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Abrasion resistant concrete mix design for cold climates

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ABRASION RESISTANT CONCRETE MIX DESIGN FOR COLD CLIMATES

by

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ABSTRACT

Studded tire usage in Alaska contributes to rutting damage on pavements resulting in high maintenance costs and safety issues. In this study, binary, ternary, and quaternary highly-abrasion resistant concrete mix designs using supplementary cementitious materials (SCMs) were developed. The properties of fresh concrete and mechanical, and durability properties of hardened concrete for these mix designs were then tested to determine an optimum highly-abrasion resistant concrete mixture which could be placed in cold climates to reduce rutting damage. SCMs used included silica fume, ground granulated blast furnace slag, and type F fly ash. Tests including workability, air content, drying shrinkage, compressive strength, flexural strength, and chloride ion permeability were conducted. Resistances to abrasion, freeze-thaw cycles, and scaling due to deicer exposure were also measured followed by a preliminary cost analysis to compare different concrete mix designs. Within the scope of this study a quaternary mix design, containing primarily silica fume and slag, provided the overall best performance in terms of strength, durability, abrasion resistance, and cost.

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1. INTRODUCTION

1.1. PROBLEM STATEMENT

Wearing course rutting that causes progressive loss of surface material is a typical pavement distress occurring in the Central Region of Alaska and other northern states such as Washington and Oregon (Zubeck et al., 2004). This type of pavement damage is mainly due to the use of studded tires, which are thought to improve traction on compact snow and ice, but also tend to wear away the pavement surface in the wheel path and create safety issues such as depressions (Cotter and Muench, 2010). Millions of dollars in road maintenance is expended annually to address surface course wear and deformation of existing pavements (Malik, 2000; Zubeck et al., 2004). Using the best possible materials and construction practices is essential to optimizing pavement service. This has led to extensive research into developing numerous experimental features deployed nationwide to evaluate various innovative concrete materials, and construction practices for concrete that may yield better performance than traditional asphalt mixture, especially for pavements that are more resistant to studded tire wear.

In Alaska, concrete has been used in heavy traffic areas such as some intersections, portions of roads, and weigh-in-motion slabs on high-volume highways. Currently there are new mix design technologies proposed to reduce rutting due to studded tire wear, such as adding crumb rubber and steel fiber to concrete mixtures. In the meantime, concrete with commonly used additives is already in production and appears to be more durable and cost-effective. The key is to identify the optimum concrete mix design, and produce and implement cost-effective, abrasion-resistant, and durable concrete for cold region highway applications which are competitive with flexible pavement in terms of performance.

1.2. OBJECTIVE

The objective of this research was to identify and select an abrasion-resistant concrete mix design with good workability, mechanical properties, and durability which could provide the longest service life.

1.3. RESEARCH METHODOLOGY

To meet the objective of this study, the following major tasks were completed:

- Literature review and survey
- Laboratory testing and optimization of mix design
- Preliminary cost analysis
- Final report and recommendations

1.3.1. Literature Review and Survey. A comprehensive literature search of published materials (nationally and internationally) and on-going research projects on relevant materials practice and construction techniques for improving abrasion resistance and durability of concrete pavements was completed. In addition, interviews with Alaskan materials suppliers, public works directors, contractors and Alaska DOT&PF engineers was completed. A critical analysis of the practices and information collected from these interviews was used in the development of the mix designs used in this study.

