DEMONSTRATION OF POZZOLAN MODIFIED CEMENT TO MITIGATE ASR

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ABSTRACT

In 1991, the South Dakota Department of Transportation (SDDOT) commissioned a research study to determine the feasibility of producing a cement that would not induce alkali-silica reactivity (ASR). The South Dakota School of Mines and Technology (SDSM&T), under contract to SDDOT, successfully identified suitable deposits of natural pozzolans near Rapid City, SD and demonstrated that ASR could be stopped by incorporating the material in cement.

Since the research completed in 1994, Dacotah Cement, a cement manufacturer in Rapid City, SD, has produced trial quantities of pozzolan modified cement. Based on laboratory test results, the cement - designated Type IPM - shows potential to prevent concrete deterioration from ASR. Because the pozzolan does not require calcining before intergrinding, the modified cement is economical. Strength and durability testing was completed to verify the cements usefulness in widespread use.

Through the testing involved in this project, it has been shown that concrete made with Dacotah Type IPM cement has compressive and flexural strengths approximately 10 - 12% lower than standard Type I/II cement mixtures. This is probably due to the fact that 10% of the cement is replaced with a natural pozzolan. Cores taken from field test sections showed compressive strengths 20 to 40% lower than strengths of laboratory concrete specimens. Time of set for the Type IPM cement is lower than that of Type I/II, mainly due to Type IPM cement being finer than Type I/II. Freeze/thaw and deicer scaling durability of this new cement is equal to the standard Type I/II used as a control.

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1 - INTRODUCTION

1.1 Background

In 1991, the South Dakota Department of Transportation (SDDOT) commissioned a research study to determine the feasibility of producing a cement that would not induce alkali-silica reactivity (ASR). Ramakrishnan, et. al., (8) of the South Dakota School of Mines and Technology (SDSM&T), under contract to SDDOT, successfully identified suitable deposits of natural pozzolans near Rapid City, SD, and demonstrated that ASR could be stopped by incorporating such materials in cement.

Since the research was completed in 1994, Dacotah Cement, a cement manufacturer in Rapid City, SD, has produced a small production run of this pozzolan modified cement. The natural pozzolan is directly interground with the cement, without calcining, which yields an economical product. Based on preliminary laboratory test results, the cement - designated Type IPM - has shown potential to prevent concrete deterioration due to ASR. (8) The next step in the development of this cement was to test various strength and durability properties in order to demonstrate the usefulness of this new cement in widespread use.

1.2 Research Objective

Before South Dakota and other states in the region could routinely use this new pozzolan modified cement, standard laboratory tests were necessary to verify its strength and durability properties when used with reactive aggregates normally encountered in each state. The objective of this research was to determine these strength and durability properties. Laboratory tests were completed to verify Type IPM cement's strength and durability properties combined with aggregates from surrounding states. Field test sections were constructed to verify constructability and finishing properties. With successful laboratory tests and field samples, transportation agencies would be able to use the cement confidently and receive the benefit of longer lived pavements and structures.

In order to measure the strength and durability properties of concrete made with Type IPM cement, standard tests were run. Compressive strength, flexural strength and static modulus of elasticity tests were conducted on specimens cast from mixtures containing both Type IPM and standard Type I/II cements along with reactive aggregates from surrounding states, including Minnesota, Nebraska, South Dakota, Colorado and Wyoming. Each state involved in the project supplied aggregates to SDSMT for testing. Concrete was made for each aggregate combination, with both types of cement, and tested for strength properties. Sulfate durability, freeze/thaw durability, and deicer scaling durability tests were conducted to measure the durability of concrete made with each cement. These durability tests were performed with local aggregates only. The results from all of these tests will verify whether or not Type IPM cement is a useful product in regards to strength and durability aspects.

2 - LITERATURE REVIEW

A literature review was conducted into the various strength and durability aspects of this project. Specifically, areas of concentration include cement chemistry, strength properties of Portland cement, time of set for Portland cement, alkali-aggregate reactivity, sulfate durability, freeze/thaw durability and deicer scaling durability. All of these considerations are interrelated and affected by cement chemistry. This review serves as a background for analyzing laboratory results and compiling these results to form viable conclusions and make recommendations.

2.1 Basic Cement Chemistry

Variations in cement from producer to producer or even from silo to silo can greatly change the quality of concrete. Thus, it is necessary to understand what variations will affect the durability of concrete, and to what extent these variations can be tolerated. In order to discuss the various chemical aspects of cement, the general chemical equations involved in the hydration of cement need to be understood.

Though the exact chemical reactions, and their respective order, are still as yet relatively unknown, it is worthwhile to understand the basic reactions involved in the hydration of cement. The following reactions are taken from the PCA Design and Control of Concrete Mixtures (23):

$$2(3CaO\bullet SiO_2) + 6H_2O \Rightarrow 3CaO\bullet 2SiO_2\bullet H_2O + 3Ca(OH)_2 \qquad \text{(Equation 2.1.1)}$$

$$2(2CaO\bullet SiO_2) + 4H_2O \Rightarrow 3CaO\bullet 2SiO_2\bullet H_2O + Ca(OH)_2 \qquad \text{(Equation 2.1.2)}$$

$$3CaO\bullet Al_2O_3 + 12H_2O + Ca(OH)_2 \Rightarrow 3CaO\bullet Al_2O_3\bullet Ca(OH)_2\bullet 12H_2O \qquad \text{(Equation 2.1.3)}$$

$$4CaO\bullet Al_2O_3\bullet Fe_2O_3 + 10H_2O + 2Ca(OH)_2 \Rightarrow 6CaO\bullet Al_2O_3\bullet Fe_2O_3\bullet 12H_2O \qquad \text{(Equation 2.1.4)}$$

$$3CaO\bullet Al_2O_3 + 10H_2O + CaSO_4\bullet 2H_2O \Rightarrow 3CaO\bullet Al_2O_3\bullet CaSO_4\bullet 12H_2O \qquad \text{(Equation 2.1.5)}$$

In Equation 2.1.1, tricalcium silicate (alite) reacts with water to produce calcium silicate hydrate (tobermorite gel) and calcium hydroxide. Calcium silicate hydrate (C-S-H) is the primary binding material in hydrated cement paste and is usually 50% of the hydrated paste by weight. In Equation 2.1.2, dicalcium silicate (belite) reacts with water to produce calcium silicate hydrate and calcium hydroxide. Tricalcium silicate hydrates and hardens rapidly, and is responsible for initial set and early strength of the cement mortar. Dicalcium silicate hydrates slowly, and adds to strength development at ages greater than one week. In Equation 2.1.3, Tricalcium aluminate reacts with water and calcium hydroxide to produce tetracalcium aluminate hydrate. Tricalcium aluminate liberates a large amount of heat during the first few days of hydration and hardening. This reaction would cause rapid set of the cement paste, and is controlled through addition of gypsum during final grinding of the cement. In Equation 2.1.4, tetracalcium aluminoferrite reacts with water and calcium hydroxide to produce calcium aluminoferrite hydrate. Tetracalcium aluminoferrite reduces the clinkering temperature during cement production, and adds very little to the final strength of a cement paste. In Equation 2.1.5, tricalcium aluminate reacts with water and gypsum to produce calcium monosulfoaluminate hydrate. This reaction delays the rapid set possible in Equation 2.1.3, and may also produce ettringite. Ettringite is a calcium sulphoaluminate with a long, needle-like structure. A standard chemical formula for ettringite is $6CaO \bullet Al_2O_3 \bullet 3SO_3 \bullet 32H_2O$. (11)

Research done at Purdue University has shown some very interesting and promising results. (2) Most new cements contain high alkali concentrations, most of which are immediately soluble in mix water. In the first few minutes of the mixing action, alkali-sulfates are dissolved which are the necessary reagent for the alkali silica reaction. Additionally, sulfates present in the pore solution will tend to be consumed by ettringite formation that rapidly occurs during the initial hydration process prior to final set of the cement paste. In cements containing a soluble gypsum as a sulfate source, it is proposed that the gypsum will act as a reservoir for the sulfate concentration of the pore solution, and will with time maintain this sulfate concentration at a constant until it is depleted. The alkali sulfate remains dissolved until this gypsum source is used up, and does not begin to convert to alkali hydroxide until this time. Thus, as the eventual decrease in sulfate concentration occurs, so does the accompanying increase in hydroxide concentration. The final concentration of hydroxide is equivalent to the sum of the sodium and potassium concentrations. Based on these observations, the pH of the resulting pore solution can be estimated, if measurements are not possible. It has been shown that for a pH of about 12.75, alkali-aggregate reactivity (AAR) will probably not be a problem, but with a pH of 13.67, AAR will definitely be a problem, given that other necessary factors are present in the concrete.

This phenomenon is not always present, especially in gypsum rich environments. Further research into this topic showed that if enough undissolved gypsum is present in the cement paste, then the alkali sulfates may not form alkali hydroxides, significantly reducing the hydroxide concentration in the final pore solution. This would reduce or eliminate the AAR potential, but may introduce other problems, such as lower strength or delayed time of set. As a conclusion, research has shown that normal cements with high alkali sulfate concentrations will quickly produce high hydroxide concentrations upon mixing, which instigates ASR. As the undissolved sulfate contained in the gypsum is depleted by ettringite formation, the alkali sulfates are converted to alkali hydroxides which raises the pH of the pore solution. To counter this action, addition of greater amounts of gypsum might prove to substantially reduce or eliminate the ASR potential. (2)

2.2 Strength Properties of Portland Cement

According to the Portland Cement Association (23), compressive strength of concrete depends on many different factors, of which, cement type and chemical composition are very important. Differences in cement composition and the compounds found in different cements create variation in compressive strengths between different cements. ASTM standards specify a minimum compressive strength for mortar made from certain types of cement. Because of this, different cement producers will have varying safety factors by which they satisfy this minimum requirement, and strengths will be higher or lower depending on the size of this safety factor. Variation will also be evident within a single producers cement, as environmental conditions and raw materials change. Therefore, it should not be assumed that any two cements will have identical strength properties given that both cements meet the ASTM minimum requirements.

Another factor affecting the strength properties of a cement is storage and handling. If cement is allowed to get wet, or is stored improperly, hydration will occur. Hydration will permanently reduce the ultimate strength potential of the cement. The PCA suggests proper storage and handling techniques. (23) Bags of cement should be stored on pallets, not damp floors, and kept close together as to reduce air circulation. These pallets should be arranged so that the first cement delivered to a storage facility is the first used in production. Also, the relative humidity of a storage facility should be as low as possible as cement stored in contact with damp air sets more slowly and has less strength than that kept in dry air. Under ordinary circumstances, cement is not stored for long periods of time, but long storage time is possible without appreciable deterioration, given that proper storage precautions are employed.

2.3 Time of Set for Portland Cement

Important to proper placement and finishing of concrete is the time of set for the cement used. The reactions involved in hydration and hardening of the cement must be slow enough to allow for transportation and proper placement, but must be, in certain circumstances, quick enough to reach final set prior to freezing. Regulation of this time of set is accomplished by adding gypsum at the same time that the clinker is ground at the cement plant. Additional factors that affect the rate of hydration include cement fineness, admixtures used, water-to-cement ratio, and temperature of the materials at the time of mixing. Addition of pozzolans and other admixtures affect the setting time of a concrete mixture by altering the water requirement, the fineness, or other properties of the mixture. (23)

2.4 Alkali-Aggregate Reactivity

Alkali-aggregate reactivity (AAR) is a deleterious process in which certain minerals found in aggregates react with soluble alkalis in concrete. This process produces an expansion caused by crystal formation around the aggregate, which in turn develops map cracking in the structure. Some aggregates are aggressively reactive and will produce cracking in as little as a year, while some can take as long as 20 years to react. The effects of AAR can cause serviceability problems and compound other problems such as freeze/thaw durability, sulfate resistance, and deicer scaling. Diagnosis of alkali-aggregate reactivity can be difficult and expensive. New methods are currently under development that will aid in classification of reactive problems in concrete structures.

2.4.1 AAR Mechanisms

Alkali-aggregate reactivity currently has three known mechanisms. (1) One such mechanism is alkali-silica reactivity (ASR). ASR is the expansion caused by a reaction between certain forms of silica in aggregates and potassium and sodium alkalis, mostly from cement, but also found in pozzolans, admixtures, and mixing water. External sources of alkalis, such as deicers and soils containing alkalis, can also add to the ASR problem. The result of this reaction is a gel product that draws water out of the aggregate and the surroundings. The gel in turn expands and causes deleterious map cracking. (1) More specifically, silicates in various constituents of the aggregate react with the alkalis (measured as a Na_2O equivalent) of the cement. Cements containing alkalis in excess of 0.6 percent equivalent Na_2O can cause significant expansion cracking due to ASR. Various blended cements used in a study by Rasheeduzzafar (3) partially or completely prevented the effects of ASR. Tests conducted on silica fume and blast furnace slag blended cements yielded results well within the ASTM standards, while the fly ash blended cement was close to the limit. As a control, ground Pyrex glass was used as aggregate with a standard Type I cement. The control failed the expansion test eightfold. (3)

Another mechanism of AAR is termed alkali-carbonate reactivity (ACR). ACR deterioration has the same ultimate result as ASR, but the reaction occurs between the cement born alkalis and certain fine-grained dolomitic limestones. This reaction is rare due to the limited supply of aggregates that also comply with other requirements for concrete aggregates. The main reaction involved in ACR is referred to as dedolomitization. This reaction consists of the dolomitic aggregate, under given circumstances, converting to calcite and brucite, which take up a greater volume than the dolomite, causing expansion. (3)

One factor affecting AAR potential of a concrete is the chloride ion concentration in the mixture environment. It has been shown that appreciable chloride concentrations significantly affect AAR. Despite this fact, it has also been shown that hydroxide concentrations affect the AAR potential, which are balanced by positive potassium and sodium ions. These ions can be traced back to the source of alkali bearing components used to produce the cement. Different forms of the alkalis present have little effect on the relative ASR potentials. This is because that the alkalis end up in the pore system either by dissolution of alkali sulfates or by release from hydrating cement compounds. This alkali concentration balanced by the hydroxide present in the pore solution creates an environment susceptible to AAR problems later. (2)

