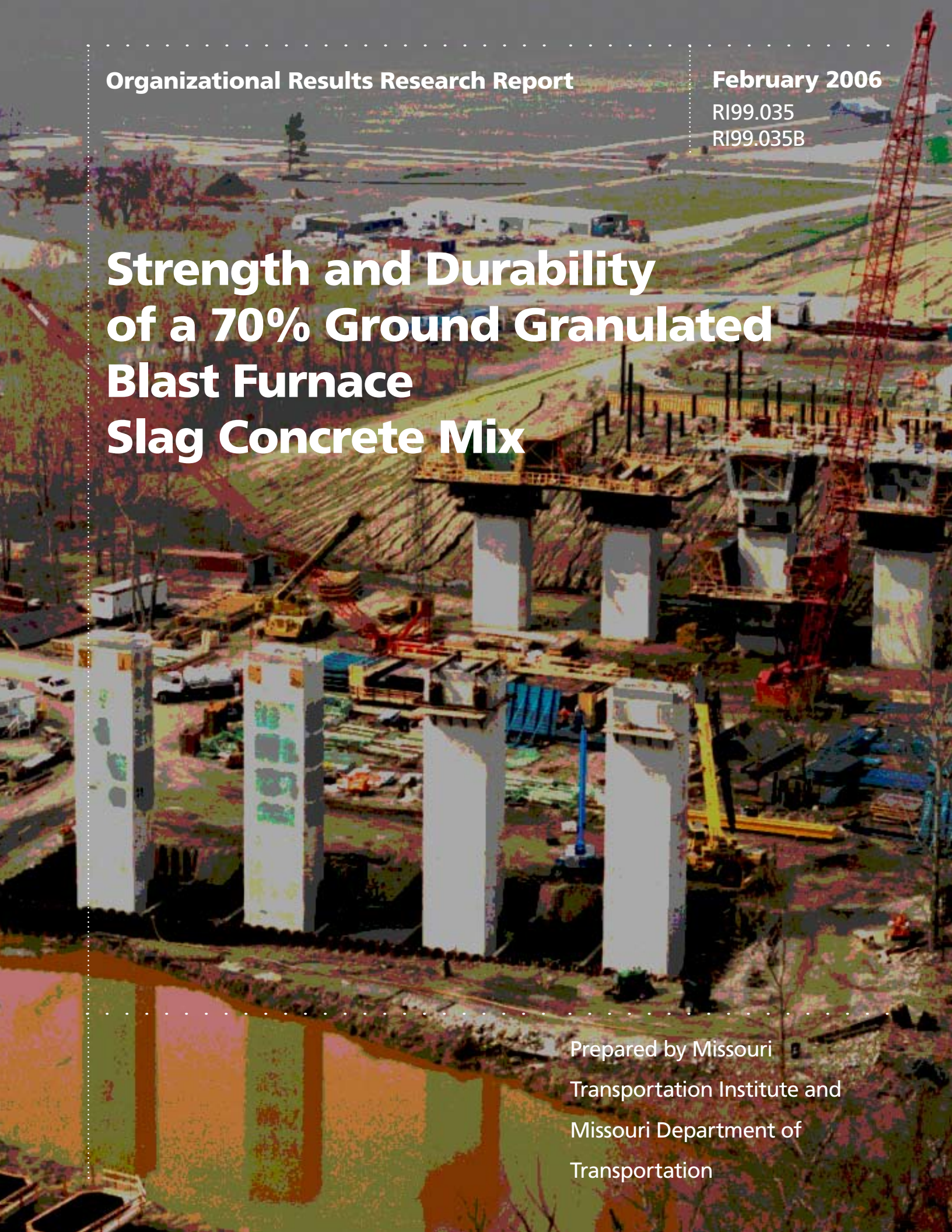

Organizational Results Research Report

February 2006

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Strength and Durability of a 70% Ground Granulated Blast Furnace Slag Concrete Mix

Prepared by Missouri Transportation Institute and Missouri Department of Transportation

FINAL REPORT

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Strength and Durability Characteristics of a 70% Ground Granulated Blast Furnace Slag (GGBFS) Concrete Mix

Prepared for the

Missouri Department of Transportation
Organizational Results

By

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16. Abstract

For a bridge pier and abutment mass concrete project, three mixes were studied: an ordinary portland cement (OPC) mix (Type I PC) and two 70% by weight ground granulated blast furnace slag (GGBFS) mixes (Type II Low Heat PC). One of the slag mixes contained a high range water reducer (HRWR). Tests for compressive strength, freeze-thaw durability, rapid chloride permeability, and salt scaling were conducted on field samples. Results showed that the strengths of the slag mixes were lower than the OPC mix at all ages up to one year, although the use of HRWR did improve the strength somewhat. Freeze-thaw durability and salt scaling resistance of both slag mixes were inferior compared to the OPC control. However, under optimized wet plus dry curing periods, slag mix durability did approach that of the OPC mix. Chloride permeability of both slag mixes was significantly lower than the OPC mix. Analysis of past studies indicates that, although slag replacements up to 80% have been used successfully, in order for slag mixes to reach strength parity with OPC mixes, the optimum replacement of PC by slag usually falls between 40 and 60%, depending on the nature and proportions of the materials. Laboratory mixes were made which reflected variables of age, PC type, w/cm, and total cementitious material content. It was concluded that although the optimum slag proportion for strength was 50%, slag replacement levels of up to 70% could be used to achieve moderate strength levels. Strength parity with zero slag mixes is possible with 70 % slag under proper conditions, which include sufficient activity of the slag-PC system.

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Executive Summary

Large placements of ordinary portland cement produce high temperatures as they cure. This in turn can result in cracks. Controlling curing temperatures is particularly challenging in bridge applications. For years the construction industry has used ground granulated blast furnace slag (GGBFS) as a replacement for ordinary portland concrete cement (OPC) to lower curing temperatures. However, MoDOT specifications only allowed low levels of blast furnace slag in concrete mixes. Higher concentrations warranted further investigation for strength and durability.

GGBFS is a by-product of the iron production process, and consists mostly of calcium silicates and aluminosilicates. This cementitious material has been touted for both its strength and durability enhancing characteristics when used in concrete. Ground granulated blast furnace slag also has a lower heat of hydration and, hence, generates less heat during concrete production and curing. As a result, GGBFS is a desirable material to utilize in mass concrete placements where control of temperature is an issue. Percentage replacements by weight of GGBFS for cement have ranged from 10 to 90%.

Conclusions

- Compressive strengths of the 70% GGBFS-Type II PC field mix at all ages up to one year were about 2000 psi lower than the plain Type I PC mix. The addition of a high range water reducer (HRWR) to the slag-PC mix narrowed the difference to about 1300 psi.
- When using the same PC type for both the control and slag-PC laboratory mixes, all slag-PC mixes had greater strengths than the plain PC mix.
- For slag-PC mixes to obtain strengths equivalent to Type I PC mixes, the data suggests that sufficient activators need to be present to activate the slag.
- Slag proportions of 40 to 60 % appear to be the optimum level for highest strength development.
- High slag content mixes can achieve typical specified strengths under proper circumstances.
- Freeze-thaw durability was lower for the slag-PC field mixes than the plain PC field mix. However, under optimum wet plus dry curing periods, the slag-PC mix Durability Factors approached that of the plain PC mix.
- Rapid chloride permeability test values were significantly lower for the slag-PC field mixes compared to the plain PC mix. Both slag-PC mixes were rated as “low”, while the plain PC mix result was “high.”
- The plain PC field mix had good salt scaling resistance. Both slag-PC field mixes exhibited significantly greater laboratory-induced salt scaling.
- The air void systems of the plain PC and both 70% slag field mixes were similar.

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Introduction

Ground granulated blast furnace slag (GGBFS) has been used in the construction industry for years as a replacement for ordinary portland cement (OPC). GGBFS is a by-product of the iron production process, and consists mostly of calcium silicates and aluminosilicates. This cementitious material has been touted for both its strength and durability enhancing characteristics when used in concrete. Ground granulated blast furnace slag also has a lower heat of hydration and, hence, generates less heat during concrete production and curing. As a result, GGBFS is a desirable material to utilize in mass concrete placements where control of temperatures is an issue. Percentage replacements by weight of GGBFS for cement have ranged from 10 to 90%.

The project reported herein was the substructure construction of the Creve Coeur Memorial Bridge as part of the Page Avenue Extension project in St. Louis County, Missouri. At the time of the project, Section 501 of the Missouri Department of Transportation (MoDOT) standard specifications allowed the use of GGBFS at a maximum of 25% replacement for Type I or II cement. However, because of its low heat generating characteristics, GGBFS was approved for use at a 70% replacement of Type II cement. The determination and evaluation of the strength and durability characteristics of the GGBFS mixes used in the Page Avenue project provide both documentation of such characteristics and information for future incorporation of GGBFS in MoDOT concrete construction.
The goals of the use of GGBFS in the Creve Coeur Bridge were to:

1) Lower the heat-of-hydration in the mass concrete footings and piers to reduce excessive temperature differentials, therefore achieving a reduction in cracking.
2) Achieve levels of ultimate strength, workability, and durability comparable to conventional mixes.

Mix constituent factors that affect the temperature of concrete include the type of cementitious material (chemistry and fineness), amount of cementitious material, presence of admixtures, and initial constituent temperatures. The mixtures chosen to accomplish the stated goals and address the temperature concerns were characterized by:

1) Replacement of 70% PC with GGBFS
2) Use of a moderate amount of cementitious content
3) Use of Type II Low Heat cement
4) Use of Grade 120 GGBFS
5) Use of high range water reducers (HRWR)

Literature Search

General

The general GGBFS literature indicates that the replacement of OPC by GGBFS typically results in lower early strengths (7 to 28 days), greater long term strengths, lower chloride ion permeability, less creep, greater sulfate attack resistance, greater alkali silica reactivity (ASR) durability, enhanced workability, less bleeding, lower heat of hydration, and increased steel corrosion resistance. Results for drying shrinkage and freeze-thaw durability are somewhat mixed, although in general, the use of slag appears to be non-detrimental. Besides lower early strength, the downsides to the use of GGBFS include extended curing times, increased salt scaling, increased plastic shrinkage cracking, and increased air entrainment required dosage. The literature reports mixed results in regard to increased or decreased required HRWR dosages[1-4].

The proportion of GGBFS in a mix should be dependent on the following: 1) the purpose for which the concrete is being used, 2) the curing temperature, 3) the grade (activity) of the slag, and 4) the characteristics of the cement or activator. GGBFS has been commonly used for cement replacement to reduce the maximum temperature rise in mass concrete. When Grade 120 (highest activity) slag is used, at least a 70% replacement may be needed to meet specification requirements. Most ready-mix concrete producers use 50% replacement with highly reactive slag during warm weather[4]. In a 1995 survey of 20 states, Duos and Eggers[5] found that 13 state DOT's allowed the use of GGBFS. Depending on weather conditions, one state limited the replacement rate to 25%, three at 40%, eight at 50%, and one at 70%.

MoDOT defines that the conditions for mass concreting exist when the minimum dimension of the concrete exceeds five feet and the volume-to-surface area ratio is equal to one. When this is in effect, the contractor is required to keep the temperature differential equal to or less than 22.2 ˚C [40˚ F] between any point deeper than 300 mm [12 in.] in the mass and the surface. Corrective measures should be applied when the differential nears 20 ˚C [35˚ F][6].

During the early hydration of the slag cement, the portland cement releases alkali metal ions and calcium hydroxides (CH). The glassy slag structure is broken down and dissolved by the hydroxyl ions. Initially, the reaction of the slag is with alkali hydroxide; later, the reaction is primarily with calcium hydroxide[4]. As hydration continues long–term, the PC continues to precipitate calcium hydroxide and grow rings of calcium silicate hydrate (CSH) inward from the original grain surface. Slag, on the other hand, develops more CSH, contributing to strength, density, and chemical resistance[7]. The filling of pores with additional CSH is called pore size refinement, while the replacement of CH with CSH (a denser structure) is termed grain size refinement[2]. Thus because of the pozzolanic reaction, slag pastes contain less calcium hydroxide than OPC pastes. It has been shown, in at least one study[8], that for slag-OPC mixes containing 80% slag, the CH is depleted. Higher CH contents tend to produce inferior concrete because of the following: 1) production of an inhomogeneous body with poor bonding between CSH

and CH, 2) a greater likelihood of cracks propagating from the interface of CSH and CH, and 3) CH is weaker than CSH[9]. The slag retains the alkali and calcium hydroxides in its hydration products (i.e. CSH). This results in a hardened cement paste that has a greater density and smaller pore sizes than an equivalent OPC paste, thus permeability and ionic diffusivity is reduced[4]. Smaller pore size relates to lower permeability, although does not necessarily mean lower total porosity. Slag-PC and OPC mixes may result in similar total porosities, but the slag-PC pore structures tend to be finer[10]. Total porosity is important to mechanical properties such as compressive strength, but is less critical to properties that are associated with durability such as permeability. Durability seems to be related to larger pores[9]. However, it has been reported[11] that a 60% slag mortar mix not only had smaller pore sizes but also a somewhat smaller pore volume. Higher CH contents are associated with greater permeability and lower durability[9]. The clinker-CSH bond appears to be stronger than the slag-CSH bond[10] which would tend to offset the denser pore structure of slag-cement mixes.

The primary factors that affect the slag-cement reaction are as follows: 1) chemical composition, fineness, glass content, and age of the slag, 2) fineness of the cement, 3) alkali concentration of the reacting system and 4) temperature[4,12] .

Glass content (degree of vitrification) is considered a primary factor[12,13], and the structure of the glass is also significant[8]. In general, an increasing glass content results in greater pozzolanic activity[12,13]. The glass content is a function of the preparation process of the slag, with granulation resulting in a higher glass content than pelletization[12].

The effect of aging on slag reactivity has been noted by Metso and Kajaus[12]. Slags that have been in the silo for more than a month tend to lose some reactivity, sometimes quite significantly, and thus require more effort at activation.

The alkali content of the slag seems to assist in the activity of the slag. In a comparison of two slag sources, it was shown that there was a significant gain in compressive strength as the alkali content increased[12].

Although slag can self-activate, the reaction is relatively slow and the use of some type of activator is warranted. Typically, OPC is used as the activator, but others have been tried. The principal activators of slag that stem from OPC are gypsum and CH[8]. Sufficient alkali is also necessary for development of significant strength. Slag cement by itself is usually deficient in alkali and thus requires an additional source. This is usually supplied by the cement. However, in cases of low OPC content mixes, additional alkali can be added. Several authors have used sodium silicate to promote strength of low OPC content mixes (zero and 5%). Strengths of the alkali activated mixes were superior to the OPC control mixes[14,15]. Sodium sulfate has also been used as an activator in pozzolanic systems. The sulfate forms gypsum nuclei for ettringite and CSH gel. The pH is raised, dissolving silica and alumina oxides which react with calcium hydroxide to form CSH[16]. Slag activators are usually termed alkaline (sodium hydroxide, sodium carbonate, and sodium silicate) or sulfate (gypsum, hemihydrate, anhydrite, phosphogypsum, and sulfur in the slag)[8] . In a study of numerous activators that included OPC, OPC clinker, sodium

hydroxide, calcium hydroxide, calcium sulfate, sodium silicate, sodium carbonate, calcium chloride, ash of sulfite sludge, hydrochloric acid, aluminate cement, fly ash, phosphogypsum, and a modified accelerating lignosulfonate admixture, the most successful were OPC, OPC clinker, sodium hydroxide, calcium hydroxide, and the ligonosulfonate admixture[12].

The use of a high range water reducer (HRWR) will act to increase the homogeneity of the slag particle distribution, thus resulting in a superior cement paste[17].

Admixture Dosage Requirements

Dosage and air retention characteristics of slag mixes are reported to be similar to OPC mixtures[18]. However, ACI 233[4] states that dosage rates of air entraining agent (AEA) may have to increase if the slag is finer than the cement. Sivasundarum and Malhotra[19] found that it took a considerably greater AEA dosage for slag cement mixes than OPC mixes, and that as the slag replacement level increased, the required dosage also increased. It has been postulated[20] that this may be due to the greater fineness of the slag and/or the presence of interground coal. The air system of slag mixes should be evaluated. An acceptable air void system is considered to include a spacing factor less than 0.2 mm [0.008 in.] and a specific surface of at least 24 mm^2/mm^3 [600 in^2/in^3][21].

It has been reported that the required HRWR dosage in a slag mix may decrease by as much as 25%[4]. Retarders become more efficient as GGBFS content increases.

Workability

It has been reported that water demand for a given slump is about 1 to 10% lower than plain concrete[4,18]. Numerous authors have reported improved workability with slag mixes. This is thought to be because the surface of the slag, which is smooth and dense, creates smooth slip planes and low absorption of water as opposed to portland cement. Also, when substituted on a mass basis, due to the lower specific gravity of GGBGS, more paste is present which would aid in workability. Because of greater workability, coarse aggregate content can be increased and the resulting decrease in paste will often render the paste less sticky[4]. However, Bush, et al.[2] found a moderate slump reduction with the introduction of 25% Grade 120 GGBFS.

For most of the mixes tested by Sanjayan and Sioulas[22], the slag mixes exhibited 20 to 50% greater slumps at constant *w/cm's* (water to cementitious material ratio by weight) indicating that the *w/cm* could have been lowered with consequent enhancement of hardened properties. Even more pronounced results were seen by Lane and Ozyildirum[23]. Bleszynski et al.[1] found that rheological properties were enhanced. However, in the Duos and Eggers[5] study, at 23˚ C [73˚ F], slump decreased with increasing slag content.

Setting Time

In general, the use of GGBFS will retard setting time[18,24]. However, experiences have been mixed in regard to setting time. Duos and Eggers[5] found that setting time was a function of the amount of cementitious material and mixing/curing temperatures. In general, setting time of richer mixes at 23˚ C [73˚ F] was not significantly affected by increasing amounts of slag. However, leaner mixes at 40˚F exhibited slower setting as slag content increased. Bush et al. [2] found little change in setting times with 25 and 50% GGBFS contents. Saika et al.[25] found that although initial setting times were not impacted much by slag replacement, final setting times increased. Hogan and Meusal[26] found that both initial and final set times were not affected appreciably. ACI 233[4] states that slag mixes tend to have longer initial setting times by one half to one hour.

Heat of Hydration

Because slag cement reacts more slowly than Type I Portland cement, it is used for mass concreting situations[20]. An alternate to Type IV and low heat of hydration Type II is a Type II PC with GGBFS. Replacement levels range from 50 to 85%. In a study of adiabatic temperature rise, it was shown that as slag replacement level increased up to 70%, temperature rise was reduced, although the reduction was only significant at the 70% replacement level. However, an increase in either binder content (250 to 350 kg/m^3 [420 to 590 lbs/cy]) or fineness (300 to 400 m^2/kg) increased temperature rise[27].

Compressive Strength

ASTM C 989[28] divides GGBFS into three strength grades in accordance with their Slag Activity Index (SAI) values: Grade 80, 100, and 120, with Grade 120 being the most active. The SAI is the ratio of the strength of a 50-50 blend of slag and PC to the strength of a plain PC mix at 7 and 28 days. The SAI is considered the best criterion for assessing the relative cementitious potential of slag[4]. However, the PC used as a reference material must meet minimum requirements of compressive strength and alkali content. The PC used in a particular project may be less reactive. In general, the early strengths of Grade 120 slag mixes are lower than OPC mixes, but usually catch up and then surpass at 7 days and beyond. It is commonly believed that the other two grades typically exhibit lower strengths than 100% OPC concrete at all ages. Factors which affect slag mix performance are as follows: 1) proportions of cementitious materials, 2) physical and chemical characteristics of the slag, 3) curing conditions, 4) presence and dosage rate of admixtures, 5) characteristics of the aggregate, and 6) characteristics of the portland cement.

Proportions of cementitious materials. High slag replacement mixes have been studied previously. In most cases, replacement of portland cement with slag ultimately lead to increases in strength. Unfortunately, the grade of slag was usually not reported. Using superplasticized constant *w/cm* mixes, Lim and Wee[29] used GGBFS replacement at 30, 50, 65, and 80% levels. By 7 days, the 50 and 65% mixes had surpassed the control mix, and by 91 days all slag mixes were stronger. At 91 days, the optimum replacement

amount was 50%. Lane and Ozyildirum[23] investigated constant *w/cm* Grade 120 GGBFS-PC binary mixes with 25, 35, 50, and 60% replacement. The only admixture used was air entrainment. Early strengths of the replacement mixes were lower than the control, but by 56 days, the 50 and 60% slag mixes exceeded the OPC mix, and at the one year mark all slag mixes were stronger than the OPC mix. From 28 days on, increasing slag percentage increased the strength. Blomberg[30] varied GGBFS replacement levels at 25 and 50%. The mixes contained a durable limestone aggregate, 357 kg/m^3 [602 lbs/cy] cementitious content, Type I cement, air entrainment, and a type A water reducer (WR). Early strengths were reduced with the slag mixes, but the 50% slag mix overtook the non-admixture OPC control mix at 14 days. Strengths of all slag mixes were greater compared to the zero-slag mix containing the WR at all ages. The 25% slag mix lagged below the control at all ages. Hogan and Meusal[26] studied 40, 50, and 65% blends. In general, at ages from 28 days to one year, blends between 40 and 50% gave the highest strengths. Early rate of strength gain was inversely proportional to slag content. Mixes with slag contents above 40% were all superior in strength to the OPC mixes. Using GGBFS contents of 25 and 50%, a study by Bush et al.[2] exhibited 38 and 51% increases in strength, respectively, compared to the control.

In at least one study, slag replacement did not result in an ultimate increase in strength, however, longer-term strengths were essentially equal to OPC concrete. Sanjayan and Sioulas[22] studied a 100 MPa [14,500 psi] strength level, superplasticized, non-air entrained mix at a constant *w/cm*. For 7, 28, 56, and 91 day ages, the percent strength of the 70% slag mix compared to the OPC control mix was 46, 71, 85, and 96, respectively. At 91 days, the percent strengths for 30, 50, and 70% replacement were 100, 95, and 96, respectively. For the 40 MPA [5800 psi] 50% replacement mix, the 91 day percent strength was 97. Strengths were from standard laboratory-cured cylinders. Thus, strength of slag mixes was initially lower than the OPC mix, but eventually caught up by 91 days at all slag replacement levels.

In several cases, slag replacement resulted in lower strengths compared to OPC control mixes. Bleszinski et al.[1] varied slag replacement levels at 35 and 50%. As the replacement level increased from zero to 50%, strength at 28 days was reduced significantly. Working with high slag replacement values, Tomisawa and Fujii [31] found that as slag percentage increased from 50 to 90%, strengths dropped. Using a Grade 100 slag, for a 55% slag mix, Zhang et al.[32], in comparison to OPC concrete, saw strength losses of 13% and 25% at 28 and 91 days, respectively. Sivasundarum and Malhotra[19] varied slag replacement at 65% to 75%. Strengths were reduced at all replacement values compared to the OPC control mix at ages up to 91 days. Li et al. [33] varied slag replacements from zero to 70%. They observed a slight loss in strength at 50%, but a more significant loss at 70%. Using a 60% slag replacement mix which featured a *w/cm* of 0.45, Grade 120 slag, and 155 kg/m^3 [260 lbs/cy] OPC, Peterson and Hale[34] were able to achieve 48 MPa [7000 psi] , but the slag mix strength fell somewhat below the OPC mix; 28 day and 90 day results were 88% and 98% of the control strengths, respectively. Mak & Lu[35] studied replacement percentages of 50 and 70. At 50%, with a PC content of 250 kg/m^3 [420 lbs/cy], the slag mix achieved a strength of 98% of the OPC mix at 28

days and 99% at 91 days. The 70% mix (150 kg/m^3 [252 lbs/cy]) reached 85% at 28 days and 93% at 91 days.

A number of projects have successfully utilized large slag proportion mixes. Unfortunately, comparisons to plain OPC control mixes were not reported for any of these projects. Luther, et al.[36] reported on 18 case histories where slag replacements of 65 to 75% resulted in successful projects that achieved 28 day strengths of 28 to 83 MPa [4100 to 12,000 psi] with *w/cm* of 0.41 to 0.56 and total cementitious contents of 194 to 478 kg/m^3 [326 to 803 lbs/cy] (OPC contents of 126 to 324 kg/m^3 [212 to 544 lbs/cy]). Most of the projects utilized a Grade 100 slag. Ozyildirim[37] reported on a mass concrete case history where a 75% slag mix achieved strengths exceeding 28 MPa [4000 psi]. Bognacki[38] reported on a mass concrete project where two slag mixes were used, both at 80% slag content. One mix contained 60 kg/m^3 [100 lbs/cy] OPC and 240 kg/m^3 [400 lbs/cy] slag which resulted in 28 day strengths averaging 40 kg/m^3 [5880 psi]. The second mix contained 48 kg/m^3 [80 lbs/cy] OPC and 190 kg/m^3 [320 lbs/cy] slag with resulting strengths of 35 MPa [5040 psi] at 28 days. The *w/cm* for both mixes was 0.50.

In general, the optimum blend of slag for greatest strength at 28 days seems to be about 50%[4]. The literature seems to be evenly split between high slag proportion mixes that exceed the control mixes and slag mixes that do not achieve parity with the controls at any ages. Fig. 1 depicts the effect of slag proportion on compressive strength at 28 and 90 or 91 days. It includes data from the present study, as discussed later in the report.

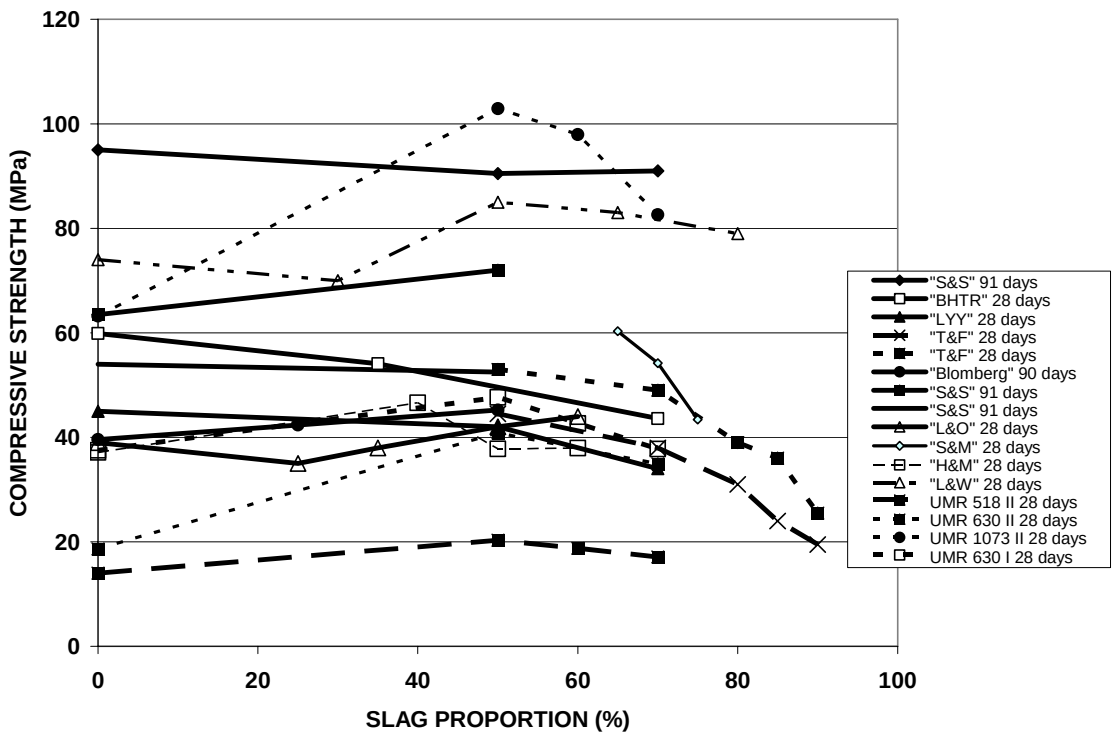

Fig. 1- Effect of slag proportion on compressive strength

Effect of slag characteristics. Because GGBFS cement reacts more slowly, especially at lower temperatures, it is typically ground finer than OPC. Hamling and Kriner[39] found that as fineness increased from 4080 to 6230 cm^2/g, 28 day strength increased significantly. Lim and Wee[29] found that more finely ground GGBFS had greater early strengths, but after 28 days the greater fineness caused little difference. Jin and Yazdani[40] studied the effect of five different slag sources on 28 day compressive strength of hot weather concreting mixes. All mixes had a 60% slag content, 0.37 *w/cm*, and contained 149 kg/m^3 [251 lbs/cy] Type II cement. The maximum difference from brand to brand was 8.3 MPa [1209 psi], with the lowest strength exhibited by Lonestar's AUCEM®.