1.3.2. Laboratory Testing and Optimization of Mix Design. The key for successfully using ternary mixtures is that a number of concrete mixes need to be formulated and tested to ensure their performance; the proportions of various ingredients should be tested to demonstrate that all the required concrete properties for a specific project meet the requirements (Schlorholtz, 2004). Hence, optimizing and finalizing a concrete mix design was completed by refining existing mix designs provided to Alaska DOT&PF (the silica fume mix designs developed by Anchorage Sand & Gravel served as a reference). This was achieved by producing different mixes with varying combination and contents of SCMs (i.e. silica fume, fly ash and slag) currently used in ready-mix applications. The experimental matrix was finalized upon discussions between the research team and professionals from Alaska DOT&PF and the Alaska concrete industry. Using primarily American Society for Testing and Materials (ASTM) standards, a series of lab tests for fundamental engineering properties and durability performance of concrete were conducted. These tests included:

- Workability (slump test for fresh concrete mixes, ASTM C143)
- Air content (ASTM C231 for Standard Air Meter and American Association of State Highway and Transportation Officials (AASHTO) TP118 for Super Air Meter)
- Mechanical properties related tests
    - compressive strength (ASTM C39)
    - flexural strength (ASTM C78)
    - shrinkage potential (ASTM C157)
- Durability tests
    - wear resistance (ASTM C944 and Abrasion by Studs, Method A: Prall Method)
    - freeze-thaw cycling resistance (ASTM C666)

- resistivity – concrete's ability to resist chloride ion penetration (ASTM C1202)
- frost scaling resistance after freezing-thawing cycle (ASTM C672)

All mechanical properties were tested at 7, 14, and 28 days. In addition, as a basic performance indicator, compressive strength was tested at one and three days as well to capture the early age characteristics of the material and to compare results at standard test ages, such as the 28 day test age. The effects of design parameters on mechanical properties were investigated to narrow the selection of parameters and determine the optimum mix designs.

Durability tests were conducted at 28 days except freeze-thaw (F-T) cycling resistance which was tested at 14 days as per ASTM C666. The air content of the screening test mixtures was measured using a Super Air Meter following AASHTO TP118. The air content of the performance test mixtures were measured using an Air Meter and ASTM C231.

1.3.3. Preliminary Cost Analysis. A preliminary cost analysis was used to estimate and compare the costs of constructing a concrete pavement in Alaska using the optimum mix designs.

1.3.4. Final Report and Recommendations. A final report was completed upon the completion of the previous tasks. The report included a summary of literature review and survey responses, descriptions of procedures and results from the laboratory testing, the optimization process for determining the optimum mixture designs and a preliminary cost analysis comparing the optimum concrete mixtures determined through the analysis. The project's findings were also outlined and future areas of research were recommended.

2. LITERATURE REVIEW AND SURVEY

2.1. LITERATURE REVIEW

A literature was completed which reviewed studded tire wear rates in Alaska and their effects on pavement. The properties of concrete relating to abrasion-resistance and strength, as well as the effects on concrete from deicer exposure, chloride ion penetration, and F-T cycles, were reviewed. A summary on supplementary cementitious materials (SCMs), including fly ash, slag and silica fume, was also completed.

2.1.1. Studded Tire Wear. Studded tires degrade pavements, cause rutting and depressions (Zubeck et al., 2004), and contribute to dust emissions (Kupiainen and Pirjola, 2011). Studded tires also help improve driving safety on snow and ice and have been found to have a positive impact on Alaska’s economy (Zubeck et al., 2004). Without studded tires crash rates have been found to increase. One Norwegian study found that a 25% decrease in studded tire usage correlated with a 5% increase in crashes (Elvik et al., 2013). If 25-50% of vehicles use studs, safety is improved for both studded and non-studded vehicle by limiting ice buildup and road polishing (Do et al., 2007). Estimates in Washington State put studded tire wear rates on concrete pavements at 0.01 per million studded tire vehicle passes (Cotter and Muench, 2010).

Studded tire use, by Alaska State Law, is allowed only from September through either May or April, depending on location (Alaska Statutes, 2018). Despite seasonally limits on studded tire usage, pavement rutting due to studded tire wear is still a problem. 1992-1993 Anchorage investigations on rutting rates found that 67-78% more rutting occurred during the winter than the summer (Frith et al., 2004). Frith et al. (2004)

estimated that if studded tire wear rates were minimized, pavement longevity in Anchorage and Juneau could be extended by 40% and 90%, respectively, which could potentially result in substantial construction and maintenance cost savings.