2.4.2 AAR Mitigation Methods

To avoid the problems caused by AAR, there are a limited number of options. First, and most obvious, simply do not use aggregates that have proven to be reactive. This is becoming more of a problem as the sources of non-reactive aggregates are being depleted. Mining and quarrying of new non-reactive sources is becoming increasingly expensive, thus escalating the AAR problem. Aggregates that are prone to ASR include the following: opal, chert and chalcedony, tridymite and cristobalite, strained or micro-crystalline quartz, granite gneiss, graywackes, argillites, phylites, siltstones, some natural sands and gravels, and volcanic glasses. (1) Aggregates that are prone to ACR include argillaceous dolomitic limestones containing calcite and dolomite along with small amounts of clay and reactive silica. (3) To avoid AAR on an aggregate level, if elimination of the aggregate is not an option, certain precautions should be followed. Known reactive aggregates should be treated with a diluting technique. This involves selective quarrying to avoid the reactive materials followed by mixing the cheaper reactive aggregate with a more expensive, yet unreactive aggregate. Also, bulk heavy media separation based on differences in specific gravities can be applied to physically remove the unwanted reactive aggregates. (1)

A second option is to use pozzolans to mitigate the detrimental effects of the reaction. Pozzolans include fly ash, silica fume, slag, and some naturally occurring materials. It should be noted that not all pozzolans will always reduce the reactive potential of a concrete mixture, and some may actually aggravate the deterioration of the structure. Research done by Ramakrishnan, et al., (8) has shown the effect of various natural pozzolans in varying concentrations on the mitigation of AAR, especially ASR. Twenty-six natural pozzolan sources were tested for their mitigation potential. Of the pozzolans tested, a silicified volcanic tuff and an expanded brown shale showed the most promising results. A volcanic ash, however, increased expansion to unacceptable levels. According to ASTM P214 (now ASTM C1260), an expansion exceeding 0.20% at 16 days after casting is considered deleterious. (5) Mortars made with some of the previously mentioned natural pozzolans had less than 0.06% expansion for 15% and 25% replacement of cement. Overall strength of these pozzolan modified mixtures was close to standard cement with only 10-17% strength loss without calcining of the pozzolan. This is an acceptable tradeoff for increased AAR durability. (4)

A third option is to use low alkali cement. Cement with alkali content of 0.6% or less of sodium oxide equivalent is said to be low alkali cement. This type of cement meets the criteria set forth in ASTM C595. There is a common misconception here that low alkali cements will completely cure the problem. But, the cement may not be the only source of alkalis. Aggregates and surrounding environments can also feed the reaction. Application of low alkali cements can also be a problem in that they are not readily available in all areas due to lack of proper resources used in production of such cements. (1)

2.5 Sulfate Durability

Sulfate attack is a severe problem in parts of the country. Deterioration and failure of structures have been attributed to this deleterious attack. The sources of sulfate attack have been identified and researched extensively in the recent past, including sulfates from deicing salts, naturally sulfate rich environments, and cement and aggregate compositions. Attack is also aggravated by mechanical means such as increased porosity caused by freezing and thawing. Higher porosity increases the rate at which the sulfates are permeated throughout the cross section. To combat the effects of sulfate attack, the concrete industry must enhance the concrete subjected to these conditions to insure a quality product. There are different methods for mitigation of sulfate attack, which are discussed further.

2.5.1 Sulfate Attack Mechanisms

Sulfate attack of a concrete at a chemical level is due to sulfate salts, such as sodium sulfate, magnesium sulfate, and calcium sulfate. Calcium sulfate reacts directly with tricalcium aluminate in cement to produce ettringite. Sodium sulfate reacts in two stages, first with calcium hydroxide early in hydration which produces calcium sulfate, which in turn reacts as before. Magnesium sulfate, the most deleterious type of sulfate, reacts with the tricalcium aluminate to form ettringite and magnesium hydroxide, which in turn lowers the alkalinity of the pore solution creating an unstable environment for the calcium silicate hydrate binder. The result of this reaction is a cohesionless, limeless silica gel. Thus, as shown in the study, an increased tricalcium aluminate content increases the potential for deterioration due to sulfate attack. (1)

An additional factor that has been defined is the tricalcium silicate/dicalcium silicate ratio. It has been shown that tricalcium silicate produces more than two times the amount of calcium hydroxide than does dicalcium silicate. In lime saturated environments, increased gypsum formation is present as well as larger, and more damaging ettringite crystals. To combat this problem, it has been shown that choosing a cement with low tricalcium aluminate content that is modified with a suitable admixture that renders a resistance equivalent of a Type V cement will work quite effectively. Such admixtures include silica fume, fly ash, blast furnace slag and other natural and synthetic pozzolans. (1)

Rock salt deicers used by highway departments in cold regions can be very detrimental to the life of a concrete structure. Commercially used salts are produced to conform with ASTM requirements. The only limit placed upon the salt is that it must have at least 95 percent sodium chloride, or other salt content by weight. This leaves 5 percent impurities with no limitations on what constitutes an impurity. Brine solutions made from such common salts can contain as much as 4,000 ppm sulfate, which is more than enough to deteriorate concrete. Much research has been done into sulfate resistance and durability, but little had been done on the influence of sodium chloride and calcium sulfate solution. The Iowa Highway Research Board in conjunction with Iowa State University has completed research into this area. (2)

Deicing situations are especially damaging due to existing cracks and joints in a pavement. These act as reservoirs for the brine solution and collection areas for settling gypsum and salts. Cracking caused by this compounded sulfate attack can appear similar to aggregate related D-cracking, which makes diagnosis quite difficult in early stages of awareness. Research done by Iowa State University has shown that small amounts of sulfate impurities in rock salt are damaging to the durability of concrete mortar. With as little as 2 percent gypsum in a solution, maximum deterioration occurred in lab tests, which was associated with a 40 percent loss in tensile strength. (2)

The physical mechanisms involved in sulfate-enhanced chloride deterioration include several modes. Most important is thought to be void filling from formation of a salt and additional ettringite. The presence of gypsum also increases the formation of the salt. Since the void structure is necessary for freeze/thaw durability, any additional substances filling the needed space will greatly reduce the expected durability. The combined effects of these phenomena will lead to premature deterioration and eventual failure.

2.5.2 Engineering Considerations

Environments will contain sulfates may deteriorate the concrete prematurely. Such environments are widespread, and include low lying areas that contain moist soils, naturally sulfate rich soils. The actions needed to prevent or at least reduce the effects of sulfate attack have been studied for many years. The results of this research and practical applications have given some fairly straight forward conclusions. (3)

First, pozzolans play a major part in preventing sulfate attack. Fly ash is a major constituent used in today's concrete mixes. Class F fly ash has been proven to be effective in preventing sulfate attack, most likely due to the reduced permeability of the concrete produced with this fly ash which in turn prevents the sulfates from reaching the core of the concrete structure. Typical mixture proportions will contain 15 to 25 percent fly ash as a replacement for the cement. It has also been proposed that in environments containing higher concentrations of sulfates, the fly ash could be added directly to the mix, without removing any of the cement. (3)

A second factor that influences sulfate resistance is the chemical composition of the cement. Cements with low tricalcium aluminate content (less than 0.6% sodium oxide equivalent) will be a good choice for sulfate resistance. Certain pozzolanic cements will also be resistant, given that they too contain a low tricalcium aluminate content. Notice should be taken that pozzolans or other admixtures will not necessarily reduce the effects of sulfate attack, and could result in complete failures if the cement itself is not resistant. A common practice is to insure that ample cement is present in a mixture proportion. This is usually achieved by adding an extra bag (94 lbs) of cement to a cubic yard of concrete. Generally, this would be a Type V cement. A guideline for determining when to use extra cement, a pozzolan, or both is as follows: environments containing less than1500 ppm sulfates (as SO_3) can be controlled by using Type V cement alone, environments containing up to 10,000 ppm sulfates can be controlled by adding an extra bag of cement or by replacing about 25% of the cement with a pozzolan, and environments containing more than 10,000 ppm sulfates can be controlled by adding an extra bag of cement as well as replacing about 25% of the cement with a pozzolan. (3)

2.6 Freeze/Thaw Durability

Freezing and thawing of a concrete can be detrimental to a given structure. As the water trapped in the pore system, in the aggregates and in the voids freezes, pressure is exerted on the internal structure of the concrete. If this pressure overcomes the tensile strength of the concrete, cracking will result. This cracking will lead to increased water absorption and more freeze/thaw damage as water penetrates the large pathways created by cracking in the previous cycle. To reduce the effects of the freezing and thawing process, air entrainment is specified for concrete exposed to a freezing environment. This air entrainment acts as a pressure relief system for the expansion of the freezing water. Generally, 4 to 6% air entrainment is suggested for durable concrete. (6)

2.6.1 Environmental Considerations

When performing freeze/thaw tests in a laboratory setting, it is necessary to completely understand the environment in which the tests are being run. High cooling rates, like those found in the laboratory, can tend to decrease the durability factor for a given specimen. This is due to the increase of hydraulic pressures and thermal shock of the specimen. On the other hand, lower cooling rates tend to draw water into the specimen, which in turn increases the osmotic pressure and ice lens production. Laboratory conditions produce relatively high cooling rates, whereas field conditions are usually much lower. For this reason, it has been determined that more research and development is needed to more accurately predict the freeze/thaw durability of concrete in the field. The presence of salts (such as deicers) tends to increase the severity of the freezing and thawing cycle. These salts provide a source of additional water by melting surface ice and also create osmotic pressures that tend to increase the degree of saturation. Given this information, it is stated that care should be taken when analyzing concrete using current laboratory procedures. (6)

2.6.2 Effects of Deicing Salts

Ice is a major problem for motorists in many cold regions, and there are few environmentally friendly deicing solutions. One of the most common of these solutions is salt, normally a sodium or calcium chloride. Application of salt to concrete, combined with the cycles of freezing and thawing can cause scaling and eventual failure of the concrete structure. The variables controlling scaling durability are not well understood, and there is a need to study what components in concrete affect its deicer scaling durability. One of the most prominent factors is strength, which is enhanced in high performance concrete. (7)

High performance concretes can be produced with high deicer-scaling resistance. Research was reported in the literature on 27 high performance concrete mixes to determine their respective durabilities. (7) The water-to-cementitious ratios of each mix were varied from 0.23 to 0.30 and the air void structure was varied from high entrainment to no entrainment. The mixes all contained 6% silica fume as a direct replacement for the cement. Results of these tests showed that the high performance mixes had excellent deicer scaling durability, regardless of air entrainment or curing time. Weight losses were determined for each specimen, and results after 150 cycles showed that less than 0.50 kg/m^2 total scaling occurred. Given this information, it is determined that it is possible to produce high performance concretes with excellent deicer scaling durability, with or without air entrainment. (7)

3 - EXPERIMENT DESIGN

As a means of comparing Type I/II and Type IPM cements, strength testing results and durability testing results were analyzed for trial mixtures made using each cement. These mixtures varied by aggregate used, and were based on mixture designs submitted by each state. Strength properties tested include compressive strength, flexural strength, and static modulus of elasticity. Durability properties tested include sulfate durability, freeze thaw durability, and deicer scaling durability. Results of these tests, as well as field testing of trial sections, were analyzed to determine the usefulness of Type IPM cement in comparison with Type I/II cement. Descriptions of the materials used and tests performed are included in the following sections.

3.1 Aggregate Properties

Aggregates from five states, listed in Table 3.1, were used in this study. Standard physical properties per ASTM C136 (30), ASTM C566 (32), and ASTM C127 (33) (including moisture content, specific gravity, absorption, and gradation) for each were obtained from aggregate analysis and compared with values reported by the supplying state. The values of specific gravity and absorption used in this research are included in Table 3.1, with aggregate gradations located in Reference 34. Moisture content for each aggregate was determined during batching sequences as each aggregate had variable moisture contents. Each aggregate was used as received, with no modifications for possible changes due to shipping.

Table 3.1 - Aggregate Properties

Aggregate	Specific Gravity	Absorption
Colorado Coarse	2.60	0.70%
Colorado Fine	2.61	0.80%
Minnesota - Meeker Litchfield Sand	2.60	1.10%
Minnesota - Johnson Lesseur Sand	2.62	1.10%
Minnesota - North Con. Aggregate	2.64	1.60%
Minnesota - New Ulm Quartzite	2.63	0.50%
Minnesota - North Star Kasota Sand	2.57	1.60%
Minnesota - Wallner Sand	2.57	1.20%
Wyoming - Starr Gravel	2.64	0.61%
Wyoming - Starr Sand	2.60	1.24%
Nebraska Sand	2.63	1.50%
South Dakota - Sioux Quartzite	2.66	0.15%
South Dakota - Opperman Sand	2.63	1.50%
South Dakota - Limestone	2.67	0.70%

3.2 Cement Properties

The cement used in the mixtures was either Dacotah Type I/II or Type IPM. The two cements were used with equal dosages for each pair of mixture proportions, with no modification for specific gravity or fineness. Table 3.2 contains results from Dacotah Cement mill reports. As shown in this table, the fineness of the Type IPM cement is higher than the fineness of the Type I/II cement. Also shown in Table 3.2 for Type IPM cement, the mortar expansion at the age of 14 days is 7 times greater than the ASTM C595M-95 limit.