Effect of Curing. Swamy and Bouikni[41] experimented with various curing regimes. In a comparison to moist-cured specimens at 180 days, curing in air lowered the strength 21% and 47% for 50% and 65% slag replacement mixes, respectively. However, an initial seven day moist cure followed by air curing resulted in strengths comparable to the moist-cured conditioning for the 50% slag mix strength, although the 65% slag mix still lost 17% strength. Regardless of curing method, the 50% slag mixes had greater strengths than the 65% slag mixes at all ages, although the difference between the 50 and 65% mix moist-cured strengths was relatively small. Slag mixes are more susceptible than OPC mixes to poor curing conditions at higher slag contents due to reduced formation of hydrate at early ages[42].

In regard to strengths of in-situ concrete, Sanjayan and Sioulas[22] cored lab-cast columns and found that, in general, 91 day strengths were lowest at the top of the columns and at their surfaces for plain and 50% slag replacement mixes. It was concluded that the interior had a higher curing temperature which accelerated strength gain, and the evaporation of moisture near the surfaces impeded strength gain. Slag mixes were more sensitive to these factors.

In comparing lab-cured cylinders to in-situ strength, it is commonly known that in-situ strength is lower than strength determined from lab-cured cylinders under standard conditions. A factor of 0.85 is typically assumed, although more recent work indicates the factor may be as low as 0.7[43]. In the Sanjayan and Sioulas[22] study, standard cured cylinders overestimated effective column compressive strength of slag mixes by about 20% at 50% slag replacement, but increased to 40% at 80% replacement.

Effect of Admixtures. Jin and Yazdani[40] studied the effect of five different air entrainment agents on 28 day compressive strength of hot weather concreting mixes. All mixes had a 60% slag content, 0.37 *w/cm*, and contained 149 kg/m^3 [251 lbs/cy] Type II cement. Dosages were varied to produce approximately uniform air contents. The maximum strength difference from brand to brand was 12.6 MPa [1827 psi].

In addition to strength gains resulting from lower *w/cm's*, HRWR's are known to increase the strength of slag concrete because of the dispersing action of the admixture on the cement particles, resulting in a superior microstructure. In a study of 30% cement replacement, it was found that the use of a HRWR resulted in superior strengths beginning at age 7 days with *w/cm's* held nearly constant[24].

Effect of Aggregate Type. Duos and Eggers[5] studied constant *w/cm* structural mixes with GGBFS Grade 120 cement replacements of 15, 30 and 50%. The mixes had constant dosages of air entrainment and water reducer. In general, with limestone aggregate, 7, 28, and 56 day strengths increased with increasing slag replacement. However, with gravel aggregate and lower cement contents, strength tended to decrease with increasing slag contents.

Expectations. It appears that for slag replacements above 80%, strengths usually will not be on par with 100% OPC mixes, although it is possible for slag values between 50 and 65% to equal OPC mixes at 28 days and exceed at later ages. Slag proportions of 40 to 60 % appear to be the optimum level for highest strength development. Some slag-PC combinations will not reach strength levels commensurate with control mixes at any slag proportion. Greater glass content, fineness, and alkali content of the slag and greater fineness and alkali content of the OPC, use of HRWRs, and possibly shorter slag detention times, increase strength development. Slag replacement of up to 80% can meet 28 MPa [4000 psi] specified strengths.

Permeability

Use of GGBFS as a partial replacement of portland cement has been found to reduce permeability[44] and has shown to result in good resistance to chloride penetration[45]. The pore structure of the paste is changed through the reaction of the slag with the calcium hydroxide and alkalis released during hydration of the PC[7]. The pores are filled with calcium silicate hydrates instead of calcium hydroxide. Additionally, because workability is enhanced, the *w/cm* can be lowered, thus resulting in a denser paste structure. It has been postulated that slag replacement of portland cement will decrease the permeability by producing a finer pore size distribution even though the total porosity may increase[45]. Bakker[46] has theorized that the reason the permeability of slag-PC mixes is less than that of OPC mixes is that the precipitation of CH in OPC mixes will not necessarily result in a total blocking of pores, whereas in slag-PC mixes, the Al_2O_4 and SiO_3 set free by the hydration of slag will meet the released CH from the PC clinker and the resulting precipitation of CSH and C_4AH_{13} will tend to fully block the pores, for the same porosity.

Lane and Ozyildirum[23] used ASTM C 1202[47] electrical resistance as an indicator of ionic transport properties. Mixes with slag replacement levels from 25% to 60% were investigated. The 25% replacement mix had comparable values to the OPC mix, while all other slag mixes had superior properties. The 60% mix resulted in an electrical resistance of about half of that of the OPC mix. Rose[48] used slag contents of 0, 40, 50, and 65% and found that as slag content increased, Rapid Chloride Ion Penetration (RCIP) significantly decreased. Additionally, a chloride soaking test was performed and it was determined that increases in *w/cm* had less effect on RCIP for the slag mixes than on the OPC control. Using somewhat higher slag contents (50% to 75%), Sivaundarum and Malhotra[19] found similar results. Zhang et al.[32] saw a significant reduction in chloride permeability using 55% slag replacement. Duos and Eggers[5] noted that chloride permeability decreased with increasing slag contents. Blomberg[30] found that as slag content increased from zero to

50%, chloride permeability dropped significantly at 90 days. Bleszinski et al.[1] found that the 35% and 50% slag mixes significantly reduced chloride permeability, with the 50% blend somewhat lower than the 35% blend. Bush et al.[2], using a 25% replacement with GGBFS, saw RCIP values reduced by half. Additionally, saltwater ponding test results at 90 days had about 50% lower chloride penetration in the top 0.5 in. than the OPC control. Using a 60% slag replacement, Peterson and Hale[34] measured a reduction in RCIP by about two-thirds.

Expectations. Overall, in a *w/cm* range of 0.39-0.45, OPC control mix coulombs passing ranged from 2100 to 5800, while at a 60% slag replacement level, the range was reduced to 800 to 2600. Slag replacement reduces permeability from a reduction in pore size, not necessarily from a reduction in porosity. Because strength is a function of porosity, this means that it could be expected that permeability and strength may not correlate well.

Freeze-Thaw Durability

Freeze-thaw resistance of concrete is a function of the interaction of the cement paste microstructure, the aggregates, the electrolytes in the pore system, the system humidity, the characteristics of the temperature cycles, the degree of saturation, and the effectiveness of the air void system. The quality of the air void system in some instances has been degraded by the use of HRWR's[17]. Durability of concrete mixes is commonly evaluated by use of ASTM C 666[49] (AASHTO T 161)[50] in which concrete prisms are subjected to freezing and thawing cycles, then periodically tested for loss in weight, increase in length, or reduction of relative dynamic modulus. The relative dynamic modulus is expressed as the Durability Factor (DF), which also takes into consideration the relative number of cycles that the specimen survives.

Numerous studies have shown that the freezing/thawing resistance of PC-slag mixes and OPC mixes is about the same[4], even at greater than 60% slag contents[20, 51]. However, for equal performance, the following conditions must be met: 1) equal compressive strengths, 2) adequate entrained air system, 3) properly cured, and 4) air-dried one month before exposure to saturated freezing conditions[18, 20]. Using mixes with 377 kg/m^3 [637 lbs/yd^3] total cementitious material at a *w/cm* of 0.45 and an air content of 6.5±0.5%, Lane and Ozyildirum[23] found excellent results for OPC and 60% slag mixes (Durability Factors all in excess of 100), with all slag mixes somewhat higher than the OPC mix. The highest Durability Factor occurred with the 50% slag mix. Pigeon and Regourd[10] compared OPC and 66% slag mortar mixes (0.5 *w/cm*) and found good durability characteristics for both mixes as determined by a method similar to ASTM C 666 Method B. Both mixes resulted in essentially the same length change, mass loss, and dynamic modulus change (which was negligible). The compressive strengths, air void systems, and air void contents were similar. All specimens were cured 28 days in water prior to testing.

Blomberg[30] found that there was no significant difference in Durability Factors among the control, 25%, and 50% slag mixes when using a durable aggregate. Strengths increased with the use of slag. Using ASTM C666, Hogan and Meusal[26] found that their

100% OPC specimens DF was 98 compared to the 50% slag specimens 91. Weight loss and expansion were negligible. They concluded that both mixes were durable. Although DF diminished somewhat, compressive strengths increased for the slag mix. Duos and Eggers[5] found that the DF dropped significantly with slag contents increasing from 15% to 50%. However, specimen length change had an inverse relationship with DF. The relationship between strength and durability was mixed. In both studies by Malhotra[52, 53], with slag replacements of 65% and 75%, both strength and durability decreased.

Expectations. Thus, for slag replacements of 50% to 70%, results have been mixed in regard to whether freeze-thaw durability increased or not, and whether the relationship of strength and DF was directly or inversely proportional.

Salt Scaling

Laboratory studies generally indicate that slag concrete has lower scaling resistance than OPC concrete[1, 4, 5, 17, 23, 30, 54 55, 56]. The root cause seems to be related to the effects of carbonation.

In comparison to field-conducted investigations, laboratory tests tend to over-predict the loss of scaling resistance, most likely because of differences in finishing and curing, and because of the greater severity of laboratory testing conditions compared to field conditions[1, 4, 23, 54,57]. Some authors have found differences between slag and OPC concrete scaling resistance to be minor[26, 30].

Carbonation effects have been studied in regard to carbonation depth and rate, effects on porosity and permeability, available calcium hydroxide, and carbonation end products. It has been reported in many cases that slag concrete can have high frost resistance, if properly air entrained. However, the resistance to salt scaling of concrete with high (greater than 60%) slag contents cannot be improved by proper air entrainment alone[51]. Stark and Ludwig[51] state that slag concrete can be frost resistant as long as the degree of hydration has progressed sufficiently, but this is not true for salt scaling resistance. They showed that surface scaling occurred in the zone of carbonation, and ceased when the depth of non-carbonation was reached. Although carbonation slightly densified the matrix, the slag concrete microstructure became coarser. Thus pore size distribution (which relates to permeability) was more important than total porosity (which relates to density). However, the authors attributed the main reason for lower salt scaling resistance to a chemical cause. When OPC concrete carbonates, only calcite is formed; when slag concrete carbonates, modifications of calcite (vaterite and aragonite) are formed. Vaterite and aragonite are subsequently dissolved by the combined attack of frost and chloride[51]. One product of the hydration of cement is calcium hydroxide (CH). There is less CH in slag mixes than OPC mixes because the CH is converted to CSH. Thus, there is less CH available near the surface of the concrete to combine with CO_2 from the air to form calcium carbonate (carbonation). The calcium carbonate tends to seal the surface, limiting the ingress of chloride. The carbonation process occurs at greater depths in slag mixes because of the diminished blockage action at early ages[17, 55]. Sulapha et al.[56] found that for low Blaine fineness slags (4500 cm^2/g), although the slag concretes were denser,

the carbonation rate increased with increasing amounts of slag. This was thought to be attributable to less CH being available due to the pozzolanic reaction, so the carbonation must progress deeper to get at the available CH. There is less material available prone to carbonation per unit area to react with CO_2. Initially the pozzolanic reaction is slow, thus porosity is higher and CO_2 diffusion is rapid. However, they found that for higher fineness slags (6000-8000 cm^2/g), carbonation rates were lower than that for OPC, thus pore modification was more dominant than the change in CH content. So, slag mixes tend to have greater depths of carbonation and are more permeable (although not necessarily more porous) and therefore would tend to scale more, especially under severe environmental conditions.

Differences in specimen finishing sometimes seem to affect the outcome of scaling tests. One possible problem is that slag delays the setting of concrete, therefore increasing the period of bleeding. If the bleed water is finished into the surface of the concrete, the durability of the surface could be lowered. In their study, Bleszinski et al.[1] found that slag mixes showed increased salt scaling with the 50% blend exhibiting the most severe damage in both the lab and field studies. Scaling was measured by mass loss. Part of the study involved turning the scaling specimens over and testing the formed surfaces to get away from the finishing issue. Hooton and Boyd[57] found that the method of finishing and curing the test slabs had a significant influence on the results. Again, premature finishing was detrimental to the scaling resistance.

Laboratory test conditions tend to be more severe than field conditions. Lane and Ozyildirum[23] ran ASTM C 666 with a 2% sodium chloride solution. The mass loss results were considered as a type of salt scaling test. The 25% slag mix was comparable to the OPC mix, while the 35%, 50%, and 60% slag mixes were less durable, with performance worsening with increasing slag percents. However, all mixes passed the criteria of acceptance except for the 60% slag mix, which barely failed. The authors pointed out that the test has a more severe environment than would be expected in the field because of the relative immaturity of the lab specimens, the more intensive freezing test cycles, and the higher saturation level in the lab specimens. Any scaling that occurred at the level of mass loss observed would most likely be limited to exposure of coarse aggregate with no progressive internal damage[23]. Additionally, shrinkage could occur at 50% relative humidity (which is the specified humidity after the moist cure period for salt scaling testing) during carbonation. Thus salt scaling, a surface phenomenon, could be more prevalent [17, 55].

Expectations. Lab scaling tests usually indicate a reduction in scaling resistance when slag is used as a replacement for PC due to carbonation effects and testing conditions.

Research Significance

There is a need for research into the use of high levels of GGBFS in concrete structures. Typical specifications are conservative, limiting use of GGBFS to 25% in typical applications. This study will provide further insight into the subject. If research can demonstrate that significantly greater amounts can be used successfully, specifications may be able to be altered to include greater percentages of GGBFS.

Experimental Program

The study is divided into two phases. The first involves the sampling and testing of the field-produced concrete. The second phase involves laboratory experimentation undertaken to clarify the effects of job cementitious materials on compressive strength.

Phase I: Field Investigation

General

The objective of this study is to determine strength and durability characteristics of a Missouri Class B-1 concrete mix using ground granulated blast furnace slag (GGBFS) at a 70% replacement of cement. Two GGBFS mixes were evaluated in the study: one mix containing high range water reducer (HRWR) and one mix without HRWR. For comparison, a conventional or standard Class B-1 concrete mix was also evaluated. Mix evaluation entailed compressive strength (AASHTO T 22-97)[58], freeze-thaw durability (AASHTO T 161-97)[50], rapid chloride permeability (AASHTO T 277-96)[54], and salt scaling (ASTM C 672-92)[60] testing. Air-void analysis (ASTM C 457-98)[21] was conducted to determine the air void system characteristics.

Materials

The coarse aggregate source for all three mix designs was a standard Gradation D crushed limestone (St. Louis formation). The fine aggregate was a Class A Missouri River sand. All mixes contained an air entraining agent.

Table 1-Material Suppliers

Material	Supplier
Type I cement	River Cement Co.
Type II cement	Lonestar, Inc.
GGBFS (Aucem)	Lonestar, Inc.
Limestone coarse aggregate	Weber North Quarry
Class A sand	St. Charles Quarry
Water	St. Louis County
Air Entraining agent (Daravair 1400)	Grace Construction Products
HRWR (Daracem-19)	Grace Construction Products

Table 2-Gradation specifications for crushed stone and river sand

Size mm (in.)	Gradation D % Passing	Class A sand % Passing
25.0 (1)	100	100
19.0 (3/4)	90-100	100
9.5 (3/8)	15-45	100
4.75 (#4)	0-8	95-100
0.850 (#20)		40-75
0.300 (#50)		5-30
0.150 (#100)		0-10

The control mixture was a B-1 mix. The mixture proportions as reported in the Plant Inspector's Daily Reports are shown in Table 3.

Table 3-Control (B-1) mix proportions

Material	Amount		
	12-14-99 (am)	**12-14-99 (pm)**	**3-23-00**
Type I cement, Kg/m^3 (lbs/cy)	374 (630)	374 (630)	374 (630)
Grade D crushed limestone, Kg/m^3 (lbs/cy)	1092 (1841)	1092 (1841)	1092 (1841)
Class A sand, Kg/m^3 (lbs/cy)	685 (1155)	685 (1155)	685 (1155)
Water, L/m^3 (lbs/cy)	157.6 (266)	157.6 (266)	132.8 (224)
Air entraining agent, L/m^3 (oz / sack)	0.249 (0.96)	0.249 (0.96)	0.211 (0.81)
w/cm	0.421	0.421	0.355

Mix proportions for the GGBFS without HRWR ("plain GGBFS") and with HRWR ("GGBFS-HRWR") are shown in Tables 4 and 5, respectively. The slag was a Grade 120 GGBFS.

Table 4-GGBFS without HRWR mix proportions

Material	Amount		
	11-8-99 (am)	**12-2-99**	**3-7-00**
Type II cement, kg/m^3 (lbs/cy)	112 (189)	112 (189)	112 (189)
GGBFS kg/m^3 (lbs/cy)	262 (442)	262 (442)	262 (442)
Grade D crushed limestone, kg/m^3 (lbs/cy)	1032 (1743)	1032 (1743)	1032 (1743)
Class A sand, kg/m^3 (lbs/cy)	694 (1170)	694 (1170)	694 (1170)
Water, L/m^3 (lbs/cy)	158.8 (268)	162.0 (273)	148.9 (251)
Air entraining agent, L/m^3 (oz / sack)	0.696-0.773 (2.68-2.98)	0.696 (2.68)	0.774 (2.98)
w/cm	0.425	0.433	0.398

Table 5-GGBFS with HRWR mix proportions

Material	Amount			
	11-5-99	**12-13-99**	**2-7-00**	**8-17-00**
Type II cement, kg/m^3 (lbs/cy)	112 (189)	112 (189)	112 (189)	112 (189)
GGBFS kg/m^3 (lbs/cy)	262 (442)	262 (442)	262 (442)	262 (442)
Grade D crushed limestone, kg/m^3 (lbs/cy)	1056 (1780)	1056 (1780)	1056 (1780)	1056 (1780)
Class A sand, kg/m^3 (lbs/cy)	703 (1185)	703 (1185)	703 (1185)	703 (1185)
Water, L/m^3 (lbs/cy)	160.8 (271)	149.8 (252)	147.3 (248)	140.2 (236)
Air entraining agent, L/m^3 (oz / sk)	0.735 (2.83)	0.696 (2.68)	0.812 (3.13)	0.812 (3.13)
HRWR L/m^3 (oz / sack)	1.24 (4.78)	1.24 (4.78)	1.24 (4.78)	1.24 (4.78)
w/cm	0.430	0.401	0.394	0.375

Several things are noted from the mix design values. The weight of total cementitious material was kept constant in each mix type, with the GGBFS replacing 70% of the portland cement. However, because slag has a lower specific gravity, paste volume was actually greater in the slag mixes. The control mix utilized a Type I cement, while the slag mixes contained Type II. The average *w/cm* ratio was lowest for the control (0.399) and GGBFS-HRWR mixes (0.400), while the plain GGBFS mix was the highest (0.419). Air entraining agent dosage was significantly higher for both the GGBFS mixes than the control. The percent sand was slightly lower (38.5%) for the control mixture as opposed to the two GGBFS mixtures (40%).

The Lonestar monthly production averages analysis of the GGBFS is shown in

Table 6.

Table 6-GGBFS analysis

Date	%Ret. #325	Blaine, cm^2/g	Strength 7 day, psi	Strength 28 day, psi	SAI 7 day, %	SAI 28 day, %	Total Alk. *
Nov '99	0.35	5870	4580	7500	104	135	0.59
Dec '99	0.30	5750	4520	7600	100	132	0.55
Jan '00	0.24	6140	4550	7520	99	131	0.69
Feb '00	0.39	5410	5200	7920	114	139	0.59
Mar '00	0.51	5300	4890	7690	107	132	0.58
Apr '00	0.38	5340	4720	8090	103	139	0.59
May '00	0.43	5470	4690	7880	101	135	0.66
Jun '00	0.49	5230	4610	7740	100	133	0.58
July '00	0.52	5370	4500	7650	98	130	0.58
Aug '00	0.51	5060	4710	7780	102	133	0.52

* $Na_2O + 0.658\ K_2O$

Lonestar's analyses of the Type II low heat Portland cement are shown in Table 7.

Table 7-Type II low heat portland cement analysis

	Blaine, cm^2/g		Strength 28 day, psi		Strength 56 day, psi		Strength 90 day, psi		Total Alk. *	
Year	1999	2000	1999	2000	1999	2000	1999	2000	1999	2000
Minimum	282	302	4200	4210	5210	4950	5860	5500	0.33	0.41
Maximum	329	325	5400	5140	6600	6280	7090	6810	0.42	0.43
Average	303	310	4755	4782	5750	5672	6437	6278	0.36	0.42

* $Na_2O + 0.658\ K_2O$

Sampling

Three concrete placements per mixture type were sampled by MoDOT Research, Development, and Technology division (RDT) personnel. The placements spanned a period from late fall 1999 to August, 2000.

The following specimens were fabricated onsite per sampled placement:

- 150 x 300 mm [6 x 12 in.] cylinders for compressive strength
- 90 x 115 x 355 mm [3 ½ x 4 ½ x 14 in.] beams for freeze-thaw durability
- 100 x 200 mm [4 x 8 in.] cylinders for rapid chloride permeability
- 150 x 300 mm [6 x 12 in.] cylinders for air void analysis
- 300x 300 x 75 mm [12 x 12 x 3 in.] panels for salt scaling analysis

Upon fabrication, the specimens were covered with wet burlap, plastic, and curing blankets, and warmed with a heat lamp when necessary. The 150 x 300 mm [6 x 12 in.] cylinders were also placed in insulated forms. The specimens were brought to MoDOT's RDT central lab for testing after 48 hours curing in the field. In the lab, the specimens were demolded, then cured in a moist room at 23 ± 1.7˚ C [73 ± 3˚ F] in 100% humidity.

Slump and air content testing was performed onsite during each of the concrete placements.

The sampling schedule is shown in Table 8.

Table 8- Sampling Schedule

Sampling Date	Mix	Test Type	Specimen Type	No. Test Replicates
11-5-99	GGBFS-HRWR	Compressive Strength	150 x 300 mm cylinder	3 at 5 ages =15 cylinders
11-5-99	GGBFS-HRWR	F/T Durability	90x115x355 beam	3
11-5-99	GGBFS-HRWR	Air Voids	150 x 300 mm cylinder	2
11-5-99	GGBFS-HRWR	RCP	100 x 200 mm cylinder	2
11-8-99	GGBFS	Compressive Strength	150 x 300 mm cylinder	3 at 5 ages =15 cylinders
11-8-99	GGBFS	F/T Durability	90x115x355 beam	3
11-8-99	GGBFS	Air Voids	150 x 300 mm cylinder	2
11-8-99	GGBFS	RCP	100 x 200 mm cylinder	2
12-2-99	GGBFS	Compressive Strength	150 x 300 mm cylinder	3 at 5 ages =15 cylinders
12-2-99	GGBFS	F/T Durability	90x115x355 beam	3
12-2-99	GGBFS	Air Voids	150 x 300 mm cylinder	2
12-2-99	GGBFS	RCP	100 x 200 mm cylinder	2
12-13-99	GGBFS-HRWR	Compressive Strength	150 x 300 mm cylinder	3 at 5 ages =15 cylinders
12-13-99	GGBFS-HRWR	F/T Durability	90x115x355 beam	3
12-13-99	GGBFS-HRWR	Air Voids	150 x 300 mm cylinder	2

Table 8 (cont'd.)				
Sampling Date	**Mix**	**Test Type**	**Specimen Type**	**No. Test Replicates**
12-13-99	GGBFS-HRWR	RCP	150 x 300 mm cylinder	2
12-14-99 am	B-1	Compressive Strength	150 x 300 mm cylinder	3 at 5 ages =15 cylinders
12-14-99 am	B-1	F/T Durability	90x115x355 beam	3
12-14-99 am	B-1	Air Voids	150 x 300 mm cylinder	2
12-14-99 am	B-1	RCP	100 x 200 mm cylinder	2
12-14-99 pm	B-1	Compressive Strength	150 x 300 mm cylinder	3 at 5 ages =15 cylinders
12-14-99 pm	B-1	F/T Durability	90x115x355 beam	3
12-14-99 pm	B-1	Air Voids	150 x 300 mm cylinder	2
12-14-99 pm	B-1	RCP	100 x 200 mm cylinder	2
2-7-00	GGBFS-HRWR	Compressive Strength	150 x 300 mm cylinder	3 at 2 ages =6 cylinders
2-7-00	GGBFS-HRWR	F/T Durability	90x115x355 beam	3
2-7-05	GGBFS-HRWR	Salt Scaling	300x300x75 mm panel	3
3-7-00	GGBFS	Compressive Strength	150 x 300 mm cylinder	3 at 3 ages =9 cylinders
3-7-00	GGBFS	F/T Durability	90x115x355 beam	3
3-7-00	GGBFS	Salt Scaling	300x300x75 mm panel	3
3-23-00	B-1	Compressive Strength	150 x 300 mm cylinder	3 at 2 ages =6 cylinders
8-17-00	GGBFS-HRWR	Compressive Strength	150 x 300 mm cylinder	3 at 3 ages =9 cylinders
8-17-00	GGBFS-HRWR	F/T Durability	90x115x355 beam	3

Compressive Strength

Compressive strength testing was done in accordance with AASHTO T 22-97 by Central Lab personnel. The specimens were tested using neoprene caps and tested in a 90,800 kg [300,000 lb] capacity machine.

Specimens were tested at 7, 28, 56, 90, and 365 days. Three specimens were tested per specimen age.

Freeze-Thaw Durability

Durability testing was done in accordance with AASHTO T 161-97, Method B by Central Lab personnel. Three replicate beam specimens were tested per concrete placement. Specimens were cured under a variety of conditions: 28, 35, 56, 90, 97, and 120 days wet, with some specimens receiving an additional 7 day dry period before testing. Thus, the effects of additional wet curing and effects of a subsequent drying period were explored. Wet curing consisted of submersion in lime-saturated water, and drying was defined as 7 days under room air temperature and humidity conditions. Every 12 to 18 cycles of freezing and thawing the beams were removed from the freeze-thaw chamber and the relative dynamic modulus, change in length, and change in weight were determined. The cycling was terminated after 300 cycles or when durability dropped below 50%. Figs. 2, 3, and 4 depict the testing equipment used in this study.