Recent measurements by Abaza et al. (2019) in Anchorage found studded tire usage to be 35%, suggesting usage rates have dropped since the earlier 53% rate determined by Zubeck et al. (2004). These decreases are partially due to technological advances and increased usage of studless tires and all-season tires. The costs associated with studded tire damage in Alaska is estimated to be $13.7 million, over 40 times the State of Alaska's revenue from studded tire sales and installations (Abaza et al., 2019).

2.1.2. Concrete Properties. There are numerable factors which can affect a concrete's resistance to abrasion, deicer scaling, chloride penetration, and F-T cycles. Some of the research regarding these factors is summarized below.

2.1.2.1. Abrasion-resistance. Numerous environmental and design factors can affect a concrete pavement's performance in the field. One important parameter for concrete pavements is their ability to resist abrasion from studded tires especially in Alaska where ruts can sometimes exceed 25 mm (Zubeck et al., 2004). In general it has been found that rigid pavements have lower wear rates, as shown by research in Oregon which found that asphalt pavements had wear rates over four times those of concrete pavements (Brunette and Lundy, 1996). Other research by Lundström et al. (2009) measured rutting on 19 test sections of rigid, semi-rigid and flexible pavements on a Swedish road. Similar to Alaska, Sweden allows studded tires only during certain times of the year. After seven years, rutting from abrasion averaged 1.7 mm on rigid pavements, 3.4 mm on semi-rigid and 3.1 mm on flexible pavements.

One factor which affects abrasion resistance is the water to cementitious material (w/c) content. Liu (1981) found that reducing the w/c ratio from 0.72 to 0.40 improves 72-hour abrasion resistance by 43% and recommends both hard aggregates and a low w/c ratio be used for good abrasion resistance. Compressive strength should also be considered. Liu (1981) also found a positive correlation between 72-hour compressive strength and abrasion resistance.

A substantial amount of research has been done on measuring the abrasion resistance of concrete mixtures containing SCMs. When looking into the effects of adding slag, Fernandez and Malhotra (1990) tested the abrasion resistance of air-entrained concrete mixes containing slag at 25% and 50% replacement levels at w/c ratios of 0.45, 0.55, and 0.70. The slag mixes, no matter their w/c ratio or slag content, had lower abrasion resistance than the control. This lower resistance was assumed to be partially due to the low compressive strength of the slag mixtures.

Researchers have also investigated the effect of adding fly ash to concrete and its effect on abrasion resistance. Harwalkar and Awanti (2014) tested the abrasion resistance and compressive strength of 60% class F fly ash samples with a 0.3 w/c ratio versus two all-cement mixes with w/c ratios of 0.3 and 0.35. Following the Australian MA20 test method, which is similar to ASTM C779, 28 day abrasion resistance was measured. The fly ash mixes were found to have 90% the abrasion resistance of the control mixes.

Naik et al. (1995) found that as the air content of mixtures containing fly ash increased, compressive strength decreased, but that compressive strength played a larger role in abrasion resistance rendering the impact of air content insignificant. When testing abrasion resistance at 28, 91, and 365 days, resistance was found to decrease with age.

Similarly Yen et al. (2007) also found that by reducing w/c ratio and increasing compressive strength, the abrasion resistance of class F fly ash mixes increased. After testing mixtures containing 15, 20, 25, and 30% fly ash, the 15% fly ash mix had similar abrasion resistance to the control while the mixtures with fly ash contents over 15% had lower abrasion resistance than the control.

Atiş (2002) tested the abrasion resistance of mixtures containing higher replacement levels of fly ash at 50% and 70% and found the fly ash mixtures had higher abrasion resistance than the control. A positive correlation was also found between compressive strength and abrasion resistance. The effects of using a superplasticizer on the mixtures was also investigated and was found to not significantly impact results.