Table 3.2 - Mill Reports for Dacotah Type I/II and Type IPM Cements

		ASTM C150-95a		DACOTAH	ASTM C595M-95	DACOTAH
CHEMICAL PROPERTIES	LIMIT	TYPE I	TYPE II	TYPE I/II	TYPE I(PM)	TYPE IPM
Silicon dioxide	min		20.00%	21.99%		26.48%
Aluminum oxide	max		6.00%	4.67%		4.99%
Ferric oxide	max		6.00%	3.27%		3.29%
Calcium oxide				64.60%		59.06%
Magnesium oxide	max	6.00%	6.00%	1.19%	6.00%	1.16%
Sulfur trioxide	max	3.00%	3.00%	2.63%	4.00%	2.48%
Sodium Oxide		---	---	---		0.11%
Potassium Oxide		---	---	---		0.55%
Loss on ignition	max	3.00%	3.00%	1.36%	5.00%	1.70%
Insoluble residue	max	0.75%	0.75%	0.38%		6.82%
Tricalcium Silicate				52.18%	---	---
Tricalcium Aluminate	max		8.00%	6.90%	---	---
Tetracalcium Aluminoferrite				9.95%	---	---
Alkalies	max	0.60%	0.60%	0.51%		0.47%

PHYSICAL PROPERTIES						
Blaine Fineness (m^2/kg)	min	280	280	385		440
No. 325 Mesh % Retained		---	---	---		95.40%
Autoclave expansion	max	0.80%	0.80%	-0.01%	0.80%	0.00%
Autoclave contraction	max	---	---	---	0.20%	-0.02%
Gillmore initial set (min)	min	60	60	155	----	----
Gillmore final set (min)	max	600	600	285	----	----
Vicat setting time (min)	not less than	45	45	100	45	110
Vicat setting time (min)	not more than	375	375		120	
3 Day compressive strength (psi)	min	1740	1740	3138	1810	3476
7 Day compressive strength (psi)	min	2760	2760	4563	2810	4499
28 Day compressive strength (psi)	min	4060	4060	6443	3510	5947
Air content of mortar, volume	max	12.0%	12.0%	7.3%	12.0%	8.7%
False set penetration	optional	50	50	86.2	----	----
Mortar Expansion	max				0.02%	0.147%

3.3 Laboratory Testing

The following tests were performed on freshly mixed concrete test batches:

ASTM C31	Standard Test Method for Making and Curing Concrete Test Specimens in the Field
ASTM C138	Standard Test Method for Unit Weight, Yield and Air Content (Gravimetric) of Concrete
ASTM C143	Standard Test Method for Slump of Hydraulic Cement Concrete
ASTM C231	Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method
ASTM C1064	Standard Test Method for Temperature of Freshly Mixed Portland Cement Concrete

All data obtained from these tests are located in Reference 34, on the same sheet as the mixture proportioning of each trial batch. The standard mixture design sheets provided by the various states included specified values for plastic properties such as slump and air content. If acceptable values were obtained for each trial batch, the specimens were cast and testing continued for that mixture. Specimens were cast according to ASTM C31 (15), demolded at the age of one day, and cured in saturated lime water for the duration of testing. If acceptable values for plastic properties were not obtained, then the batch was discarded, and adjustments made to obtain the specified results. Only four trial batches were discarded (SD IPMA, SD I/IIA, WY IPMA, WY I/IIA). For the discarded trials, cylinders and flexural beams were cast to verify strength results. Any admixtures used in a particular batch were of the brand and type as used in the respective state DOT mixture designs. If no admixtures were specified, or no longer available, those standard to the local redi-mix producers were used. These admixtures were obtained directly from the producer.

Strength testing was performed on all concrete mixtures involved in this study. Testing for compressive strength was completed according to ASTM C39 (14) using 4" x 8" cylinders. An average of three cylinders constituted one test. Testing for flexural strength was completed according to ASTM C78 (20) using 4" x 4" x 14" prismatic beams. An average of three beams constituted one test. Static modulus of elasticity was determined according to ASTM C469 (13) again using 4"x 8" cylinders. Results of two cylinders were averaged for each test. All dimensions were measured using the same digital caliper to eliminate error in measuring devices. The same testing machine was used throughout the series of strength tests in order to eliminate variation between testing machines.

Durability testing was performed on concrete made from South Dakota aggregates only. Sulfate durability was tested according to ASTM C1012. (9) A minimum of six mortar bars (1"x1"x10") were tested for each cement type. Freeze/thaw durability was tested according to ASTM C666. (10) A minimum of four beams (4"x4"x14") were tested for each cement type. Deicer scaling durability was tested according to ASTM C672. (12) A minimum of four beams (4"x4½"x16") were tested for each cement type.

3.4 Mixture Proportions

Mixture proportioning for this project was based on standard mixture proportions as supplied by participating state DOT personnel. A spreadsheet was developed using the Portland Cement Association Absolute Volume Method (23) to determine batch sizes and respective constituent weights used in each test mixture. These sheets are included in Reference 34. Each mixture was given an identification, based on state, aggregate, and cement type used. Table 3.3 contains the identification used for each mixture and a brief description of the materials used. Table 3.4 contains a summary of the mixture proportions found in Reference 34. The mixtures in a pair of mixtures are identical, except for the cement type changing from Type I/II to Type IPM for a constant cement content. Note that one pair of mixtures is not identical to another, as each mixture is based on a different standard mixture proportion from each state.

Table 3.3 - Mixture Identification

Mixture ID#	*Interpretation*
CH-1	Chamberlain, SD Field Test Section (Type IPM Cement)
COL I/II	Colorado Aggregates - Type I/II Cement
COL IPM	Colorado Aggregates - Type IPM Cement
ML I/II	Minnesota Aggregate (Meeker-Litchfield Sand with SD Quartzite) - Type I/II Cement
ML IPM	Minnesota Aggregate (Meeker-Litchfield Sand with SD Quartzite) - Type IPM Cement
MNCA I/II	Minnesota Aggregate (No. Con. Agg. with Johnson Lesseur Sand) - Type I/II Cement
MNCA IPM	Minnesota Aggregate (No. Con. Agg. With Johnson Lesseur Sand) - Type IPM Cement
MNUQ I/II	Minnesota Aggregate (New Ulm Quartzite with North Star Sand) - Type I/II Cement
MNUQ IPM	Minnesota Aggregate (New Ulm Quartzite with North Star Sand) - Type IPM Cement
NE I/II	Nebraska Sand - SD Limestone - Type I/II Cement
NE IPM	Nebraska Sand - SD Limestone - Type IPM Cement
SD I/II	South Dakota Aggregates - Type I/II Cement
SD IPM	South Dakota Aggregates - Type IPM Cement
WNU I/II	Minnesota Aggregate (Wallner Sand with SD Quartzite) - Type I/II Cement
WNU IPM	Minnesota Aggregate (Wallner Sand with SD Quartzite) - Type IPM Cement
WY-1	Cheyenne, WY Field Test Section (Type IPM Cement)
WY I/II	Wyoming Aggregates - Type I/II Cement
WY IPM	Wyoming Aggregates - Type IPM Cement

Table 3.4 - Summary of Mixture Proportions

Mix ID	Cement pcy	Type	Coarse pcy	Fine pcy	Water pcy	Air Entrainer oz/cwt	Brand Name	HRWR oz/cwt	Brand Name
WY I/II B	658	I/II	1855	1195	230	4	MB-VR	16	Rheobuild 1000
WY IPM B	658	IPM	1855	1195	230	3	MB-VR	12	Rheobuild 1000
SD I/II B	600	I/II	1740	1235	252	1.85	MB-VR	0	
SD IPM-B2	600	IPM	1740	1235	252	1.85	MB-VR	0	
NE I/II	564	I/II	905	2159	237	1.3	MB-VR	0	
NE IPM	564	IPM	905	2159	237	1.3	MB-VR	0	
COL I/II	610	I/II	1040	1829	275	1.4	MB-VR	0	
COL IPM	610	IPM	1040	1829	275	1.4	MB-VR	0	
MNCA I/II	530	I/II	1899	1200	244	1.22	MB-VR	0	
MNCA IPM	530	IPM	1899	1200	244	1.22	MB-VR	0	
MNUQ I/II	530	I/II	1868	1200	244	1.2	MB-VR	1	Rheobuild 1000
MNUQ IPM	530	IPM	1868	1200	244	1.2	MB-VR	1	Rheobuild 1000
WNU I/II	530	I/II	1883	1200	244	1.25	MB-VR	1	Rheobuild 1000
WNU IPM	530	IPM	1883	1200	244	1.25	MB-VR	1	Rheobuild 1000
ML I/II	530	I/II	1897	1200	244	1.2	MB-VR	0	
ML IPM	530	IPM	1897	1200	244	1.2	MB-VR	0	

4 - LABORATORY TESTING RESULTS

Laboratory testing for this research consisted of preparing laboratory mixed concrete specimens, compressive strength testing, flexural strength testing, static modulus of elasticity testing, sulfate durability testing, freeze/thaw durability testing, and deicer scaling testing. Sixteen different concretes were mixed in the laboratory, with specimens cast for strength testing. Durability tests were performed on South Dakota mixtures only. Following are the results of all laboratory testing.

4.1 Strength Properties

The following figures (Figure 4.1.1 - 4.1.8) contain compressive strength curves for the mixtures listed in Table 3.3. Results of the static modulus of elasticity test are found in Table 4.1.2. Numerical results for compressive and flexural strength testing along with data sheets for these tests are located in Reference 34.

