

Route	Year of Construction	Station
SR-91	2002	100+50
SR-175	1999	269+80
SR-175 @ Concord	1999	270+80
SR-176	1994	107+00

Table 44. Properties of Cement and Concrete for the ACBFS Sections (Lankard 2010b)

Route	Cement/Fly Ash	Water-to Cement Ratio	Cement Paste Content (%)
SR-91	Portland cement only	0.50	29.2
SR-175	Portland cement only	0.42	35.2
SR-175 @ Concord	Portland cement only	0.42	35.2
SR-176	Portland cement and fly ash	0.46	28.3

The petrographic evaluation found that the four concretes examined were in “reasonable compliance” with the requirements of the mixture design with regard to cementitious constituents, cement content, the *w/cm*, coarse aggregate type, and coarse aggregate content. In general, the air content was also found to be in compliance with the specifications, except at SR-175 at Concord, where one core had low air content (3.7 percent) and both had nonuniform air distributions.

Of greatest interest to Lankard (2010b) was the type and amount of microcracking observed in lapped sections. Microcracking was both subsurface horizontal and vertical that was observed under a stereomicroscope. Similar subsurface horizontal cracking is seen in all cores from distressed locations and in two of the four cores extracted from “no distress” locations.

Based on the petrographic analysis, which included stereomicroscopy, petrographic microscopy, and limited scanning electron microscopy, Lankard (2010b) hypothesizes that the subsurface horizontal cracking in the “no distress” locations is simply in an “early stage” of the more extensive cracking observed in the “distressed” locations. In addition, he states his belief that the vertical surface cracking is simply a consequence of the subsurface horizontal cracking which occurred as a result of expansive stresses. Lankard (2010b; 2011) concludes that there is strong evidence that the observed pavement distress in three of the four pavements studied was due primarily to a type of internal sulfate attack, the source of the sulfate being the calcium sulfide from the ACBFS aggregate. ASR activity associated with chert and siltstone constituents of the fine aggregate was also observed, but he considers it to be of secondary importance.

Lankard (2011) based this conclusion on petrographic evidence that destructive internal sulfate reactions have been and are operative in the Ohio pavements evaluated, based not only on the extensive secondary infilling of air-voids with secondary ettringite, but also his observation of ettringite filling gaps around ACBFS aggregate that is intimately combined with the hydrated cement paste.

As described previously, three ACBFS pavements were evaluated in the Phase B study (Lankard 2010a), and after the Phase C study, Lankard re-evaluated concrete from the Phase B work. Two of the pavements studied were from Jefferson County, Ohio, where the source of the ACBFS was Weirton Steel, Weirton, West Virginia. None of the Jefferson County pavements showed the pattern of cracking observed in the pavements examined in Phase C, nor did any have diagnostic microstructural features consistent with internal sulfate attack or ASR, although secondary infilling of the air-void system with secondary ettringite was observed.

The one Cuyahoga County pavement included in Phase B was from the same SR-176 included in Phase C, but the core was from the mainline, not the shoulder, as is the case with the cores studied under Phase C. All of the ACBFS used in the Cuyahoga County pavements is from LTV Steel in Cleveland, Ohio. Petrographic analysis of the Phase B cores was originally conducted in December 2009, and when these same specimens were re-examined in May 2010, Lankard (2010b) observed that new cracks had formed during the storage period, concluding that it was the same subsurface horizontal cracking that is unique to the “distress mechanism in slag aggregate concretes.”

Lankard (2010b; 2011) concludes that the variability in the chemical and physical properties of the ACBFS likely contribute to the observed variation between the Jefferson County and Cuyahoga County pavements, as well as those cited in the Michigan studies. He also found that concrete mixtures made with high cementitious content and/or Class C fly ash were more prone to deterioration, citing that this conclusion is consistent with the work reported from Michigan. He mentions that controlling the chemical and mineralogical characteristics of the ACBFS might be one approach to addressing the problem, although implementing this approach would be very difficult, as both plant-to-plant variability as well as within-plant variability would have to be addressed. Instead, Lankard (2010b; 2011) recommends that the following strategies be adopted to address the potential for destructive MRD in concrete pavements made with ACBFS aggregates: (1) use a moderate-sulfate resistant cement (Type II) and/or a supplementary cementitious material (SCM) such as fly ash, silica fume, or slag cement of sufficient quantity known to improve sulfate resistance; and (2) limit moisture accessibility through better joint sealing to keep water out.

ONTARIO

An Ontario Provincial Standard Specification states that use of ACBFS in concrete pavements is prohibited (SSP110F11 2005). Prior to this specification, there was no restriction on the use of ACBFS for concrete pavements, and there is no documented rationale for the restriction. It is believed that this restriction would have minimal impact on ACBFS suppliers in Ontario, since up to that point ACBFS was not a competitive aggregate due to its restricted distribution area. However, it would have an impact on the Michigan-supplied ACBFS in the Windsor area.

CHAPTER 6. FIELD SURVEY AND PETROGRAPHIC ANALYSIS OF SELECTED SITES IN OHIO AND INDIANA

Although numerous studies have been conducted to evaluate the performance of concrete pavements made with ACBFS coarse aggregate, these were mainly forensic investigations examining pavements that were suffering some type of distress. To take a broader look, four concrete pavements containing ACBFS (two located in Ohio and two in Indiana) were examined visually and sampled, and followup petrographic analyses were conducted to determine if some type of common distress mechanism was at work.

The results of the field evaluations and the summaries of the petrographic analyses are presented below. Detailed information on the field evaluations is presented in appendix B.

OHIO (SR-175–SECTIONS 1 AND 2)

A field evaluation was conducted on two test sites in Ohio, both located on SR-175 in Beachwood, a southeastern suburb of Cleveland. Detailed visual assessments were based on the LTPP pavement inspection procedures (Miller and Bellinger 2003) and those recently developed for the Innovative Pavement Research Foundation specifically for use in inspecting pavements affected by MRD (Van Dam et al. 2009).

The pavements studied were selected in consultation with ODOT. A 1,000-ft (305 m) representative section was selected for each detailed evaluation on each test site using the two methods cited above. During the field evaluation, coring locations were identified in one of the slabs in each selected pavement section. In total, four full-depth cores, 4 in. (100 mm) in diameter, were extracted from each evaluated pavement, two at joint locations (labeled “B” at the joint and “C” 1 ft (300 mm) from the joint) and two from the slab interior (“D” at midslab in an area free of distress and “E” at midslab through distress). Traffic control and coring were provided by ODOT.

The SR-175 test sections were JRCP 10 in. (250 mm) thick on 6 in. (150 mm) of dense-graded aggregate base. The concrete was made with ACBFS coarse aggregate and natural sand. Section 1 had 600 lb of Type I cement per cubic yard of concrete (356 kg/m^3), whereas Section 2 has 385 lb of Type I cement (228 kg/m^3) and 165 lb of slag cement per cubic yard (98 kg/m^3).

Section 1 is on the eastbound lanes of Harvard Road, and Section 2 is on the southbound lanes of Richmond Road. Both are located near the intersection of the two roads.

SR-175–Section 1, Eastbound Harvard Road (93+28 to 108+37), Beachwood, Ohio

Figure 33 shows a typical photo of the conditions observed on SR-175–Section 1. A summary of the conditions observed is as follows:

- Description: The 1,000-ft (305 m) section evaluated was located in the right lane and included 50 slabs. As shown in figure 33, not much distress was present on this section.
- LTPP Survey: There was low-severity joint seal damage recorded for all joints and a moderate amount of linear cracking also observed. There were a few joint spalls and spots of scaling, but nothing severe to note at the time. A few rigid patches were recorded around drains.
- MRD Survey: Only a rust-colored staining was observed on this section down the middle of each slab.
- Coring: No evidence of MRD was observed on this pavement, so the 1B and 1E cores were taken through some of the staining.

Figure 33. Photo. SR-175–Section 1; Eastbound Harvard Road, Beachwood, Ohio.

SR-175–Section 2, Southbound Richmond Road (223+09 to 247+70), Beachwood, Ohio

A summary of the conditions observed on SR-175–Section 2 is as follows:

- Description: The 1,000-ft (305 m) section evaluated was located in the right lane and included 50 slabs. As the overview above shows, not much distress or MRD was present on this section, but slightly more than on Section 1.
- LTPP Survey: Low-severity joint seal damage was recorded for all joints. A moderate amount of linear cracking was also observed and was more prevalent than in Section 1. There were a few joint spalls, but nothing severe to note at the time. A few rigid patches were recorded around drains.

- MRD Survey: Only a rust-colored staining was observed on this section, but in addition to running along the middle of each slab, the staining extended along all the joints and corners of each slab as well. This staining is from corroded reinforcement mesh.
- Coring: No evidence of MRD was observed on this pavement, so the 1B and 1E cores were taken through a crack in an area with staining.

Petrographic Analysis of SR-175 Cores, Ohio

Six cores extracted from ODOT SR-275 near Beechwood were subjected to petrographic examination to assess the general condition of the concrete. The petrography was limited to a stereomicroscope reconnaissance investigation. The findings from this work indicate that none of the cores show evidence of damage or deterioration of any significance from sulfate attack, ASR, or freeze-thaw damage. In some cores there appear to be limited zones of ettringite mineralization in voids around slag particles, but no cracking is associated with this mineralization. One core showed a microcrack associated with such mineralization. Several cores show evidence of sulfate mineralization and microcracking at the bottom surface. These zones are limited to the bottom 0.5 in. (12.5 mm) or so and are not associated with any surface distress on the pavement cores. Although reaction rims are commonly observed in many of the siliceous fine aggregate particles, none of the cores show evidence of cracking due to ASR or significant microcracking due to ASR. Deposits of gel were observed but are very limited to a few aggregate particles or nearby voids. None of the cores show evidence of significant carbonation.

The most significant secondary deposits are corrosion products observed on steel wires that are likely welded wire fabric used to reinforce the JRCP. Some of these deposits are quite extensive on joint and crack walls and in a few cases microcracks filled with corrosion products cut into the paste. Most of the large cracks that cut through the depth of the cores are typical of drying shrinkage. No cracking or microcracking associated with freeze-thaw damage was observed.

INDIANA (SR-19 AND SR-331–SECTIONS 3 AND 4)

Field evaluations were also conducted on two test sites in Indiana, one located on SR-19 near Elkhart and the other located on SR-331 near South Bend. The same approach for visual assessment used on the Ohio pavements was also employed on the Indiana pavements.