Rashad et al. (2014) tested the abrasion resistance of eight binary, ternary and quaternary high-volume fly ash mixtures over 180 days. For each mix the 70% fly ash content was partially replaced with either slag or silica fume or equal parts both. Although the all-cement control mix had the highest abrasion resistance, the quaternary mix of fly ash, slag, and silica fume, as well as the ternary mixture of fly ash and silica fume, did have improved abrasion resistance over the binary fly ash mix. The ternary mixtures of slag and fly ash had the lowest compressive and abrasive resistance. Similar to Liu (1981) a strong correlation was found between compressive strength and abrasion resistance (R^2=0.93, using a polynomic equation). To improve both the compressive strength and abrasion resistance in high volume fly ash mixtures, the authors suggest adding silica fume with or without slag, but not just slag due to its poor performance.

2.1.2.2. Deicers. F-T cycles deteriorate concrete due to the exposure of deicing salts on the surface of the concrete, which results in scaling and internal cracking (Pigeon et al., 1996). Deicers degrade concrete pavements by interfering with cement-aggregate interactions, resulting in chemical reactions between the cement and deicers (Shi et al., 2009). Having entrained air in concrete reduces deterioration due to F-T cycles and improves salt scaling resistance. This is possible by allowing water within the concrete to move into pores and expand, reducing potential stress. Each concrete slab has a critical air void spacing factor at which internal cracking can be mitigated. This spacing is the distance between particles in the paste and the nearest air void. Shon et al. (2018) investigated this spacing in binary and ternary mixes of fly ash and silica fume and found the critical air-void spacing to be 200 and 300 microns for binary and ternary mixtures, respectively. Although the air void spacing within a concrete system is important, Jin et al. (2013) found that the air void size-distribution has a larger effect on F-T resistance. Research has found the addition of some SCMs, such as metakaolin, silica fume and slag, also contribute to a more reasonable air void size-distribution (Duan et al., 2013).

Nehdi et al. (2004) testing the scaling resistance of binary, ternary and quaternary self-compacting concrete (SCC) mixes with and without viscosity modifying admixtures (VMA). SCMs investigated included fly ash, slag, silica fume, and rice husk ash. The control performed the best, followed by the ternary mix with a VMA. The other ternary and quaternary mixes also performed well. The binary fly ash mix without a VMA performed the worst, with six times the average cumulative mass loss of the other mixes.

Whiting (1989) used ASTM C672 to investigate the effects of scaling due to deicer salt exposure when replacing cement with 0, 25, and 50% fly ash. Fly ash samples were found to have higher levels of scaling over the all-cement control. A lower w/c ratio was also found to improve scaling resistance. Pigeon et al. (1996) corroborated this, noting that in general fly ash tends to decrease concrete's resistance to salt scaling, whereas when slag is added, researchers have mixed results.

2.1.2.3. Chloride ion penetration. Concrete is exposed to chlorides in various ways including from exposure to deicing salts or marine environments. When chlorides ingress into concrete this can reduce durability by corroding the reinforcing steel. To investigate a concrete's resistance to chloride permeability, lab tests are used to predict field performance. Tempest et al. (2017) investigated the chloride permeability of mixtures prepared in the lab and in the field and found good correlation between the two.

Chung et al. (2010) investigated binary mixtures containing either 10% silica fume or 20% fly ash. The effects of varying air content to 2, 4 or 6%, as well as varying w/c ratios of 0.4, 0.5, or 0.6 were investigated as well. Duplicate samples were made of which half were exposed to 300 F-T cycles before testing their chloride penetration. Afterwards both the F-T samples, and samples which hadn't been tested, were placed in salt solutions and their chloride ion penetration was measured. As the air content and w/c of samples increased, the chloride penetration increased as well. Samples which had been exposed to F-T cycles were also found to have higher permeability than samples which had not. As the age of the samples increased, the chloride ion penetration was reduced. The addition of fly ash was found in all but one case to reduce the chloride penetration while the addition of silica fume reduced permeability even further.