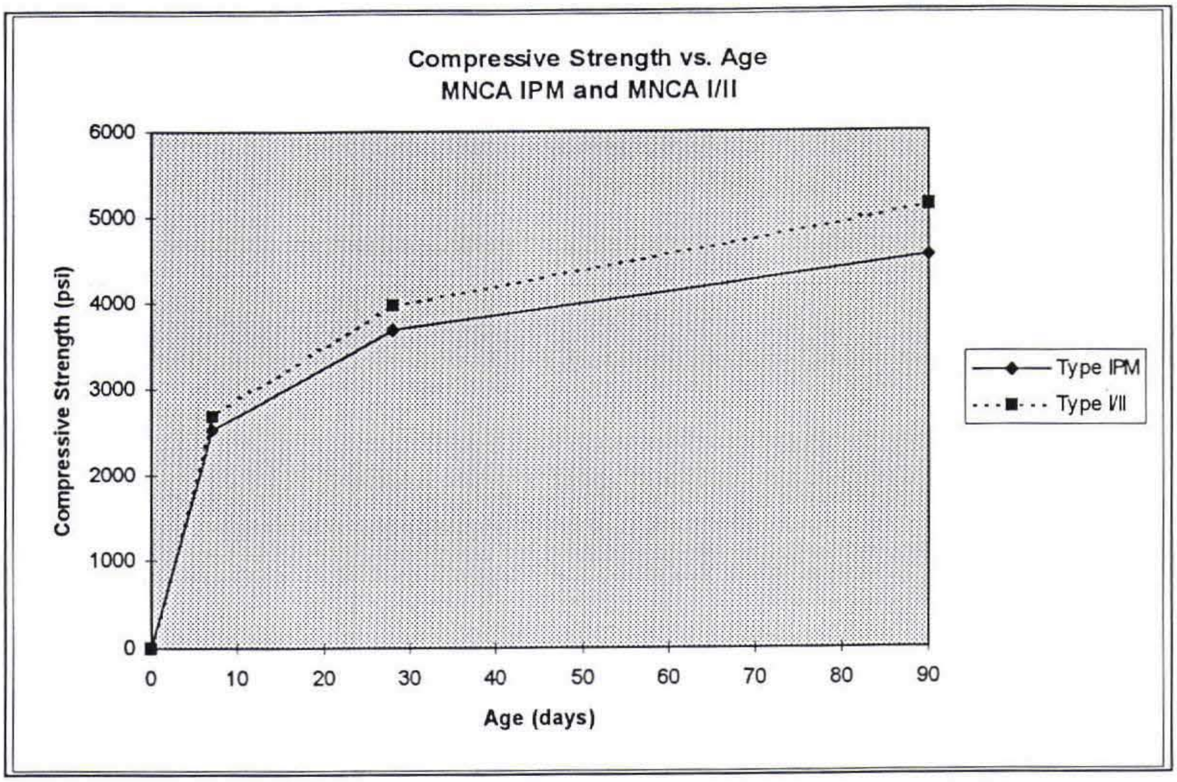

Figure 4.1.1 - Compressive Strength vs. Age MNCA IPM and I/II

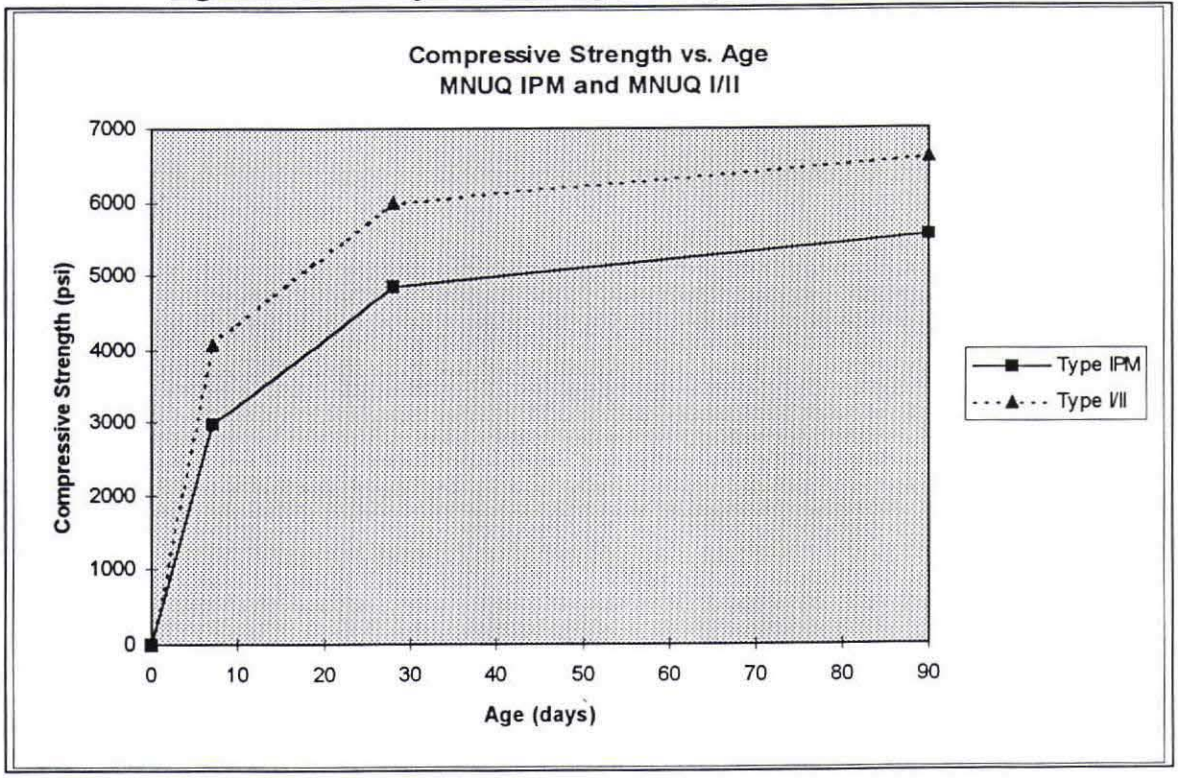

Figure 4.1.2 - Compressive Strength vs. Age MNUQ IPM and I/II

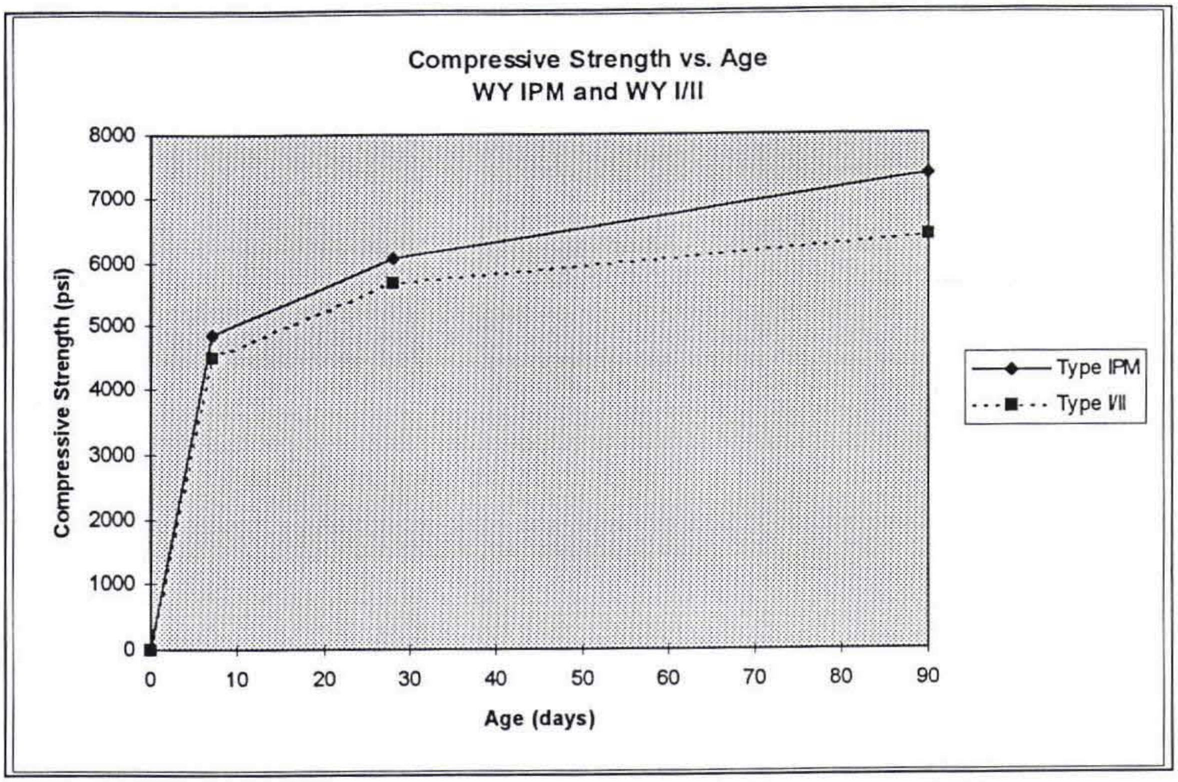

Figure 4.1.3 - Compressive Strength vs. Age WY IPM and I/II

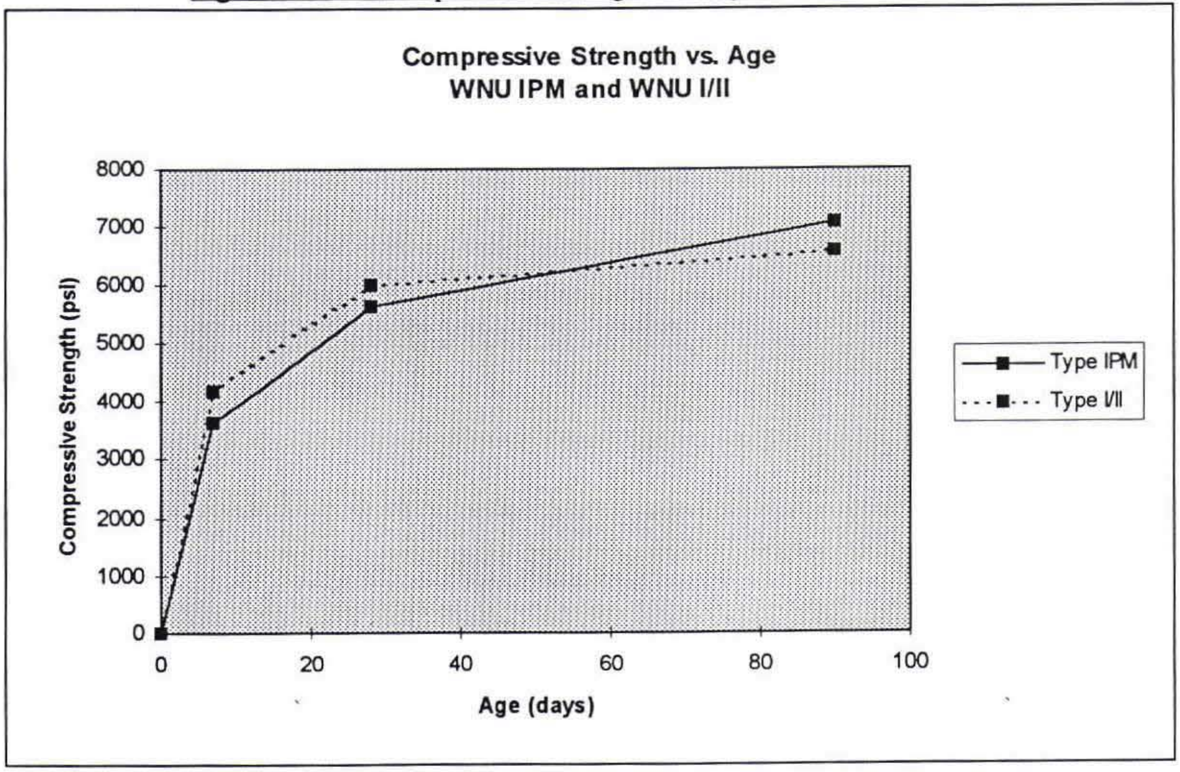

Figure 4.1.4 - Compressive Strength vs. Age WNU IPM and I/II

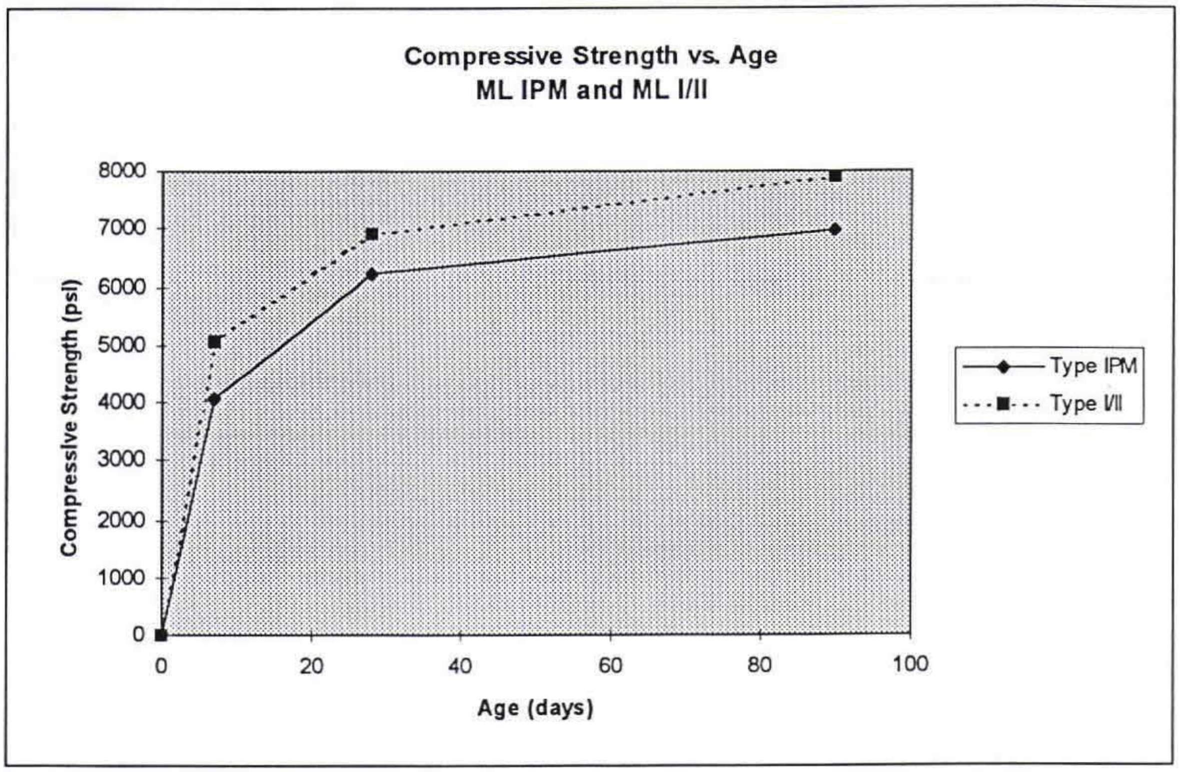

Figure 4.1.5 - Compressive Strength vs. Age ML IPM and I/II

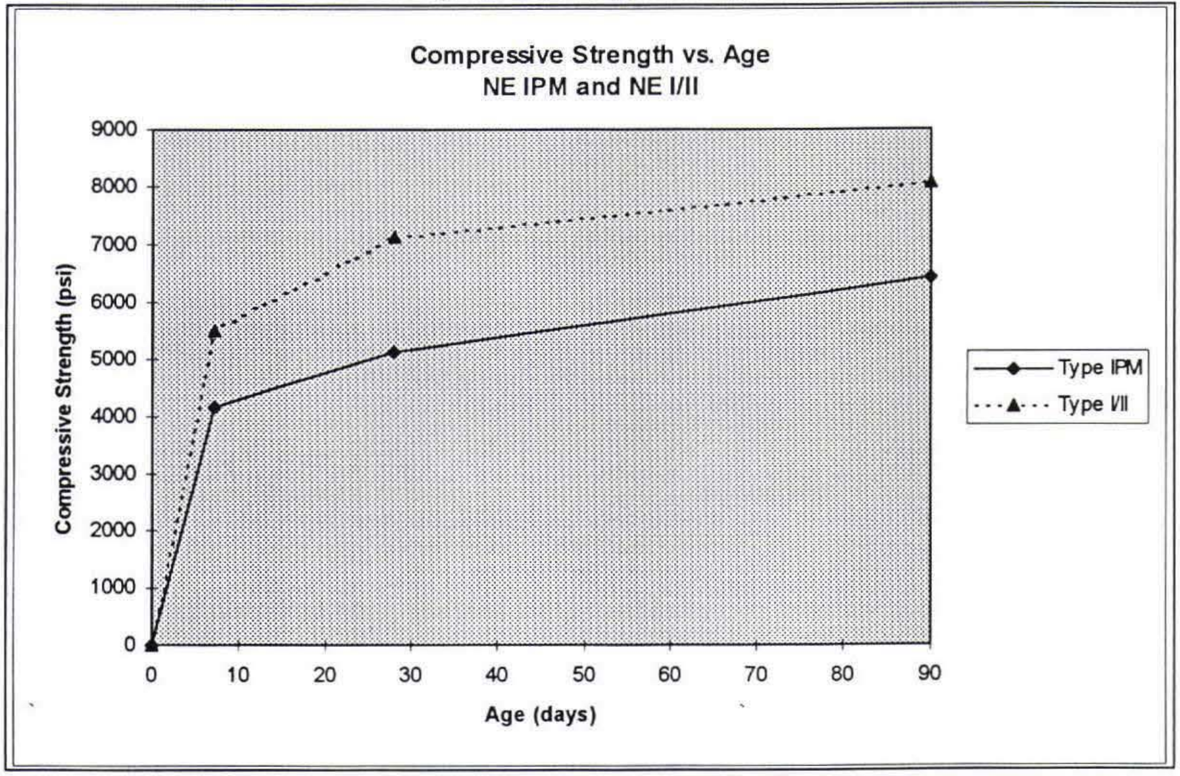

Figure 4.1.6 - Compressive Strength vs. Age NE IPM and I/II

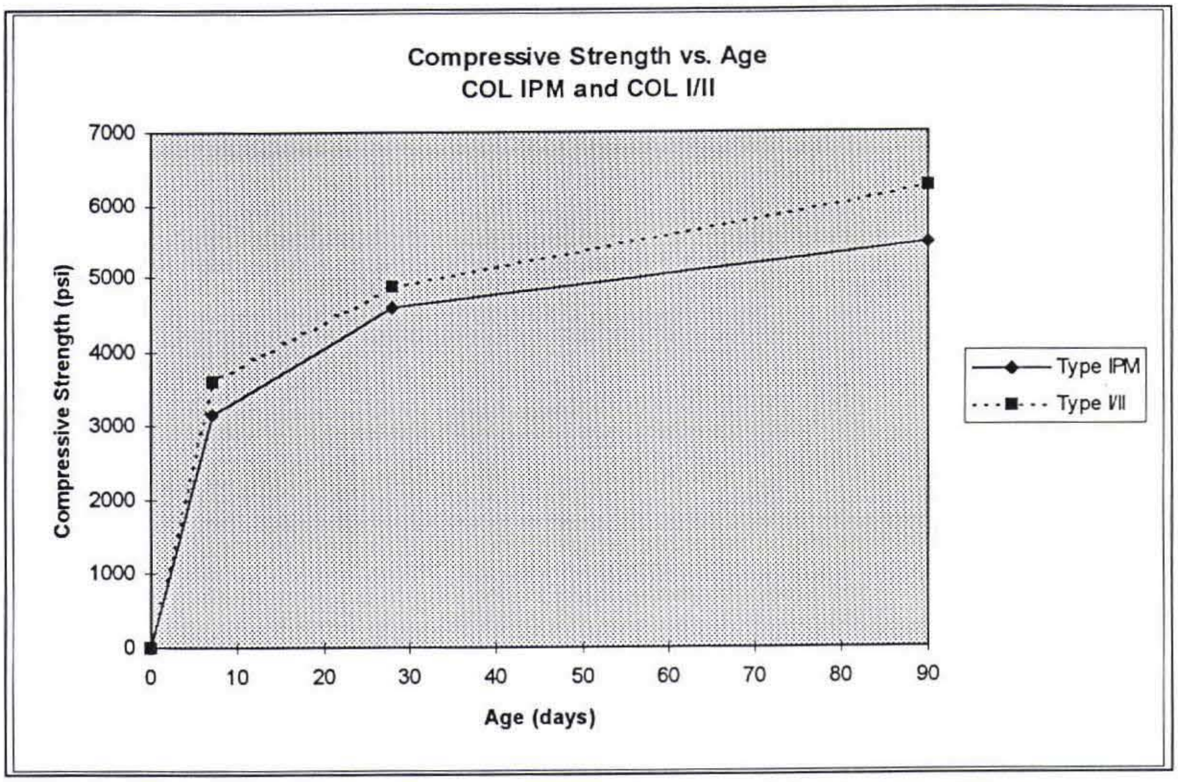

Figure 4.1.7 - Compressive Strength vs. Age COL IPM and I/II

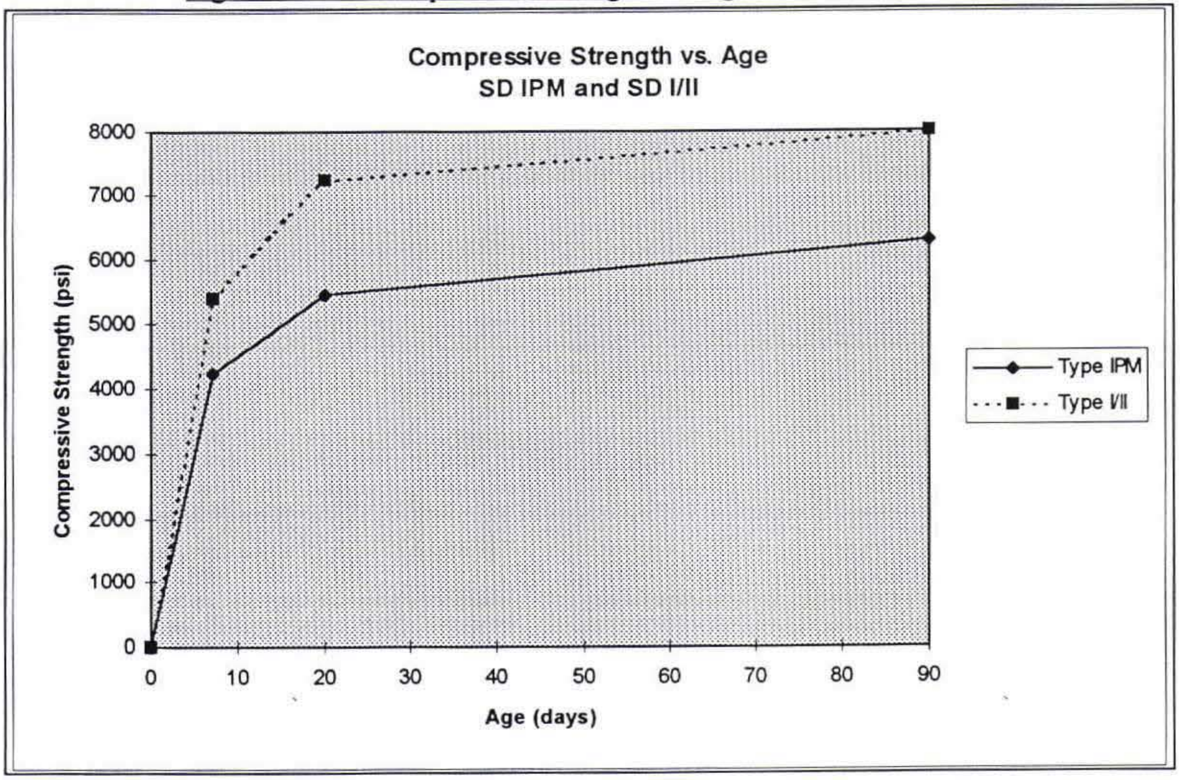

Figure 4.1.8 - Compressive Strength vs. Age SD IPM and I/II

Table 4.1.1 - Static Modulus of Elasticity Results

STATIC MODULUS OF ELASTICITY RESULTS			
	Average Modulus of Elasticity (psi)		
Mix ID	7 Day	28 Day	90 Day
WY1PMA	3.79E+06		
WY12B	3.54E+06	3.80E+06	4.31E+06
SD1PMB	5.18E+06	5.88E+06	
SD12B	5.92E+06	6.10E+06	6.42E+06
COL1PM	3.20E+06	3.44E+06	4.09E+06
COL12	3.22E+06	3.99E+06	4.52E+06
NE1PM	4.53E+06	4.58E+06	5.13E+06
NE12	5.14E+06	5.45E+06	6.10E+06
MNUQ1PM	3.78E+06	4.15E+06	4.71E+06
MNUQ12	4.16E+06	4.84E+06	4.87E+06
MNCA1PM	2.95E+06	3.03E+06	3.78E+06
MNCA12	3.20E+06	3.49E+06	4.24E+06
MWNU1PM	4.69E+06	4.95E+06	5.07E+06
MWNU12	5.06E+06	5.30E+06	5.90E+06
MML1PM	4.61E+06	5.39E+06	5.93E+06
MML12	4.84E+06	5.83E+06	5.86E+06

4.2 Durability

Durability testing was performed on concrete made with South Dakota aggregates only. Sulfate durability (ASTM C1012), freeze/thaw durability (ASTM C666, type A), and deicer scaling durability (ASTM C672) tests were performed according to their respective standards and specifications.