The pavements studied were selected in consultation with INDOT. One 1,000-ft (305 m) representative section was selected for detailed evaluation on each test site using the two methods previously described. During the field evaluation, coring locations were identified in one of the slabs in each selected pavement section. In total, four full-depth cores, 4 in. (100 mm) in diameter, were extracted from each evaluated pavement, two at joint locations and two from the slab interior, using the same “B”, “C”, “D”, and “E” designations as discussed previously. Traffic control and coring was provided by INDOT. Section 3 is on the northbound lanes of Nappanee Street (SR-19), and Section 4 is on the northbound lanes of Capital Avenue (SR-331). The following summarizes the conditions present for each section.

SR-19–Section 3, Northbound Nappanee Street (R-26114), Elkhart, Indiana

Section 3 was constructed in 2003/2004, using 480 to 501 lb/yd^3 (285 to 297 kg/m^3) cement and 89 to 119 lb/yd^3 (53 to 71 kg/m^3) of fly ash at a *w/cm* of 0.384 to 0.402. In addition to ACBFS coarse aggregate, natural sand fine aggregate was used. The following summarizes the general conditions for SR-19–Section 3:

- Description: The 1,000-ft (305 m) section evaluated was located in the right lane and included 50 slabs. The overview presented in figure 34 shows there was not much distress or MRD present on this section.
- LTPP Survey: Low-severity joint seal damage was recorded for all joints. A few linear cracks were also observed. In addition, there were a few joint spalls, but nothing severe to note at the time.
- MRD Survey: Dark staining was observed on this section running along the middle of each slab. Two slabs were observed to have parallel cracking, but that was the only other indication of MRD.
- Coring: No evidence of MRD was observed on this pavement, so the 1B and 1E cores were taken through a crack in an area with staining.

Figure 34. Photo. Overview of SR-19–Section 3, Indiana.

SR-331–Section 4, Northbound Capital Avenue (R-26937), South Bend, Indiana

Section 4 was constructed in 2004 with 440 to 657 lb/yd^3 (261 to 390 kg/m^3) of Type I cement and 120 lb/yd^3 (71 kg/m^3) of slag cement at a *w/cm* of 0.353 to 0.41. In addition to ACBFS coarse aggregate, natural sand fine aggregate was used. The following is a summary of the general conditions observed.

- Description: The 1,000-ft (305 m) section evaluated was located in the right lane and included 50 slabs. As the overview in figure 35 shows, there is not much distress or MRD present on this section. There is less distress observed than Section 3, but significantly more MRD present.

- LTPP Survey: Only medium-severity joint seal damage was recorded for all joints. There were many corners with deterioration, but they were not large enough per the distress definition of corner spalling to record.
- MRD Survey: Dark staining was observed on this section, but in addition to running along the middle of each slab, the staining extends along all the joints and corners of each slab as well. The corners of nearly every slab were deteriorated, and although this deterioration would not be picked up by the LTPP survey, it was recorded as joint disintegration under the MRD procedure.
- Coring: While MRD was observed on this pavement, it was primarily in the corners. Therefore, the 1B and 1E cores were taken through an area with staining.

Figure 35. Photo. Overview of SR-331–Section 4, Indiana.

Summary of Petrographic Analysis Conducted on SR-19 and SR-331

Six cores extracted by INDOT from SR-19 near Elkhart and SR-331 near South Bend were subjected to petrographic examination to assess the general condition of the concrete. The petrography was limited to a stereomicroscope reconnaissance investigation. The findings indicate no evidence of damage or deterioration of any significance from sulfate attack, ASR, or freeze-thaw damage. In some cores there are limited zones of ettringite mineralization in the voids around slag particles, but no cracking is associated with this mineralization. One core showed a microcrack associated with such mineralization.

The cores from SR-19 show more ettringite mineralization near the bottom surface. Although reaction rims are commonly observed in siliceous fine aggregate particles, none of the cores show evidence of cracking due to ASR or significant microcracking due to ASR. Deposits of gel were observed but are limited to a few aggregate particles or nearby voids. None of the cores show evidence of significant carbonation. Core 4E from SR-331 shows corrosion products in cracks and microcracks near the top of the core. No cracking or microcracking associated with freeze-thaw damage was observed.

SUMMARY OF FIELD AND PETROGRAPHIC EVALUATIONS FROM OHIO AND INDIANA

The pavements evaluated were all in relatively good condition, with little observation of distress. They all shared some similarities, in that ASR was present in all but had not led to any pavement deterioration at the time of the evaluation. Limited secondary mineralization of ettringite was observed in a few voids, but there was no evidence of sulfate attack or freeze-thaw damage. In all cases, the concrete appeared to be in good condition. Only SR-331–Section 4 in Indiana has significant staining in the vicinity of joints and the presence of joint disintegration at the corners. Yet no significant problems were identified in the petrographic analysis, which was conducted on cores taken through stained areas but not directly at the deteriorating corners. This pavement should be monitored over time to determine if the observed distress continues to develop.

CHAPTER 7. MECHANISTIC-EMPIRICAL PAVEMENT DESIGN EVALUATION

This chapter considers the appropriateness of existing pavement design methods in use by Michigan, Indiana, and Ohio for JPCP pavements containing ACBFS aggregates. The evaluation is based on information obtained from the design and construction plans and specifications used by MDOT, INDOT, and ODOT, with the practices of each agency considered individually.

The objective of this effort is to consider the adequacy of JPCP design thickness produced by the individual States for concrete pavement containing ACBFS coarse aggregate and to provide recommendations to the States regarding unique pavement design practices that might be required when ACBFS is used as an aggregate. Two basic questions are addressed in the evaluation:

- What is the predicted performance of JPCP made with ACBFS coarse aggregate designed using the current practices of each State?
- Do design procedures for concrete containing ACBFS aggregates need to be different than for natural aggregates?

Two types of analysis were performed to conduct this evaluation: a mechanistic analysis of stress ratios and a mechanistic-empirical analysis of predicted pavement performance. For the mechanistic analysis, the maximum tensile stresses predicted in JPCP slabs for combinations of CTE by aggregate type, joint spacing, and slab thickness were developed on the basis of the typical design, construction, and loading criteria for Michigan, Indiana, and Ohio. For the mechanistic-empirical performance prediction analysis, the percent of slabs cracked at the end of the pavement's design life were developed using the same input conditions. The computer software, inputs, analysis approach, and results are discussed in the next section.

COMPUTER SOFTWARE AND INPUTS

A mechanistic pavement finite element analysis software tool (EverFE, version 2.24, http://www.civil.umaine.edu/EverFE/) was used to calculate the maximum predicted stresses generated in concrete slabs as a result of the combined effect of temperature and traffic. For the mechanistic-empirical analysis, the MEPDG, version 1.100, was used to predict the percentage of slab cracking as a function of environmental and traffic loading factors.

A number of pavement slab dimensions and thicknesses representing typical sections for pavements in each State were modeled. In addition, the allowable loading for each State (based on vehicle weight restriction policy) was considered to determine typical truck loading. Additionally, a more damaging special permitted loading was also considered.

The 20,000-lb (9,072 kg) single axle is the heaviest legal standard axle load in all three States except for permitted overloads and a maximum load of 24,000 lb (10,886 kg) on Indiana interstates. The response of the 20,000-lb axle was compared with two- and three-axle load groups, but because the maximum legal weight of individual axle loads within load groups are reduced by the States, the single axle induces the highest legal standard axle stress response. A potential overload axle weight of 26,000 lb (11,793 kg) was also considered in the EverFE analysis.

Typical slab dimensions and thicknesses were determined from the design policy of each State.

The slab size/thickness criteria provided by MDOT are shown in table 45.

Table 45. MDOT's Joint Spacing for JPCP
(MDOT 2010)

Pavement Thickness (in.)	Joint Spacing (ft)
6.5 to 8.75	12
9.0 to 11.75	14
12 or more	16

The INDOT policy for joint spacing design recommends that the initial trial value for joint spacing is 15 ft (4.6 m) for slab thicknesses less than 10 in. (250 mm), 18 ft (5.5 m) for slab thicknesses between 10 (250 mm) and 12 in. (300 mm), and 16 ft (4.9 m) for slab thicknesses of 12 in. (300 mm) or greater. The designer may conclude the design process by recommending other slab size to thickness relationships (INDOT 2010).

ODOT's design manual states that a 15-ft joint (4.6 m) spacing will be used for all nonreinforced concrete pavements (ODOT 2008).

The slab dimension and thickness combinations evaluated were based on the above mentioned recommendations. A slab width of 12 ft (3.7 m), corresponding to a typical lane width, was assumed for all cases. Pavement life would be extended if a widened lane (13.5 to 14 ft (4.1 to 4.3 m)) were used.

EverFE Inputs

In all three States, the dual-wheel, single-axle load was found to be the most damaging axle configuration for this investigation of the effect of slab dimensions on predicted cracking damage. Loading was also considered at two different locations, interior and edge, for each combination of load level, CTE, slab dimension, and slab thickness.

The inputs used in the EverFE model are presented in table 46. The typical CTE of concrete containing ACBFS as coarse aggregate ranges from 5.1 to 5.9 με/ºF (9.2 to 10.6 με/ºC) (according to the MEPDG documentation (AASHTO 2008). A CTE value of 4.3 με/ºF (7.7 με/ºC) representing a nonslag aggregate (an average limestone value of 3.4 to 5.1 (6.1 to 9.2) was also considered in the analysis.

Table 46. EverFE Inputs Used in the Analysis

Input	Value
Concrete elastic modulus	4,940 ksi*
Concrete Poisson's ratio	0.2
Concrete coefficient of thermal expansion	4.3 ×10-6/ºF (limestone) 5.1 ×10-6/ºF (ACBFS) 5.9 ×10^{-6}/ºF (ACBFS)
Concrete density	150/ft^3
Base thickness	12 in.
Base modulus	30 ksi
Slab/Base interface	Unbonded
Temperature change on top of the slab	3.5 ºF/in. of slab thickness
Dual-wheel single-axle loads	20, 26 kips
Joint spacing	12, 15, 18 ft.
Slab thickness	8, 10, 12, 14 in.

* The E of 4,940 ksi corresponds to a modulus of rupture of 650 lbf/in^2according to ACI equations.
1 ksi = 6.895 MPa; 1 ºF = 0.56 ºC; 1 ft^3 = 0.026 m^3; 1 kip = 4.45 kN; 1 ft = 0.305 m; 1 in. = 25.4 mm.

MEPDG Inputs

The same combination of CTE, joint spacing, and slab thickness values mentioned above were employed in the MEPDG analysis. For the environmental effects, Detroit's climate was selected. For traffic, annual average daily truck traffic of 9,600 (based on Detroit's I-96 traffic) was selected. Table 47 shows the inputs used in the MEPDG analysis.