Fig. 2-Freeze-thaw cabinet

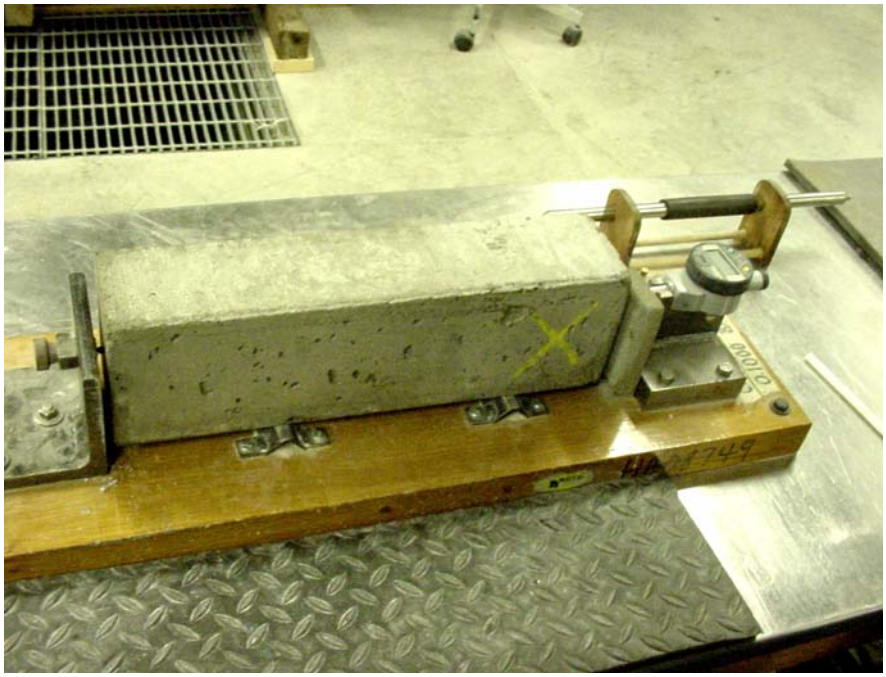

Fig. 3- Durability specimen length change device

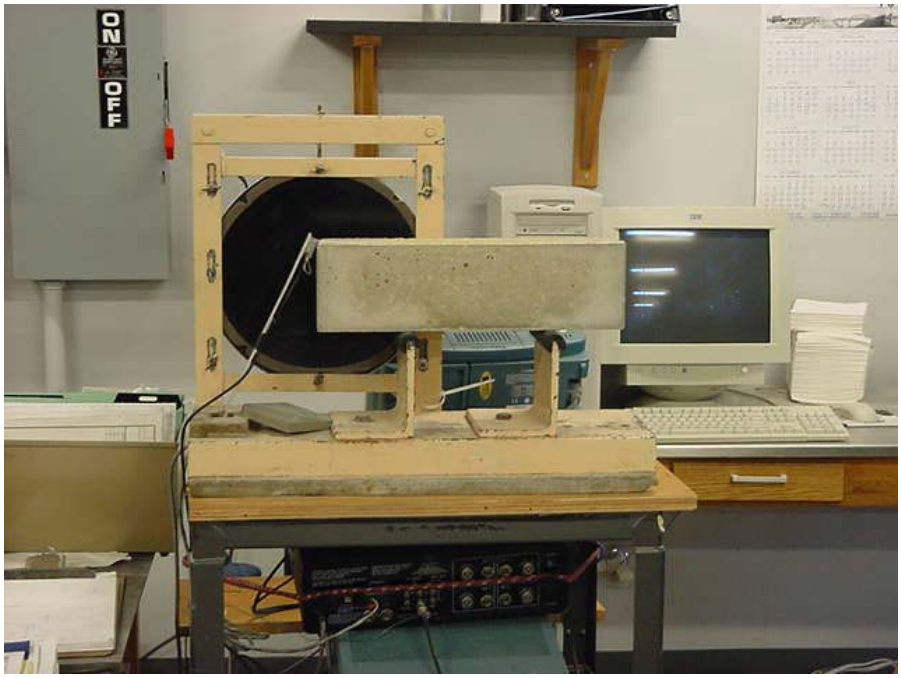

Fig. 4- Durability specimen dynamic modulus determination

Salt Scaling

Salt scaling resistance was determined by utilizing the procedure in accordance with ASTM C 672-92. Three replicate test panels were made per mix type at the construction site. After casting, the concrete was struck off using a wooden strike-off board. After the bleed water had disappeared, the concrete surface was finished with three passes of the wooden strike-off board, followed with a medium-stiff brush finish and immediately covered with plastic sheeting. The panels were demolded after 20 to 24 hours and moist cured for 14 days at 23 ± 1.7˚ C [73 ± 3˚ F]. After moist curing, the panels were cured in air at the same temperature and at 45% to 55% relative humidity for 14 days. A calcium chloride solution was ponded in the test panels to a depth of 6.4 mm [¼ in.]. The ponded area of each panel was determined. The panels were then subjected to 16 to 18 hours of freezing followed by 6 to 8 hours of thawing at 23 ± 1.7˚ C [73 ± 3˚ F] and 45% to 55% relative humidity. Every 5 cycles the panels were flushed, weighed, and the panels were inspected visually. The loose fines were also weighed. The cycling was continued to only 25 cycles because the panels had exceeded the terminal rating of 2.5 Equivalent Visual Rating (EVR). Panel mass loss in grams was accumulated and converted to a square foot basis. The mass loss was then converted to EVR. The conversion factor is one EVR= 65 g/ft^2/cycle. In Figure 5 is shown a typical salt scaling test specimen.

Fig. 5-Salt scaling specimen

Rapid Chloride Permeability

Permeability to chloride was determined in accordance with AASHTO T 277-96 by RDT personnel. Two concrete placements per mixture type were sampled and tested. For each placement, two 100 mm (4 in.) diameter, 200 mm (8 in.) long cylinders were cast, cured at the jobsite for 48 hours under site conditions of temperature, and transported to the RDT central laboratory where they were cured at 23 ± 1.7˚ C [73 ± 3˚ F] in a moist room until the time of testing. One cylinder was tested at 28 days and the other at 56 days. Three 50 mm (2 in.) thick slices from the top, middle and bottom portions of each cylinder were sawn. Chloride permeability is expressed in terms of total coulombs passed through the specimen in a 6 hour period. Calculations of the coulombs passed were based on 4.0 in diameter specimen. In Figure 6 is shown the rapid chloride permeability test equipment.

Fig. 6-Rapid chloride permeability device

Air Void Analysis

Concrete specimens in the hardened state were examined for air void system characteristics in accordance with ASTM C 457-98, Procedure A (linear traverse method) by RDT personnel. Two concrete field placements per mixture type were sampled and tested. Two 150 mm (6 in.) diameter, 300 mm (12 in.) long cylinders were cast, cured at the site for 48 hours under site conditions of temperature, and transported to the central laboratory where they were stored in a moist room at 23 ± 1.7˚ C [73 ± 3˚ F] at 100% humidity until the time of testing. Two cylinders from each concrete placement were prepared as follows: a vertical slice was taken, then sawed in half. One of the halve's flat surfaces was surface prepared. Operation of the linear traverse device and data collection

were done manually by RDT personnel. In Fig. 7 is shown the air void analysis linear traverse testing equipment.

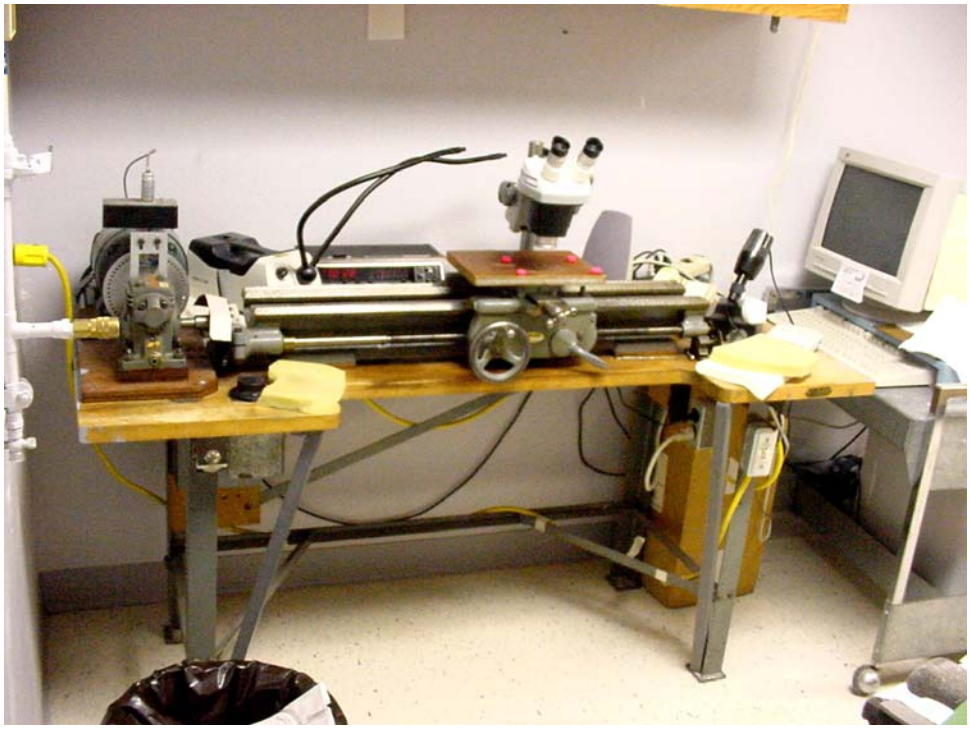

Fig. 7-Air void analysis station

Results

Resources

Information for this report was obtained from the following:

Data:
From MoDOT: RDT, District 6, Lonestar Cement, Fred Weber, Inc., Geotechnology, Inc., SCI Engineering, Inc.

Personal communication:

Patty Lemongelli	MoDOT RDT
David Amos	MoDOT RDT
Frank Reichart	MoDOT Construction and Materials Division
Gary Branson	MoDOT District 6
Bruce Kates	Jacobs Civil, Inc.
Chris Gottman	Fred Weber, Inc.

Field Testing: Plastic Concrete

Results of the field testing are shown in Table 9.

Table 9- Concrete placement field results

Date	Mix Type	Structure	Slump Average, [Range], mm (in.)	Air Content Average, [Range], %	Mix Temperature, Average, [Range], ˚C (˚F)
12-14-99 Am	Control	End Bent 10 Abutment seat	93 (3.66) [88-100] (3.46-3.94)	6.3 [5.5-6.9]	21 (70) [19-22] (66-72)
12-14-99 Pm	Control	“	“	“	“
3-23-00	Control	End Bent 1 Abutment	94 (3.70)	5.2 [4.9-5.7]	19 (66) [17-21] (63-70)
11-08-99	GGBFS without HRWR	W.B. Footing, Bent 3	94 (3.70) [88-100] (3.46-3.94)	5.3 [4.0-6.5]	20 (68) [17-23] (63-73)
12-02-99	GGBFS without HRWR	E.B. Footing, Bent 3	91 (3.58) [75-100] (2.95-3.94)	5.3 [4.0-6.9]	20 (68) [14-26] (57-79)
3-7-00	GGBFS without HRWR	E.B. Footing, Bent 4	98 (3.86) [95-102] (3.74-4.02)	5.9 [4.3-6.6]	24 (75) [18-27] (64-81)
11-05-99	GGBFS with HRWR	Bent 10, Toe wall	138 (5.43) [125-150] 4.92-5.91)	5.0 [4.6-5.6]	17 (63) [14-19] (57-66)
12-13-99	GGBFS with HRWR	Pier 9 W.B. columns	131 (5.16) [100-175] (3.94-6.89)	5.1 [4.6-6.2]	21 (70) [20-21] (68-70)
2-7-00	GGBFS with HRWR	E.B. Pier 3 columns, first lift	143 (5.63) [120-150] (4.72-5.91)	5.4 [5.2-5.5]	17 (63) [16-20] (61-68)
8-17-00	GGBFS with HRWR	Bent 8, column 3, first lift	161 (6.34) [137-182] (5.39-7.17)	6.1 [5.6-7.0]	29 (84) [28-31] (82-88)

From observation of Tables 3-5 and 9, several things can be noted. First, the slump of the B-1 and plain GGBFS mixes was about 95 mm [3 ¾ in.] while the slump of the GGBFS-HRWR mix averaged about 140 mm [5 ¾ in.]. It took more water to get the same slump for the plain GGBFS mix compared to the B-1 mix (averages of 156.6 vs 149.3 L, respectively), although air content was slightly lower for the GGBFS mix, thus the *w/cm* had to be increased (0.419 vs 0.399, plain GGBFS vs B-1). Water contents for the B-1 and the GGBFS-HRWR mixes averaged the same, thus *w/cm's* were the same. The results are somewhat surprising, as the literature indicates that water demand is typically less for slag mixes, although there have been some exceptions reported.

Required air dosage was higher for the plain GGBFS mix compared to the OPC mix. This is expected, as the literature indicates that required air dosage typically should increase with slag present, if the slag is finer than the portland cement. Of greater interest is the fact that the air dosage rate was higher in the GGBFS-HRWR mix than the other two mixes, yet average air content was a bit lower. This is especially surprising because HRWR's usually entrain additional air.

The presence of the HRWR explains why the slump is greater than the plain GGBFS mix, even with a lower water content in the HRWR mix.

Compressive Strength

Research, Development, and Technology Results

Measured compressive strength is a function of the mixture characteristics, degree of hydration at the time of test, curing conditions, specimen preparation, and test conditions. Mixture characteristics of significance include characteristics of each component, mixture proportions, and interactions among components. Completeness of mixing is included in the mixture characteristics category. Degree of hydration is a function of time and curing conditions. These two factors were kept constant for all types of mixes. Specimen preparation and test conditions are assumed to be essentially equivalent for all three mixture types in this study. Thus, differences in behavior for this study reduce to mixture characteristics and the interaction with curing conditions.

It is assumed that mix proportioning was done on an absolute volume basis, and that the increase in paste volume due to the combined effect of the substitution of slag for OPC on a equal weight basis coupled with the lower specific gravity of the slag (compared to OPC) was equally offset by a decrease in aggregate volume. Thus the actual mix design weights shown in Tables 3-5 are correct. Even so, the slag mixes will have a greater volume of paste (say, about 0.5 cf/cy). This would tend to increase strength, but to a limited degree. Air content can also be a factor in strength. The common rule of thumb is a 5% loss in strength for every 1% increase in air content. Finally, the single-most important factor governing strength is the *w/cm*. In this study, efforts were made to keep both air content and *w/cm* constant.

Characteristics of the components include chemical and physical characteristics of the cementitious materials, aggregate characteristics, and presence of HRWR. The type of

aggregate materials were held constant among mixes. HRWR effects will be discussed below. Effect of cementitious materials is the focus of this study.

Compressive strength results for the three types of mixes are shown in Table 10.

Table 10. Compressive strength of mixes sampled by RDT

	Compressive Strength (psi)						
Mix	**Date Cast**	**7 Days**	**14 Days**	**28 Days**	**56 Days**	**90 Days**	**365 Days**
B-1	12-14-99 am	4430		5710	6080	6380	6990
	12-14-99 pm	4340		5520	5930	6180	6950
	3-23-00			4800	5130		
	Average	4385		5343	5713	6280	6970
GGBFS	11-8-99	2620		3800	4060	3950	4500
	12-2-99	3120		4590	4750	5010	5520
	3-7-00			4020	4140	4310	
	Average	2870		4137	4317	4423	5010
GGBFS HRWR	11-5-99	2610		3470	3760	4010	4520
	12-13-00		4830	5560	5970	6140	6770
	2-7-00			5580	5980		
	8-17-00		3720	3990	4130		
	Average	2610	4275	4650	4960	5075	5645

Compressive strength results for individual placements are shown in Figs. 8-10.

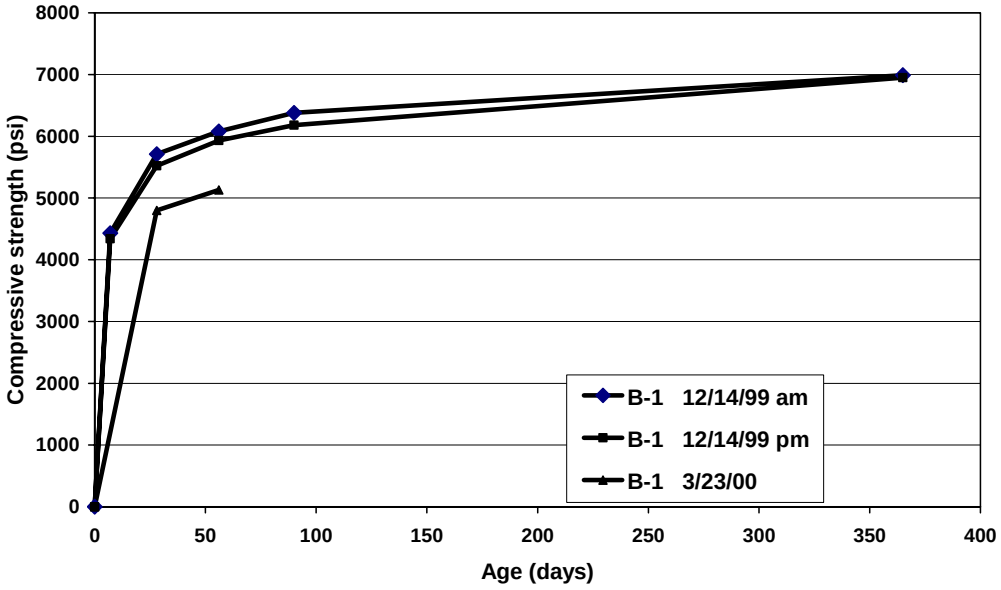

Fig. 8-Compressive strength of B-1 mixes

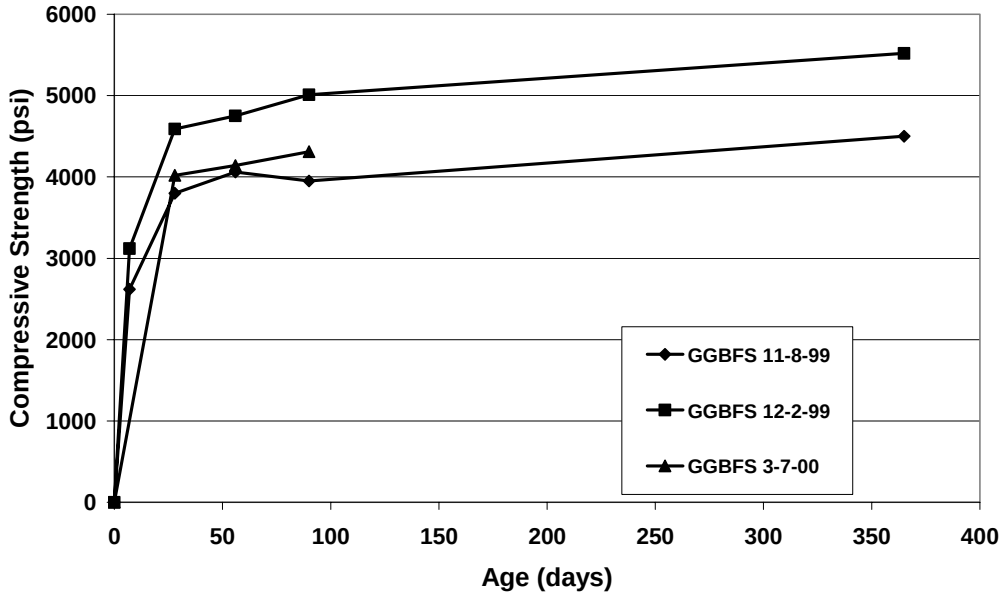

Fig. 9-Compressive strength of GGBFS mixes without HRWR

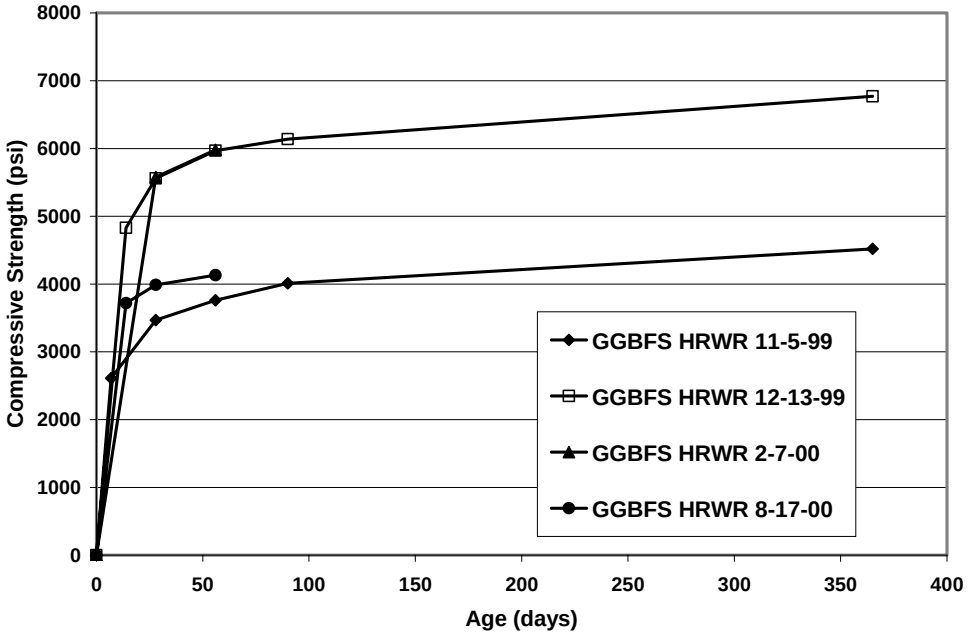

Fig. 10-Compressive strength of GGBFS mixes with HRWR

As can be seen, there are significant differences from placement-to-placement. Fig. 11 shows the average strength gain for each mixture type.

As expected, the GGBFS mixes exhibited slower strength gains at early ages (7 days). This behavior can be explained by the slower pozzolanic reaction of the slag compared to normal portland Type I cement. Also, the slag mixes contained Type II cement, which also typically exhibits lower early strengths. Two of the GGBFS mixes with HRWR were on par with the control mix by day 28. However, results at 90 and 365 days show that the strength of all the slag mixtures remained below the control mix.

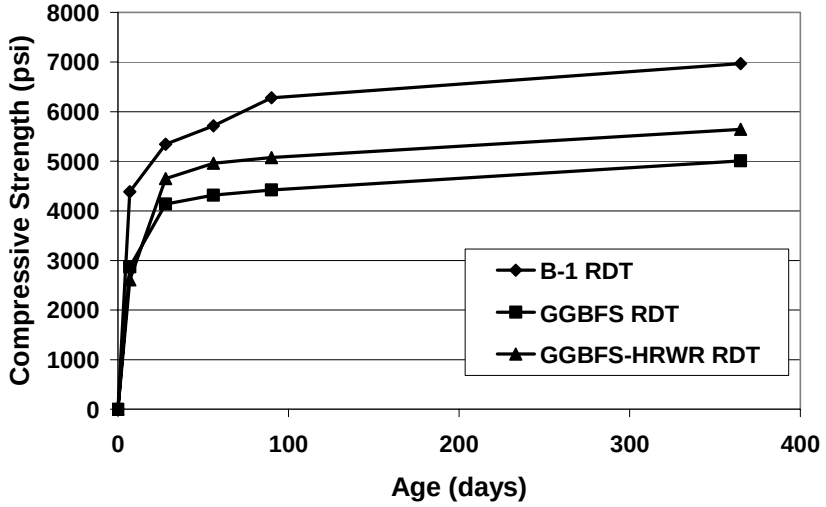

Fig. 11-Average compressive strength of all mixes

The GGBFS mix with HRWR did result in higher strengths than the slag mix containing no HRWR. This is expected because of the superior microstructure that results from using HRWR.

Other mixture proportion factors affect strength besides the presence of pozzolans and type of cement. Fig. 12 depicts the effect of *w/cm* on 56 day strengths. As shown, there is considerable scatter in the results. Worse yet, within each type of mix, the traditional *w/cm*-strength relationship is not evident. In fact, the trends for the control mix and the plain GGBFS mix are actually backwards. Finally, the range of *w/cm's* is small, but the range in strengths is large. Thus, other factors are at play and/or the *w/cm's* are inaccurate.

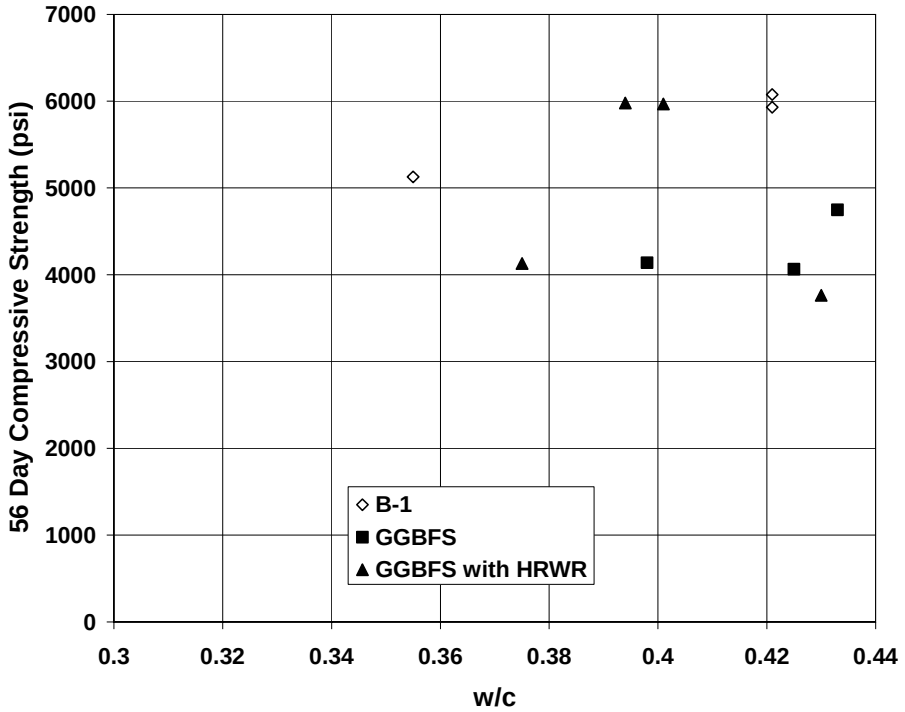

Fig. 12-Relationship of w/cm and 56 day compressive strength

In regard to air content, usually the presence of air lowers strength. The average plastic concrete field air content for the B-1 mix was actually a bit higher than the two slag mixes, so air content cannot explain the greater strength of the B-1 mix. Fig. 13 shows the relationship between field-measured air content of plastic concrete and strength in this study. Apparently, the effect of air content is not a significant factor in these mixes, or else the air content values are inaccurate. However, it should be noted that the air contents shown are averages for a particular day, rather than actual air measurements on the sample from which the cylinders were cast. Batch-to-batch air content varied through the day by as much as ± 1.0%.

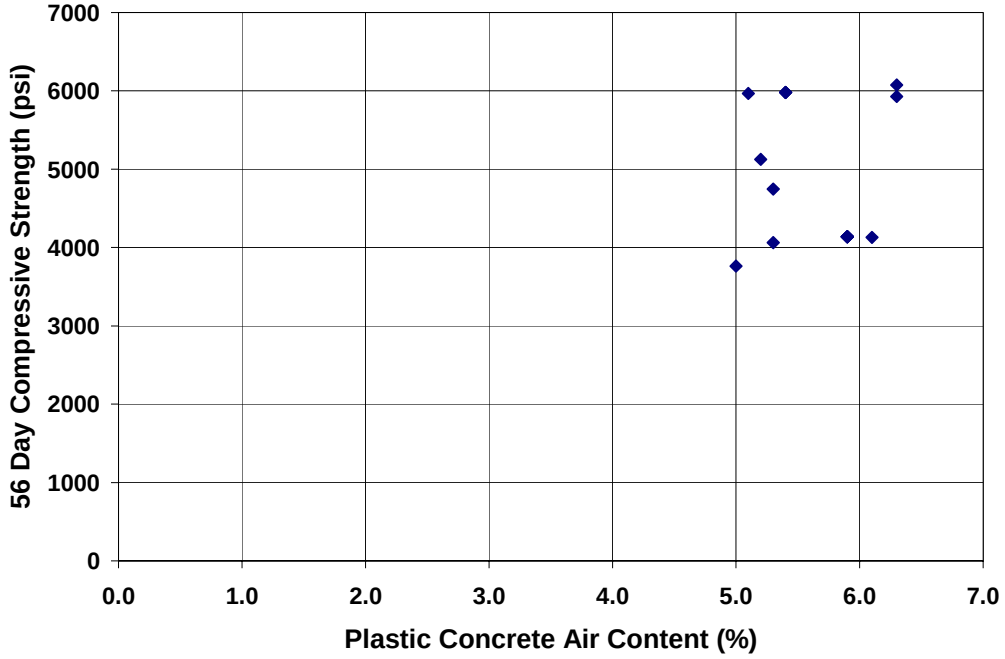

Fig. 13-Relationship of field-measured air content and 56 day compressive strength

Early in the project there was concern expressed over the coarseness of the air system in the HRWR mixes and that air content should be increased to assure freeze-thaw protection. However, examination of field inspectors' reports indicate that the air entraining agent dosage was not much different between the plain and GGBFS-HRWR mixes, and corresponding plastic concrete air contents were about the same (5.4% for the HRWR mixes and 5.5% for the plain GGBFS concrete). Average air content for the B-1 mixes was 5.9%. Hardened concrete air content results are shown in Fig.14. As air content increased, the strength in both the slag mixes decreased. But, the opposite was true for the B-1 mix. Thus, the effect of air content is not well established for this data. Additionally, assuming a loss in strength of 5 percent for every one percent air, the loss in strength shown in Fig. 7 is considerably in excess. Thus, the loss in strength is not explained by air content increase alone.