4.2.1 Sulfate Durability

Sulfate durability of concretes made from Type IPM and Type I/II cements using Opperman sand was determined according to ASTM C-1012. (9) The use of this reactive sand is a deviation from the ASTM standard of using the inert, Ottawa sand. For this test, six mortar bars (1" x 1" x 10") were cast for mortars made from each cement according to ASTM C-440, and cured in a warm water bath. Cubes were also cast and tested to determine when strengths of the mortar bars were greater than the specified 2850 psi. After this limit was reached, the bars were measured for initial length using the length change comparator and placed in a sulfate solution made by dissolving 40g of anhydrous sodium sulfate (Na_2SO_4) per 1000 ml of distilled water. The total amount of solution used was approximately 3000 ml, enough to fill the container used for storing the specimens. Specimen length was then determined at 1, 2, 3, 4, 8, 13, and 15 weeks. The data sheet including the length change comparator readings is located in Appendix D. These results are plotted in Figure 4.2.1.

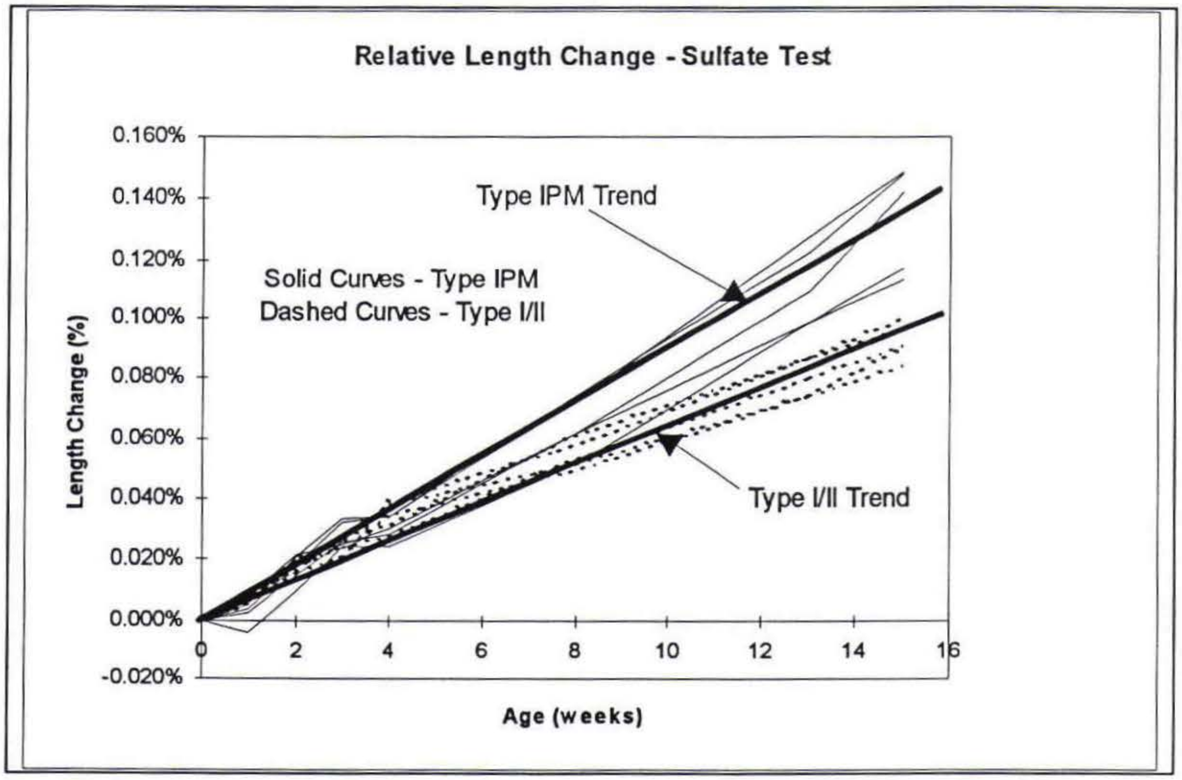

Figure 4.2.1 - Sulfate Durability Results – Reactive Sand

Further study of the mortar bars from this test using a Scanning Electron Microscope (SEM) produced the photographs in Figure 4.2.2 through Figure 4.2.5. Table 4.2.1 contains the chemical analysis obtained from the SEM, along with the chemical composition of ettringite that should be found in the sample.

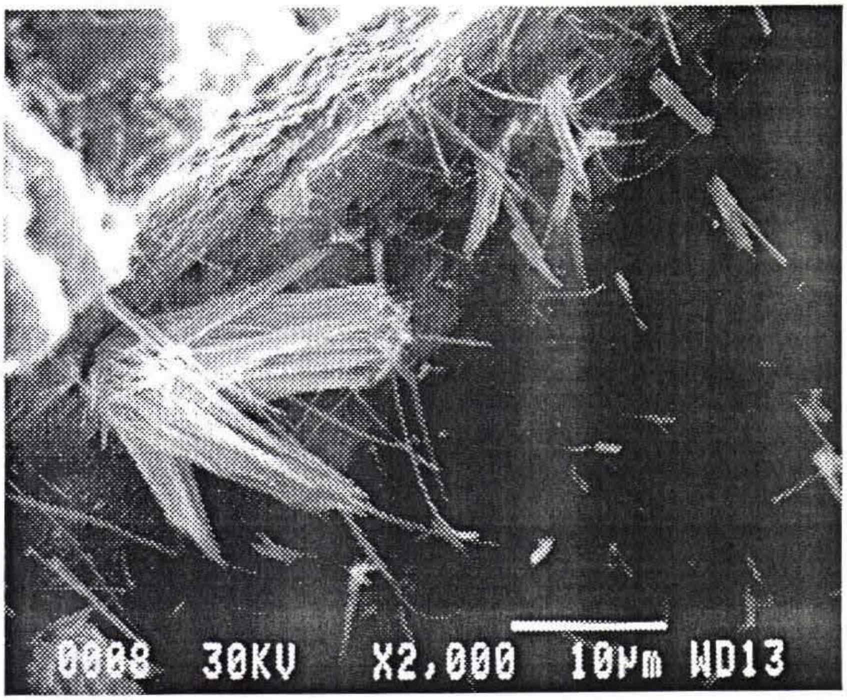

Figure 4.2.2 - SEM Photograph of Type I/II Mortar Bar (2000x)

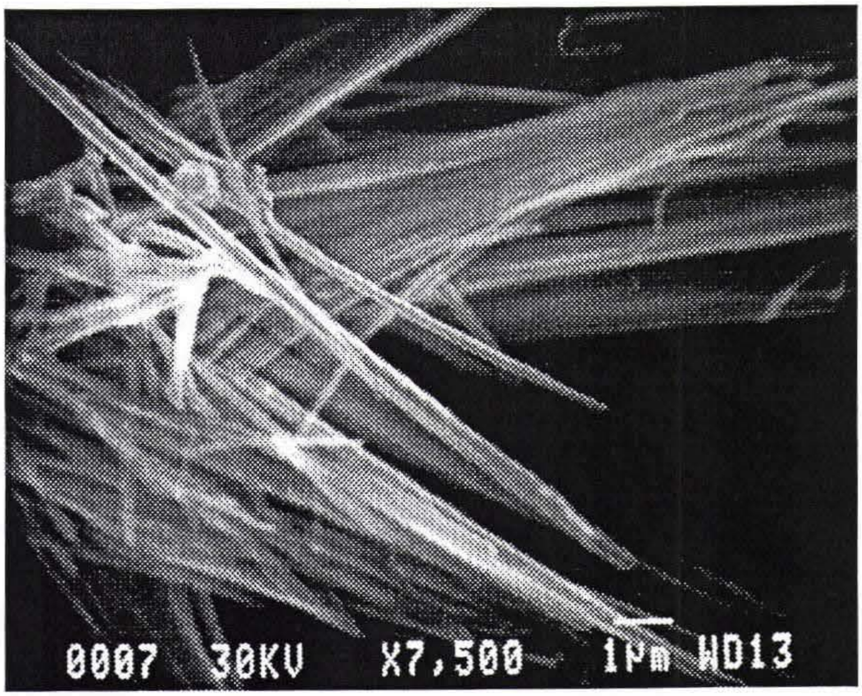

Figure 4.2.3 - SEM Photograph of Type I/II Mortar Bar (7500x)

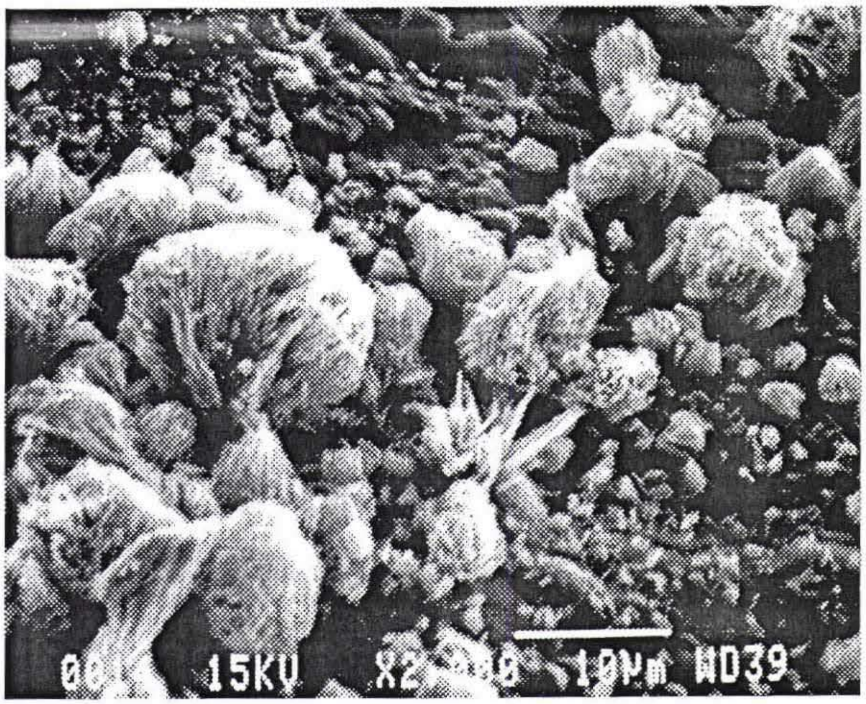

Figure 4.2.4 - SEM Photograph of Type IPM Mortar Bar (2000x)