Table 47. Inputs Used in the MEPDG Analysis

Input	Value
Concrete modulus of rupture	650 lbf/in^2
Concrete Poisson's ratio	0.2
Concrete coefficient of thermal expansion	4.3 ×10-6/ºF (limestone) 5.1 ×10-6/ºF (ACBFS) 5.9 ×10-6/ºF (ACBFS)
Concrete density	150/ft^3
Base thickness	12 in.
Base modulus	30 ksi
Climate	Detroit, MI
Traffic	9,600 AADTT
Traffic growth	4%, Compound
Joint spacing	12, 15, 18 ft
Slab thickness	8, 10, 12, 14 in.

AADTT=annual average daily truck traffic

ANALYSIS APPROACH

EverFE/Mechanistic Stress Ratio Analysis

For this analysis, the 20,000-lb (9,072 kg) axle load was evaluated along with an assumed permit overload of 26,000 lb (11,793 kg). The maximum tensile stress for the various combinations of slab dimension and thickness were determined for each load level from multiple analyses conducted using EverFE. As expected, the maximum tensile stress corresponded to the edge loading condition for all slab size and load-level combinations. This condition represents the most critical loading location in terms of pavement damage, and would result in transverse slab cracking. An inservice JPCP would have a relatively low frequency of occurrence of this critical loading.

A stress ratio (tensile stress divided by the flexural strength) of 0.75 is considered an acceptable damage level for the design of concrete structures (Mehta and Monteiro 2006), whereas a stress ratio less than 0.55 is considered to provide virtually unlimited fatigue performance (Packard 1973). In this investigation, the maximum tensile stress in the slab is computed from EverFE and the flexural strength (or modulus of rupture, MOR) is assumed to be 650 lbf/in^2 (4.48 MPa). The results of this analysis are plotted in figures 36 through 41 and summarized in tables 48 and 49.

This discussion first considers the interior slab loading condition, as this is the most common loading experienced by pavements in the field. Table 48 indicates that for the commonly used joint spacing of 15 ft (4.6 m), the required slab thickness needed for acceptable performance (in terms of either stress ratio) is dependent upon the CTE of the coarse aggregate. Indeed, it is observed that the required slab thicknesses for the ACBFS pavements are about 0.4 in. (10 mm) (for a CTE of 5.1 x 10^{-6}) to about 0.8 in. (20 mm) (for a CTE of 5.9 x 10^{-6}) greater than the slab thickness for limestone pavement.

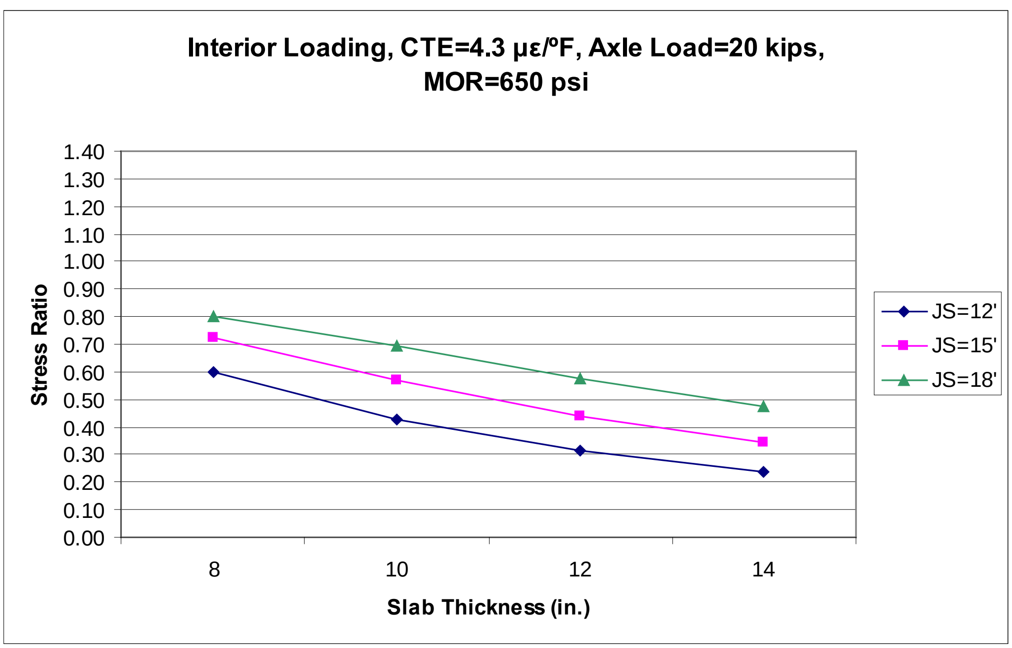

Figure 36. Stress ratios for interior loading with CTE of 4.3 με/ºF and 20-kip axle load.

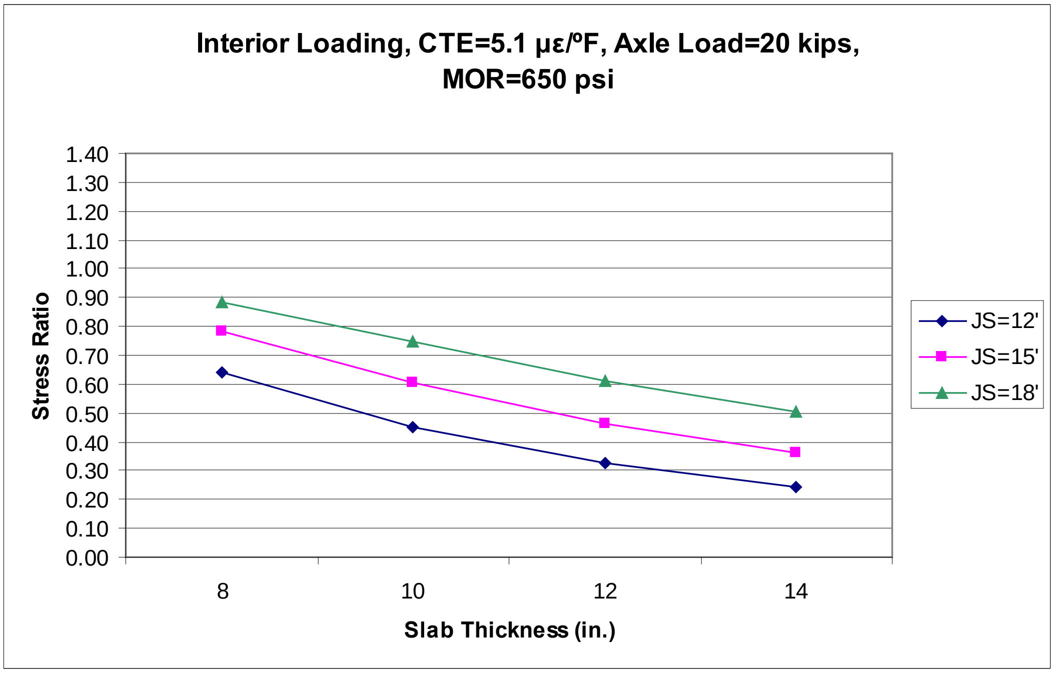

Figure 37. Stress ratios for interior loading with CTE of 5.1 με/ºF and 20-kip axle load.

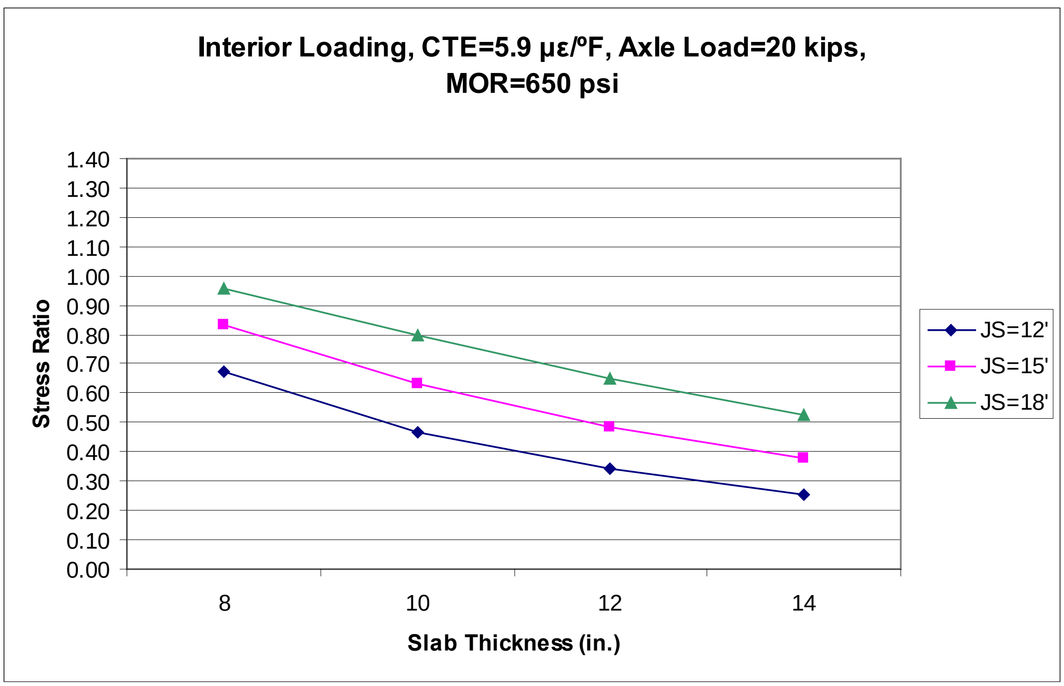

Figure 38. Stress ratios for interior loading with CTE of 5.9 με/ºF and 20-kip axle load.