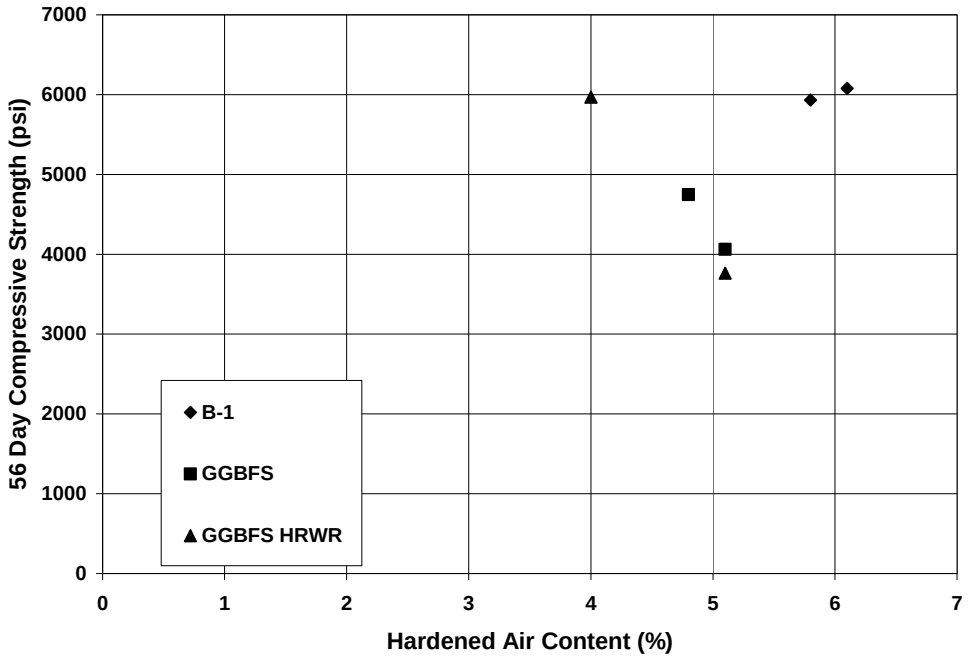

Fig. 14- Effect of hardened air content on compressive strength

Thoroughness of mixing was a concern early in the project, especially when compressive strength results seemed to be excessive. Also, variability of the uniformity of cementitious materials was of concern. However, there are no concrete uniformity tests on record, and no examination of cement and slag mill reports was reported.

Fig. 15 is a plot of slag and OPC variations in properties as reported by the producer for the years 1999 and 2000.

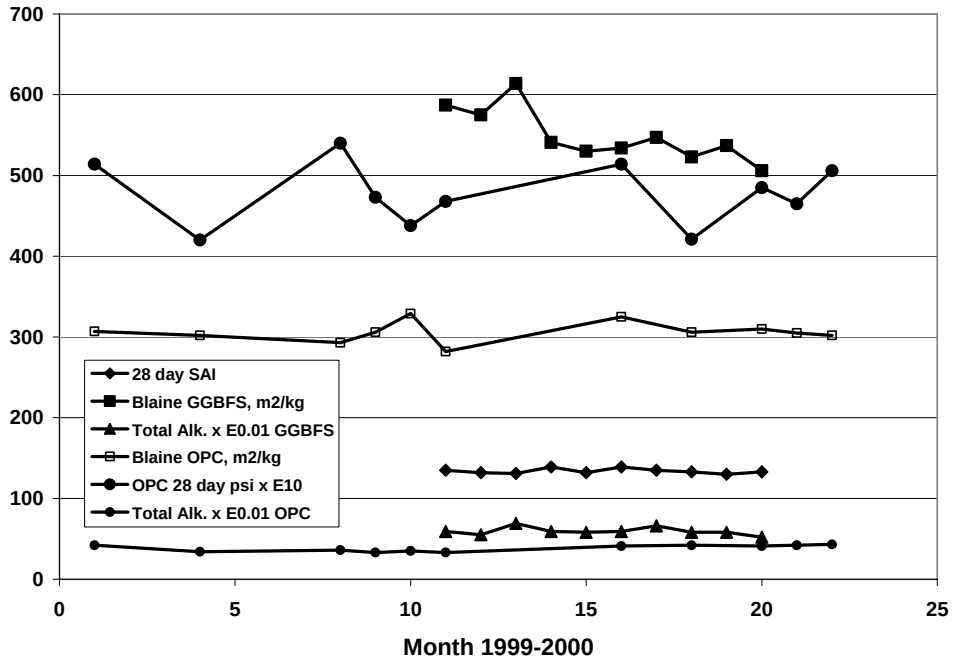

Fig.15- Variation in GGBFS and OPC properties

It appears that the Slag Activity Index was well above the required minimum of 115% (as stated in ASTM 989) and variation was not pronounced. However, the OPC used for the SAI quality control tests may not be the same as the OPC used in this project. The Type II low heat PC in this project had a relatively low total alkali content (average of 0.39%). For the determination of slag activity, ASTM C 989 requires that the total alkali content of the reference cement be between 0.6% and 0.9%. In addition to the low alkali content of the PC, GGBFS typically has a low alkali content, as did the one in this study. Thus, the low alkali content in the cementitious system could explain a lower reactivity between the slag and the PC used on this project, which could lead to lower strengths.

Another variable that could affect strength is curing condition. The cylinders were brought into the lab after 48 hrs in the field and cured under standard laboratory conditions, so the only differences in curing would have been during the first 48 hours. Attempts were made to protect the specimens from freezing in the field. The temperature of the specimens during field curing was not monitored, however, no evidence of freezing damage was observed. Unless freezing damage occurred, it is not believed that 48 hours at moderately different temperatures would affect 56 day and older measured strengths.

It is assumed that differences in conditions of testing were essentially insignificant, although numerous differences could have occurred, such as surface moisture condition of the specimens at the time of testing and the uniformity of the condition of the cylinder capping system.

District 6 and Consultant Results

During the course of the project, strength tests were performed by both MoDOT District 6 and SCI Engineering personnel. Fig. 16 is a plot of 28 day strength sets (mostly averages of two replicates) over a two year period. Also shown is the initial test mix strength achieved by Geotechnology, Inc.

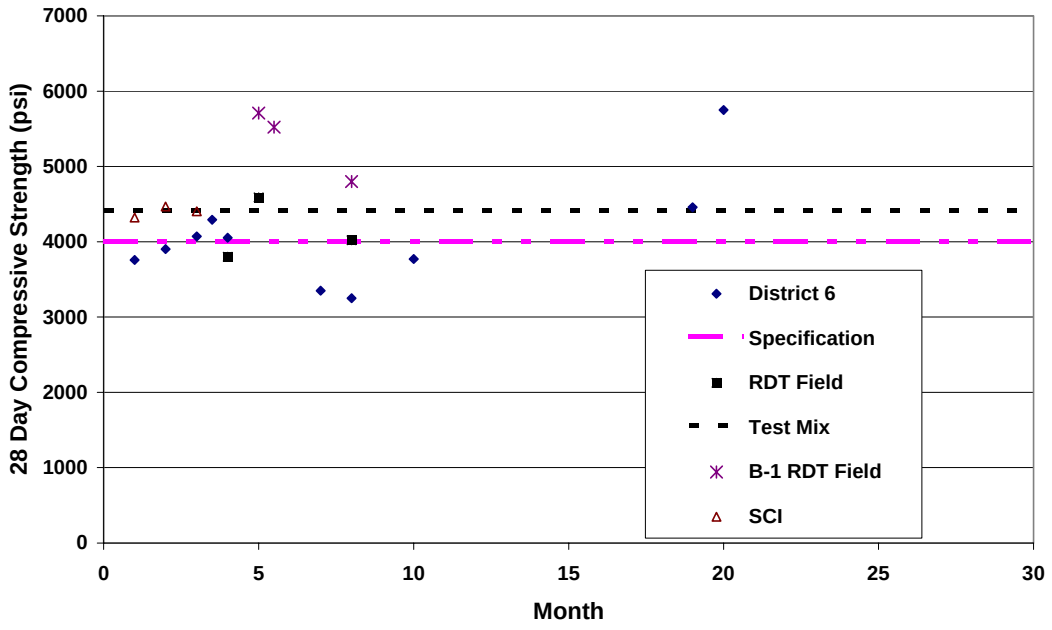

Fig.16- Field compressive strength test results, GGBFS mix (no HRWR) 1999-2001

Several things are apparent from the plot. First, the results were quite variable, ranging from about 22 to 40 MPa [3200 to 5800 psi]. A significant percent of the test results were lower than the required 28 MPa [4000 psi]. District 6 results tended to be lower than the others. This may have been due in part to the manner in which the cylinders were cured. The specimens were left in their molds in a building and thus were not moist cured or temperature regulated. At some point, this procedure was changed to include curing in a temperature-controlled water tank. The second construction season showed higher strengths, although there were only two data points involved. During the first season, all of the slag mixes were weaker than the B-1 mixes. Very few of the slag mixes met or exceeded the mix design test mix strengths.

Fig. 17 is a similar plot, showing the GGBFS mixes with HRWR.

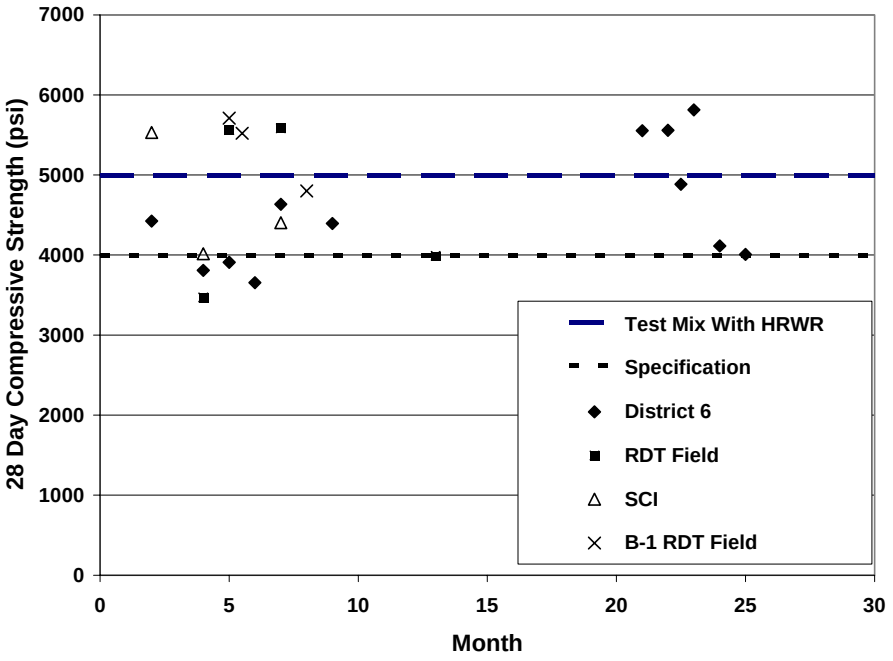

Fig. 17- Field compressive strength test results, GGBFS mix (with HRWR) 1999-2001

Similar trends are noted. However, the first season included some higher strength values as well as the second season, and, there were several slag mixes that equaled or exceeded two of the three B-1 mix strengths.

Fig. 18 shows a comparison of GGBFS mixes with and without HRWR for all data. The HRWR mixes tend to be greater in strength than the non-HRWR mixes (average of 32 and 28 kg/m^3 [4595 and 4126 psi], respectively).

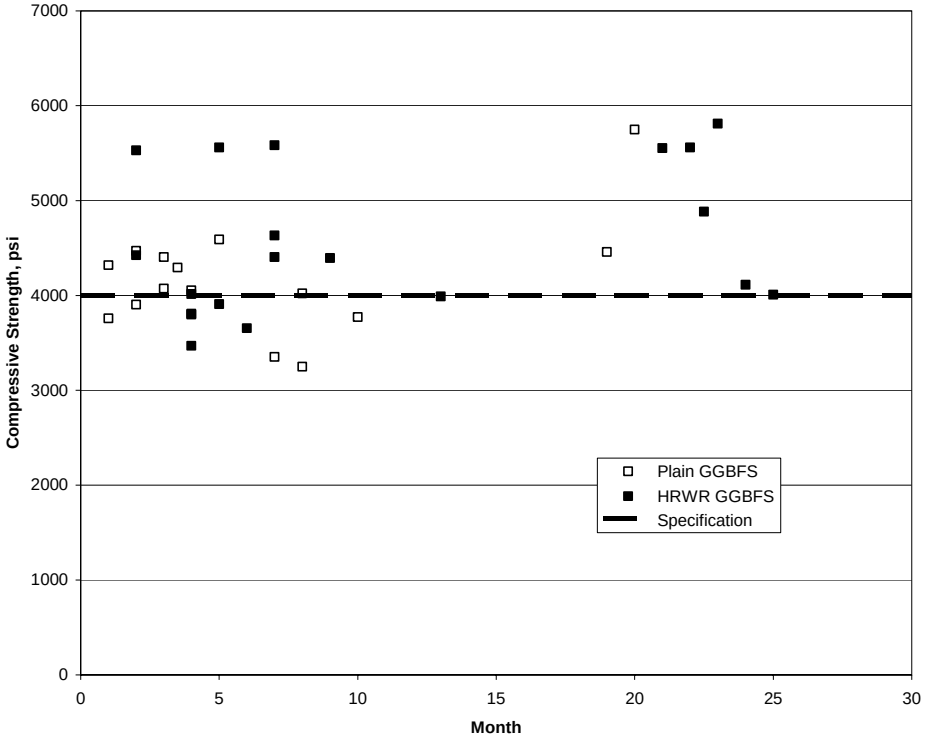

Fig. 18- Effect of HRWR on compressive strength

Freeze-Thaw Durability

The results of freeze-thaw testing are shown in bar charts in Figs. 19-21. The original wet curing time was 35 days with no subsequent drying period. Fig. 19 shows that the Durability Factor of the B-1 mix was significantly greater than both slag mixes. The B-1 mix met the recommended minimum of 90, but both slag mixes were significantly lower. The slag mix with HRWR average DF was less than the non-HRWR slag mix, but the averages were based on only two or three concrete placements (average of three replicates per test) with a significant amount of scatter in the test results.

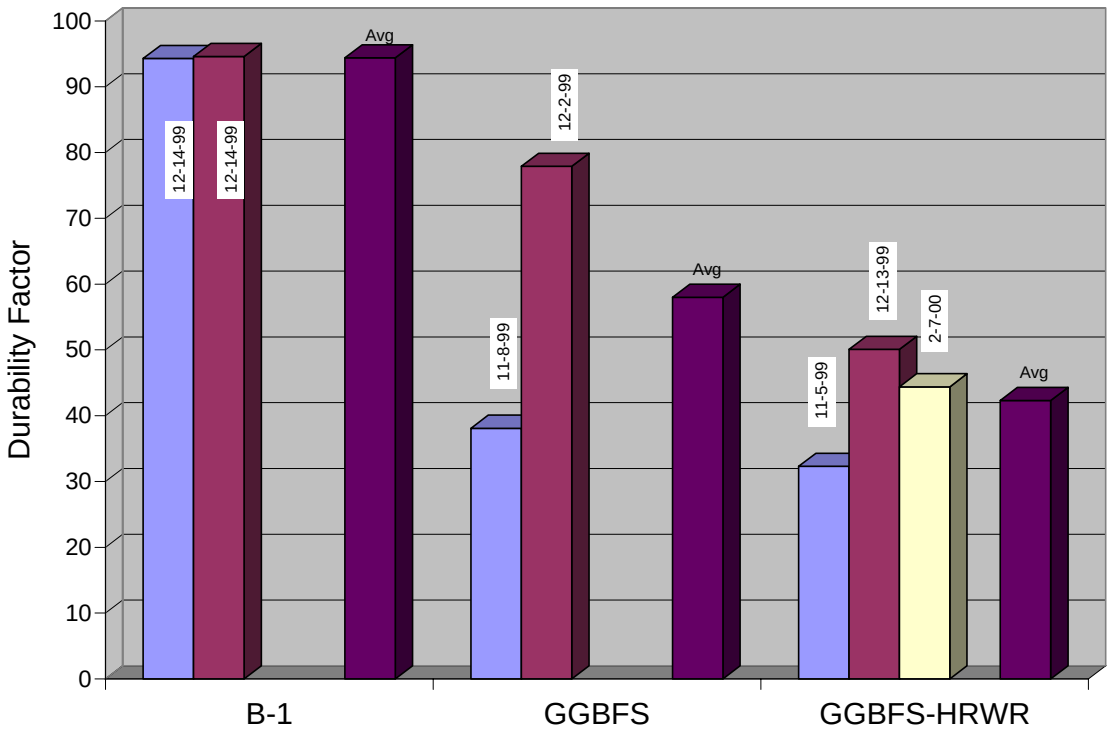

Fig. 19-Durability Factors for all three mix types-35 days wet curing

The effect of curing interval was explored to see if a longer curing time for GGBFS mixes would increase durability. Fig. 20 shows that increasing wet curing time from 35 to 56 days had little effect.

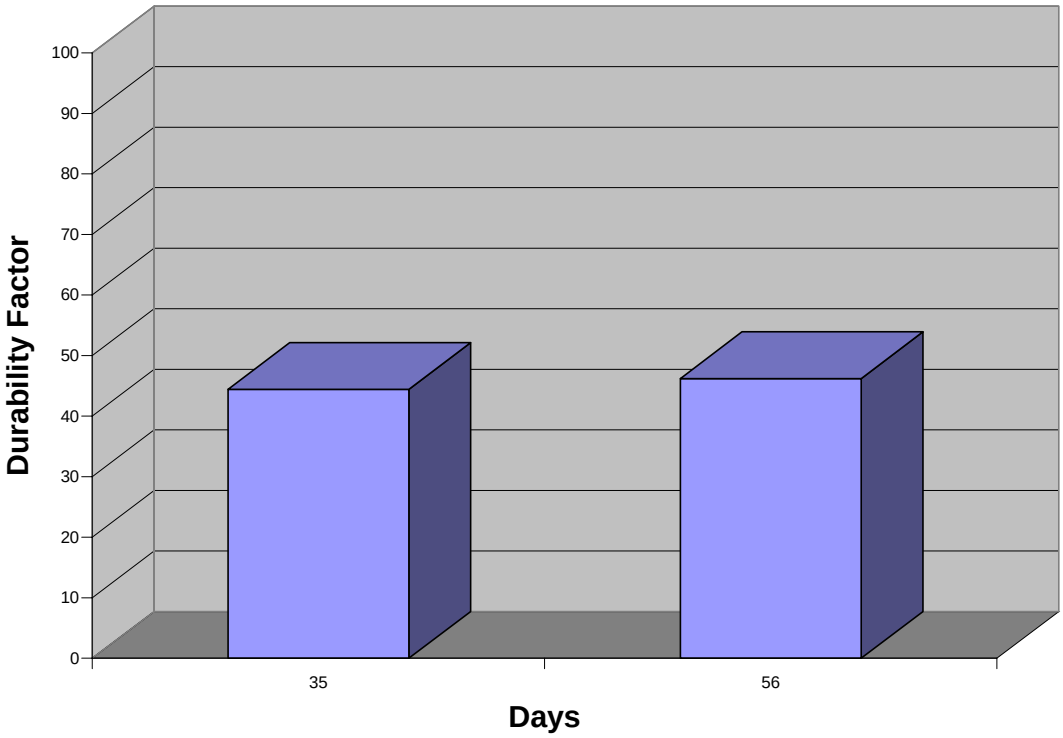

Fig. 20- Effect of additional wet curing time on Durability Factor-GGBFS/HRWR mix, cast 2-7-00

It appears from Fig. 21 that adding a subsequent 7 day drying period to a 56 day wet curing period does little to benefit tested durability.

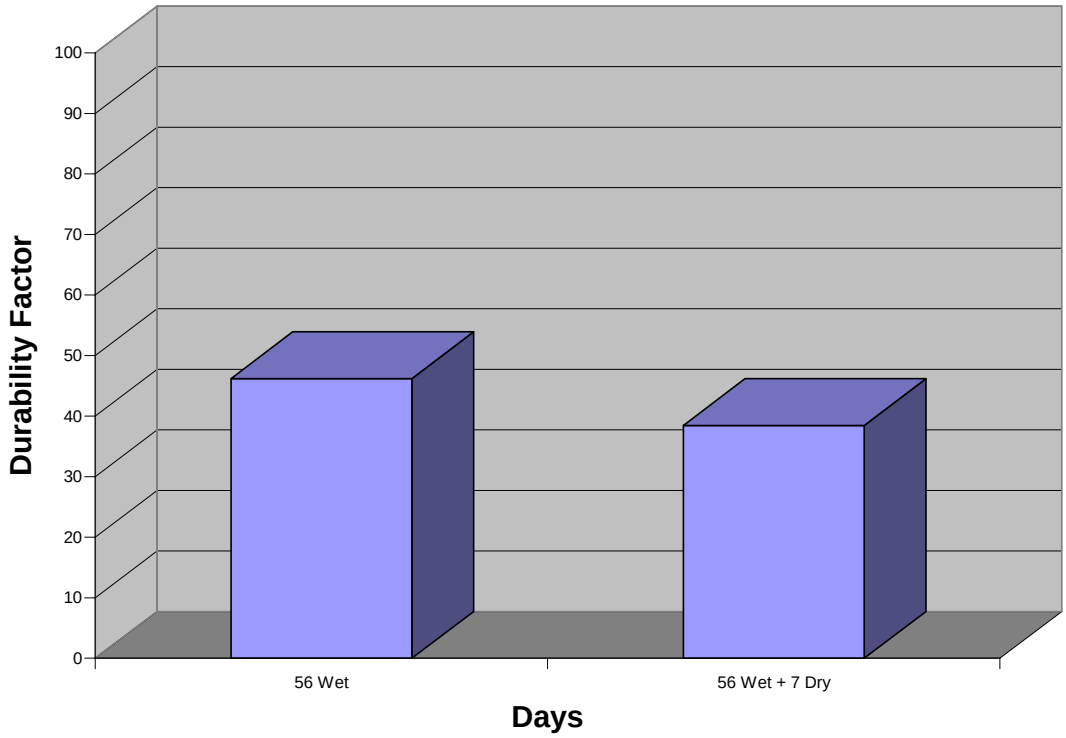

Fig.21- Effect of additional 7 day drying period on Durability Factor-GGBFS/HRWR mix, cast 2-7-00

Fig. 22 shows that the effect of replacement of the last 7 days of a 97 day wet curing period with 7 days of dry curing again seemed to make little difference in the Durability Factor.

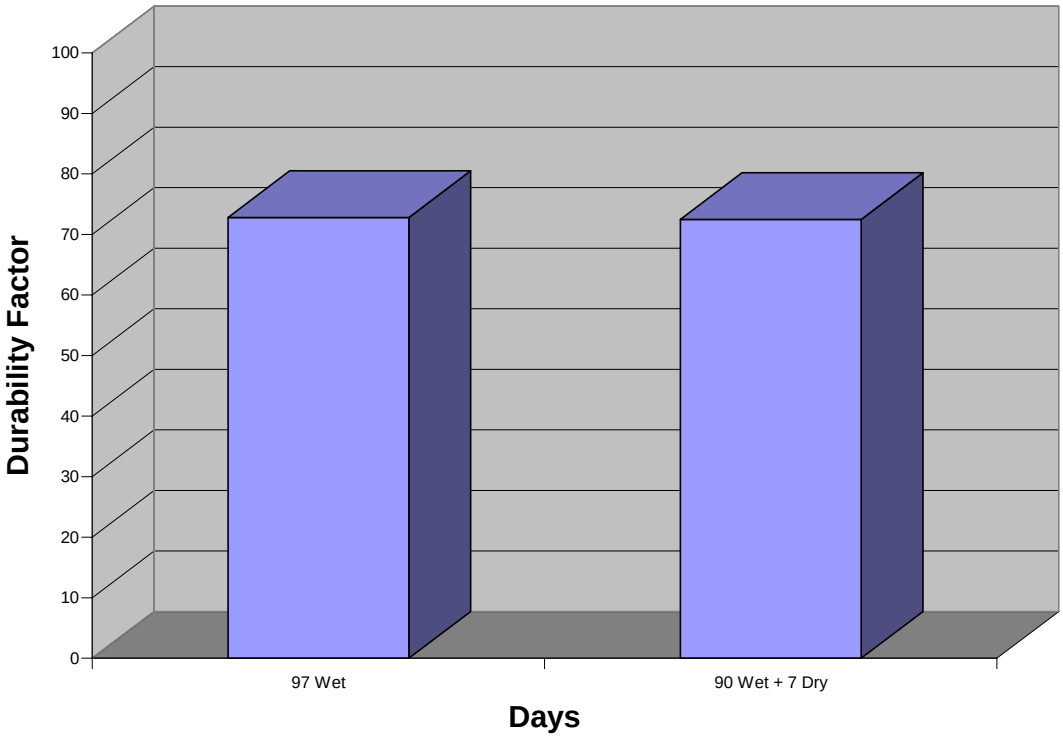

Fig. 22-Effect of replacement of 7 days wet curing with 7 days drying on Durability Factor-GGBFS mix, cast 3-7-00

And, the effect of a combination of increased wet curing time plus a subsequent 7 day drying period again made little improvement, as seen in Fig. 23.

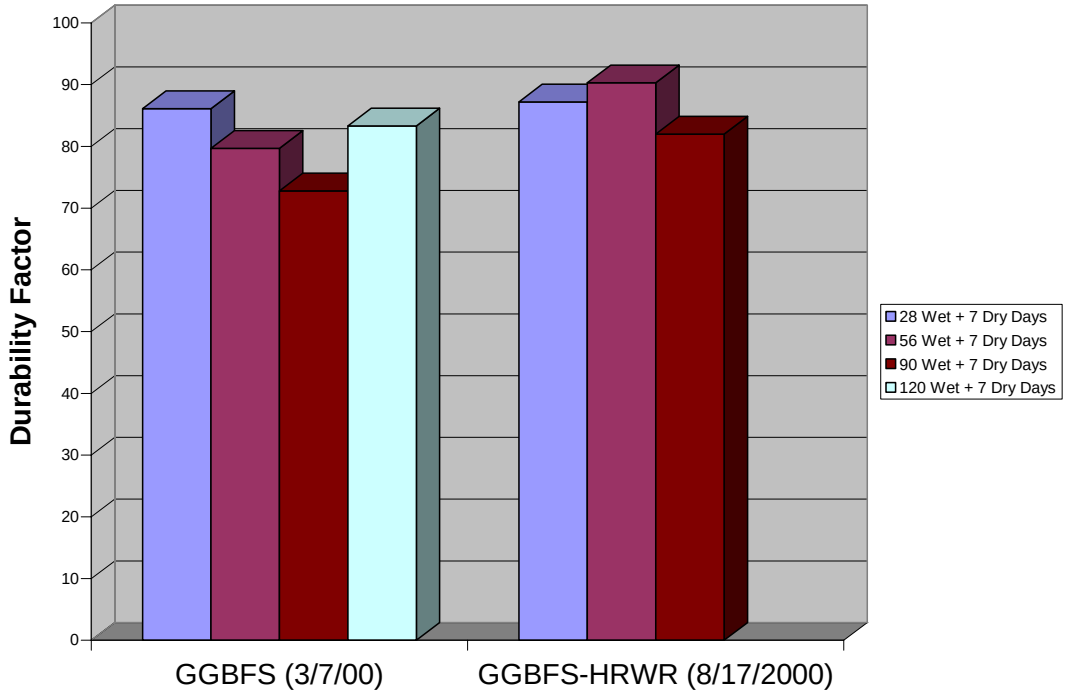

Fig. 23-Effect of increased wet curing period with subsequent 7 day drying period on Durability Facto

Looking at Figs. 19 to 23, when comparing the various curing regimes across different sets, in four out of six cases, longer wet curing periods did not increase DF; in two out of two cases, additional 7 day drying periods did not increase DF; but in five out of seven cases longer wet curing periods followed by a 7 day dry period did increase DF. However, the results are difficult to assess because these are different sets and had no common control specimens. Table 11 shows the minimum and maximum Durability Factors for each mix type and associated curing regimes. The median values reflect all curing methods. Note that under optimum curing conditions, the slag mix durability approached that of the OPC mix.

Table 11-Minimum, maximum and median Durability Factors

Mix	**DF, min.**	**Curing Mode, min. (days)**	**DF, max.**	**Curing Mode, max. (days)**	**DF, med., all methods**
B-1	94*	35 wet	94*	35 wet	94
GGBGS	58**	35 wet	86**	28 wet + 7 dry	78
GGBFS-HRWR	42***	35 wet	90***	56 wet + 7 dry	48

* single placement
** 2 placements
*** 3 placements

Correlations of durability factor, weight change, length change, and strength are shown in Figs.24-27.