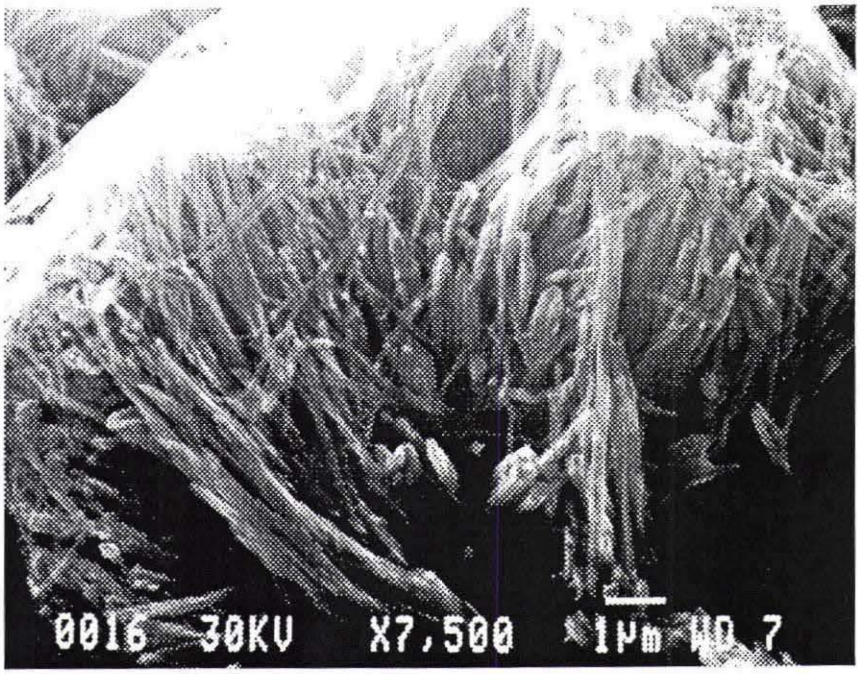

Figure 4.2.5 - SEM Photograph of Type IPM Mortar Bar (7500x)

Table 4.2.1 - Chemical Analysis of SEM Sample

ELEMENT	ATOM%	ETTRINGITE%	WEIGHT%	FORMULA	COMPOUND%
Ca	20.15%	20.15%	34.58%	CaO	48.38%
Al	6.00%	3.36%	6.93%	Al_2O_3	13.09%
S	9.50%	10.08%	13.04%	SO_3	32.55%
Si	1.36%		1.63%	SiO_2	3.49%
Fe	0.13%		0.31%	FeO	0.40%
Mg	0.11%		0.11%	MgO	0.19%
Na	0.45%		0.45%	Na_2O	0.60%
P	0.43%		0.57%	P_2O_5	1.30%
K	0.00%		0.00%	K_2O	0.00%
O	61.88%		42.39%	-----	-----

The sulfate durability test was repeated using inert sand and the results are shown in Figure 4.2.1. The expansion of the mortar made with Type IPM was slightly higher than the expansion of the mortar made with Type I/II cement, but both were within acceptable limits.

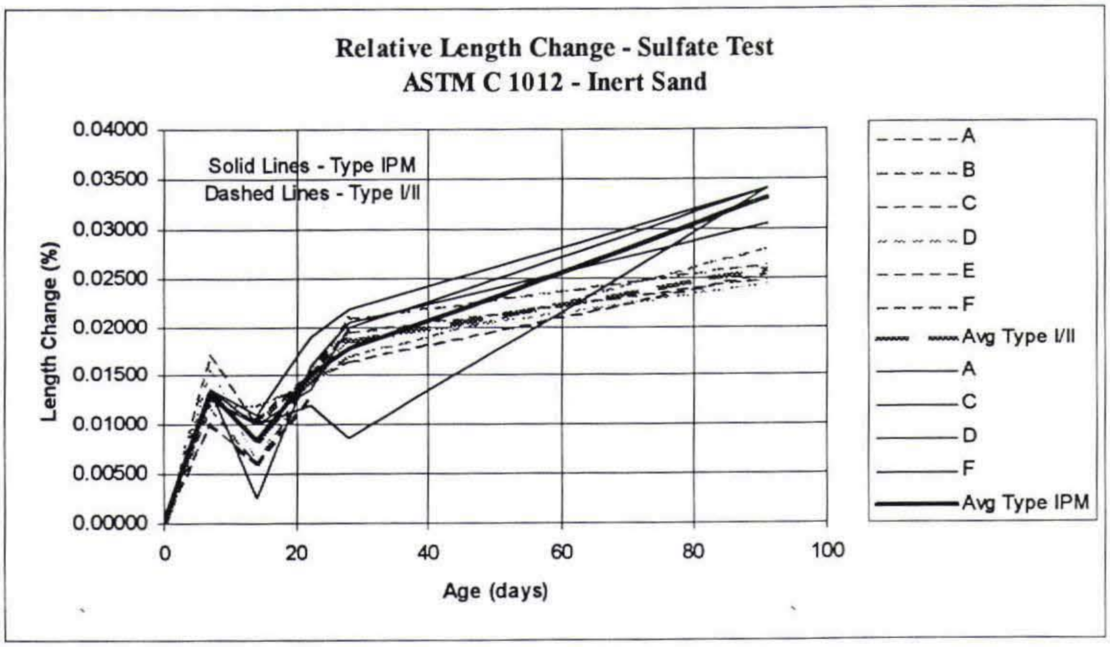

Figure 4.2.1 – Sulfate Durability Results – Inert Sand

4.2.2 Freeze/Thaw Durability

Freeze/thaw durability for concrete made from Type IPM and Type I/II and South Dakota aggregates was determined according to ASTM C-666, type A method. (10) For this test, beams (4" x 4" x 16") were cast according to ASTM C-666 and cured in saturated lime water for 14 days. After 14 days the beams were labeled the initial dynamic modulus was determined using resonant frequency methods. The beams were then placed in the freeze/thaw cabinet, and subjected to cycles of freezing and thawing according to ASTM C-666. The cycle rate was approximately 20 cycles every 72 hours. Cycles were counted using a data recorder. At the end each 20 consecutive cycles, the beams were again tested for resonant frequency, and returned to the cabinet. Testing was stopped at 300 cycles. The data sheet used to record the dynamic modulus test results is located in Reference 34. Results for each set of beams (Type IPM and Type I/II) are plotted in Figure 4.2.6 and Figure 4.2.7.

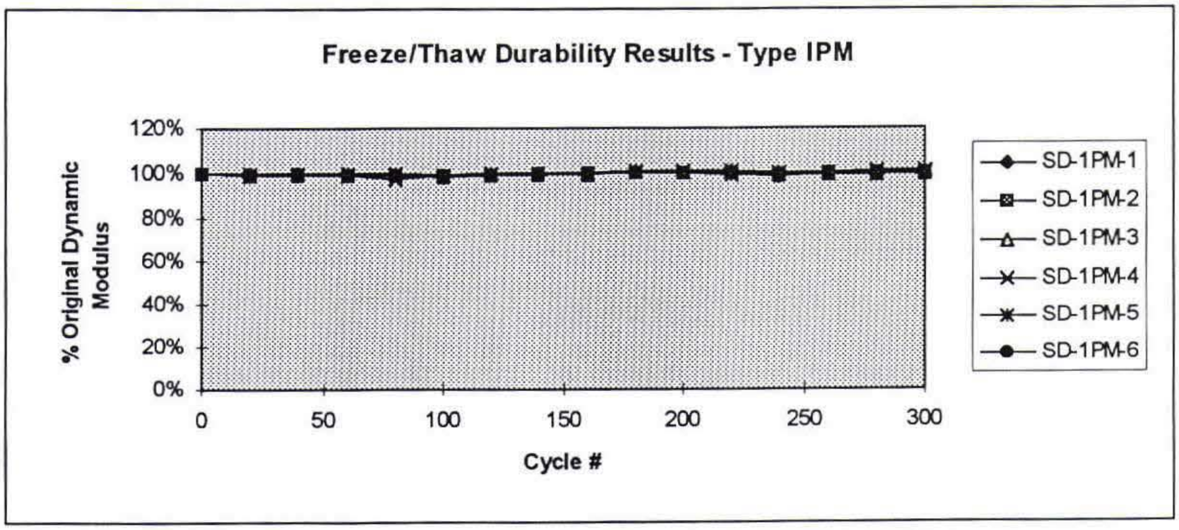

Figure 4.2.6 - Freeze/Thaw Durability Results - Type IPM

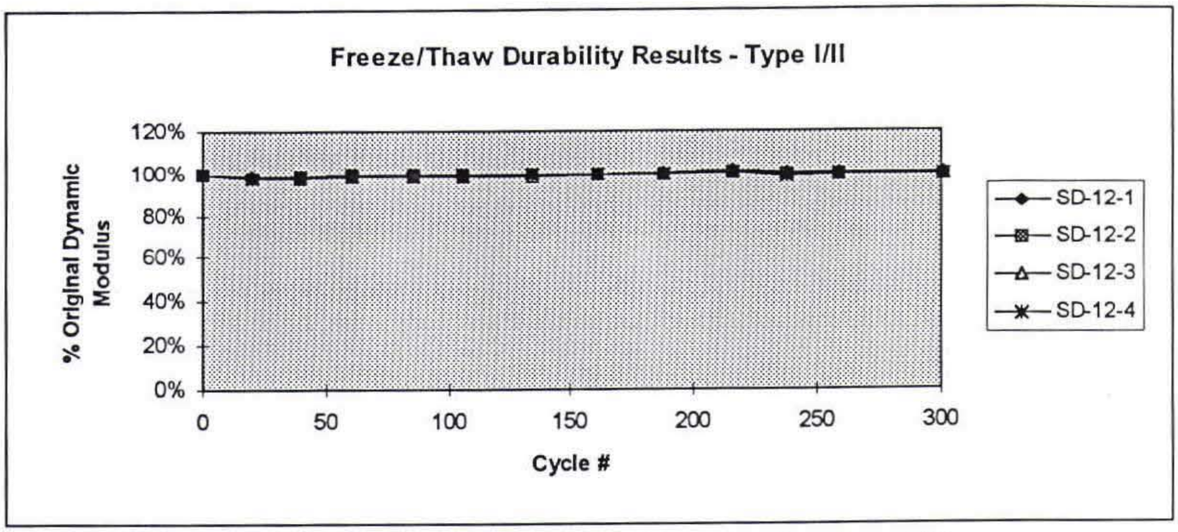

Figure 4.2.7 - Freeze/Thaw Durability Results - Type I/II

4.2.3 Deicer Scaling Durability

Deicer scaling durability of concretes made from Type IPM and Type I/II cement and South Dakota aggregates was determined according to ASTM C-672. (12) Results of the deicer scaling test are plotted in Figure 4.2.9. The data sheet used to record the progression of the deicer scaling test is located in Reference 34.

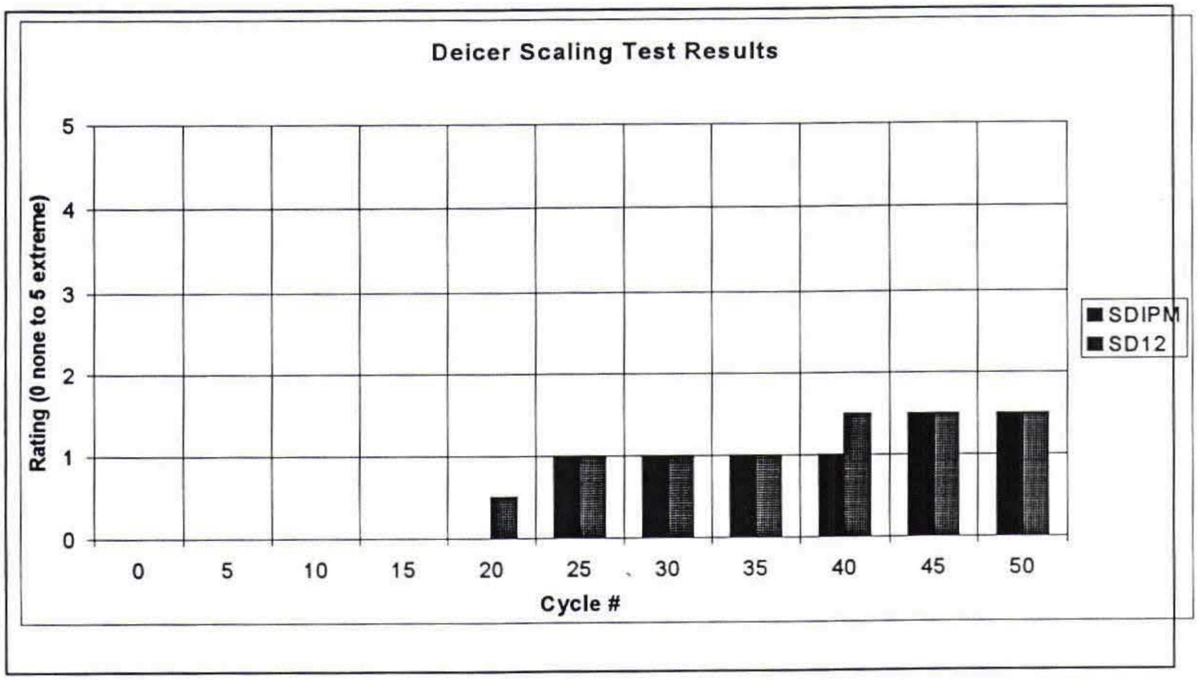

Figure 4.2.8 - Deicer Scaling Test Results - South Dakota Aggregates

5 - FIELD TESTING

Field testing of concrete made with Type IPM cement was performed as part of this research project. Two test sections were placed, one in I-90 near Chamberlain, SD, and one in I-25 near Cheyenne, WY. Discussion of these field tests are included in the following.

5.1 Chamberlain, South Dakota

On Aug. 30, 1996, field tests were performed at mile marker 260 on the eastbound lane of I-90 west of Chamberlain, SD. The initial mixing operations were observed and the operator's reactions to the Type IPM cement were recorded. Air content and slump were measured at the plant by SDDOT personnel and are shown in Reference 34. Site conditions are also recorded on the data sheet located in Reference 34.

Air content and slump were again taken at the job site. The slump dropped one-quarter inch, and the air remained steady. These tests were done using SDDOT equipment. Samples were taken from the fifth truck to allow time for setup and site preparations. Additional air content and slump tests were conducted on subsequent trucks. These results are found on the data sheet in Reference 34. The air content measured by the SDSM&T air meter, calibrated in Rapid City, SD, was slightly lower than that of the SDDOT air meter. This small deviation is attributed to the elevation difference between Rapid City and Chamberlain.