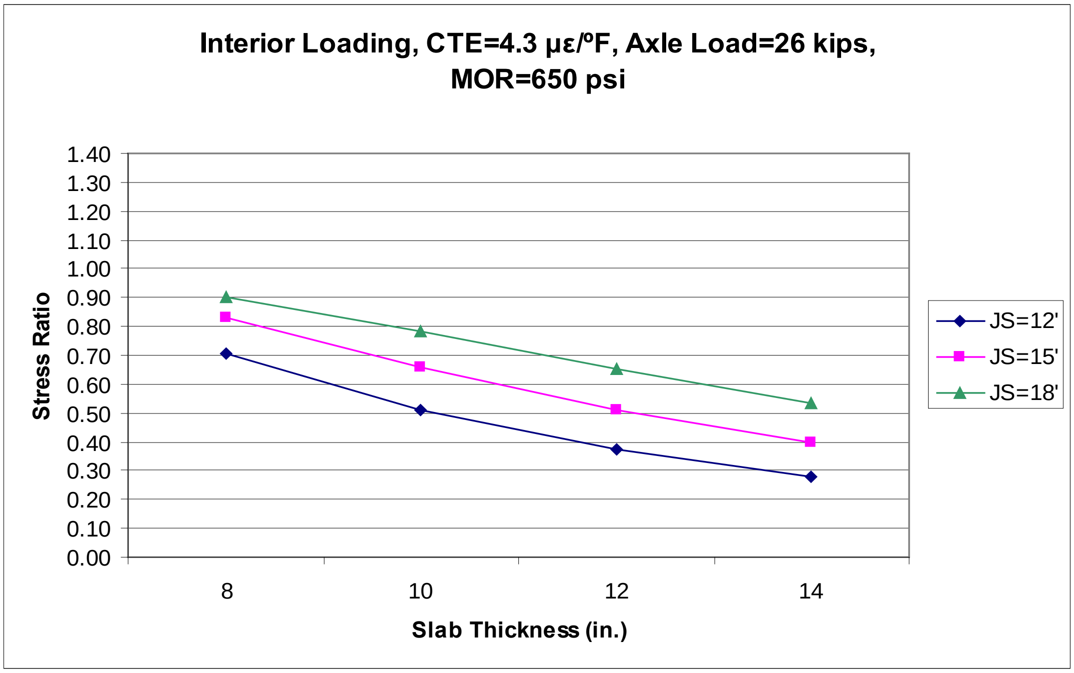

Figure 39. Stress ratios for interior loading with CTE of 4.3 με/ºF and 26-kip axle load.

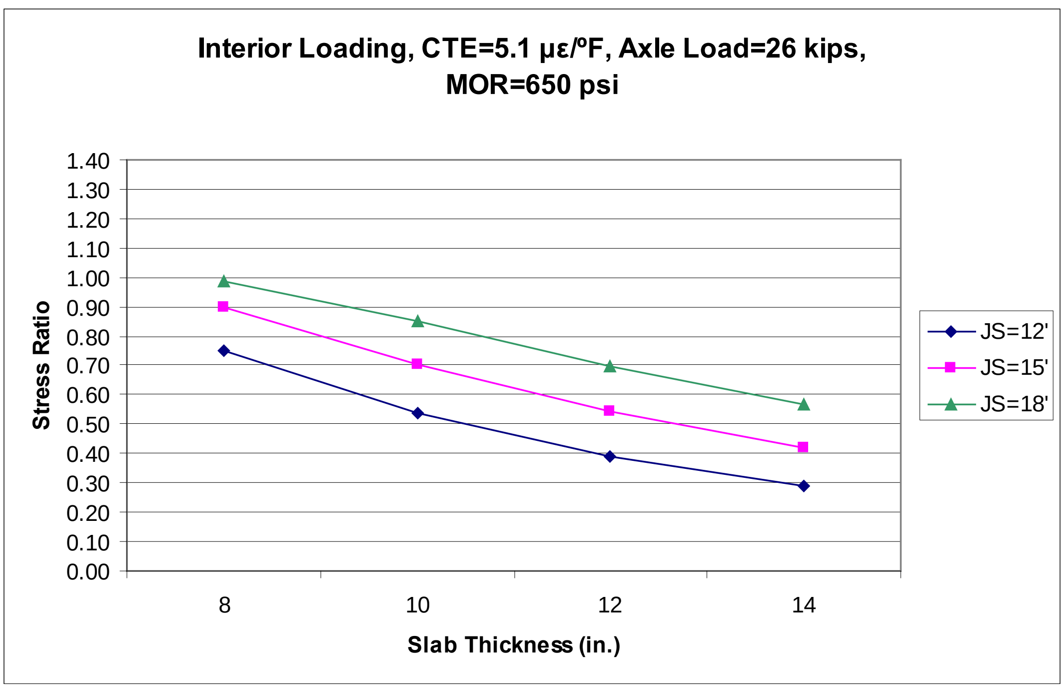

Figure 40. Stress ratios for interior loading with CTE of 5.1 με/ºF and 26-kip axle load.

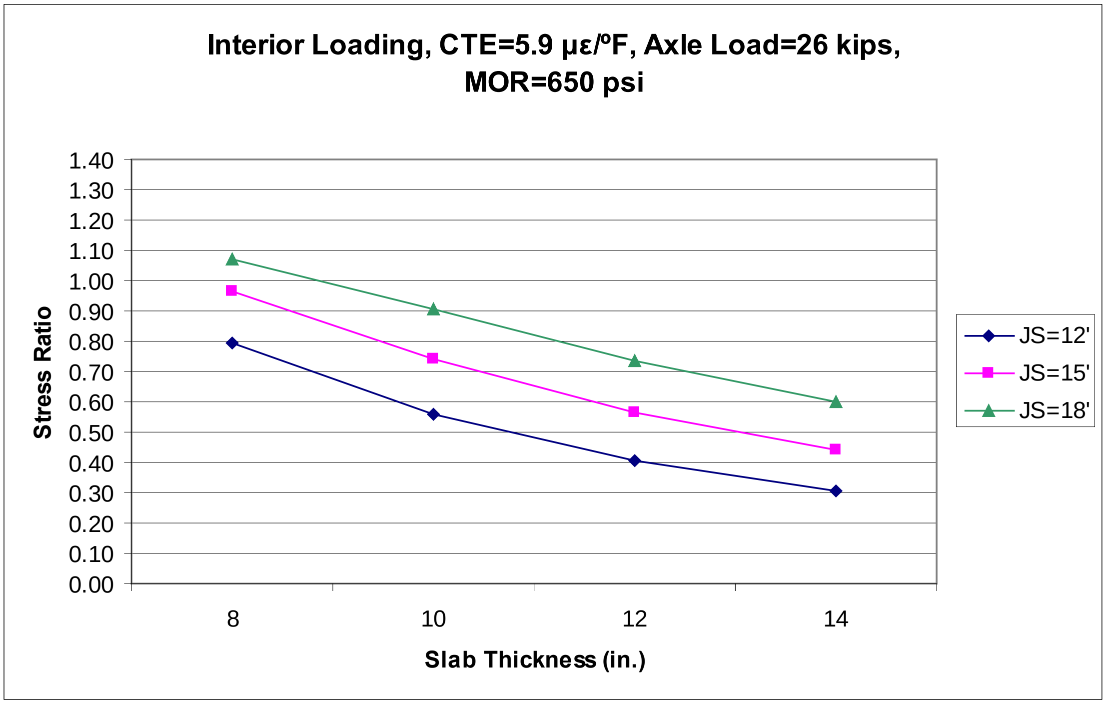

Figure 41. Stress ratios for interior loading with CTE of 5.9 με/ºF and 26-kip axle load.

Table 48. Summary of Interior Slab Loading, 20 kips

Interior Axle Load (kips)	CTE (με/ºF)	Joint Spacing (ft)	Thickness @ SR = 0.75 (in.)	Thickness @ SR = 0.55 (in.)
20	4.3	12	< 8.00	8.59
20	5.1	12	< 8.00	8.95
20	5.9	12	< 8.00	9.20
20	4.3	15	< 8.00	10.31
20	5.1	15	8.33	10.71
20	5.9	15	8.86	11.07
20	4.3	18	8.90	12.40
20	5.1	18	10.00	13.09
20	5.9	18	10.67	13.54

CTE = coefficient of thermal expansion; SR = stress ratio

Table 49. Summary of Interior Slab Loading, 26 kips

Interior Axle Load (kips)	CTE (µε/°F)	Joint Spacing (ft)	Thickness @ SR = 0.75 (in.)	Thickness @ SR = 0.55 (in.)
26	4.3	12	< 8.00	9.60
26	5.1	12	8.00	9.90
26	5.9	12	8.35	10.13
26	4.3	15	8.94	11.47
26	5.1	15	9.50	11.88
26	5.9	15	9.91	12.17
26	4.3	18	10.46	13.82
26	5.1	18	11.33	> 14.00
26	5.9	18	11.88	> 14.00

None of the three States considered here is currently using significantly longer or shorter joint spacing, but the analysis results are included to demonstrate the importance of the relationship that exists between the CTE and joint spacing. For example, if Indiana uses 18-ft (5.5 m) joint spacing instead of 15 ft (4.6 m), the increase in required slab thickness would range from approximately 1.5 to 2.0 in. (38 to 50 mm).

When the 0.55 stress ratio is considered, this difference becomes even greater, with the thickness difference ranging from about 0.5 in. (13 mm) for the minimum ACBFS CTE to 1.5 in. (38 mm) for the maximum ACBFS CTE value.

When the 26-kip (115.7 kN) overload is considered for the interior loading case with the 15-ft (4.6 m) joint spacing, the difference in slab thickness to achieve acceptable performance with a stress ratio of 0.75 between the average limestone CTE aggregate value and the range minimum ACBFS CTE value is about 0.5 in. (13 mm). Relative to the maximum CTE value for ACBFS aggregate, the required thickness increases approximately 1 in. (25 mm).

Note that for the 18-ft (5.5 m) joint spacing case, the slab thickness required is greater in all cases than for shorter joint spacing. The range in thickness between the CTE of 4.3 for limestone and the ACBFS values is on the order of 1 in. to 1.5 in. (25 to 38 mm).

As expected, the slab edge loading increases the stress developed in the pavement, as shown in figures 42 through 44 and tabularized in table 50. However, it must be remembered that edge loading is not common in most pavements, and, therefore, may not control the thickness design.

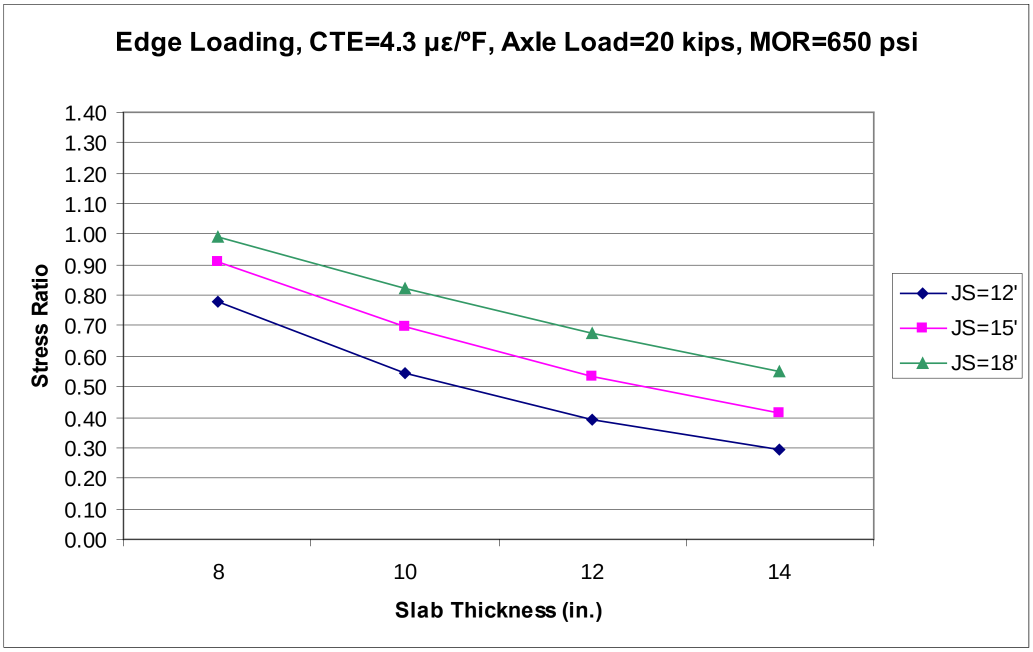

Figure 42. Stress ratios for edge loading with CTE of 4.3 με/ºF and 20-kip axle load.