Investigations into the effects of air entrainment on the chloride permeability of concrete mixtures exposed to F-T cycles found that irrespective of air entraining admixture (AEA) content, the permeability of all-cement concrete mixes varied widely as samples were exposed to over 600 F-T cycles (Saito et al., 1994). Conversely, when AEA was added to mixtures containing fly ash or slag to obtain minimum air contents of 5.3% and 3.4%, respectively, changes in permeability after exposure to F-T cycles was minimal, but substantial increases in permeability, of two to four times the initial permeability, were seen in SCM mixtures without an AEA.

Nehdi et al. (2004) studied SCC binary, ternary and quaternary mixes containing various SCMs. Chloride ion penetration was measured at 28 and 91 days. At 28 days all the SCM mixtures had low to moderate penetration of 1,000-4,000 Coulombs while the control had a high penetration of over 4,000 Coulombs. By 91 days the penetration rates decreased and the SCM mixtures had very low penetration (less than 1,000 Coulombs) while the control mix had moderate penetration (2,000-4,000 Coulombs).

Yang et al. (2017) tested binary mixtures containing either 40% slag or fly ash with 0.42 and 0.50 w/c ratios. They also investigated different wet curing times by wet curing samples for two, five and eight days before dry curing. Chloride permeability was tested at 28 and 360 days. The lowest permeability was seen in samples wet cured for eight days, followed by those cured for five and two days. Regarding the effects of SCMs, overall the slag mixtures had the lowest permeability, followed by the fly ash mixtures, and lastly the control.

2.1.2.4. Freeze-thaw resistance. To minimize the effects of frost action and F-T cycles on concrete AEAs are usually added to the paste during mixing. Naturally occurring entrapped air voids are too large and spaced too far apart to provide such benefits (Bassuoni and Nehdi, 2005). Unfortunately the addition of an AEA usually results in reduced strength, and although AEAs decreases the number of large pores, their addition does not decrease total porosity (ACI, 2012).

Air entrained (AE) concrete is used in cold regions to help mitigate the effects internal cracking and pressure due to entrapped water. When microscopic air bubbles are formed inside the concrete, water can move to these areas, which allow them to freeze and expand, and subsequently reduce the pore pressure of the pores within the concrete. This reduced potential pressure then contributes towards mitigating cracking and other durability issues. The chemical mechanisms which occur when adding AEAs to fresh concrete to improve the F-T durability are complex, and innumerable factors including the materials used, mixing procedure, and type of AEA used effect the final air content (Du and Folliard, 2005). The two main processes necessary for entrainment include the formation of the air bubbles and subsequently their stability as the concrete hardens. A minimum air content of 6% was found to provide adequate air entrainment (Wang et al. 2009).

The downside to adding an AEA to increase air content is reduced compressive strength. For example, Zhang et al. (2018) found that when adding AEAs to normal concrete to obtain up to a 5% air content, compressive strength was not affected, but once the air content surpassed 7%, compressive strength was reduced significantly. They also found that the compressive strength of the fly ash mixtures investigated were even more severely affected by high air contents. Additional AEA has been found to be necessary for mixtures containing fly ash compared to that of an all-cement mix, but if too much AEA is added this may result in air voids combining to form bigger voids which reduce permeability and durability. Therefore care should be taken when determining proper dosage of an AEA especially in mixtures containing SCMs. In addition, for every 1% AEA added, a 5% decrease in strength can be expected (Korhonen, 2002).

To investigate the effects of SCMs on the F-T resistance of concrete, Bleszynski et al. (2002) placed concrete slabs consisting of binary and ternary mixes of slag and silica fume on a road in Ontario which heavy trucks used. The slabs were exposed to both deicer salts and annual F-T cycles. Overall the ternary blends had improved durability over both the plain portland cement concrete (PCC) and binary mixes.