Specimens were cast from the fifth truck. Two separate samples were taken from the truck, carried in two different wheel barrows. One of the samples was accidentally dumped from its container and replaced. This sample was used to mold flexure specimens. The other sample was used to take slump and air content samples, and to mold compressive strength specimens. Before all the specimens were cast, approximately 45 minutes had passed, and the concrete was starting to get stiff. Specimens were covered with plastic overnight, removed from the site the next day and taken to the SDSM&T campus, demolded and placed in lime water.

A field survey was taken of finishers, foreman, and other persons involved in this project. A copy of this survey is located in Reference 34. The overall impression was "that the concrete was very good, had good finishability, and acted similar to other quality concretes." Strength testing results for this test section are included in Table 5.1, and plotted in Figure 5.1. The single point at 193 days of age is the average of two cores submitted by SDDOT.

Table 5.1 - Strength Testing Results - Chamberlain, South Dakota

	Compressive Strength			
	Individual Strengths		Average Strengths	
Age	(psi)	(MPa)	(psi)	(MPa)
7 day (#1)	3750	25.9		
7 day (#2)	3440	23.7	3590	24.8
28 day (#1)	3830	26.4		
28 day (#2)	4240	29.2	4030	27.8
90 day (#1)	5530	38.1		
90 day (#2)	5270	36.3	5400	37.2
190 day (core)	3810	26.3		
190 day (core)	2800	19.3	3300	22.8
	Flexural Strength			
	Individual Strengths		Average Strengths	
Age	(psi)	(MPa)	(psi)	(MPa)
7 day (#1)	425	2.9		
7 day (#2)	425	2.9	425	2.9
28 day (#1)	550	3.8		
28 day (#2)	495	3.4	525	3.6
90 day (#1)	580	4.0		
90 day (#2)	635	4.4	605	4.2

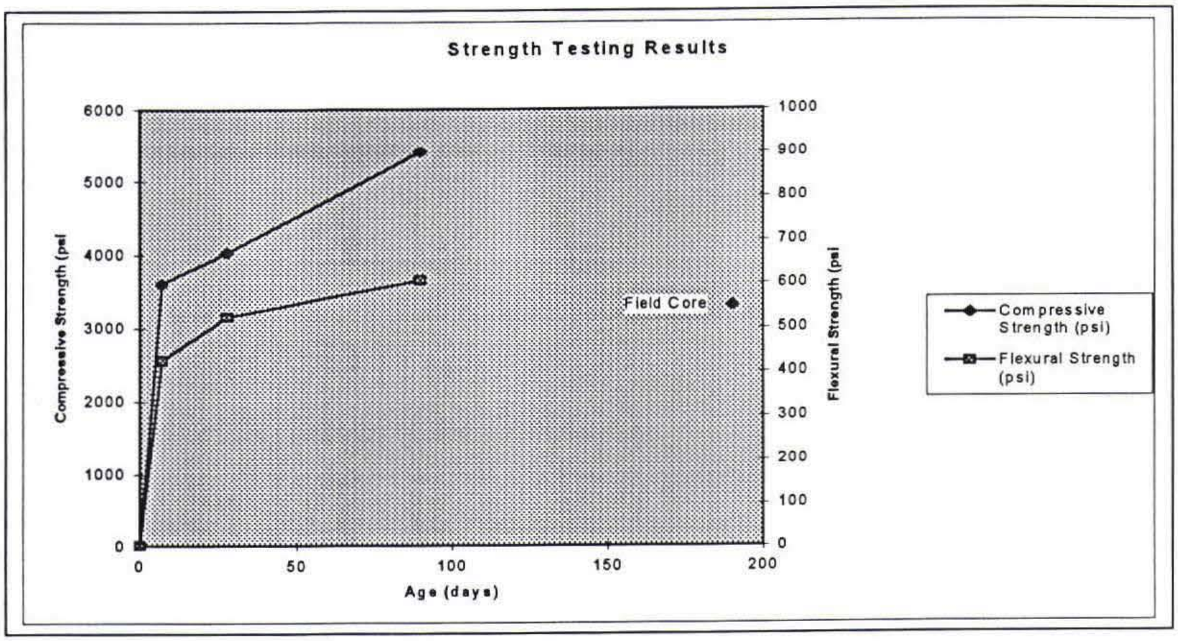

Figure 5.1 - Strength Testing Results - Chamberlain, South Dakota

5.2 Cheyenne, WY

On September 5, 1996, a test placement of the Type IPM cement in an interstate highway patching operation on I-25 south of Cheyenne (mile marker 7.4), was monitored. This placement is part of a larger project being conducted by Wyoming Department of Transportation (WYDOT) to find an appropriate ASR mitigation application in Wyoming. The cement had been shipped in bags and was to be hand loaded into the trucks. Each truck was loaded, by hand, with 51 bags of Type IPM cement, along with aggregates and water from the plant. Site conditions are shown on the data sheet in Reference 34.

A WYDOT representative performed slump and air content tests. All data from the tests are included in the data sheet in Reference 34. Samples were taken from second truck, using two wheelbarrows, sampling from the middle of the load. Cylinders and beams were cast and placed under plastic for protection. The molding process was completed in 30 minutes. The specimens were removed the next morning for transport back to the SDSM&T campus where they were demolded and placed in lime water.

Again, a field survey was conducted of those involved in the project with similar reactions to those found in Chamberlain, SD. Strength testing results for this test section are included in Table 5.2, and plotted in Figure 5.2. The single point at 94 days is the compressive strength test result for a single field core from the WYDOT.

Table 5.2 - Strength Testing Results - Cheyenne, Wyoming

	Compressive Strength			
	Individual Strengths		Average Strengths	
Age	(psi)	(MPa)	(psi)	(MPa)
8 day (#1)	5320	36.7		
8 day (#2)	5300	36.5	5310	36.6
28 day (#1)	6520	45.0		
28 day (#2)	6490	44.7	6510	44.9
90 day (#1)	7530	51.9		
90 day (#2)	7720	53.2	7620	52.5
Core (90 day)	6620	45.6		
	Flexural Strength			
	Individual Strengths		Average Strengths	
Age	(psi)	(MPa)	(psi)	(MPa)
8 day (#1)	595	4.1		
8 day (#2)	545	3.8	570	3.9
28 day (#1)	660	4.6		
28 day (#2)	600	4.1	630	4.3
90 day (#1)	720	5.0		
90 day (#2)	705	4.9	715	4.9

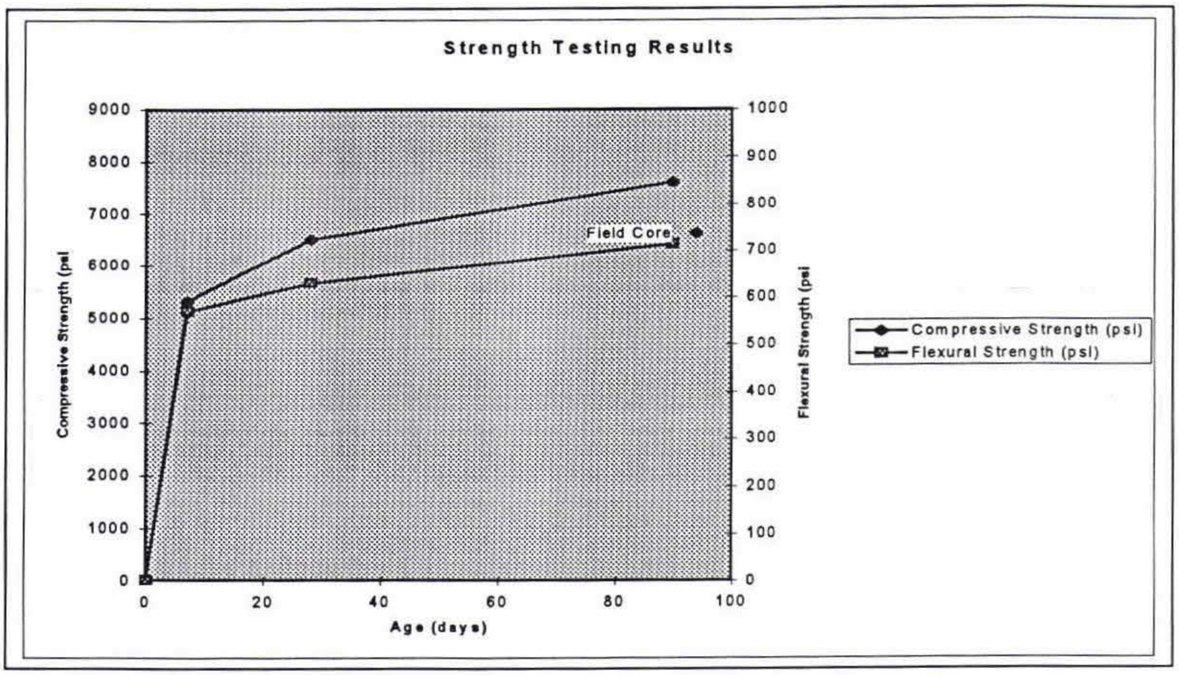

Figure 5.2 - Strength Testing Results - Cheyenne, Wyoming

6 - DISCUSSION OF LABORATORY RESULTS

Analysis and discussion of laboratory testing data presented in the previous sections is included in the following.

6.1 Time of Set

The time of set for both Type I/II and Type IPM cements was determined according to ASTM C403. (21) A modification was used in this test in that only mortar was mixed, instead of sieving mortar from an actual concrete mixture. This was the suggestion of a SDDOT Research Engineer (29) The mortar consisted of Oral sand (2.75 parts), cement (1 part), and a water-to-cementitious ratio of 0.40. Results of this test are shown in Table 6.1. The reported time of set values on Dacotah Cement's mill reports show that the Type IPM cement was slightly longer to reach the Vicat setting time. As shown using the penetration resistance test, the Type IPM had a slightly faster setting time. These tests are completely different, and do not measure the same property, so no direct comparison is present. Type IPM may set sooner because it is ground finer than the Type I/II.

Table 6.1 - Time of Set by Penetration Resistance

Time of Set - ASTM C403

	Initial Set	Final Set
Type IPM	3:25	5:00
Type I/II	4:00	6:00

6.2 Strength Testing Discussion

Strength testing data presented in Section 4.1 show a very distinct trend. Results show that concrete mixtures made with Type IPM cement have approximately 10% to 12% less compressive and flexural strength at all ages, as compared with Type I/II. This trend is evident in all mixtures with two exceptions. The data for laboratory mixtures using Wyoming aggregates show that the Type IPM strengths are actually higher than the Type I/II. Also, data for the Minnesota - Wallner New Ulm mixtures showed that the Type IPM cement mixture gained strength and actually surpassed the Type I/II somewhere between 28 and 90 days. There is a possible reason for these anomalies. The aggregates used in these mixtures are prone to aggressive ASR deterioration, especially the Wyoming aggregates. In these mixtures, ASR deterioration may be great enough to reduce the strength of the Type I/II mixtures within the test duration while, in the Type IPM mixtures, the ASR deterioration was mitigated.

Excluding these anomalies, the general trend is now analyzed. The exact reason for the trend of lower strengths is not possible to define exactly, but there are a few factors that may influence the strength gain of the Type IPM mixtures. First, the age of the Type IPM cement used in testing was from nine months to one year. Cement will lose strength potential over time as the cement particles will partially hydrate, permanently losing strength when exposed to humidity. (23) Additionally, the intergrinding of 10% to 15% of the volcanic tuff pozzolan reduces the amount of cement present per cubic yard, since the cement was used as a direct one to one replacement for Type I/II in the mixture proportioning. This changes the water-to-cement ratio, while the water-to-cementitious ratio stays the same, thus providing another possible explanation for the reduction in strength assuming that the pozzolan has limited cementitious properties.

The ratio between flexural strength and compressive strength for the laboratory data is shown in Table 6.2. As shown in the table, on average, the flexural strength of the concrete specimens tested was approximately equal to 12% of the compressive strength. Large amounts of variation are present in flexural strength testing, thus 2 to 3% either side of 12% is an acceptable error in using this estimate. Typically, this ratio is about 10%. Any difference in this typical value and the reported value could come from the aggregates used, variation in testing methods, or other statistical phenomenon.

Table 6.2 - Flexure/Compression Ratio

Flexure/Compression Ratio		
Mix ID	Type IPM	Type I/II
MNCA	14.7%	13.8%
MNUQ	14.0%	11.4%
WY	11.2%	11.4%
WNU	13.2%	10.3%
ML	11.7%	11.2%
NE	12.5%	11.6%
COL	12.0%	9.9%
SD	8.1%	11.5%

6.3 Sulfate Durability Discussion

As shown in Section 4.2.1, the expansion of mortar bars made from Type IPM cement was approximately 50% higher than the expansion of the bars made from Type I/II cement. It is hypothesized that the cause of this higher expansion might be from the combined effects of ASR and sulfate attack. But, logically, if Type IPM cement reduces or eliminates the effects of ASR as expected, then the opposite trend of lower expansion for the Type IPM bars would be expected. Thus, there must be some other explanations for this phenomenon. Possible explanations may include a higher tricalcium aluminate content in the Type IPM cement, which as discussed in the literature review, will tend to aggravate sulfate attack. Also, as discussed in the literature review, in sulfate rich environments (>10,000 ppm sulfates), it is recommended that an extra bag of cement along with addition of a pozzolan be used in the mixture proportioning to mitigate the effects of sulfate attack. In this test, the sulfate concentration was 40,000 ppm. The Type IPM cement is produced by replacing approximately 12% of the parent cement (Type I/II) with a natural pozzolan. According to the literature (3), for this case more cement should be present in the mixture. Since there is not enough cement in the mixture, higher expansions due to sulfate attack can be expected, which with the added expansion of ASR, even if only small, will compound the problem.

Scanning electron microscopy analysis was done on a specimen taken from the Type IPM and Type I/II mortar bars. Fewer ettringite bundles (shown in Figure 4.2.2) were found in the Type I/II bars compared with Type IPM mortar bars. This shows that ettringite formation was higher in the Type IPM bars, which corresponds with the higher expansion observed in the length change measurements. Also, the size and development of the ettringite bundles seemed to be smaller in the Type I/II specimen, as can be seen by comparing Figure 4.2.3 with Figure 4.2.5. Chemical analysis of the bundles showed that they actually contained ettringite. Based on the chemical analysis shown in Table 4.2.1, the crystal bundles in Figures 4.2.2 through 4.2.5 are filled with ettringite. ETTRINGITE% in the table is a ratio with respect to the measured calcium based on the following chemical formula for ettringite: (11)

$$\text{Ettringite} = 6CaO \bullet Al_2O_3 \bullet 3SO_3 \bullet 32H_2O$$

The variation between the ATOM% and the ETTRINGITE% may be attributed to experimental error which comes from background interference and accuracy of testing equipment. (30) Both samples (Type I/II and Type IPM) showed similar chemical compositions, according to the SEM spectrum analysis.