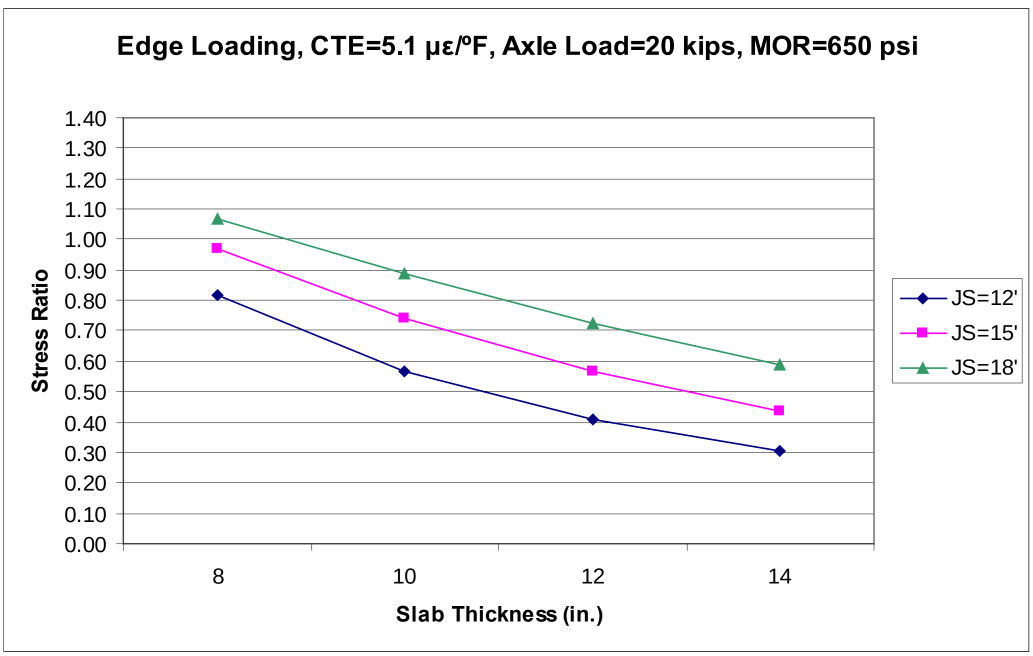

Figure 43. Stress ratios for edge loading with CTE of 5.1 με/ºF and 20-kip axle load.

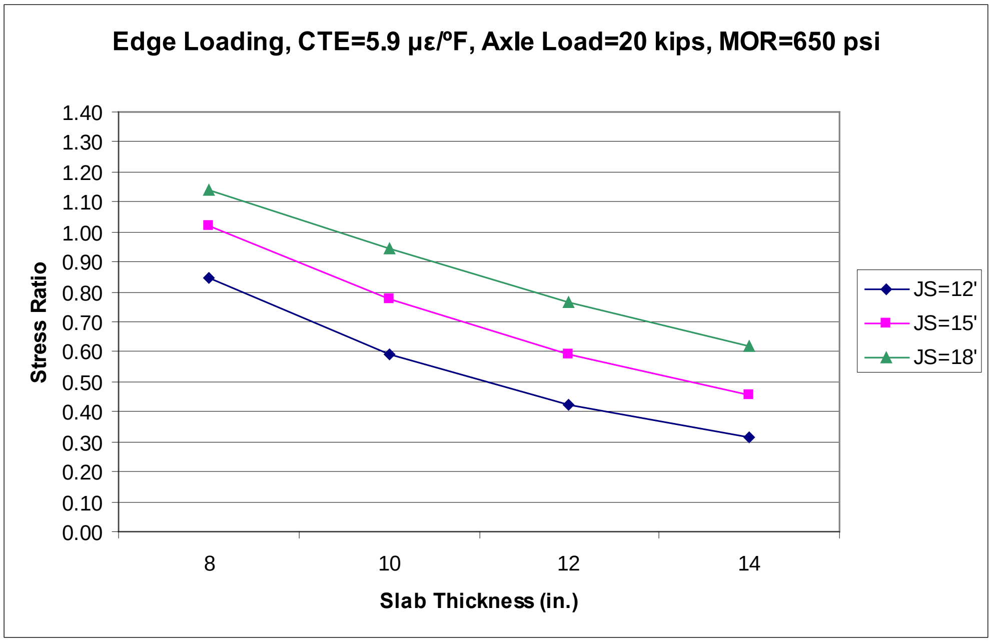

Figure 44. Stress ratios for edge loading with CTE of 5.9 µε/ºF and 20-kip axle load.

The trends illustrated in table 50 are similar to those described in the previous section for interior slab loading. Again, for the common joint spacing of 15 ft (4.6 m), the required slab thicknesses needed for acceptable performance for the ACBFS pavements are from about 0.4 in. (10 mm) (for a CTE of 5.1 x 10^{-6}) to 0.8 in. (20 mm) (for a CTE of 5.9 x 10^{-6}) greater than the slab thickness for the limestone pavement. As shown in figures 45 through 47, and summarized in table 51, when the 26-kip (115.7 kN) overload is considered for the edge loading case, thickness requirements increase, but differences resulting from CTE are slightly less dramatic, probably because the stress levels are all similarly high.

Table 50. Summary of Edge Slab Loading, 20 kips

Edge Axle Load (kips)	CTE (µε/ºF)	Joint Spacing (ft)	Thickness @ SR = 0.75 (in.)	Thickness @ SR = 0.55 (in.)
20	4.3	12	8.25	9.92
20	5.1	12	8.56	10.25
20	5.9	12	8.77	10.47
20	4.3	15	9.52	11.76
20	5.1	15	9.91	12.17
20	5.9	15	10.32	12.62
20	4.3	18	11.00	14.00
20	5.1	18	11.65	> 14.00
20	5.9	18	12.14	> 14.00

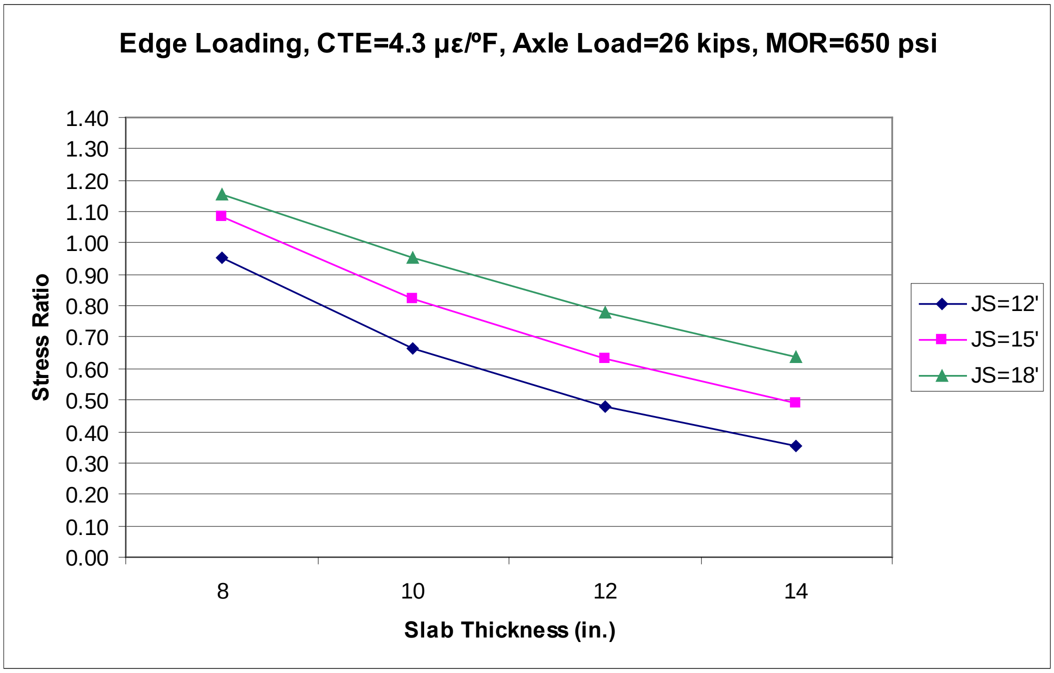

Figure 45. Stress ratios for edge loading with CTE of 4.3 με/ºF and 26-kip axle load.

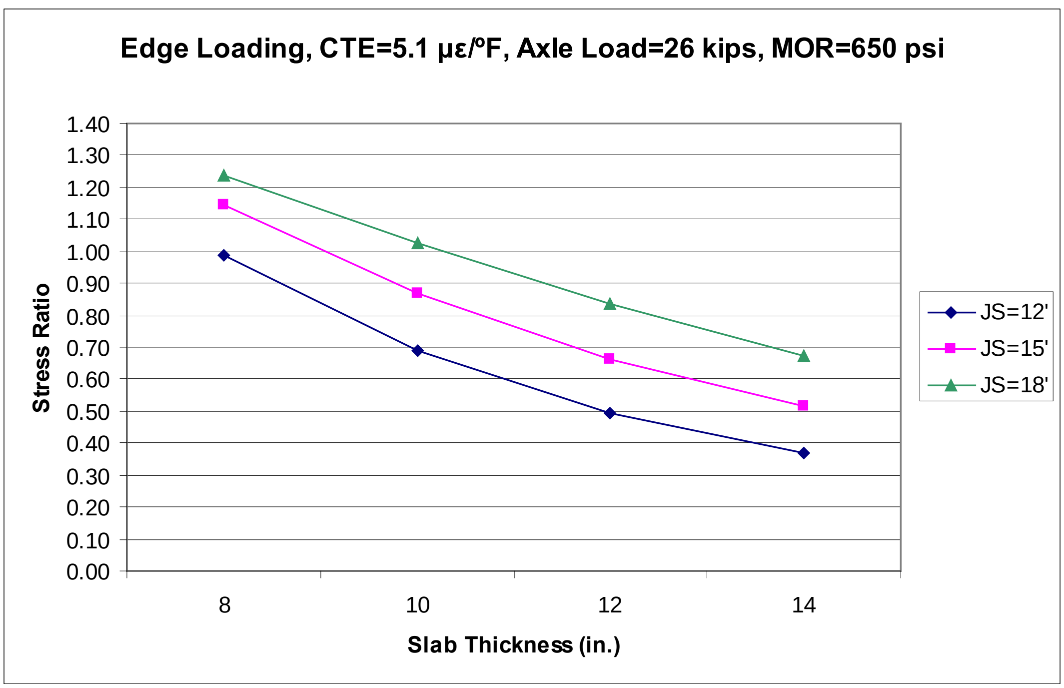

Figure 46. Stress ratios for edge loading with CTE of 5.1 με/ºF and 26-kip axle load.