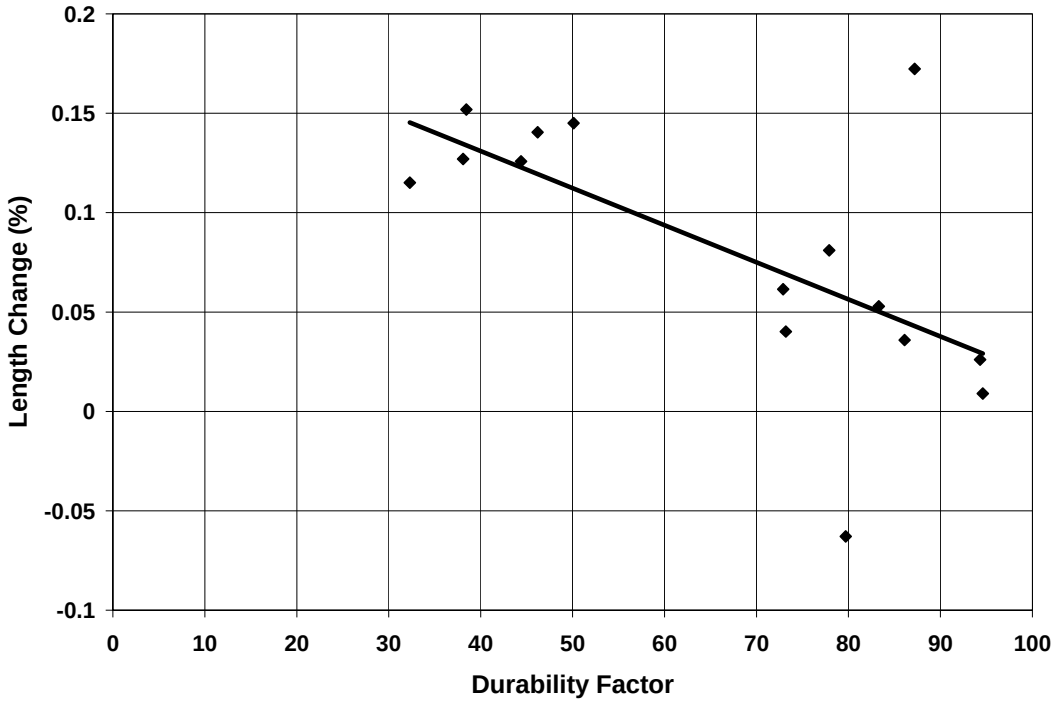

Fig. 24-Relationship of Durability Factor and length change

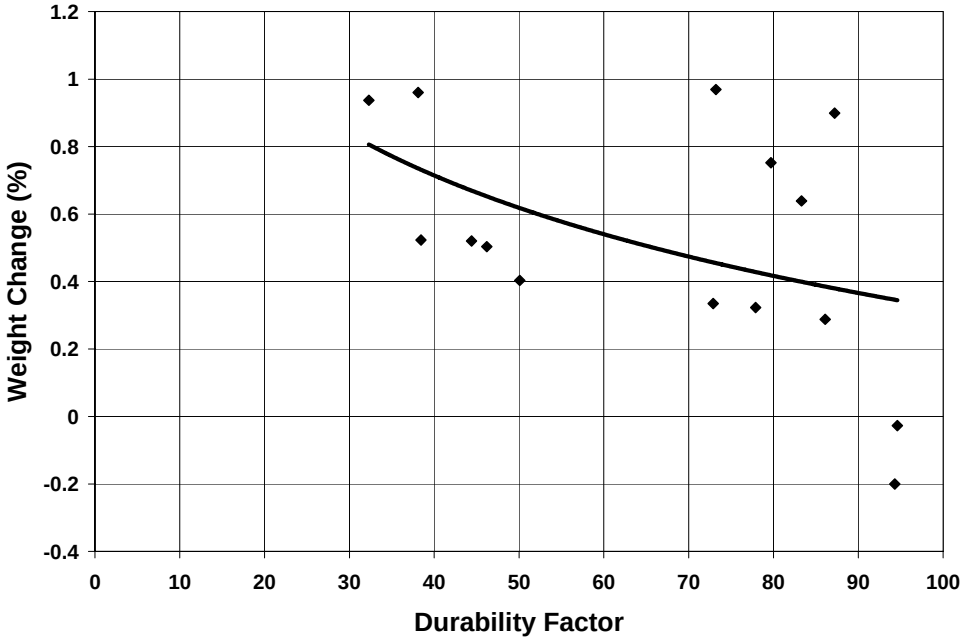

Fig. 25-Relationship of Durability Factor and weight change

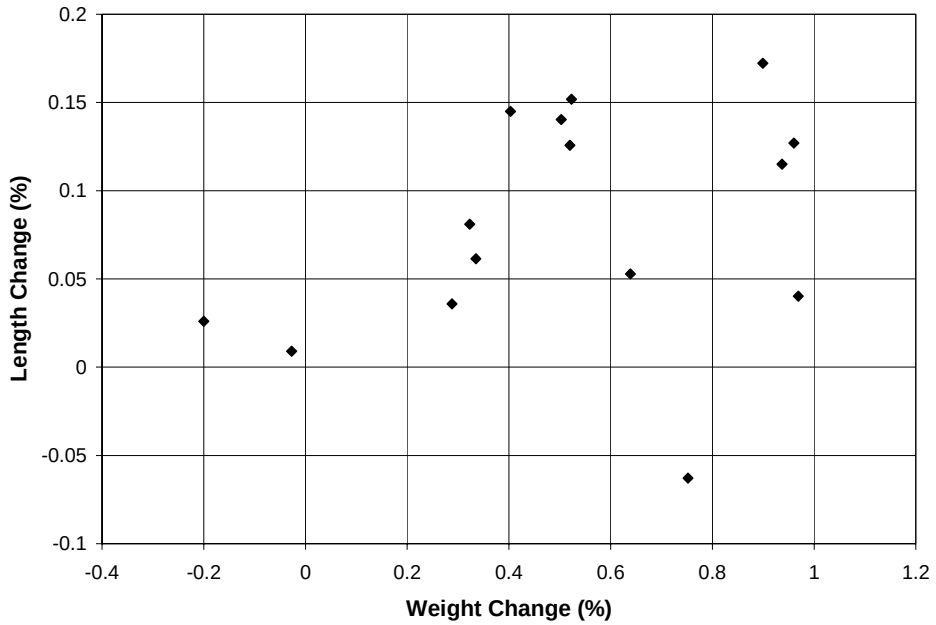

Fig. 26-Relationship of weight change and length change

As can be seen, if all methods of curing are included, the different types of durability measurements correlate poorly with each other. And, as seen in Figs. 27-29, various measures of durability do not correlate well with compressive strength.

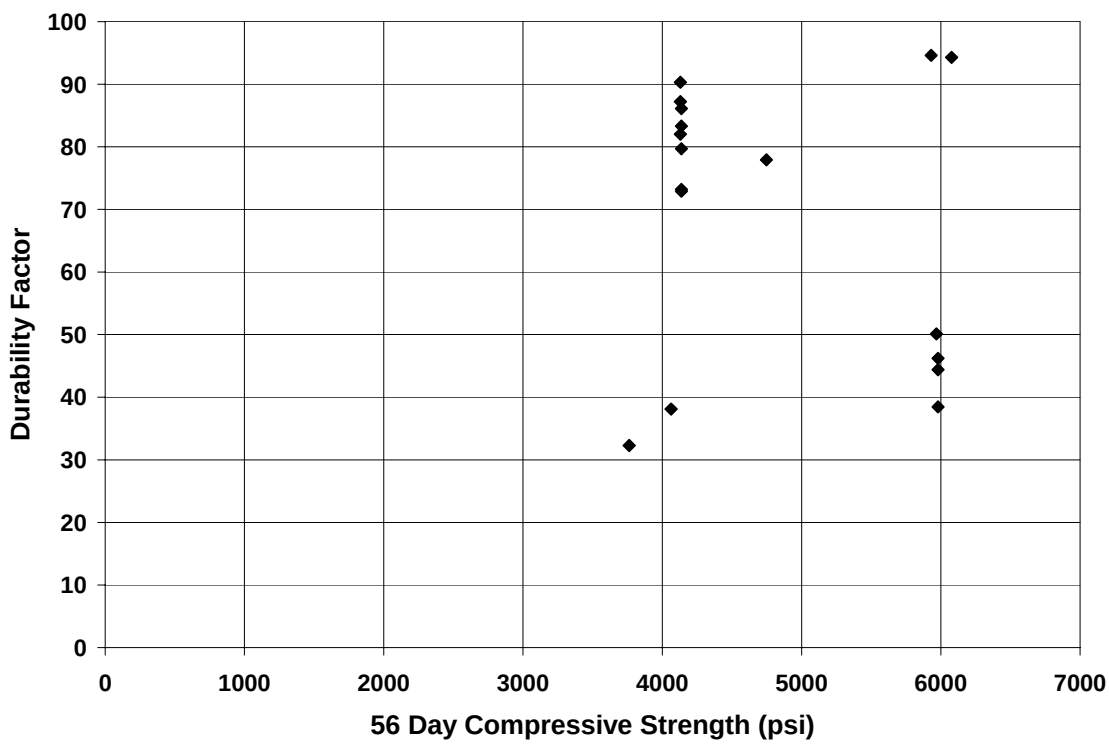

Fig. 27-Relationship of 56 day compressive strength and Durability Factor, all methods of curing

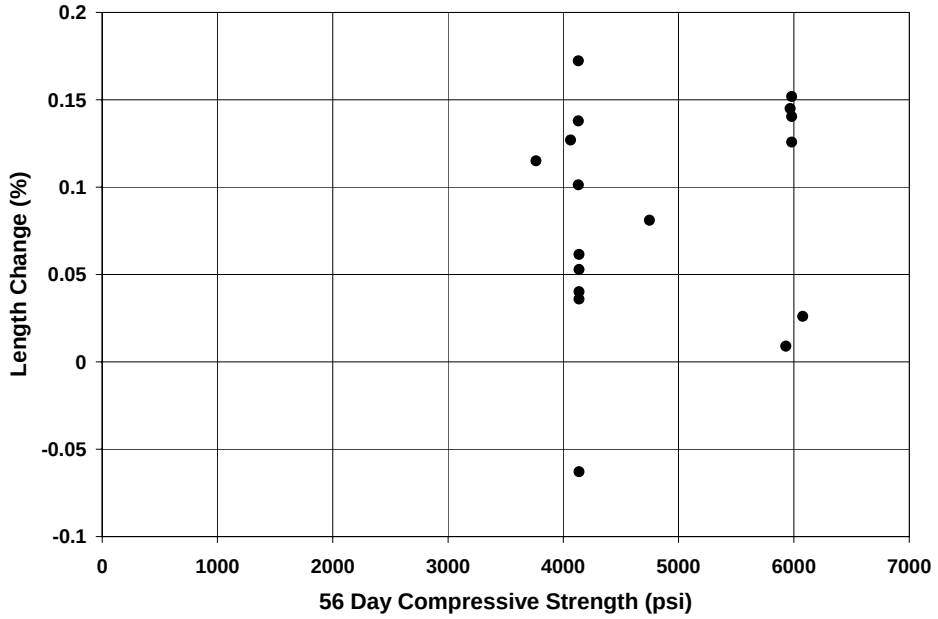

Fig. 28-Relationship of 56 day compressive strength and length change, all methods of curing

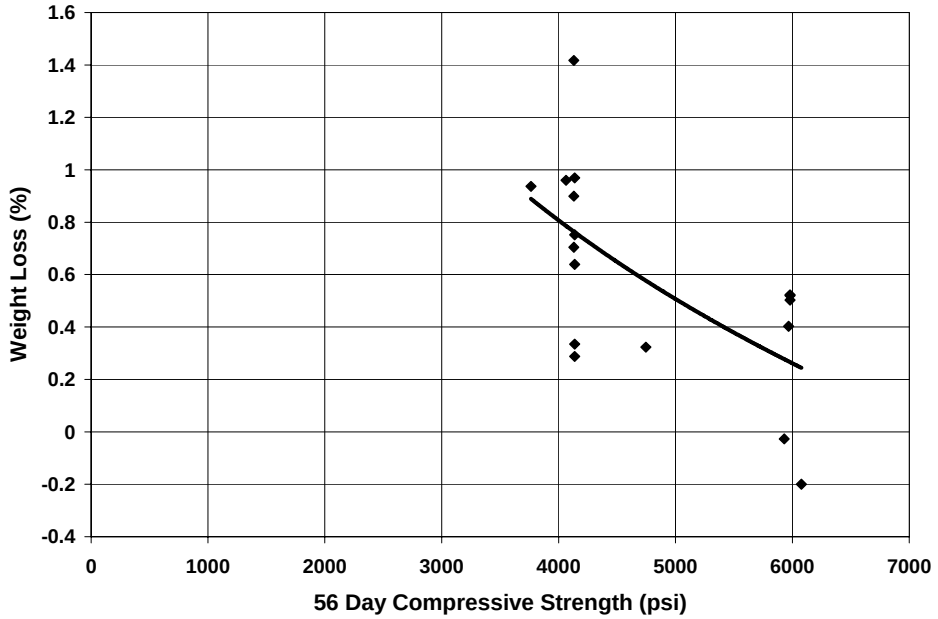

Fig. 29-Relationship of 56 day compressive strength and weight change, all methods of curing

However, as shown in Figs.30-32, correlations of various durability measures with a common 35 day curing period show much stronger relationships with each other.

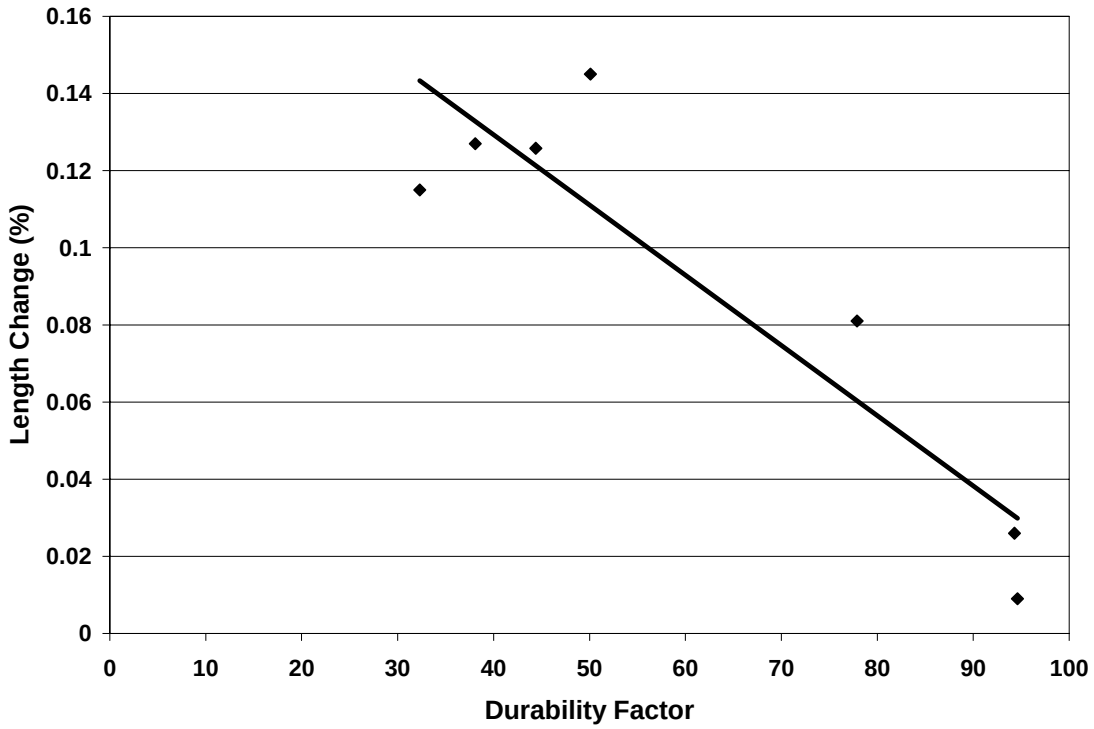

Fig. 30-Relationship of Durability Factor and length change, 35 day wet curing

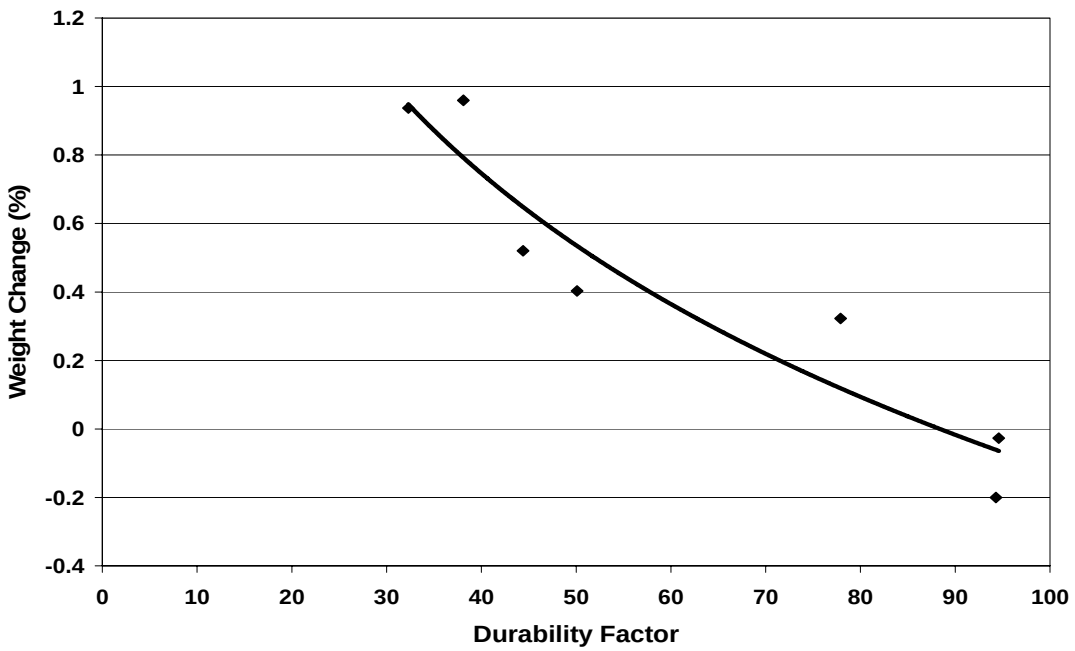

Fig. 31-Relationship of Durability Factor and weight change, 35 day wet curing.

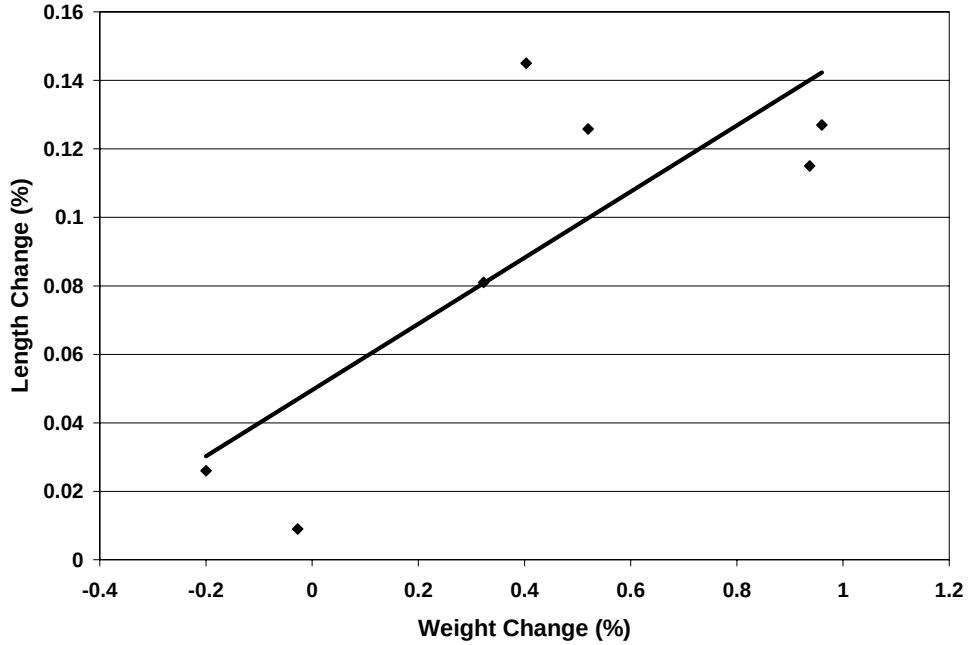

Fig. 32-Relationship of weight change and length change, 35 day wet curing.

But, even by limiting the data set to include durability specimens with a common curing type, compressive strength test result correlations with durability measures are only fair to poor, as seen in Figs. 33-35.

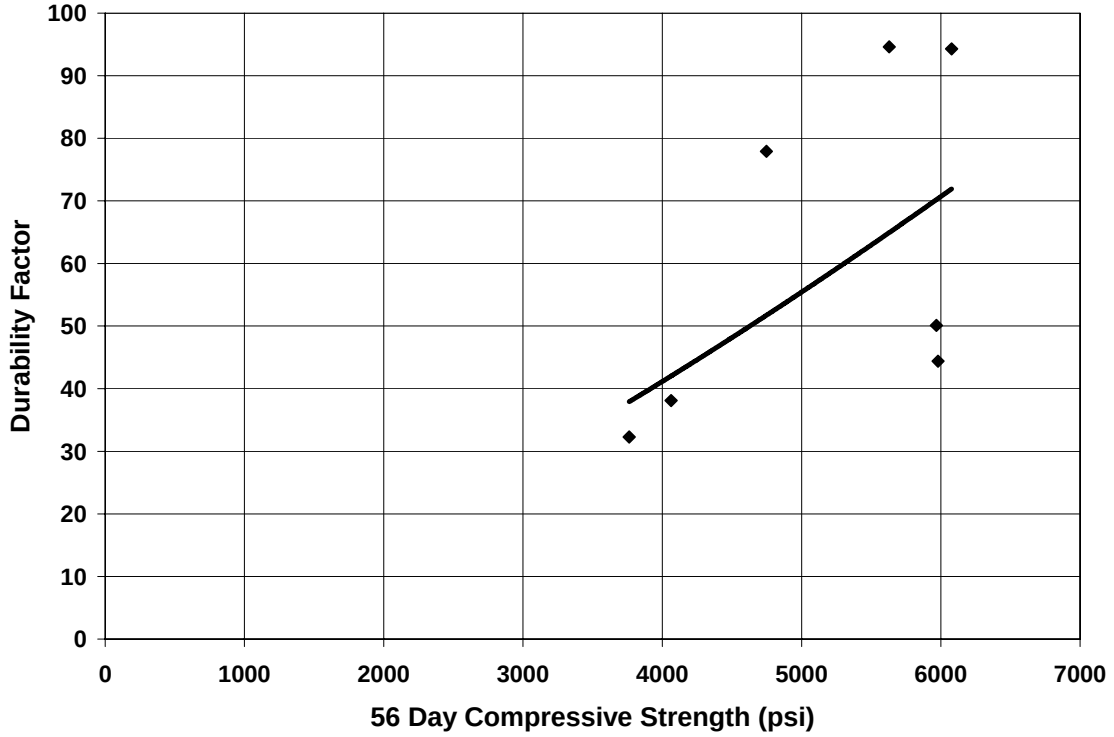

Fig. 33- Relationship of 56 day compressive strength and Durability Factor, 35 day wet curing for durability specimens

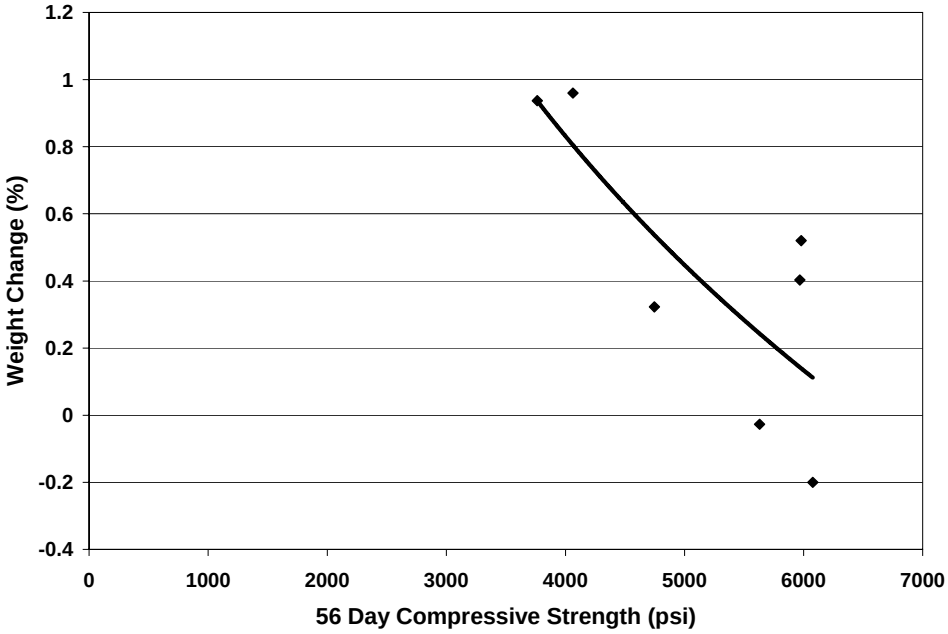

Fig. 34- Relationship of 56 day compressive strength and weight change, 35 day wet curing for durability specimens

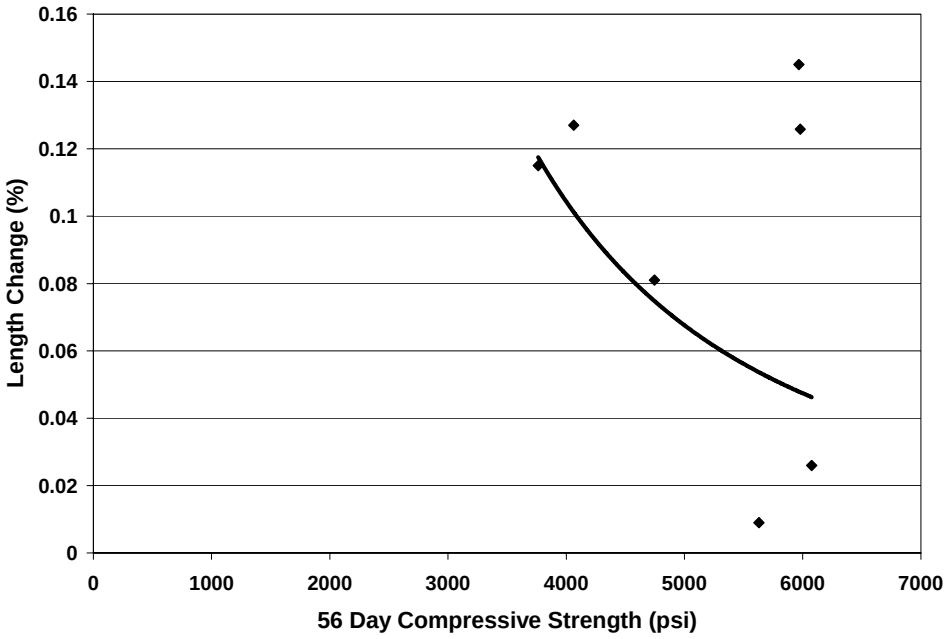

Fig. 35- Relationship of 56 day compressive strength and length change, 35 day wet curing for durability specimens

Salt Scaling

The results of the salt scaling testing are shown in Fig. 36. Larger EVR values indicate greater scaling. The desirable maximum EVR level is two. It can be seen that the B-1 mix met the level easily, while both slag mixes failed. The EVR rating for the B-1 mix is significantly smaller than the slag mixes, with the GGBFS-HRWR mix somewhat greater than the plain GGBFS mix. The plain GGBFS mix (cast 3-7-00) was a relatively low strength batch. However, the GGBFS-HRWR batch (cast 2-7-00) exhibited a rather high strength, thus there does not seem to be a strong connection between salt scaling and strength in this study. Overall, the low salt scaling resistance results were expected, as most researchers have found a decrease in resistance in laboratory studies of slag mixes.