6.4 Freeze/Thaw Durability Discussion

Analysis of freeze/thaw durability test data as presented in Section 4.2.2 shows that no deterioration of either concrete specimens made with Type IPM or Type I/II cement occurred after 300 cycles. As discussed earlier, air entrainment is essential to freeze/thaw durability. The mixtures used for this test, made from South Dakota aggregates, yielded fresh concretes with air contents of 6% and 3.4% for Type IPM and Type I/II, respectively. Commonly, 4 to 6% air entrainment is suggested for durable concrete. The Type IPM mixture has this suggested level of air entrainment, but the Type I/II mixture has a slightly lower air content than suggested. Durability is also affected by the strength of the concrete. The resulting Type I/II mixture was tested to have compressive strengths near 8,000 psi at 90 days. This is considered a high strength concrete. As discussed in the literature review, concretes with high strengths may not need as much air entrainment to be freeze/thaw durable.

6.5 Deicer Scaling Durability Discussion

Analysis of deicer scaling durability testing data as presented in Section 4.2.3 shows that little or no scaling and spalling occurred on any test samples for the duration of the test. A rating scale is used to classify the degree of deterioration in deicer scaling tests. (12) On this scale, a rating of 1 is defined to have “very slight scaling (1/8” depth, max, no coarse aggregate visible.” A rating of 2 is “slight to moderate scaling.” Specimens tested for deicer scaling durability had a rating somewhere between 1 and 2 at the end of testing. Both sets of specimens (Type IPM and Type I/II) showed equivalent amounts of deterioration over the duration of the test. The relative quality of this concrete in terms of durability depends on design specifications and regional conditions. For means of this research, it has been determined that concrete made from Type IPM cement is at least as durable when exposed to deicing solutions as concrete made from Type I/II cement.

6.6 Field Testing Discussion

On Figure 5.1, notice that the cores submitted by the SDDOT had significantly lower strength than 90 day specimens stored in the lab. Several factors play a role in this reduction in strength. The cores were cut from an in-service structure, thus some strength loss due to fatigue can be expected, but not as much as shown in the results (approx. 40% strength loss). Additionally, the section the cores were taken from was subjected to freezing and thawing, which could add to the strength reduction. Finally, the slab was not laboratory cured which could account for additional deviation from the laboratory samples. As in the Chamberlain field test, the single core cut from the test section in Wyoming, was tested to have lower strength than the laboratory samples, but not as severe.

7 - CONCLUSIONS AND RECOMMENDATIONS

Conclusions:

1. Both compressive and flexural strength for concrete made using Type IPM cement is lower than standard Type I/II mixtures. On average, strengths were 10-12% lower in Type IPM mixtures than Type I/II mixtures.
2. Freeze/thaw durability tests showed excellent performance for both Type IPM and Type I/II mixtures, with no deterioration evident after 300 freeze/thaw cycles.
3. Deicer scaling durability tests showed good performance for both Type IPM and Type I/II mixtures, with only minimal scaling of the surfaces after the test duration.
4. Results from cores taken from field trials showed a much lower strength over time as compared with lab cured specimens cast at the same time as placement of the trial slabs.
5. Ettringite formation in mortar bars, using reactive sand, made from Type IPM cement was more advanced than mortar bars made from Type I/II as shown through scanning electron microscopy.
6. Expansion of mortar in the sulfate test (ASTM C 1012), made with inert sand, was slightly higher for type IPM cement than for Type I/II cement, but both were within acceptable limits.

Recommendations:

1. Investigate high expansions in Type IPM cement as reported on cement test report.
2. Compare test results from SDDOT and CTL with results presented herein.
3. Perform additional field testing of concrete made with Type IPM cement.
4. Perform Rapid Chloride Permeability testing to determine relative permeability of Type IPM mixtures as compared with Type I/II mixtures.
5. Perform additional research into minor modification of current Type IPM cement formulation to improve strength characteristics and address other areas of concern.
6. Perform ASTM C 1293 (Canadian Prism Test) on concrete made with Type IPM and Type I/II cement, to check for ASR.

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APPENDIX

AGGREGATE MINERALOGY AND CHARACTERIZATION

Aggregate Summary Data

			< 0.371 inch sieve	>0.371 inch sieve		
	Lab no.	Sample name	Weight Sand (grams)	Weight Gravel (grams)	wt% sand	wt% gravel
1	967.0376	Nebraska Sand	292.9	11.2	96.3	3.7
2	967.0377	Johnson Lesueur	na	na	100.0	0.0
3	967.0378	New Ulm Quartzite	78.6	142.0	35.6	64.4
4	967.0379	Meeker Sand	na	na	100.0	0.0
5	967.0380	North Con. Agg.	92.4	227.7	28.9	71.1
6	967.0381	Wallner New Ulm	na	na	100.0	0.0
7	967.0382	Colorado Fine	226.8	18.4	92.5	7.5
8	967.0383	Colorado Coarse	na	na	0.0	100.0
9	967.0384	North Star Kasota	na	na	100.0	0.0
10	967.0385	Wyoming Sand	na	na	100.0	0.0
11	967.0386	Wyoming Gravel	11.9	880.9	1.3	98.7
12	967.0387	SD Sand	na	na	100.0	0.0
13	967.0388	SD Quartzite	115.8	631.8	15.5	84.5
					Visual Estimates	
		Sample name	Weight Gravel (grams)			
		Nebraska Sand	11.2			
		Johnson Lesueur	na			
		New Ulm Quartzite	142.0			
		Meeker Sand	na			
		North Con. Agg.	227.7			
		Wallner New Ulm	na			
		Colorado Fine	18.4			
		Colorado Coarse	na			
		North Star Kasota	na			
		Wyoming Sand	na			
		Wyoming Gravel	880.9			
		SD Sand	na			
		SD Quartzite	631.8			

Percent by Volume

			Percent by Volume for Each Rock or Mineral Constituent - Optical Microscopy																	
		Aggregate Type	**monocrystalline quartz**	**polycrystalline quartz and quartzite**	**granite and granite gneiss**	**microcline**	**plagioclase**	**limestone and dolostone**	**chert and chalcedony**	**gabbro and amphibolite**	**silicic volcanic (rhyolite)**	**graywacke**	**phosphatic or opalline sedimentary rock**	**biotite**	**amphibole (hornblende)**	**opaque Fe-oxide, hematite and hematitic sandstone**	**epidote-rich rock**	**other metamorphic**		
1	**967.038**	**Nebraska Sand**	28.0	12.7	30.6	9.8	9.8	0.7	2.1	1.1	2.4	0.2	0.0	0.0	0.0	1.5	0.0	0.5	100.0	
2	**967.038**	**Johnson Lesueur**	20.7	4.9	27.0	2.0	8.5	21.6	1.8	0.9	1.1	4.3	0.7	0.0	0.0	3.1	0.2	2.9	100.0	
3	**967.038**	**New Ulm Quartzite**	83.6	0.0	0.0	0.6	0.0	0.3	12.5	0.0	0.0	0.0	0.0	0.0	0.0	1.7	0.0	0.0	100.0	(percent of materials in quartzite)
		New Ulm Quartzite	0.0	99.7	0.0	0.0	0.0	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	100.0	(percent quartzite)
4	**967.038**	**Meeker Sand**	29.5	2.4	21.9	2.0	9.8	21.0	1.7	3.3	0.9	3.3	0.9	0.0	0.4	1.3	0.2	0.2	100.0	
5	**967.0380**	**North Con. Agg.**	2.7	0.2	21.3	1.1	0.2	65.2	1.5	4.8	0.0	2.3	0.0	0.2	0.0	0.0	0.6	0.0	100.0	
6	**967.038**	**Wallner New Ulm**	35.7	3.2	21.4	3.4	10.7	16.3	1.8	1.4	0.0	1.8	2.0	0.0	0.0	1.6	0.6	0.0	100.0	
7	**967.038**	**Colorado Fine**	44.0	6.0	23.0	12.0	12.0	0.0	1.0	0.0	0.0	0.4	0.0	0.2	0.2	0.8	0.0	0.0	100.0	
8	**967.038**	**Colorado Coarse**	8.8	0.4	64.0	1.0	6.4	0.0	0.0	6.8	0.0	0.0	0.0	0.0	3.4	0.4	8.8	0.0	100.0	
9	**967.038**	**North Star Kasota**	47.2	2.3	12.8	3.1	9.0	12.2	2.0	0.9	3.5	0.8	1.5	0.7	0.0	2.4	0.0	0.7	100.0	
10	**967.039**	**Wyoming Sand**	27.6	5.8	38.9	9.1	12.9	0.6	0.0	0.0	1.2	0.0	0.0	0.0	0.6	0.8	0.4	1.5	100.0	
11	**967.039**	**Wyoming Gravel**	13.8	2.0	51.9	6.1	11.4	3.5	0.3	3.2	0.0	0.0	0.0	1.2	3.0	0.5	2.2	0.0	100.0	
12	**967.039**	**SD Sand**	46.5	10.8	19.6	11.2	6.9	0.0	1.0	0.6	3.3	0.0	0.0	0.0	0.0	0.2	0.0	0.0	100.0	
13	**967.039**	**SD Quartzite**	0.0	100.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	100.0	

Aggregate Type	Major Constituents	Percent Reactive
Nebraska Sand	40.7% quartz and 30.6% gneiss	76.7
Johnson Lesueur	27.0% gneiss and 25.6% quartz	81.3
New Ulm Quartzite	83.6% quartz and 12.5% chert	96.4
New Ulm Quartzite	99.7% quartz	100.0
Meeker Sand	31.9% quartz, 21.9% gneiss, and 21.0% limestone	80.7
North Con. Agg.	65.2% limestone and 21.3% gneiss	93.2
Wallner New Ulm	38.9% quartz and 21.4% gneiss	80.2
Colorado Fine	50.0% quartz and 23.0% gneiss	74.4
Colorado Coarse	64.0% gneiss	73.2
North Star Kasota	49.5% quartz, 12.8% gneiss and 12.2% limestone	80.9
Wyoming Sand	38.9% gneiss and 33.4% quartz	74.0
Wyoming Gravel	51.9% gneiss	71.6
SD Sand	57.3% quartz and 19.6% gneiss	81.2
SD Quartzite	100% quartz	100.0

	Percent by Weight for Each Mineral Constituent - X-ray Diffraction									
Aggregate Type	**quartz**	**kaolinite**	**hematite**	**oligoclase**	**calcite**	**microcline**	**muscovite**	**hornblende**	**dolomite**	**chlorite**
Nebraska Sand	32.7	0.0	0.0	20.9	3.2	39.3	2.0	0.0	2.0	0.0
Johnson Lesueur	29.0	2.6	0.0	19.7	14.3	12.6	2.5	1.2	18.1	0.0
New Ulm Quartzite	79.3	19.6	1.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Meeker Sand	24.1	0.0	0.0	14.7	19.6	10.4	2.9	1.3	22.7	4.2
North Con. Agg.	9.1	0.0	0.0	7.8	25.6	5.9	1.7	1.4	42.6	5.9
Wallner New Ulm	23.5	0.0	0.0	15.0	19.5	13.2	1.6	1.6	19.1	6.5
Colorado Fine	42.2	0.0	0.0	19.2	1.0	35.4	2.2	0.0	0.0	0.0
Colorado Coarse	32.0	2.0	0.0	37.3	0.8	24.3	3.7	0.0	0.0	0.0
North Star Kasota	27.2	2.2	0.0	16.8	17.4	13.4	0.0	0.0	23.0	0.0
Wyoming Sand	32.9	0.0	0.0	27.7	2.6	31.9	4.9	0.0	0.0	0.0
Wyoming Gravel	28.3	0.0	0.0	36.2	1.9	24.8	2.7	6.1	0.0	0.0
SD Sand	46.6	0.0	0.0	19.8	1.3	30.8	1.7	0.0	0.0	0.0
SD Quartzite	97.8	1.8	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0

	Compound Reduction from X-ray Diffraction												
Aggregate Type	SiO2	Al2O3	Na2O	K2O	CaO	FeO	Fe2O3	MgO	H2O	CO2	MnO	TiO2	NiO
Nebraska Sand	71.8	12.8	2.2	6.1	3.2	0.1	0.1	0.5	0.3	2.3	0.0	0.1	0.0
Johnson Lesueur	52.3	8.8	1.9	2.3	14.1	0.8	0.2	3.8	0.6	14.8	0.2	0.1	0.0
New Ulm Quartzite	88.2	7.6	0.0	0.1	0.0	0.0	1.1	0.0	2.9	0.0	0.0	0.1	0.0
Meeker Sand	43.1	7.3	1.4	1.9	18.2	1.5	0.2	5.7	0.8	19.3	0.4	0.1	0.0
North Con. Agg.	20.8	4.8	0.8	1.1	27.2	2.2	0.2	10.1	0.9	32.1	0.6	0.0	0.0
Wallner New Ulm	44.6	7.9	1.5	2.2	17.1	1.7	0.3	5.5	1.0	17.5	0.5	0.1	0.0
Colorado Fine	77.9	11.8	2.0	5.5	1.4	0.1	0.1	0.1	0.3	0.5	0.0	0.0	0.0
Colorado Coarse	73.5	15.1	3.6	4.2	1.9	0.1	0.2	0.1	0.7	0.3	0.0	0.1	0.0
North Star Kasota	47.3	7.2	1.6	2.1	17.0	0.7	0.1	4.6	0.4	18.4	0.3	0.0	0.0
Wyoming Sand	72.9	14.0	2.8	5.4	2.6	0.1	0.2	0.1	0.5	1.2	0.0	0.1	0.0
Wyoming Gravel	71.0	14.3	3.5	4.2	3.2	1.0	0.4	0.7	0.5	0.8	0.0	0.1	0.0
SD Sand	79.4	10.9	2.1	4.8	1.5	0.1	0.1	0.0	0.3	0.5	0.0	0.1	0.0
SD Quartzite	98.6	0.7	0.0	0.0	0.0	0.0	0.4	0.0	0.3	0.0	0.0	0.0	0.0