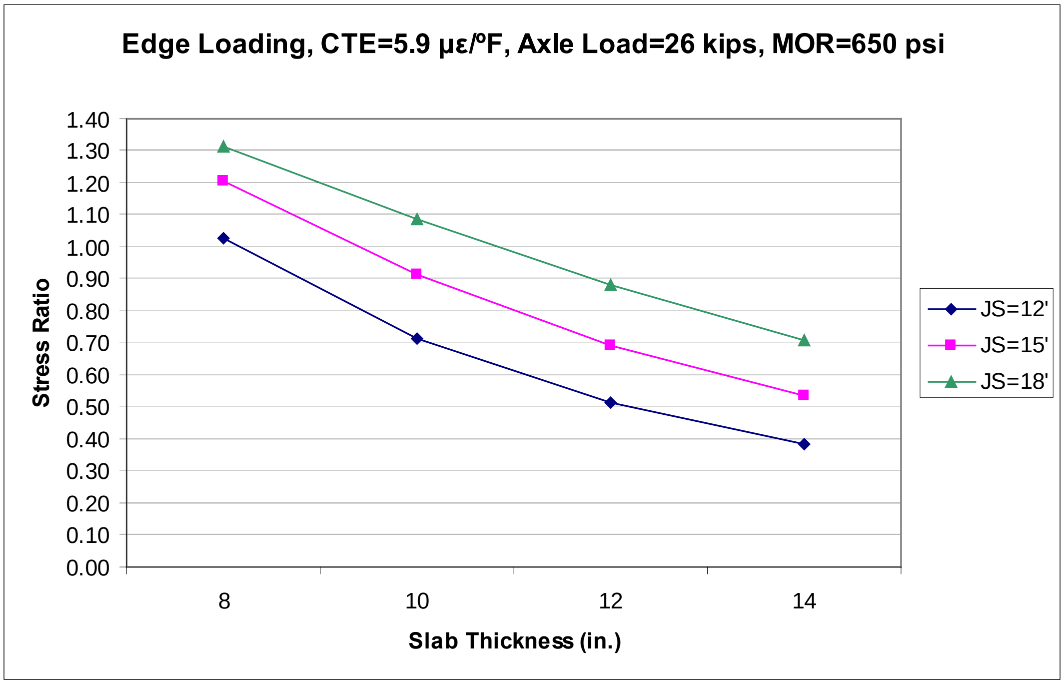

Figure 47. Stress ratios for edge loading with CTE of 5.9 µε/ºF and 26-kip axle load.

Table 51. Summary of Edge Slab Loading, 26 kips

Edge Axle Load (kips)	CTE (µε/ºF)	Joint Spacing (ft)	Thickness @ SR = 0.75 (in.)	Thickness @ SR = 0.55 (in.)
26	4.3	12	9.38	11.22
26	5.1	12	9.60	11.47
26	5.9	12	9.74	11.60
26	4.3	15	10.74	13.14
26	5.1	15	11.14	13.47
26	5.9	15	11.45	13.87
26	4.3	18	12.43	> 14.00
26	5.1	18	13.00	> 14.00
26	5.9	18	13.53	> 14.00

Mechanistic-Empirical Pavement Performance Analysis

For MEPDG analysis, slab cracking was considered the most appropriate performance indicator. The predicted transverse cracking is expressed as the "percentage of slabs cracked" as obtained from the MEPDG outputs for all combinations of CTE, joint spacing, and slab thickness values.

The results of the MEPDG slab cracking analysis are presented in figures 48 through 50 for different CTE levels. It can be observed from the graphs that when the CTE of different aggregate types is the only variable, transverse cracking can significantly change for some combinations of joint spacing and slab thickness.

For example, for an aggregate with a relatively low CTE, figure 48 shows that with a percent-slabs-cracked criterion of 15 percent, a pavement 8 in. (200 mm) thick is predicted to crack beyond this limit within the 20-year design life regardless of joint spacing. It also shows that thicker pavements (10, 12, or 14 in. (250, 300, or 350 mm)) will satisfy the 15-percent slab cracking criterion. The 15-percent slab cracking criterion was selected merely for purposes of illustration, but it is not an uncommon value used in this type of analysis.

Figure 49 shows that for an aggregate with a higher CTE, not only does the 8-in.-thick (200 mm) slab exceed the cracking limit, but so does a 10-in.-thick slab (250 mm) with 18-ft (5.5 m) joint spacing. Other combinations, however, are shown to satisfy the cracking limit.

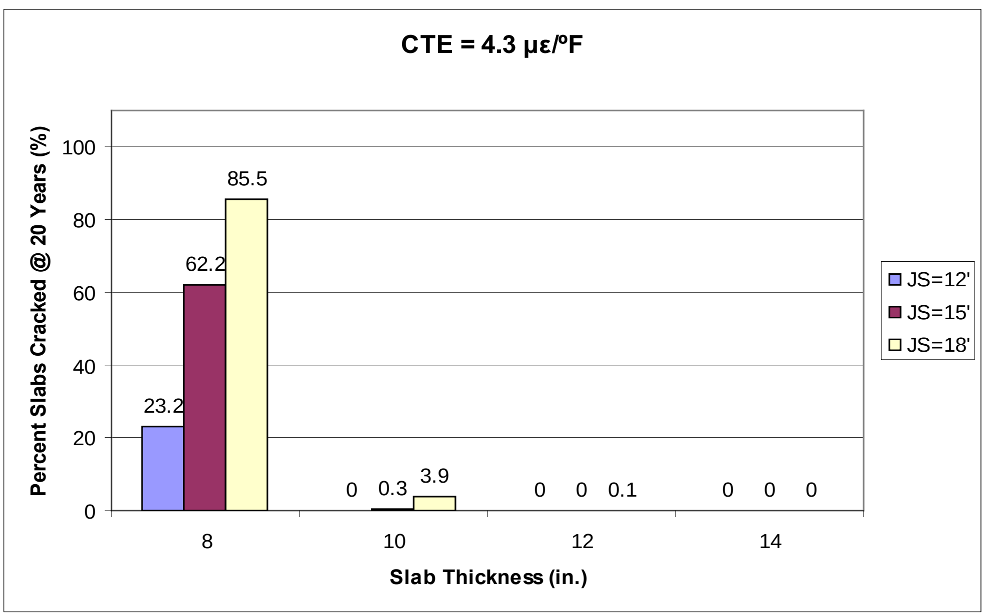

Figure 48. Percent slabs cracked for combinations of joint spacing and slab thickness for CTE of 4.3 με/ºF.

In the case of an aggregate with relatively high CTE (shown in figure 50), it can be seen that when the joint spacing is 18 ft (5.5 m), only a 14-in.-thick (350 mm) pavement will satisfy the cracking limit. By considering other joint spacing values, however, all thickness values will result in acceptable cracking levels except for the 8-in.-thick (200 mm) pavement.

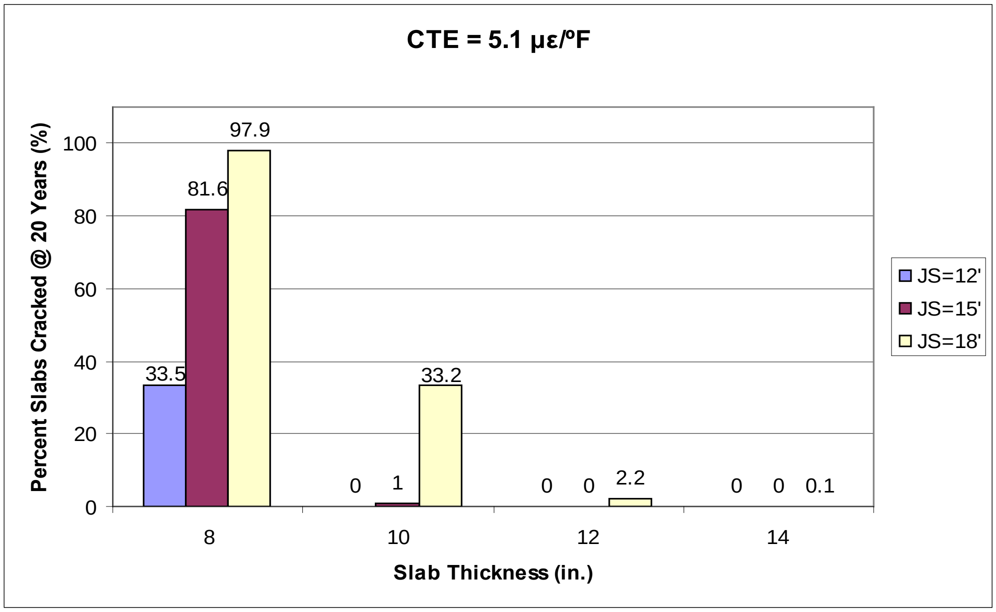

Figure 49. Percent slabs cracked for combinations of joint spacing and slab thickness for CTE of 5.1 με/ºF.