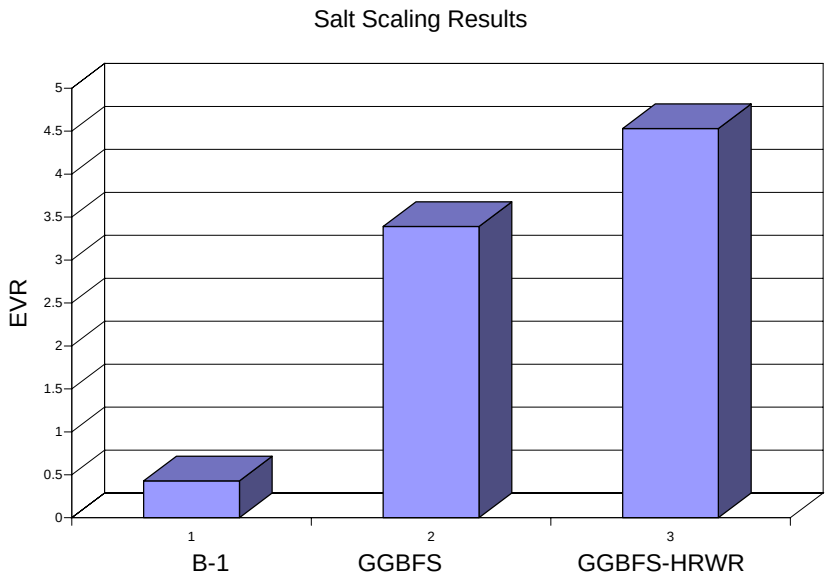

Fig. 36-Results of salt scaling testing

Rapid Chloride Permeability

Results of the rapid chloride permeability testing are shown in Fig. 37. Both slag mixtures exhibited significantly lower permeability values than the B-1 mix at both 28 and 56 days. This was expected due to the pozzolanic reaction resulting in a tighter microstructure. Both GGBFS mixes averaged about the same. However, the lowest permeability mix was the mix with HRWR which is expected because of the better quality microstructure that results from the use of a HRWR. The average 56 day coulomb values of both slag mixes were in the 1075 to 1135 coulombs passing range, which would be rated as low permeability according to AASHTO T 277, while the B-1 mix average value of 6895 coulombs passing would be rated as high.

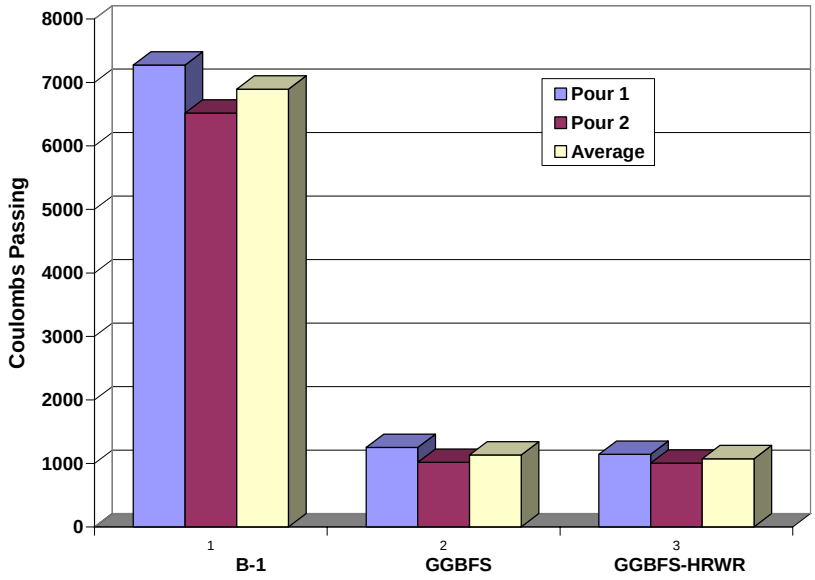

Fig. 37-Results of rapid chloride permeability testing at 56 days.

Air Void Analysis

The results of the air void analysis are shown in Figs. 38-42.

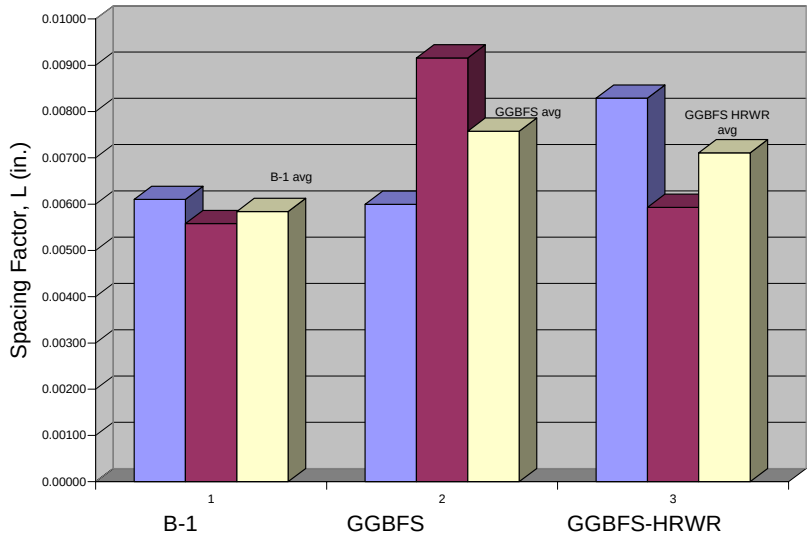

Fig. 38-Spacing Factor of air void systems.

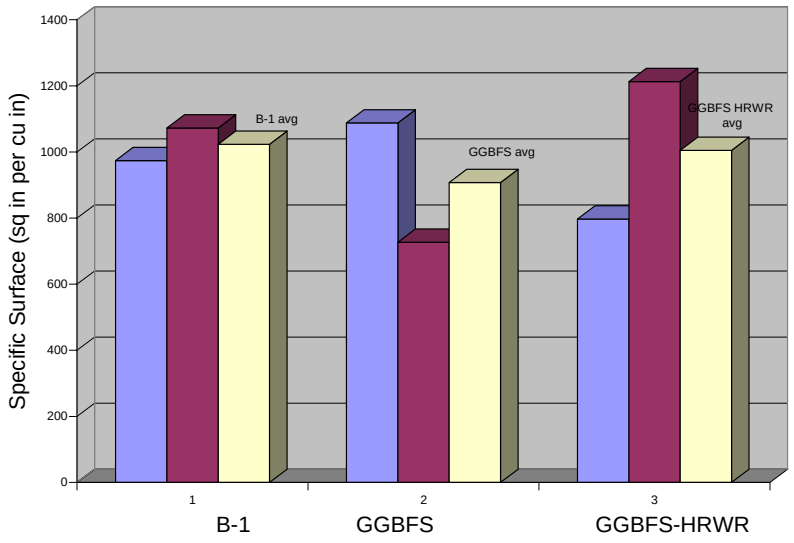

Fig. 39-Specific surface of air systems.

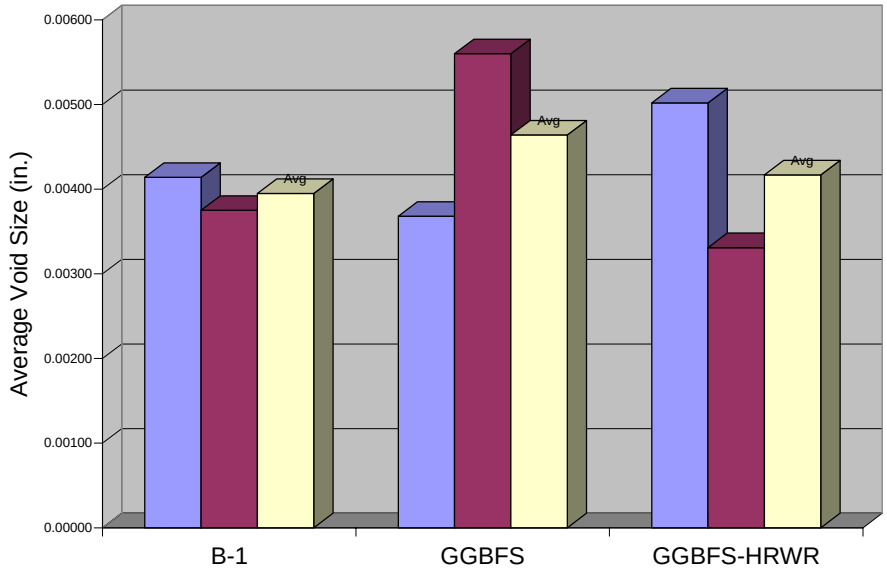

Fig. 40-Average void size of air systems.

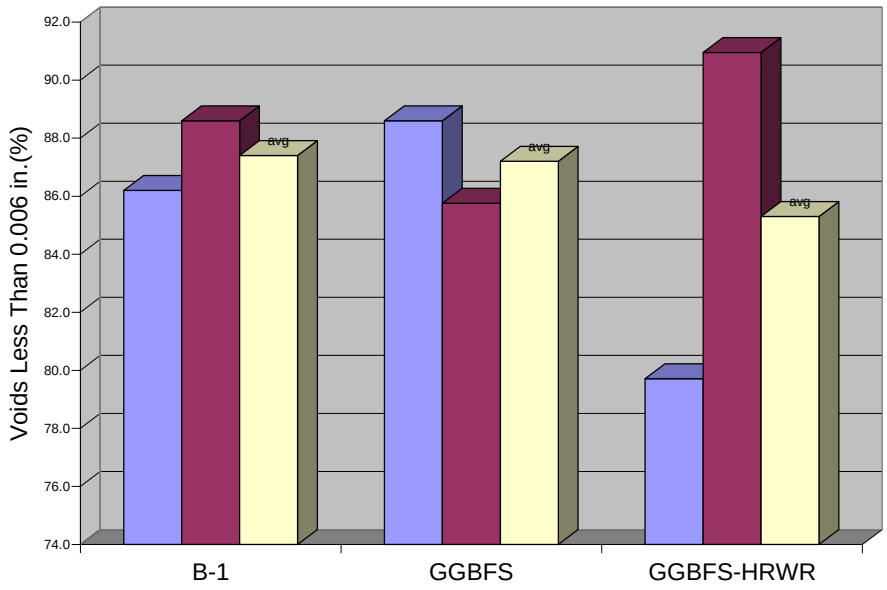

Fig. 41-Frequency distribution of air systems, percentage less than 0.006 in.

Comparing the plain GGBFS mix with the B-1 mix, the slag mix void system was somewhat inferior, with a greater bubble spacing factor, smaller specific surface, larger average bubble size, and less total air content. However, the spacing factor and specific surface values were within the recommended ranges of 0.004-0.008 in. and 600-1100 sq. in. per cu. in., respectively. And, the percentage of bubbles less than 0.006 in. was about the same.

Fig. 42 shows the average hardened air content values for the three mixes. The order of air contents for the B-1, GGBFS, and GGBFS-HRWR was 6.0%, 5.0%, and 4.6%, respectively. All were within the required 5.5 ± 1.5%.

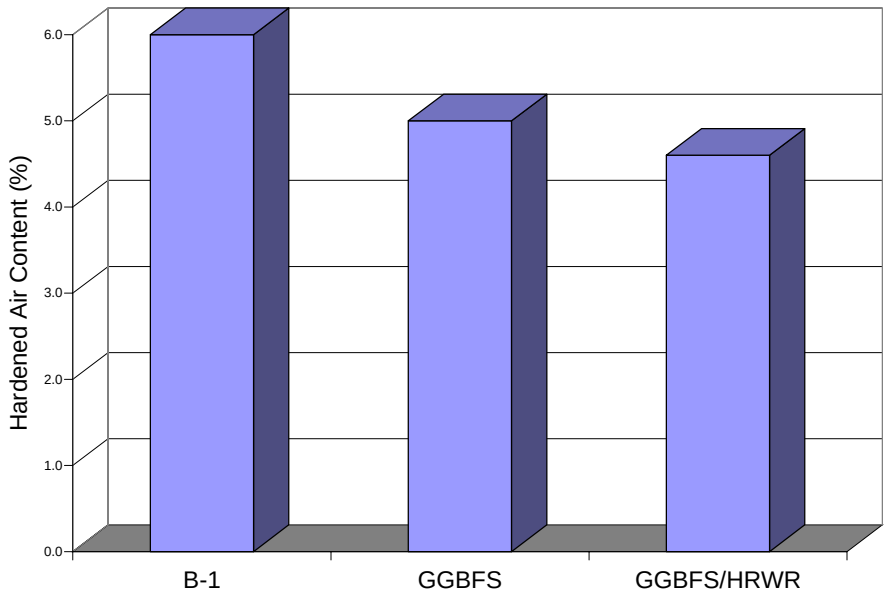

Fig. 42-Total hardened air content of all three mixtures.

In comparing the plain slag mix with the slag mix containing HRWR, differences were relatively small, with the HRWR mix actually showing a slightly smaller spacing factor, smaller average bubble size, larger specific surface, but a smaller percentage of bubble size less than 0.006 in. Thus the slag mix with HRWR did not seem to show a decrease in air void system quality compared to the plain slag mix, although there was a somewhat smaller total air content in the HRWR mix. None of the hardened air content parameters correlated with Durability Factor.

Field Temperature Data

Contractor field temperature data from concrete placements during the summer and fall of 1999 were examined. Temperature differentials are shown in Fig. 43. The range is from 18 to 50 C. On eight occasions the differential exceeded the MoDOT spec of 22.2˚ C [40˚ F].

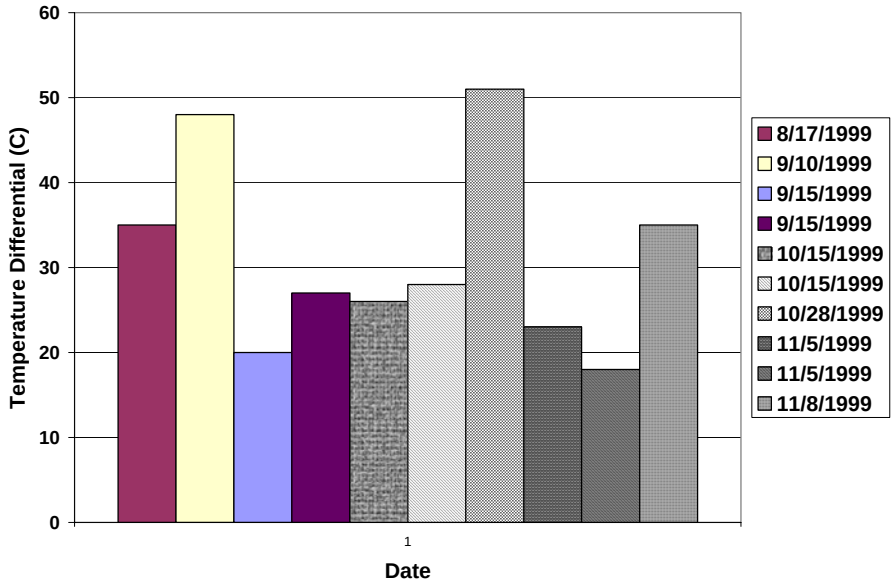

Fig. 43- Maximum temperature differentials.

Discussion

The strength of concrete is a function of *w/cm*, air content, the interaction between the slag and the PC (which involves slag and PC characteristics), effects of admixtures, degree of hydration (curing effects), aggregate characteristics, mix proportions, and testing conditions. In the study of RDT results, certain of these variables were held constant: aggregate characteristics, curing conditions, total weight of cementitious materials, and testing conditions. The remaining variables were *w/cm*, air content, effect of HRWR, percent slag (zero and 70), and the interaction between the PC and the slag. The interaction was characterized by quantifying the characteristics of the slag. In most cases reported in the literature, the fineness of the slag was the most commonly reported property. Alkali content was also mentioned as being important.

A search of the literature resulted in 12 studies (including the present study) with large slag replacement values (50-90%)[22,23,26,29,31,32,33,52,53,61,62]. These included 31 separate mixes. A linear multiple regression analysis was performed to determine which of the above variables were significant in affecting strength. A model was developed which included *w/cm*, percent slag, presence of air entrainment, presence of HRWR, and fineness of slag. Unfortunately, slag and PC chemical characteristic information was lacking in many of the studies, thus, the slag*OPC chemical interaction could not be included. Of the five main effects included in the strength prediction (estimation) model, all were statistically significant. Fig. 44 shows the resulting relationship between the observed values of 28 day compressive strength and the ones estimated from the model. Fig. 45 is similar, but data is expressed as percent OPC 28 day strength. This analysis supports the previous assertion as to which variables are significant contributors to concrete strength for high slag replacement mixes.

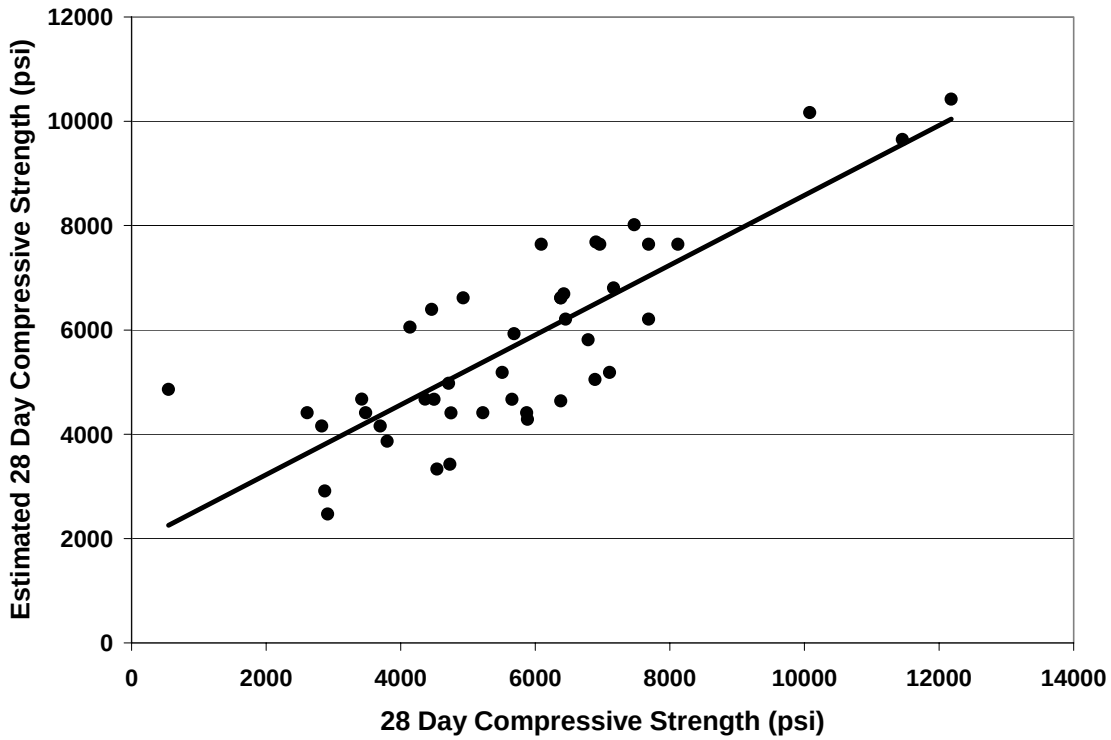

Fig.44- Observed vs. estimated 28 day compressive strengths of high slag replacement mixes

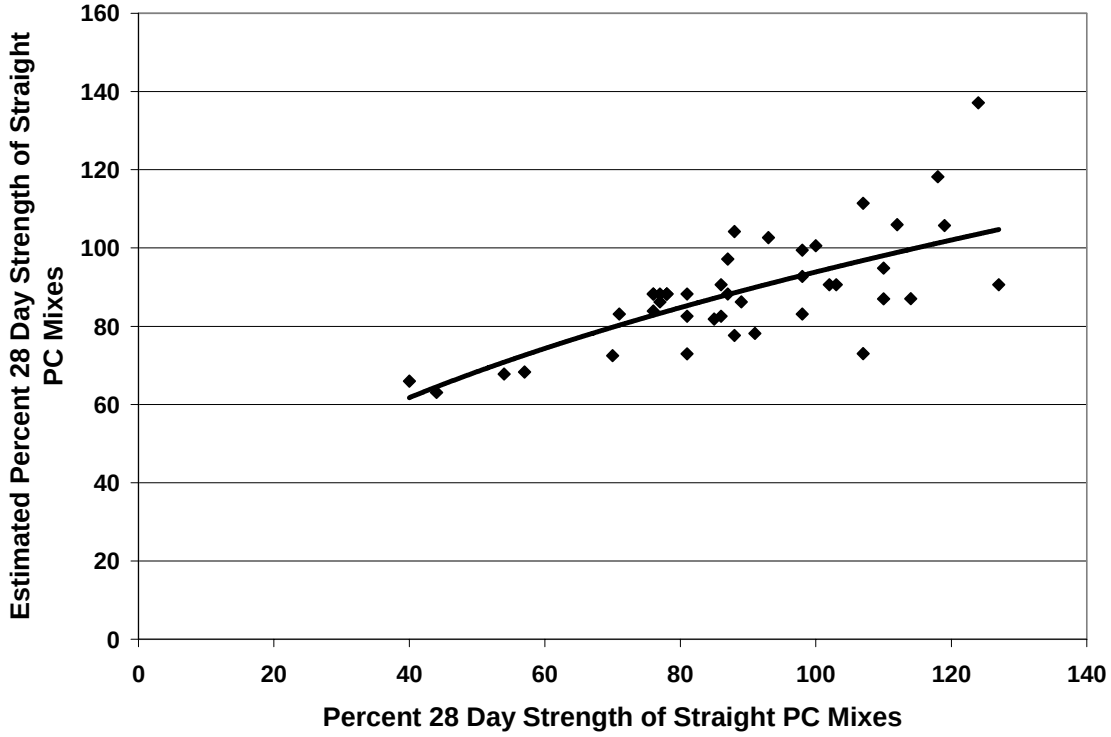

Fig.45- Observed vs. estimated 28 day compressive strengths of high slag replacement mixes as a percent of OPC mixes strength

Previously, it was pointed out that in this project, *w/cm* did not seem to have much bearing on the results, and it was concluded that the *w/cm* data may have been faulty, and thus would have to be disregarded as a variable. In regard to the effect of air content on strength, the average hardened air content was somewhat lower for both slag mixes (5.0% and 4.6 %) compared to the B-1 mix (6.0%), which does not help explain the lower strengths of the slag mixes. Also, air content did not seem to impact strength to the degree that was being observed, and did not seem totally rational, so air content data was not used to explain the results.

An attempt was made to further explain why the slag mixes in the present study did not develop the strength level of the OPC mix. To answer this, the five variables (*w/cm*, percent slag, presence of air entrainment, presence of HRWR, and fineness of slag) were examined for possible effects. A global plot of the 12 studies with percent slag plotted against percent of OPC mixes 28 day strength is shown in Fig. 46. As can be seen, as the percent slag increases from 50% to 90%, the percent of OPC 28 day strength decreases. It appears that on the average, a maximum of about 60 percent slag should be used if one expects to achieve 100% of the straight PC mix strength. A more conservative value would be about 40% slag replacement at the 95% confidence level. Only two mixes with a 70 percent or more slag proportion achieved parity with the OPC mixes. However, other factors seem to be in play because the results vary within a given percent slag replacement. Using the same materials as utilized in the field phase of this study**,** the results of the lab phase of this study reported later showed that about 50 % would be the optimum slag replacement.

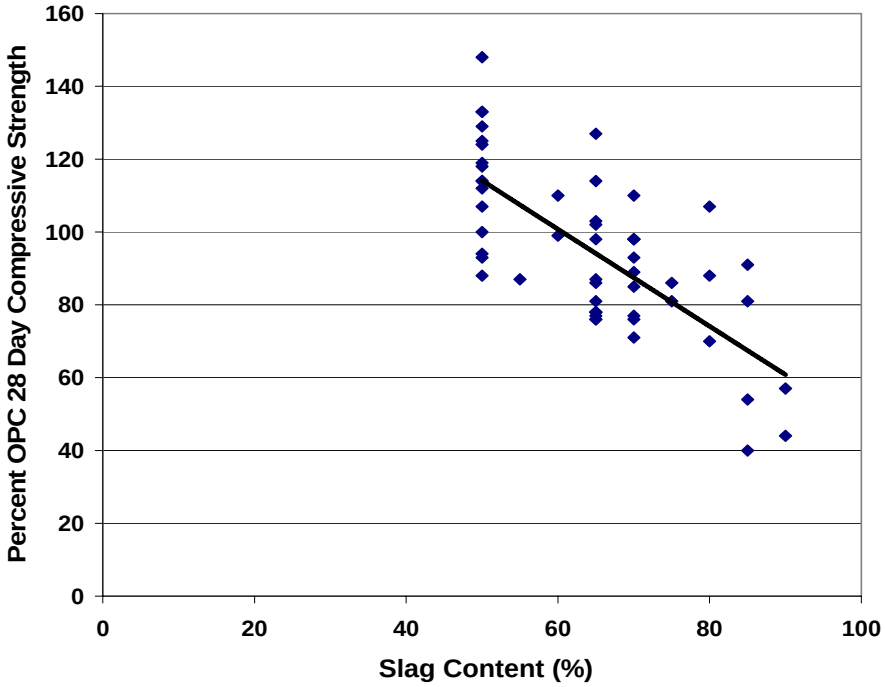

Fig. 46- Effect of slag replacement on percent of OPC mix 28 day strength

A statistical analysis was performed to determine which of the five variables was significant to the relationship between achieved strengths of the slag and OPC mixes. The results showed that percent slag and slag fineness were the only two significant variables. However, in most of the studies in the data set, *w/cm* and admixtures were held constant between the slag and OPC mixes. Additionally, the literature indicates that the effect of fineness becomes less important at ages later than 28 days.

As noted above, at any given slag replacement, other factors apparently affect the percent strength. However, it is difficult to separate percent slag replacement from amount of slag per volume of concrete. For high slag content mixes, looking at all the mixes containing Grade 120 slag, several things were examined. Fig. 47 shows the effect of cement content on percent strength. It appears that on the average, somewhere around 130 kg/m^3 [215 lbs/cy] OPC (with slag) may be required to achieve parity with an OPC (zero slag) mix, although as little as 60 kg/m^3 [100 lbs/cy] OPC might be successfully used to reach minimum specified strength. It is hypothesized that below this value, there is insufficient activator and hydroxide produced to completely activate the slag, which will result in strengths being lower than 100% OPC mixes. The MoDOT mix only contained 112 kg/m^3 [189 lbs/ cy]. However, the minimum required cement content may go up or down depending on the activity of the slag, the characteristics of the cement, and, of course, the amount of slag present. In a practical sense, if prevention of thermal cracking in mass concrete is the primary objective, and is being met while still meeting a minimum compressive strength specification, then achieving 100% parity with a zero slag mix may not be necessary. An increase in cement content may defeat the purpose of using the slag in mass concreting, which is to reduce thermal cracking. Thus, it seems that there is a limiting percent of slag replacement that is
practical.

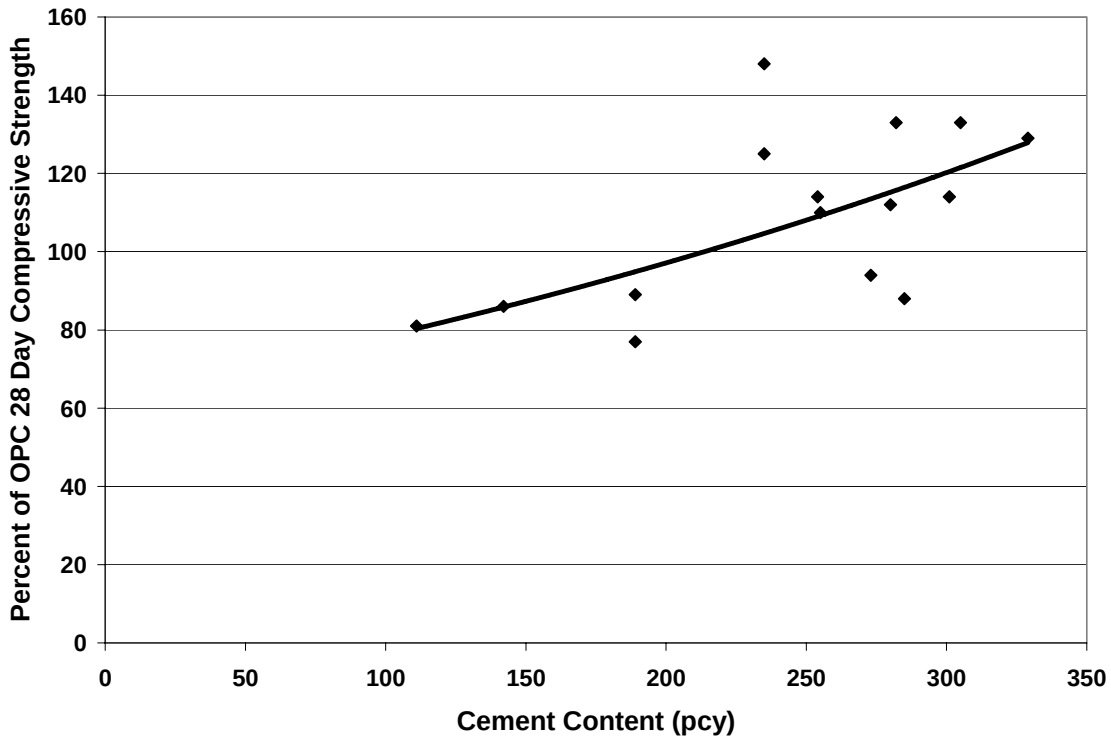

Fig. 47- Effect of cement content on percent of OPC mix 28 day strength for mixes containing grade 120 slag.