The ASTM 666 testing method measures the F-T durability of samples. Shon et al. (2018) tested 14 mixtures including binary mixtures containing 5% silica fume and up to 45% class F fly ash, as well as ternary mixtures containing both. Two duplicates of each ternary mixture were made, one with an AEA and one without. At 14 days, the compressive strength of the binary silica fume mixture had the highest strength, followed by the binary fly ash mixtures, and then the ternary mixtures. Non-air-entrained (Non-AE) mixtures had higher compressive strength than those with. As fly ash content increased, compressive strength decreased. Concerning the durability factor, which is determined based on the RDME value and the cycles passed, the ternary AE mixtures performed the best while the non-AE ternary mixtures generally had the lowest durability factors. The durability factor ranges from 0-100% and is indicative of the concrete's durability. A higher durability factor is indicative of a high resistance to F-T cycles whereas a lower durability factor suggests a resistance to F-T cycles.

Toutanji et al. (2004) investigated the F-T resistance of 14-day cured SCM mixes. 16 mixes were tested including binary mixes of 8-15% silica fume, 60-80% slag, or 20-30% fly ash, as well as ternary mixes of fly ash and slag, and quaternary mixes containing all three SCMs. Of the 13 mixes, the control had the highest durability factor (which was 89.7%), followed by the 8% silica fume (34.9%), 70% slag (26.5%), and the quaternary mixes. Binary fly ash mixes performed the worst, with binary slag mixtures performing slightly better. Despite the high durability of the 8% silica fume mix, the 15% silica fume mix had the second-lowest durability factor. Overall the quaternary mixes performed better than the binary mixes with the authors concluding that the combination of SCMs may have resulted in a more stabilized mix with better F-T resistance.

Chung et al. (2010) tested binary mixes of 10% silica fume or 20% fly ash with w/c ratios of 0.4, 0.5, or 0.6 and 2, 4, or 6% air content. After ASTM 666 F-T testing all mixes had durability factors over 95%. Nonetheless the silica fume mixes performed slightly better than the fly ash mixes, potentially due to the early pozzolanic reactions of silica fume, which contribute to early age strength. The varying air content did not seem to affect the durability factor. Another study found that using steel-fiber reinforced crumb rubber could be a solution to combat pavement deterioration in high-traffic areas due to F-T action (Abaza and Aboueid, 2018). The steel-fiber reinforced rubber concrete developed was found to have improved frost-resistance over the standard PCC tested.

Although researchers use F-T testing to predict how concrete will perform, Mehta (1991) argues that laboratory F-T tests are more extreme than what would occur in the field. F-T tests expose the concrete too early to freezing, and don’t always predict the concretes field performance, especially for samples which test poorly.

2.1.2.5. High strength concrete. High strength concrete, which usually have strengths exceeding 6000 pounds per square inch (psi), typically has a w/c ratio less than 0.4 (Mehta, 1999). To achieve workability at these lower w/c ratios, a water reducer is used. A lower w/c ratio results in lower permeability, which is the key to durability against aggressive environments. Good workability is also important and allows for better pumping and filling of forms. This can then save constructions costs, especially on large projects and those with tight reinforcing spacing. A high cement content can also cause thermal cracking, but by using mineral admixtures this effect can be lessened. High strength concrete can also be made from using high volumes of fly ash with a low w/c ratio. For example, one mixture containing almost 60% fly ash and a w/c of 0.288 had low early strength of 1200 psi at one day but 12000 psi at 28 days (Malhotra et al., 1994).

2.1.3. Supplementary Cementitious Materials. To minimize rutting and pavement degradation, SCMs can be added. For structural applications, the Alaska DOT&PF Highway Construction manual (2017) requires limiting the combination of two or more SCMs to a combined 40% replacement level. Individual replacement levels are limited to 35% fly ash, 40% slag, and 10% silica fume. For concrete pavement highway construction in Alaska, no standards exist, but for airport concrete pavement, construction standards do exist. In rigid airport pavement, fly ash content is limited to a 20%, and concrete is required to be designed to meet a 28-day 735 psi flexural strength.