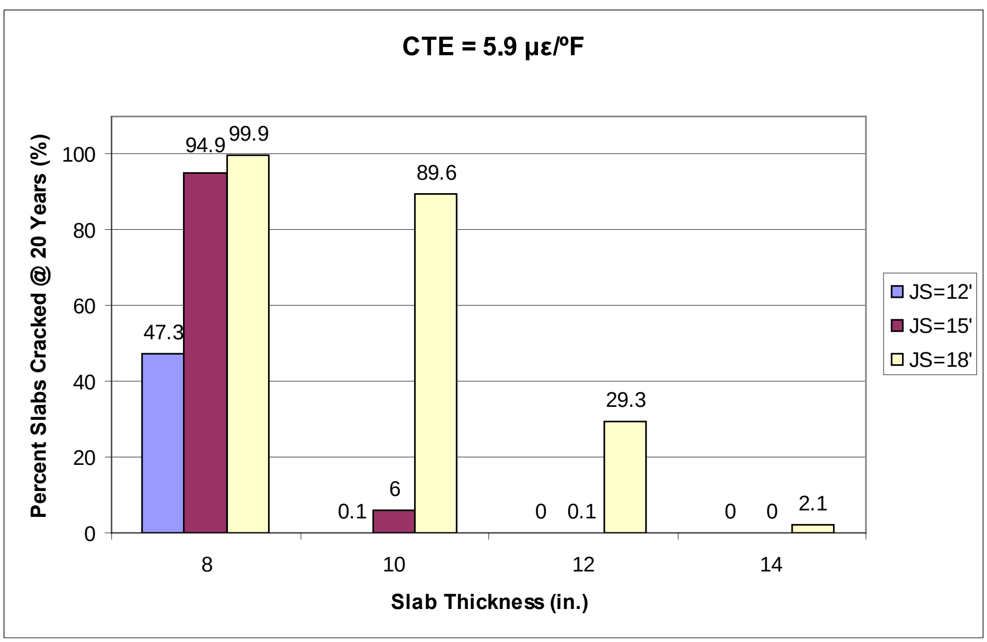

Figure 50. Percent slabs cracked for combinations of joint spacing and slab thickness for CTE of 5.9 με/ºF.

CONCLUSIONS

The results of these analyses demonstrate the importance of aggregate CTE and how it can affect the design for different joint spacing and slab thickness values. This is an important consideration in designing concrete pavements containing ACBFS aggregates. As the examples show, it is also important to identify the representative CTE value for the specific aggregate source being used on a project.

The current design guidelines used by each of the three States produce adequate results overall, but the CTE of the concrete made with different types of coarse aggregate should be considered in the design process. The implication of these findings can be important to the successful performance of concrete pavements containing ACBFS coarse aggregate.

CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

In 2006, a moratorium on the use of ACBFS aggregate in concrete pavements in Michigan went into effect as a result of studies either conducted or commissioned by MDOT that evaluated the performance of concrete pavements constructed with ACBFS aggregate. This report has reviewed the currently available related literature, examined pavements constructed with ACBFS aggregates in Ohio and Indiana, and conferred with the project's expert task group, drawing together conclusions and recommendations for the future use of ACBFS aggregates in concrete pavements.

CONCLUSIONS

Based on the information collected in this study, the following conclusions can be drawn:

- Individual particles of ACBFS aggregate are highly variable due to their vesicular nature and mineralogy, which are largely a function of how the material is cooled during processing. In general, slowly cooled slag will have fewer entrapped pores, resulting in increased density. Slow cooling will also result in the formation of crystalline phases, resulting in increased chemical stability. It is recognized that denser, more crystalline ACBFS aggregates are more desirable. The use of water in processing at the Ford Rouge River Complex was singled out in one study (Vitton, Subhash, and Dewey 2002) as negatively impacting the properties of the ACBFS aggregate from this source. Limits are set in many locales on the percentage of glassy (noncrystalline) particles allowed for use in concrete.

- Bulk physical properties of ACBFS are relatively uniform over time, having similar variability to that observed in naturally derived materials. Although absorption is expected to be higher than that of natural aggregate, it is known that the highest quality ACBFS aggregates will have less than 4 percent absorption. In addition, the higher the density, the higher the suitability of the ACBFS for use as an aggregate in paving concrete. The Japanese standards set a minimum oven-dry density of 2.4 g/cm^3 (0.0867 lb/in^3) for ACBFS aggregates to be used in normal concrete applications (JIS 2003).

- Two often-cited chemical properties of ACBFS that pose a risk to concrete performance are iron unsoundness and dicalcium silicate unsoundness. Both are considered rare in modern ACBFS and can be addressed through control of the chemical composition.

- The chemical property of greatest concern regarding the use of ACBFS aggregate in concrete pavements is calcium sulfide. It is known that the solubility of calcium sulfide increases with increasing alkalinity of the concrete pore solution, which is a function of the cement alkalinity, cement content, and the presence of other sources of internal soluble alkalis including those that might exist in SCMs (e.g., certain Class C fly ashes). It has been observed that the dissolution of calcium sulfide has led directly to the deposition of secondary ettringite in available space including air voids, cracks, and gaps. To what degree this is detrimental to concrete in service is unknown, but it has been postulated that extensive infilling of the entrained air-void system by secondary ettringite may compromise its ability to protect the paste against freeze-thaw damage, especially if the air-void system was marginal to begin with.

- Further, it has been hypothesized that if sufficient dissolution of calcium sulfide occurs, it may result in a type of internal sulfate attack causing paste expansion and cracking. European (CEN EN 12620:2002) and Japanese (JIS A 5011-1:2003) standards establish an upper limit for total sulfur of 2 percent for ACBFS aggregates and acid-soluble limits of 1.0 and 0.5 percent, respectively. Studies conducted in Michigan and Ohio suggest that the use of an SCM capable of mitigating ASR and sulfate attack (e.g., Class F fly ash, slag cement) would be effective in reducing the risk of deterioration.

- Numerous documents and international standards cite the negative impacts incurred when ACBFS aggregates are batched in concrete dry of SSD. This has resulted in loss of workability of the fresh concrete and increased shrinkage and uncontrolled random early-age cracking in newly constructed pavements. It has also been cited as causing microstructural damage due to localized desiccation of the cement paste in the immediate vicinity of the ACBFS particles. The first citation of this problem goes back to the 1940s, yet it is still commonly cited in the most recent documents reviewed.

- In hardened concrete, the mechanical property of greatest interest with regard to concrete pavement performance is the characteristics of the crack face that forms at control joints. It is commonly cited that the crack face in concrete containing ACBFS aggregate is relatively smooth, with the crack passing through, not around the aggregates as is common with many naturally derived aggregates. The smooth crack interface provides very little aggregate interlock once the crack opens beyond 0.035 in. (0.899 mm), and results in low LTE if load transfer devices, such as smooth steel dowels, are not used or if the crack occurs at midpanel.

- Concerns have also been cited regarding the toughness of concrete made with ACBFS aggregate, particularly when loaded dynamically. Some laboratory studies have concluded that the cracking tendency of concrete made with ACBFS aggregates is such that it is more prone to cracking/spalling. This has resulted in reduced serviceability and higher maintenance/rehabilitation costs on a number of long-jointed reinforced concrete pavements in Michigan.

- The Michigan DOT has performed an analysis of the maintenance costs for concrete pavements with and without ACBFS in otherwise similar designs. Data from 148 projects were analyzed (107 with natural aggregate and 41 with ACBFS aggregate). The results indicate that the average maintenance expenditure for ACBFS concrete pavements is nearly twice that for pavements with natural aggregate.

- Overall, the use of short-jointed plain concrete pavements on stiff base support is believed to lead to better performance of ACBFS concrete pavements as it reduces the reliance on aggregate interlock and minimizes movement and deflection at the joint.

RECOMMENDATIONS

The recommendations based on a review of the literature fall into two categories. The first are recommended strategies for immediate implementation of the products developed as part of this study regarding the use of ACBFS aggregate in concrete pavements:

- Distribution of the Best Practices Guide (Smith, Morian, and Van Dam 2012) to agencies that use ACBFS coarse aggregate in paving concrete. The guide document includes discussion of the chemical and physical properties of ACBFS and presents overall recommendations that should be considered in efforts to improve pavement performance. Of critical consideration is the absolute necessity of ensuring that the ACBFS aggregate is maintained wet and batched at SSD or wetter by requiring that concrete plants are equipped with sprinklers and drainage facilities.

- Development of pavement design recommendations that will help to improve the performance of pavements using ACBFS as a coarse aggregate in the concrete. These will include the adoption of short-jointed dowelled plain concrete pavements, with typical joint spacings of 15-ft (4.6 m), but perhaps slightly shorter (but no less than 12 ft (3.7 m)) as dictated by the MEPDG or applicable design procedure. Other design considerations include the provision of a stiff base course (perhaps cement- or asphalt-treated in accordance with an agency's practices) and the adaptation of an aggressive sealing program aimed at reducing the amount of free moisture entering the pavement system.

Recommendations for future research activities that are required to answer questions that are currently not addressed include the following:

- A study of pavements constructed with ACBFS coarse aggregates spanning multiple States is needed to evaluate their performance, especially with regard to pavement longevity. With the current emphasis in the industry on long-life concrete pavements that have expected lives of 35 to 50 years, factors contributing to lives in excess of 30 years need to be studied, linking longevity to the chemical, mineralogical, and physical properties of the ACBFS aggregate as well as the concrete mixture, pavement design, and construction factors.

- A study is needed to compare the chemical, mineralogical, and physical characteristics of all major sources of ACBFS used as coarse aggregate in modern concrete pavements in the United States. This study would primarily be an inventory of current supply, but should be linked to pavement performance studied as described above to determine if there is a linkage between ACBFS sources and pavement performance.

- A study evaluating the chemical stability of calcium sulfide in the presence of various mixture parameters is needed to evaluate the effects of fly ash and high-alkali cements, as well as ASR-susceptible aggregates. The use of Class F fly ash, slag cement, and blends of both should be investigated for mitigation of ASR and internal sulfate attack. As part of this study, the impact on infilling of the air-void system with secondary ettringite also needs to be evaluated.

APPENDIX A: I-94 QUALITY CONTROL PLAN[1]

Edw. C. Levy Co. Quality Control Department Concrete Coarse Aggregate for Ajax Paving Industries I94, Pelham to Wyoming Reconstruction Project

1.0 Scope

1.1 This plan outlines the procedures, responsibility and frequency for testing of concrete coarse aggregate materials (Coarse Agg Blast Furnace Slag & Intermediate Agg Blast Furnace Slag) for the Ajax Paving Industries, I94, Pelham to Wyoming reconstruction project–MDOT Project #'s 45684A, 55848A, 60412A, 60413A & 72064A

2.0 Applicable Procedure Documents

2.1 Michigan Department of Transportation (MDOT) Procedures for Aggregate Inspection

2.2 Michigan Department of Transportation (MDOT) Michigan Test Methods;

2.2.1 MTM 107 – Michigan Test Method for Sampling Aggregate

2.2.2 MTM 108 – Michigan Test Method for Materials Finer Than No. 75µm (No. 200) Sieve in Mineral Aggregates by Washing

2.2.3 MTM 109 – Michigan Test Method for Sieve Analysis of Fine, Dense Graded, Open Graded and Coarse Aggregates in the Field

2.2.4 MTM 110 – Michigan Test Method for Determining Deleterious and Objectionable Particles in Aggregate

2.3 American Society for Testing and Materials (ASTM)

2.3.1 ASTM C29 – Test Method for Unit Weight and Voids in Aggregate

2.3.2 ASTM C33 – Specifications for Concrete Aggregates

2.3.3 ASTM C40 – Test Method for Organic Impurities in Fine Aggregate for Concrete

2.3.4 ASTM C127 – Test Method for Specific Gravity and Absorption of Coarse Aggregate

2.3.5 ASTM C128 – Test Method for Specific Gravity and Absorption of Fine Aggregate

2.3.6 ASTM C136 – Test Method for Sieve Analysis of Fine and Coarse Aggregates

2.3.7 ASTM C566 – Test Method for Total Moisture Content of Aggregate by Drying

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2.3.8 ASTM C702 – Practice for Reducing Samples of Aggregate to Testing Size

3.0 Responsibilities

3.1 The Edw. C. Levy Co. will deliver the Blast Furnace Slag coarse aggregate materials to the Ajax Paving Industries concrete batch plant in a moisture condition at or above SSD (i.e. moisture level of 24 hour soak utilized in ASTM C128)

3.2 Ajax Paving Industries is responsible for the setting of a stockpile watering system on the Blast Furnace Slag batch plant stockpiles.