In an examination of 70 percent slag replacement mixes, the effect of slag fineness was explored, as shown in Fig. 48. There appears to be a rough trend of increasing 28 day strength as fineness increases.

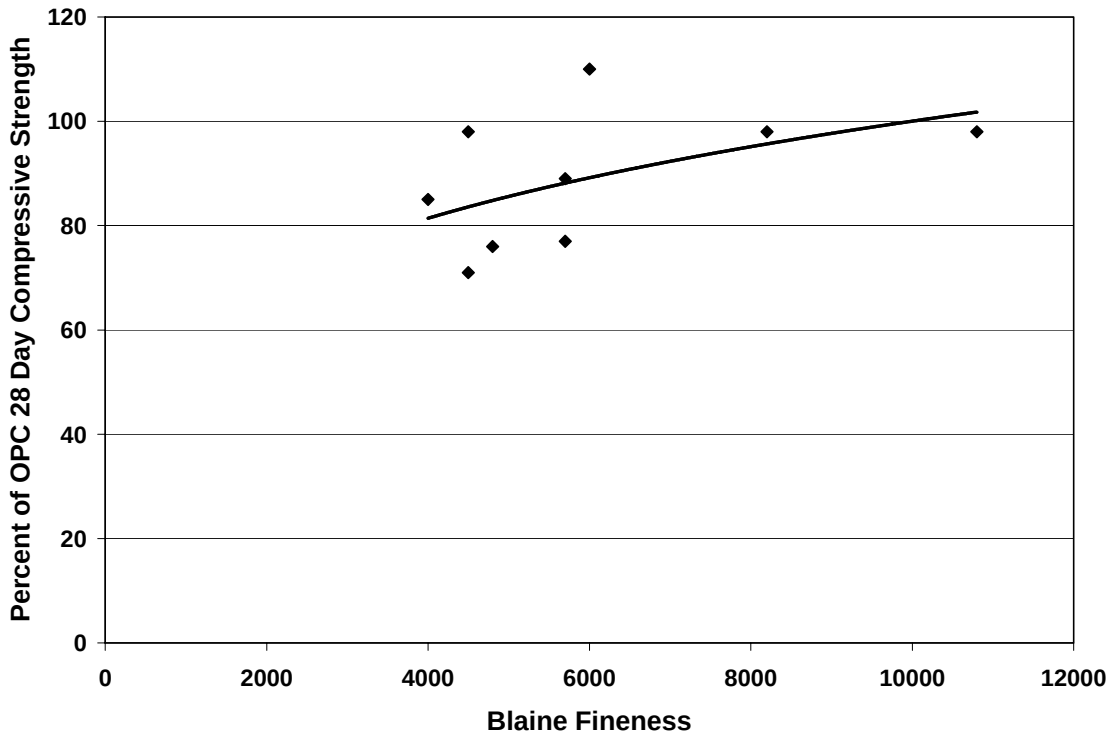

Fig. 48- Effect of slag fineness on percent of OPC mix 28 day strength for mixes containing 70 percent slag

Also, for the 70 percent slag mixes, limited data indicates that there is not a clear relationship between cement content and percent OPC 28 day strength, as shown in Fig. 49.

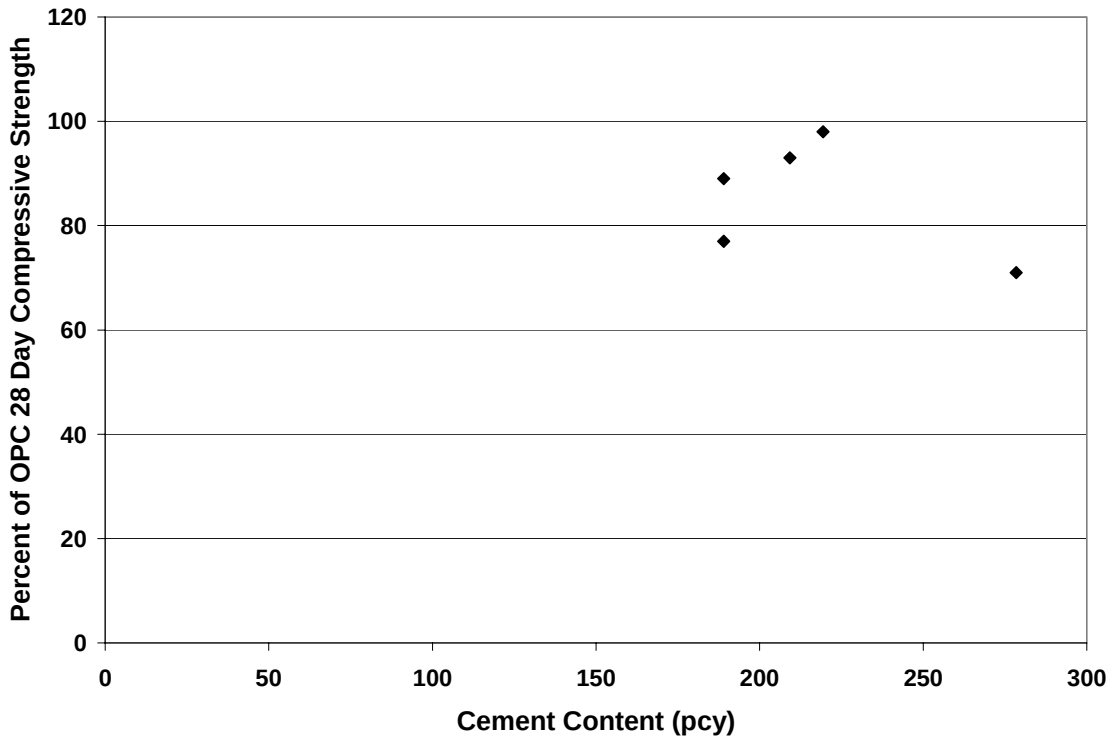

Fig. 49- Effect of cement content on percent of OPC mix 28 day strength for mixes containing 70 percent slag

Thus, the fact that the MoDOT GGBFS mixes did not achieve strength and durability levels equal or greater than that of the OPC mix is not surprising because of the high slag replacement level and, possibly, the lower combined level of activity of the specific cement and slag used in the mixes. However, as expected, the GGBFS HRWR mix did achieve greater strengths than the plain GGBFS mix.

The literature does not reveal a clear relationship between strength and freeze-thaw durability for high slag content mixes. In this study, the OPC mix exhibited good freeze-thaw resistance as measured by ASTM C 666 Method B. Freeze-thaw durability was lower for the GGBFS mixes than the OPC mix. This correlated with the lower strengths. Most of the conditioning methods utilizing extra wet and/or drying time intervals did little to improve the results for the slag mixes; however, under certain combinations of wet plus dry curing periods, the slag mix Durability Factors did approach that of the OPC mix. However, the trends in the combinations were not consistent. Overall, the air void systems of the slag mixes, which were somewhat inferior relative to the OPC mixes, could be a factor in the explanation of the GGBFS mixes' poorer freeze-thaw performance, but the differences in systems were not great, and the slag mix air void system parameters did meet ASTM and ACI recommendations. The variation in DF results as a function of curing and conditioning methods points out the difficulty in prediction of field performance from variations of the ASTM C666 procedure.

The Rapid Chloride Permeability and salt scaling test results reported herein were sensible in magnitude and trend. The literature indicates that the interpretation of what laboratory salt scaling results really mean in relation to field performance is open to question.

Phase II: Laboratory Investigation

General

In regard to the field phase of the study, the expectation was that at some interval of curing, the slag mix strengths would approach or even exceed that of the plain mix due to pozzolanic activity. However, both GGBFS mixes had lower strengths than the B-1 mix at 3 to 365 days of standard curing.

The B-1 mix had a 374 kg/m^3 [630 lbs/cy] cement content, utilizing a Type I portland cement (PC). The slag mixes contained 112 kg/m^3 [189 lbs/cy] Type II Low Heat cement and 263 kg/m^3 [442 lbs/cy] Grade 120 Aucem GGBFS. UMR researchers hypothesized that the difference in strengths between the B-1 and the slag mixes may have been due to an insufficient reaction between the slag and the Type II PC. Both the Type II PC and the slag had low amounts of activators, such as alkali and sulfate, which typically function as activators of the slag. Thus, the total available amount of activators in the system may not have been sufficient to fully utilize the potential of the slag. A second hypothesis was that there was not enough PC in the mix to provide sufficient activation.

Laboratory Investigation

To test these two hypotheses, a series of mortar mixes were tested for compressive strength. Specimens were 50 mm [2 in.] mortar cubes cast in accordance with ASTM C 109[63]. Certain of the mix combinations reflected the job mix designs: 374 kg/m^3 [630 lbs/cy] total cementitious materials (TCM), zero and 70% slag, and 0.41 *w/cm*. The experimental designs are shown in Tables 12 and 13.

Table 12- Type II Mixes

w/cm	**TCM (lbs/cy)**	**OPC Content (lbs/cy)**			
		0% slag	**50%**	**60%**	**70%**
0.783	518	518	259	207	155
0.783	630	630	315	252	189
0.41	1073	1073	536	429	322
1.23	189-630*	189	189	189	189

*Range of TCM

Table 13- Type I Mixes

w/cm	TCM (lbs/cy)	OPC Content (lbs/cy)			
		0% slag	50%	60%	70%
1.23	189	189			
0.783	518	518			
0.783	630	630	315	252	189
0.41	1073	1073			

Twenty-three mixes were designed. Of these, two mixes were made with Type II PC at four levels of slag proportion (0, 50, 60, and 70%) and two levels of TCM at one *w/cm*. The TCM's were 374 kg/m^3 [630 lbs/cy] (project design) and a leaner 5.5 sack (308 kg/m^3 [518 lbs/cy]) mix. Unfortunately, at this level of TCM, a 0.41 *w/cm* did not meet the flow requirements of ASTM C 109; correct flow was achieved at a *w/cm* of 0.783.

Another series of cubes (0, 50, 60, and 70% slag) were made at the project *w/cm* = 0.41. However, at this *w/cm*, the TCM had to be increased to 639 kg/m^3 [1073 lbs/cy] to meet flow requirements.

A third series of cubes were made at the project OPC content (112 kg/m^3 [189 lbs/cy]) with slag proportion increasing (0, 50, 60, and 70%) by adding successively larger amounts of slag, keeping the PC at 112 kg/m^3 [189 lbs/cy]. In order to meet flow requirements at the lowest TCM content, the *w/cm* had to be raised to 1.23.

To see the effect of type of PC, a fourth series (0, 50, 60, and 70% slag) was made at 374 kg/m^3 [630 lbs/cy] TCM using Type I PC. Additionally, control mixes of zero slag at the four TCM levels using Type I PC were tested, thus zero slag mixes could be compared at all *w/cm's*.

Thus, there were a total of 23 mix designs of one batch each, with 12 replicate cubes per batch. Six cubes were broken at two ages each (28 and 56 days), for a total of 276 cubes.

Materials

Cementitious Materials. Buzzi Unicem was asked to supply the same type of materials that were used on the project (Type I from River Cement Company, Type II and GGBFS from Lonestar Cement Company). Table 14 lists the characteristics of the Type I PC. In Tables 15 and 16 are comparisons of the materials used on the project versus what was used in the lab study. The information for the project was supplied by MoDOT (using supplier data); the source of information for the lab study materials were the mill certifications that came with the cement and slag samples.

Table 14- Type I OPC

Property	Lab Study
Blaine Fineness (m^2/kg)	375
7 day strength (psi)	4570
Total Alkalies (%)	0.46
SO_3 (%)	2.70

Table 15- Type II PC

Property	Project	Lab Study
Blaine Fineness (m^2/kg)	302-310	322
7 day strength (psi)	2350-3660	3070
28 day strength (psi)	4200-5400	4580
56 day strength (psi)	4950-6600	5470
90 day strength (psi)	5500-7090	5530
Total Alkalies (%)	0.33-0.43	0.38
SO_3 (%)	1.96-2.16	2.3

Table 16- Grade 120 GGBFS

Property	Project	Lab Study
Blaine Fineness (m^2/kg)	506-614	490
7 day strength (psi)	4500-5200	4240
28 day strength (psi)	7500-8090	6900
Total Alkalies (%)	0.52-0.69	Not available
SO_3 (%)	1.11-1.86	0.27
SAI (7 day avg)	98-114	101
SAI (28 day avg)	130-139	132
Specific Gravity	2.86-2.92	2.91

In looking at the 7 through 90 day strengths, the alkali and sulfate contents, and the fineness of the Type II materials, it appears that that the lab study Type II PC was similar to the project material. For the slag, comparing the slag SAI and specific gravity values, the slags appeared similar, even though the lab material was a little coarser and exhibited somewhat lower strengths and sulfate content.

Table 17 shows the requirements of ASTM C 989[28] for the reference cement used for SAI determination.

Table 17- ASTM C 989 Reference Cement Requirements

Parameter	Level
Minimum 28 day strength(psi)	5000
Minimum Total Alkalies (%)	0.60

It should be noted that the project Type II cement's strength and alkali content levels are lower than that which is required by ASTM C 989 for the reference cement used for SAI determination. And, sulfate of slag is allowed to be as high as 4.0%, which is considerably greater than the level that was present in the slag used in this study. Thus, reactivity of the cement-slag system could be expected to be lower than if a cement and slag system with more slag activator content had been used.

Sand. The fine aggregate used for the mortar cubes was an uncrushed silica sand. It was significantly finer than that required by ASTM C 778[64]. The extra surface area may have been responsible for the increased water demands that were required to meet flow requirements. Table 18 shows the C 778 sand requirements and the sand gradation used in this study as reported from the supplier, U.S. Silica.

Table 18- Graded Sands

Sieve	ASTM C778 % Passing	This Study % Passing
#16	100	100
#20	96-100	100
#30		99.9
#40	65-75	95.4
#50	20-30	80.4
#70		49.4
#100	0-4	15.4
#140		2.4
#200		0.1

The bulk specific gravity SSD was measured as 2.648 and the absorption was 0.1%.

Equipment

A detailed account of the batching, curing, and testing procedures, as well as the equipment used, is included in Appendix A.

The batches were mixed in a 4.7 l [5 qt] Hobart mixer. The mortar was cast in 50 mm [2 in.] cube rigid steel gang molds of 3 cubes each. The loading device was a 890,000 N [200,000 lb] Tinius-Olsen servo-hydraulic controlled universal testing machine with a 64 mm [2.5 in.] square bottom platen and a spherically-seated 90 mm [3.528 in.] diameter upper platen.

Procedure

Each batch was made by accumulatively weighing the sand and cementitious materials, followed by dry mixing by hand. Then the mixing bowl was inserted into the mixer and mixing begun.

The bowl was removed, sides scraped with a spatula, and the mortar hand mixed. The 12 steel cube molds were then filled and consolidated as per ASTM C 109. The cubes were cured in a moist room for until such time that they could be removed from the molds without damaging the specimens, usually 24 hours, but in some cases up to 1 to 5 days. Once removed from the molds, specimens were marked and immersed in lime-saturated water for the remainder of the time period prior to compressive strength testing (28 or 56 days). The buckets containing the lime-saturated water and specimens were stored in a room at 23 ± 2°C [73 ± 3.5°F].

On the day of testing, cube dimensions were measured with an electronic caliper. The cubes were towel dried to an SSD state, weighed, turned on their sides, and loaded to failure.

The porosity of the cubes was calculated in the following manner. Once compressive testing was completed, the specimen material was collected and dried to a constant mass in an oven at 110 ± 5°C [230 ± 9°F]. Using the oven-dried weight of a cube and its measured bulk volume, the bulk dry-density was calculated. For each mix design, a weighted apparent relative density (apparent specific gravity) for the mix solids (sand, cement, slag) was determined. Knowing the bulk dry-density of the cube and the apparent relative density of the mix solids, the porosity was calculated as follows:

$$\eta = 1 - \frac{\gamma_d}{S_a}$$

Where: η = Porosity

γ_d = Bulk Dry Density (g/cm^3)

S_a = Weighted Apparent Relative Density (specific gravity)

It should be noted that the units are inconsistent in the above equation. However, when densities are expressed in g/cm^3 the math is valid because the density of distilled water can be assumed to be 1 g/cm^3

Results and Discussion

Experimental Results

The strength data is summarized in Appendix B. Gross compressive strengths were calculated by dividing the load at failure by the cross-sectional area of the cube. Fig. 50 is a summary of all the data, including 28 and 56 day strengths.

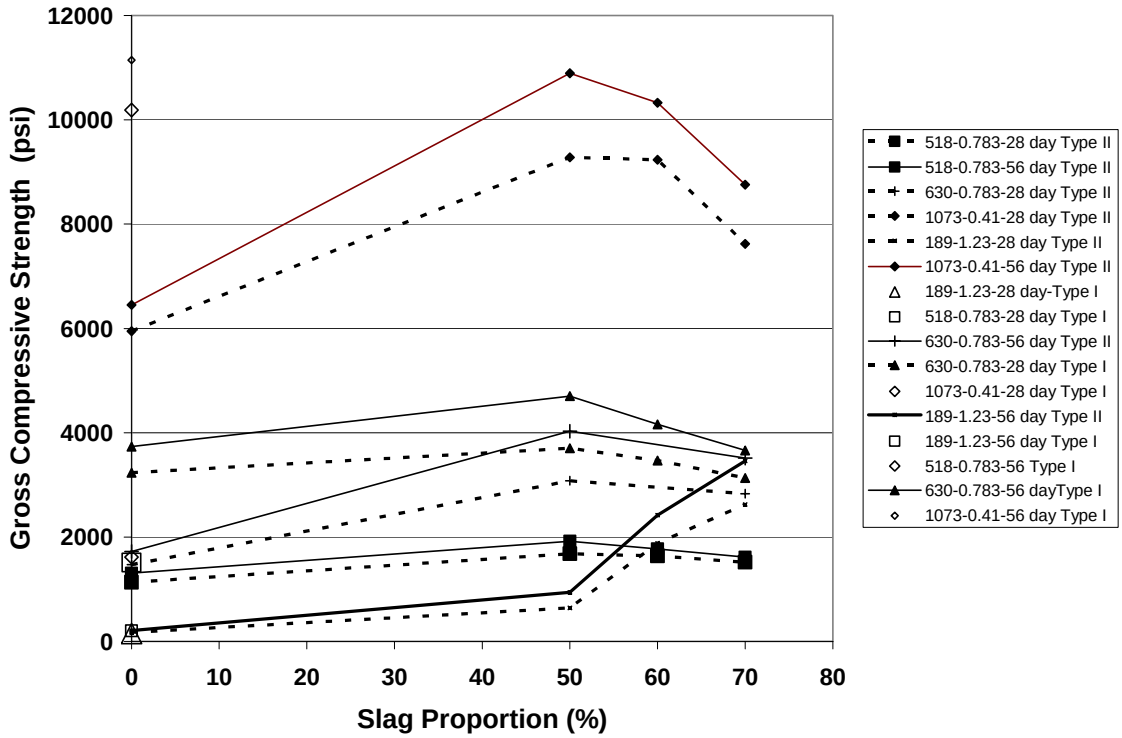

Fig. 50- Effect of percent slag, TCM, w/cm, PC type, and time of curing on strength

Several things are apparent from the figure. The four series of mixes will be referred to as the 189, 518, 630, and 1073 mixes, denoting their TCM content in English units. First, for the 518 Type II series, the 630 Type I and II series, and the 1073 Type II series at both 28 and 56 days, the shape of the curve is similar, with the optimum slag content peaking at 50%. It is hypothesized that as slag content increases and becomes more available, the pozzolanic reaction is more pronounced and strength increases, hence the ascending portion of the curve. However, at some point, this effect is overshadowed by the diminishment of the available activators such as alkali and sulfate, and thus the full potential of the increasing slag content is not realized, thus strength drops off as slag levels continue to increase. Secondly, the 630 Type II series had greater strengths than the 518 Type II series at the same *w/cm.* This could be attributed to a higher TCM. Third, the Type I 630 series exhibited greater strengths than the Type II 630 series at both ages. The lower strength is thought to be a result of the relatively low activity of the Type II cement/slag system. Fourth, the 1073 Type II series had greater strengths than the 630

Type II series; it had a lower *w/cm* and a greater TCM. Fifth, all the 70% slag mix strengths were lower than their associated optimums, but in most cases were stronger than the zero slag mixes. Sixth, the 189 series increased in strength as TCM increased.

As a general rule, as sand content increases, entrapped air increases. The mixes in this study had a considerable range in TCM content, thus the sand content, and most likely air content, varied as well. It is commonly understood that as air content increases, strength will decrease. To see the effect of sand content on strength, porosities were calculated. Fig. 51 shows the effect.

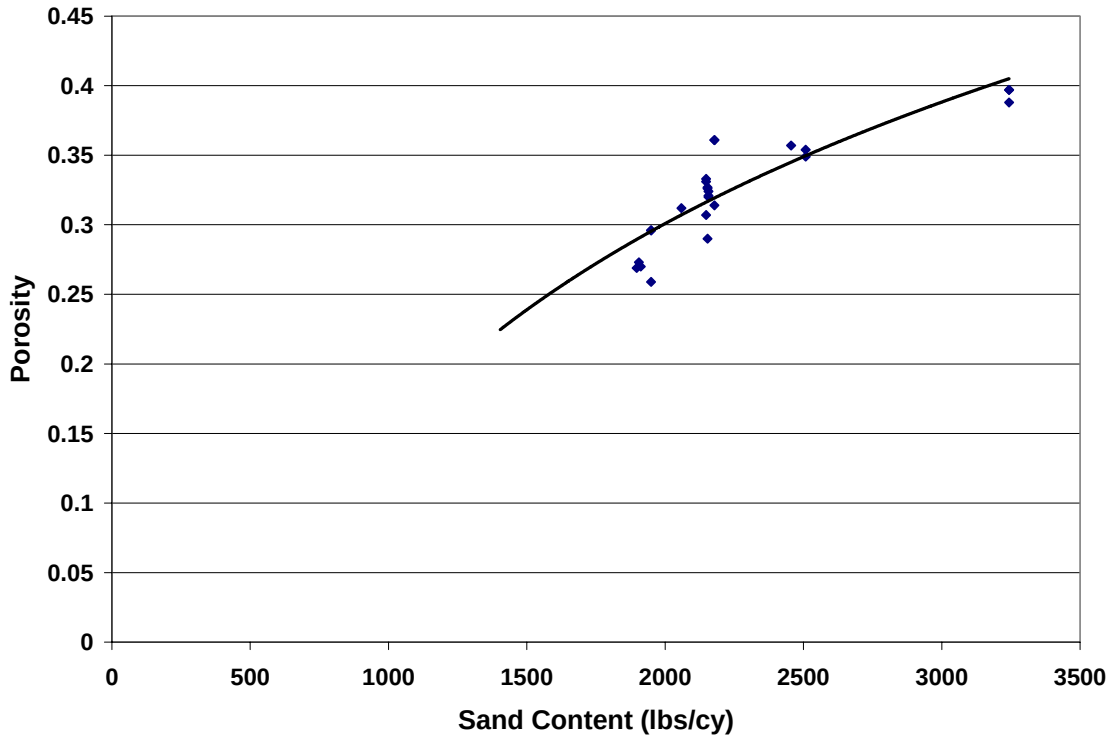

Fig. 51- Relationship of sand content and porosity

As can be seen, there is a strong relationship between sand content and porosity. The cube mixes contained quite a lot of fine sand that would tend to entrap considerable air. This would have a large and variable impact on strength as TCM changed from mix to mix. To correct for the effect of sand-induced porosity, net compressive strengths were calculated and subsequently used in the analysis. These are tabulated in Appendix C.

Plots of 56 day net compressive strengths were used to examine the effects of slag proportion, TCM content, and type of PC. Looking at mixes containing Type II PC, Fig. 52 shows that the optimum slag proportion was 50%. Also shown is the comparison of 308 and 374 kg/m^3 [518 and 630 lbs/cy] TCM at the same *w/cm*: a greater TCM content resulted in greater strengths. However, the difference in strength between the slag mixes and the zero slag mix for the 630 series was greater than the difference between the slag mixes and the zero slag mix for the 518 series, indicating that it is beneficial to increase

the PC content when working with large slag proportion mixes in order to obtain greater reactions between the PC and the slag over and above the increase in strength due to a greater TCM. Finally, in every case, the 70% slag proportion strengths were greater than those of the zero slag mixes, with decreasing significance as TCM is reduced. These trends are supported in the general literature.

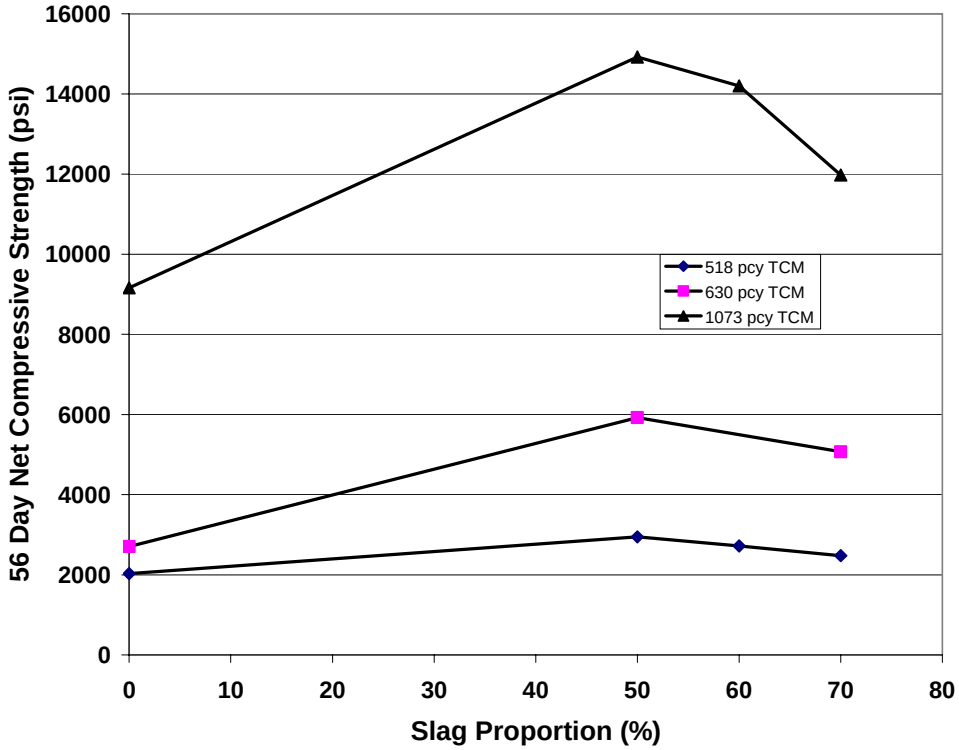

Fig. 52- Effect of slag proportion and TCM on 56 day strength

Fig. 53 depicts a comparison of PC type, while holding *w/cm* and TCM constant. For the situation of 374 kg/m^3 [630 lbs/cy] TCM, Fig. 53 shows that the Type I PC used in this study seemed to be more active than the Type II, although the effect varied with slag proportion.

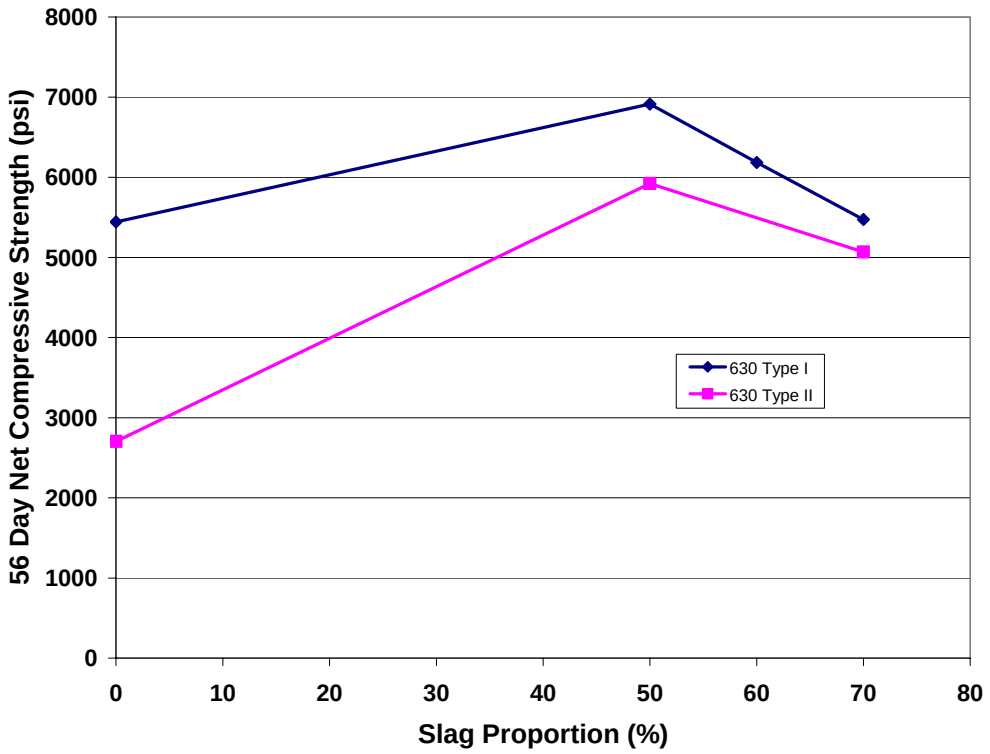

Fig. 53- Effect of slag proportion and cement type on 56 day net compressive strength

Fig. 54 shows the difference in 56 day net compressive strengths between the 70% slag mixes and the zero slag mixes as a function of TCM. As shown, the difference increases considerably as TCM increases. Thus, slag replacement does increase concrete strength even at high replacement rates.

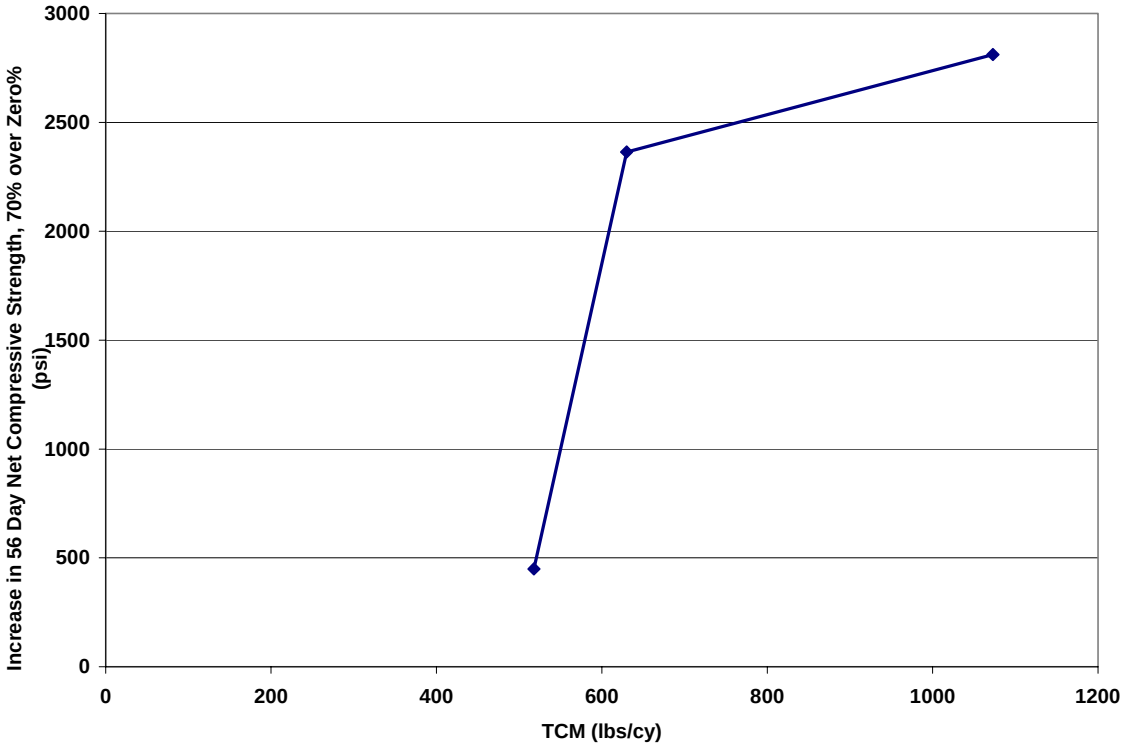

Fig. 54- Difference in net compressive strength between 70% and zero percent slag mixes

Fig. 55 shows the difference in 56 day net compressive strengths between the Type I PC mixes and the Type II mixes as a function of TCM. As shown, the difference increases considerably as TCM increases. Thus, mixes containing Type I PC's can have greater strengths than Type II PC's at higher PC contents.

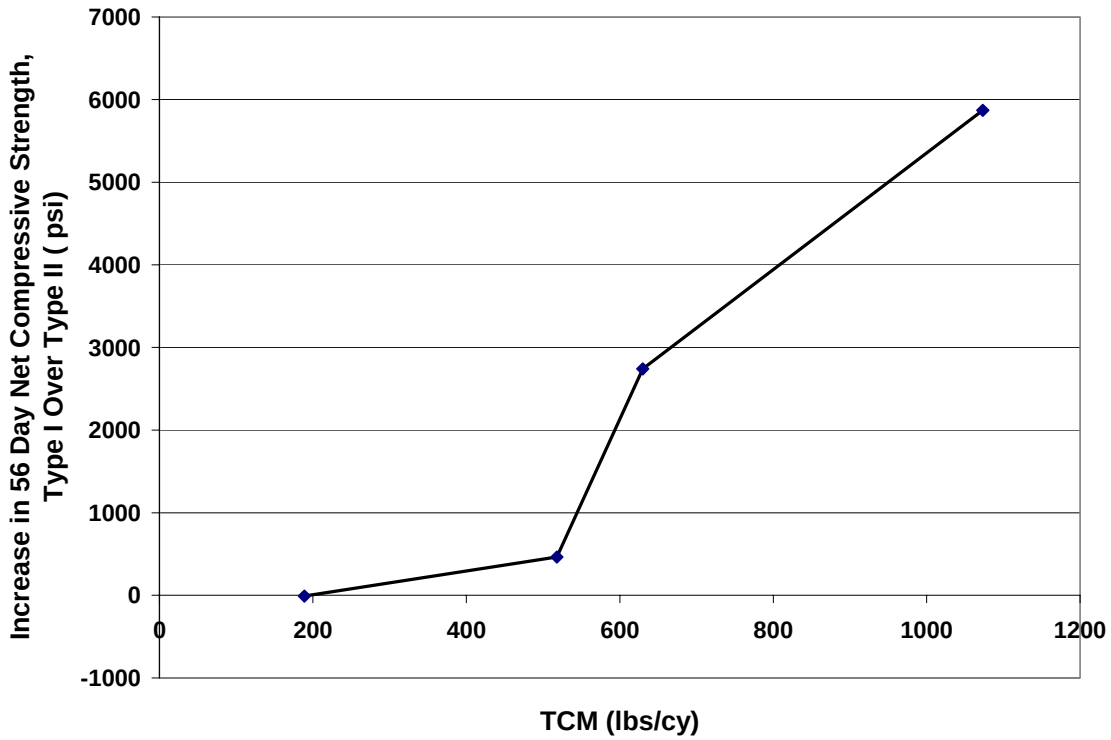

Fig 55- Difference in 56 day net compressive strength between Type I PC and Type II PC mixes at zero slag content

Fig. 56 shows the series of mixes where the PC content was held constant at the project level of 112 kg/m^3 [189 lbs/cy], while slag content was increased up to the 70% level. As shown, the strengths increased considerably as slag content (and TCM) increased, an indication that the PC was successful in reacting with the slag.

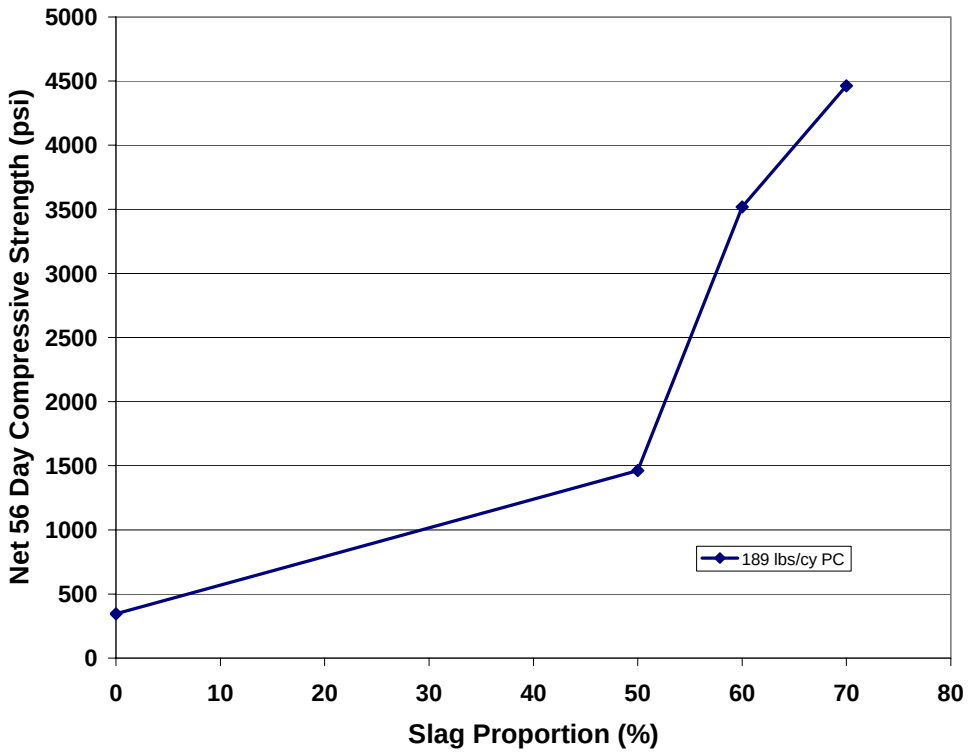

Fig. 56- Effect of slag addition at constant PC content

Statistical Analysis

A statistical analysis was performed on the results of the testing. Several questions needed answering. First, was porosity significant (and potentially clouding the analysis, thus needing correction)? To answer this, the 112 kg/m^3 [189 lbs/cy] series was examined. This series was where *w/cm* and cement type were held constant, thus only TCM (and therefore sand content and hence entrapped air, or porosity) varied. Paired t-tests of gross and net compressive strengths were performed. The analysis showed that the two types of strengths were significantly different at the 95% confidence level, thus porosity was significantly affecting the strength results. This conclusion led to the analysis to be concentrated more on net strengths rather than gross strengths.

The second question was, is the slag proportion significant in affecting strength? Regression analysis was performed. A model that encompassed all the main effects of the study was analyzed; the main effects were *w/cm*, PC type, PC content, and slag content. This model inherently accounted for porosity because PC content and slag content were related to sand content, and sand content was related to porosity. The results of the analysis showed that all four main effects were significant at the 95% level.

To further explore the role of TCM, the regression analysis was confined to looking at the net compressive strengths of the 518 series to the 630 series. Net compressive strengths

of the 518 series and the 630 series were modeled where the effects of *w/cm*, cement type, and porosity were nullified, thus only TCM at various slag proportions were varied. The results showed that strengths at the two levels of TCM were significantly different at the 95% level.

Finally, the question of the effect of cement type was further examined. Net compressive strengths of the 630 Type I PC series and the 630 Type II series were modeled, where the effects of *w/cm*, TCM, and porosity were nullified, thus only PC type was varied along with the proportion of slag. The results showed that strengths resulting from differences in cement type were significantly different at the 95% level. Also, paired t-tests showed the strengths to be different at each slag proportion.

Additionally, there was a significant interaction of PC type and amount of slag, and, an interaction of PC type and PC amount. These trends indicate that there is an activity issue involving the specific PC and slag being used.

Phase II Conclusions

The results of the Phase II laboratory portion of the study showed the following:

1) The margin in 56 day strengths over 28 day strengths increased as TCM increased as shown in Fig. 50.

2) The optimum slag proportion at constant TCM was 50% in all four cases.

3) As TCM increased, strength increased.

4) Type I PC mixes were stronger than Type II mixes, both with and without slag, at 28 and 56 days. This indicates that the chemical interaction between the Type I PC and the slag was better than that of the Type II PC and slag. At a given percent slag replacement, the difference between strengths of Types I and II PC at 28 days was about the same as at 56 days. This trend was not determined for later strengths.
5) 70% slag replacement of Type II PC resulted in increased 28 and 56 day strengths at all TCM levels (compared to zero slag mixes). The increase was more pronounced at greater TCM contents. Thus, large-scale replacement of Type II cement with GGBFS can result in significant increases of strength.

6) The 70% slag Type II PC mixes were weaker than the zero slag Type I mixes of the same series. The decrease was relatively small at lower PC contents and was more pronounced at greater TCM contents.

7) At a constant 112 kg/m^3 [189 lbs/cy] (project amount), as slag content increased and slag proportion approached 70%, strength increased. This indicates that the PC was successful in activating high levels of slag content. However, the comparison of the 518 and 630 series showed that greater slag contents require greater PC contents for increased

strengths. The difference in strength between the slag mixes and the zero slag mix for the 630 series was greater than the difference between the slag mixes and the zero slag mix for the 518 series, indicating that it is beneficial to increase the PC content when working with large slag proportion mixes in order to obtain greater reactions between the PC and the slag over and above the increase in strength due to a greater TCM.

8) At a zero slag proportion, as TCM increased, the margin of strength of Type I mixes over Type II increased at 28 and 56 days.

9) At greater levels of slag content, the type of PC became less significant, as Type II PC-slag mix strengths approached Type I-slag mix strengths at both 28 and 56 days.

General Conclusions

Based on the results of this study, the following conclusions are drawn:

1. In a comparison of Type I OPC and 70% GGBFS field mixes, compressive strengths of the slag mixes at all ages up to one year were lower. The average strength of the OPC mixes was 48.1 MPA [6970 psi] compared to the averages of 34.6 MPa and 38.9 MPa [5010 and 5640 psi] for the plain GGBFS and GGBFS-HRWR mixes, respectively. However, several test sets of the GGBFS-HRWR mixes approached the OPC control tests values. Based on a search of the literature, the results were to be expected; the maximum replacement to achieve parity with 100% OPC mixes seems to be between 40% and 80%. Slag proportions of 40 to 60 % appear to be the optimum level for highest strength development. Some slag-PC combinations will not reach strength levels commensurate with control mixes at any slag proportion. The general literature and the results of the Phase II laboratory portion of this study seem to support the hypothesis that for slag mixes to obtain strengths equivalent to Type I PC mixes at 28 to 56 days, sufficient activators such as alkali or SO_3 needs to be present to activate the slag and produce more CSH (pore refinement) and replace CH with CSH (grain size refinement). Sufficient activator content can come from having the right combination of a sufficient PC content plus sufficient activator present in the cement–slag combination. In the case of this project, a low cement content (189 lbs/cy) or a low proportion of OPC (30%) plus a low activator level (alkali content of 0.38 and SO_3 content of 0.27) in the Type II Low Heat PC, could have led to strengths that were less than the Type I PC control mix. However, 70% slag replacement mixes are capable of achieving reasonable levels of compressive strength and may even achieve parity with zero slag mixes utilizing the same type of PC. Whether the strength of the slag-Type II PC mixes with sufficient activator would ultimately exceed the strength of the Type I mix was not determined.

2. The literature does not reveal a clear relationship between strength and freeze-thaw durability for high slag content mixes. In this study, the OPC mix exhibited good freeze-thaw resistance as measured by ASTM C 666 Method B. Freeze-thaw durability was lower for the GGBFS mixes than the OPC mix. This correlated

with the lower strengths. Most of the conditioning methods utilizing extra wet and/or drying time intervals did little to improve the results for the slag mixes; however, under optimum wet plus dry curing periods, the slag mix Durability Factors did approach that of the OPC mix.

3. Rapid chloride permeability test values were significantly lower for the GGBFS mixes compared to the OPC mix. In regard to chloride permeability, both GGBFS mixes are considered to be low, while the OPC mix result was somewhat high. The literature is almost universal in supporting this result of slag replacement. Slag replacement reduces permeability from a reduction in pore size, not necessarily from a reduction in porosity. Because strength is a function of porosity, this means that it could be expected that permeability and strength may not correlate well.

4. The OPC control mix had good salt scaling resistance. Both GGBFS types of mixes exhibited significantly greater salt scaling than the OPC control. Past studies indicate that lab scaling tests usually show a reduction in scaling resistance when slag is used as a replacement for PC due to carbonation effects and testing conditions.

5. Although strength continues to increase after 28 days, qualitative relationships between different types of mixes are usually established by 28 days of curing.

Recommendations

1) High-slag content concrete mixes should continue to be considered for future projects, providing that certain conditions are met. First, specifications should be written to reflect realistic expectations for: a) the service requirements of the facility, and b) the potential of the mix design itself.
   a. The specifications should address only those parameters that are of interest. For example, the freeze-thaw durability and air void system specifications for exterior concrete should be tailored to the environment and the extent and manner in which the structure will be exposed (drainage considerations). Whether or not salt scaling resistance will be required should be considered. The level of strength necessary, as opposed to comparison to a non-slag mix, should be ascertained.
   b. Careful attention should be paid during mix design to the actual job materials that will go into the mix. It should be determined up front what can be accomplished with the specific mix components in relation to each other in the proportions anticipated. Once the variables are studied under controlled laboratory conditions, the mix can be applied under field conditions.

2) The curing regime for freeze-thaw durability (ASTM C 666) for slag mixes should be finalized, either by decree or by research.

3) For slag mixes that will be subjected to deicing salt environments, the most realistic type of salt scaling test/ specification should be adopted. This may involve research.

4) For low heat applications using high slag proportion mixes, choice of OPC type and level of slag replacement should be made after appropriate trial mixes are analyzed.

5) Levels of acceptable Durability Factors for different applications (bridge decks, substructures, pavements, etc.) should be adopted.

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Appendix A: Batching, Curing, and Testing Procedures

Cube specimens for this study were mixed, cast, cured, and tested based on pertinent sections of the procedures set forth in ASTM C 109/C 109M-02.

Mixing & Casting. Batch size was sufficient to produce 12 cubes with nominal dimensions of 50 mm [2 in.]. The mixing bowl of a Hobart Model N50 (4.7 l [5 qt] capacity) mixer was pre-moistened and the batch water was added to the bowl. The sand and cementious materials were dry-mixed by hand before introduction into the mixing bowl. Having attached the mixing bowl to the mixer, the mixing procedure (i.e. mixing time and speed, etc.) was carried out according to ASTM C 305-99. Figure A1 shows the batching and mixing station.

Fig. A1- Batching equipment

Upon completion of the mixing procedure, casting of the cubes began immediately per the procedure outlined in ASTM C 109, section 10.4.3. Figure A2 shows the cube casting station.

Fig. A2- Steel cube molds

Curing. The molded cube specimens were moved to a moist-cure room and placed on a shelf such that water would not drip on them. Cubes were left in the molds and in the moist-cure room until such time that they could be removed from the molds without damaging the specimens; this time period ranged from 1 to 5 days. Once removed from the molds, specimens were marked and immersed in lime-saturated water for the remainder of the time period prior to compressive strength testing. The buckets containing the lime-saturated water and specimens were stored in a room at 23 ± 2°C [73 ± 3.5°F].

Compressive Strength Testing. Prior to testing in compression, a set of 6 cubes was removed from the lime-water bucket and temporarily stored within a damp cloth. If necessary, the cube surfaces that were to be in contact with the loading platens (the sides of the cube as cast) were lightly sanded to remove any fins or irregularities. At this point, several measurements were made in order to calculate material properties for analysis purposes. Cube dimensions were measured using an electronic caliper device. A single length, width, and height measurement (as tested) was taken at a location across the middle of each specimen. Finally, a saturated, surface-dry (SSD) weight of each cube was obtained immediately before compression testing.

Compression testing was performed using a servo-hydraulically controlled Tinius-Olsen (T-O), 890,000 N [200,000 lb] capacity, Universal Load Frame. The system uses an Admet software program, MTestW©, for load control and data acquisition. The most recent calibration of the T-O occurred on October 28, 2004.

The lower loading platen is a rectangular-shaped steel column, 266 mm [10.5 in.] tall and 63.6 mm [2.505 in.] square. The upper platen is a spherically-seated disk, 90.0 mm [3.538 in.] in diameter and 22.9 mm [0.900 in.] thick. The disk is seated within a cylindrically-shaped steel column, 81 mm [3.200 in.] in diameter and 117 mm [4.625 in.] tall. The loading system is shown in Figure A3.

Having completed the preliminary specimen measurements, the cube was centered on the lower platen and the upper crosshead lowered until the upper platen was almost in contact with the specimen. To enclose the specimen during testing, a 400 mm [16 in.] long section of 160 mm [6.3 in.] diameter PVC pipe was split lengthwise and then, using duct tape, rejoined along one side to serve as a hinge. Once the PVC pipe enclosure was in place around the specimen, the test program was initiated and loading began. A pre-load of 222 N [50 lb] was applied. Once the pre-load was obtained, the specimen was loaded to failure at a rate of 890 N/sec [200 lb/sec]. After failure, the specimen was quickly gathered into a pan and reweighed to check that the entire cube had been retrieved for a subsequent moisture content determination.

Fig. A3- Loading system

Porosity Determination. A portion of the analysis included determination of the porosity of the cube specimens at testing. As described above, once compressive testing was completed, the specimen material was collected and dried to a constant mass in an oven at 110 ± 5°C [230 ± 9°F]. Using the oven-dried weight of a cube and its measured bulk volume, the bulk dry-density was calculated. For each mix design, a weighted apparent relative density (apparent specific gravity) for the mix solids was determined. Knowing the bulk dry-density of the cube and the apparent relative density of the mix solids, the porosity was calculated as follows:

$$\eta = 1 - \frac{\gamma_d}{S_a}$$

Where: η = Porosity

γ_d = Dry Density (g/cm^3)
S_a = Weighted Apparent

Relative Density (specific gravity)

One should take note that the units are inconsistent in the above equation. However, when densities are expressed in g/cm^3 the math is valid because the density of distilled water is 1 g/cm^3.

Appendix B: Gross Compressive Strengths

SlagDataSummary.xls

	Type II							
% slag	517-0.783-28	517-0.783-56	630-0.783-28	630-0.783-56	1103-0.41-28	1103-0.41-56	189-1.23-28	189-1.23-56
0	1156	1310	1499	1726	6004	6451	176	208
50	1680	1879	3082	4028	9281	10892	644	942
60	1639	1773	3498	4487	9233	10326	1880	2421
70	1518	1615	2834	3516	7620	8756	2626	3460
	Type I							
			630-0.783-28	630-0.783-56				
0	1513	1618	3235	3735	10187	11143	151	206
50			3703	4702				
60			3468	4160				
70			3133	3663				

Appendix C: Net Compressive Strengths

SlagDataSummary.xls Net Strengths

	Type II							
% slag	517-0.783-28	517-0.783-56	630-0.783-28	630-0.783-56	1103-0.41-28	1103-0.41-56	189-1.23-28	189-1.23-56
0	1745	2027	2303	2703	8447	9163	291	345
50	2577	2947	4515	5923	13095	14921	984	1463
60	2514	2719	5120	6315	13013	14200	2714	3519
70	2327	2476	4145	5067	10725	11975	3387	4463
	Type I							
			630-0.783-28	630-0.783-56				
0	2326	2489	4753	5443	14,458	15,032	254	337
50			5478	6913				
60			5147	6185				
70			4696	5474				

Appendix D: Deviations/Problems

Four types of problems occurred. First, some of the batches were not large enough to make 12 full cubes. Thus two batches had one cube that was short, one batch had three short cubes, and two batches had 11 short cubes and were missing one cube. However, in all cases, when turned on their sides for testing, the actual reduced cross-sectional areas

of the cubes were used in the strength calculations. In general, as cubes became shorter, the difference in strength between the cube in question and the average of the set of six became smaller. The magnitude of difference ranged from 1.1 to 2.4 MPa [155 to 354 psi], with an average of 1.6 MPa [228 psi]. Given the magnitude of the strengths being reported, it was felt that this small difference did not affect the analysis of the strength trends. For the two sets with all short cubes, the magnitude of non-standard height fell in the range of the other series' short cubes, and a statistical analysis of the whole data set with and without the outliers revealed no change in outcomes. Thus no corrections were applied.

The second type of problem dealt with non-standard age of the specimens when tested. Three sets were tested at 30 days instead of 28, and one set was broken at 49 days instead of 56. To correct for this, the strength data of each set was plotted versus time of curing, the equation of the line was determined, and the strengths for the non-standard test age specimens were corrected to the appropriate ages. Changes were nominal. A typical curve is shown in Fig.D1.

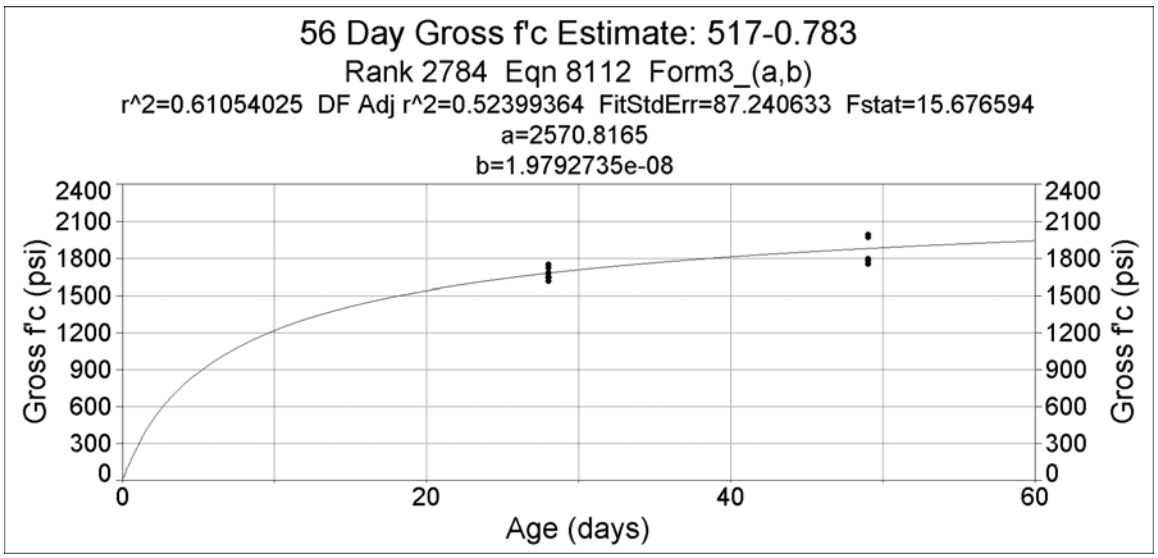

Fig. D1- Typical time-strength curve

The third possible problem concerned possible leakage of the molds. It was felt that for at least one of the sets, there was excessive leakage of paste out of the mold, thus increasing the relative amount of sand, which in turn would cause the entrapped air content to increase, thus increasing porosity.

Strength and dry density analysis indicated that a fourth problem occurred when one batch (374 kg/m^3 [630 lbs/cy] Type II PC 60% slag) apparently was made with the wrong type of cement. The results of this batch were not included in the analysis.