3.3 Ajax Paving Industries is responsible for the operation of the stockpile watering system on the Blast Furnace Slag stockpiles if it is determined that the moisture levels in the batch plant stockpiles has fallen below the SSD moisture level.

3.4 Edw. C. Levy Co. will make adjustments to and reset any sprinklers that are dislodged from the Blast Furnace Slag stockpiles when quality control representatives are at the concrete batch plant. Ajax Paving Industries will maintain the stockpile watering system at all other times.

3.5 Edw. C. Levy Co. will perform Quality Control testing of the aggregate materials in accordance with this Quality Control Plan.

3.6 Based on the results of the Quality Control testing, the Edw. C. Levy Co. will inform Ajax Paving Industries of any aggregate quality or consistency concerns.

3.7 Edw. C. Levy Co. will provide Ajax Paving Industries with a daily report of moisture results for the Blast Furnace Slag aggregates as sampled from the batch plant stockpiles.

3.8 Ajax Paving Industries will advise Edw. C. Levy Co. of any irregularities in aggregate material that may affect the quality of the finished concrete product as soon as they become apparent. A schedule of “after hours” contacts and phone numbers will be provided to Ajax Paving Industries.

4.0 Frequency of Testing

4.1 Coarse Agg Blast Furnace Slag

4.1.1 The Coarse Agg Blast Furnace Slag will be tested and certified by the Edw. C. Levy Co at the source of the material.

4.1.2 The Edw. C. Levy Co. Quality Control Department will verify the gradation and percent moisture of the Coarse Agg Blast Furnace Slag at the concrete batch plant at a minimum frequency of 3 tests per concrete production day (but not less than 1 test per 1,000t of aggregate used) taken from the face of the pile being used to feed the batch plant (shipping face).

4.1.3 The Ajax Paving Industries end loader will assist the sampling effort by preparing “Mini Stockpile” sample locations at the direction of the Edw. C. Levy Co. Quality Control Technician.

4.1.4 Percent moisture will be calculated based on the gradation samples.

4.1.5 Unit Weight will be calculated on a weekly basis during concrete production.

4.1.6 Specific Gravity and Absorption will be calculated on a monthly basis during concrete production.

4.2 Intermediate Agg Blast Furnace Slag

4.2.1 The Intermediate Agg Blast Furnace Slag will be tested and certified by the Edw. C. Levy Co at the source of the material.

4.2.2 The Edw. C. Levy Co. Quality Control Department will verify the gradation and percent moisture of the Intermediate Agg Blast Furnace Slag at the concrete batch plant at a minimum frequency of 3 tests per concrete production day (but not less than 1 test per 1,000t of aggregate used) taken from the face of the pile being used to feed the batch plant (shipping face).

4.2.3 The Ajax Paving Industries end loader will assist the sampling effort by preparing "Mini Stockpile" sample locations at the direction of the Edw. C. Levy Co. Quality Control Technician.

4.2.4 Percent moisture will be calculated based on the gradation samples.

4.2.5 Unit Weight will be calculated on a weekly basis during concrete production.

4.2.6 Specific Gravity and Absorption will be calculated on a monthly basis during concrete production.

5.0 Feeding of the concrete aggregates into the Concrete Batch Plant

5.1 If a quality problem is identified with either concrete aggregate material, the Edw. C. Levy Co. Quality Control Technician will direct the loading of material into the concrete batch plant in an effort to isolate any "out-of-specification" material.

6.0 Additional measures to promote the production and placement of high quality concrete pavement

6.1 HIPERPAV

6.1.1 Ajax Paving Industries will utilize the HIPERPAV software package to determine optimal concrete production and paving conditions.

6.1.2 If the HIPERPAV software identifies that concrete production and paving conditions are not optimal to the production of quality concrete, measures shall be taken by Ajax Paving Industries to utilize a method recognized by HIPERPAV to create optimal conditions (such as mix adjustments, curing adjustments, time of placement adjustments) or delay paving operations until such time that optimal conditions exist.

6.2 Storage of aggregate materials in concrete batch plant feed hoppers

6.2.1 At the end of each day, or during extended delays of more than 2 hours in duration, every effort shall be taken to empty the concrete batch plant feed hoppers.

6.2.2 Materials stored in concrete batch plant feed hoppers for extended periods of time are subject to the ambient environmental conditions that may result in excessive drying.

APPENDIX B: FHWA BLAST FURNACE SLAG FIELD SURVEY COMMENTS

Ohio (SR-175–Sections 1 and 2)

A field evaluation was conducted on two test sites in Ohio, both located on SR-175 in Beachwood, a southeastern suburb of Cleveland. Section 1 is sited on the eastbound lanes of Harvard Road, and Section 2 is on the southbound lanes of Richmond Road. Both are located near the intersection of the two roads.

Detailed visual assessments based on the Long-Term Pavement Performance (LTPP) program's pavement inspection procedures supplemented by those recently developed by members of the project team for the Innovative Pavement Research Foundation (IPRF) specifically for use in inspecting pavements affected by materials-related distress (MRD). The pavements studied were selected in consultation with the Ohio Department of Transportation (ODOT). Only one 1,000-ft (305 m) representative section was able to be selected for detailed evaluation on each test site using the two methods described above. During the field evaluation, coring locations were identified in one of the slabs in each selected pavement section. In total, four full-depth cores, 4 in. (100 mm) in diameter, were extracted from each evaluated pavement, two at joint locations and two from the slab interior. Cores were taken with associated labels of "B" at the joint, "C" 1 ft (300 mm) away from the joint, "D" midslab free of MRD, and" E" midslab through MRD. Traffic control and coring was provided by ODOT.

SR-175–Section 1, Eastbound Harvard Road (93+28 to 108+37), Beachwood, Ohio

- Description: The 1,000-ft (305 m) section evaluated was in the right lane and included 50 slabs. As figure B-1 shows, little distress or MRD was present on this section.

Figure B-1. Overview of SR-175, Section 1.

- LTPP Survey: Low-severity joint seal damage was recorded for all joints, and a moderate amount of linear cracking was also observed. There were a few joint spalls and spots of scaling, but nothing severe to note at the time. A few rigid patches were recorded around drains.

- MRD Survey: Only a rust-colored staining was observed on this section, down the middle of each slab.

- Coring: No evidence of MRD was observed on this pavement, so the 1B and 1E cores were taken through some of the staining.

SR-175–Section 2; Southbound Richmond Road (223+09 to 247+70); Beachwood, Ohio

- Description: The 1,000-ft (305 m) section evaluated was the right lane and included 50 slabs. Not much distress or MRD was present on this section (see figure B-2), but slightly more than on Section 1.

Figure B-2. Overview of SR-175, Section 2.

- LTPP Survey: Low-severity joint seal damage was recorded for all joints. A moderate amount of linear cracking was also observed and was more prevalent than in Section 1. There were a few joint spalls, but nothing severe to note at the time. A few rigid patches were recorded around drains.

- MRD Survey: Only a rust-colored staining was observed on this section, but in addition to running along the middle of each slab, the staining extended along all the joints and corners of each slab as well.

- Coring: No evidence of MRD was observed on this pavement, so the 1B and 1E cores were taken through a crack in an area with staining.

Indiana (SR-19 and SR-331—Sections 3 and 4)

A field evaluation was conducted on two test sites in Indiana, one project located on SR-19 near Elkhart and the other located on SR-331 near South Bend. Section 3 is located in the northbound lanes of Nappanee Street (SR-19), and Section 4 is located in the northbound lanes of Capital Avenue (SR-331).

Detailed visual assessments based on the LTPP pavement inspection procedures supplemented by those recently developed by members of the project team for IPRF specifically for use in inspecting pavements affected by MRD. The pavements studied were selected in consultation with the Indiana Department of Transportation (INDOT). Only one 1,000-ft (305 m) representative section was able to be selected for detailed evaluation on each test site using the two methods described above. During the field evaluation, coring locations were identified in one of the slabs in each selected pavement section. In total, four full-depth cores, 4-in. (100 mm) in diameter, were extracted from each evaluated pavement, two at joint locations and two from the slab interior. Traffic control and coring was provided by INDOT.

SR-19–Section 3, Northbound Nappanee St (R-26114); Elkhart, Indiana

- Description: The 1,000-ft (305 m) section evaluated was the right lane and included 50 slabs. As figure B-3 shows, there is not much distress or MRD present on this section.

Figure B-3. Overview of SR-19, Section 3.

- LTPP Survey: Low-severity joint seal damage was recorded for all joints. A few linear cracks were also observed. In addition, there were a few joint spalls, but nothing severe to note at the time.

- MRD Survey: Dark staining was observed on this section running along the middle of each slab. Two slabs were observed to have parallel cracking, but that was the only other indication of MRD.

- Coring: No evidence of MRD was observed on this pavement, so the 1B and 1E cores were taken through a crack in an area with staining.

SR-331–Section 4, Northbound Capital Avenue (R-26937); South Bend, Indiana

- Description: The 1,000-ft (305 m) section evaluated was the right lane and included 50 slabs. As figure B-4 shows, not much distress or MRD was present on this section. Less distress was observed than on Section 3, but there was significantly more MRD present.

Figure B-4. Overview of SR-331, Section 4.

- LTPP Survey: Only medium-severity joint seal damage was recorded for all joints. There were many corners with deterioration, but they were not large enough per the distress definition of corner spalling to record.

- MRD Survey: Dark staining was observed on this section, but in addition to running along the middle of each slab, the staining extends along all the joints and corners of each slab as well. The corners of nearly every slab that were deteriorated, but not able to be picked up by the LTPP survey, were recorded as joint disintegration.

- Coring: While MRD was observed on this pavement, it was primarily in the corners. Therefore, the 1B and 1E cores were taken through an area with staining.

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