Many SCMs, including fly ash, silica fume and slag, are pozzolanic and contain high amounts of amorphous silica and alumina. When added to hydrating cement the silica and alumina in pozzolans react with calcium hydroxide (CH) products to product additional strength-contributing products such as calcium silicate hydroxides. If properly proportioned and cured, these products help improve ultimate strength and durability, reduce shrinkage and improve resistance to chemical shrinkage and ASR (Shi and Day, 2001). As the reactivity of an admixture increases, so does the early age strength of the concrete (Li and Zhao, 2003). Silica fume’s pozzolanic reactivity is 1.29, fly ash’s is 0.875, and ground granulated blast furnace slag’s (GGBFS) is 0.040 (Khan et al. 2014).

This pozzolanic reactivity was determined through the Chapelle test, which measures the pozzolanic activity based on the CH consumed after being placed in a diluted slurry of the pozzolan. Swamy (1997) emphasizes the importance of moist curing for concrete mixtures containing either fly ash or slag. Without proper curing samples usually don’t achieve their target 28-day strength.

2.1.3.1. Silica fume. Due in part to their high amorphous silica content and small size, silica fume particles act as pozzolans, helping to improve long term strength and durability. Size varies, but a rough estimate puts the diameter of a silica fume particle at a tenth of a micron (Aïtcin, 2016). Because of their small size, with a 15% cement replacement level, there are approximately two million silica fume particles for each cement particle (Cohen et al., 1990). Although the addition of silica fume helps improve strength through increased packing density and pozzolanic reactions, its limitation would be its price point of roughly 10 times the cost of cement (Ženíšek et al., 2016).

At lower w/c ratios, silica fume has been found to help mitigate chemical attacks due to decreased permeability and reduced CH content (ACI, 2012). Silica fume also improves resistance to alkali-silica reaction (ASR), and electrical resistivity. Higher electrical resistivity potentially reduces corrosion of reinforcing steel placed in concrete with silica fume (ACI, 2012). Mehta (1985) exposed concrete samples containing 15% silica fume to six different acids and sulfates. Of the six, concrete containing silica fume had improved chemical resistance over the control for all solutions except ammonium sulfate. Similar results were found in the field when measuring the chlorine penetration on the IL 4 bridge in Illinois. The deck overlay containing 10% silica fume had higher resistance to chloride penetration than the control (Detwiler et al., 1997).

Adding silica fume typically increases water demand and particle packing due to its high surface area of 15,000-25,000 m^2/kg, which is over triple that of cement particles (King, 2012). To maintain a low w/c ratio, water reducers can be added. When using silica fume, a low w/c ratio is “the single most important factor” (Jahren, 1983).

When mixing, silica fume should be mixed in as soon as possible to ensure dispersion throughout the mix and for particles to wet (Jahren, 1983). Because the addition of silica fume results in a sticky paste, slump should also be increased 20 to 30 millimeters in order to maintain a similar workability to all-cement mixtures (Jahren, 1983). When using silica fume, the AEA demand increases 125% to 150% (ACI, 2012).

Because of the fineness of silica fume particles, the heat of hydration is increased (ACI, 2012). At higher w/c ratios (such as 0.50), silica fume accelerates cement hydration while at lower w/c ratios (such as 0.35) the addition of silica fume retards both the start of hydration and the acceleration period (Langan et al., 2002).

As silica fume content increases, bleeding decreases (ACI, 2012). This is in part due to the increase in fines which increases cohesiveness, and the high surface area of the silica fume particles which get coated in water (Panjehpour et al., 2011). Although bleeding is reduced, shrinkage cracking may increase since the water may evaporates faster than the concrete bleeds, leaving behind a drier surface (ACI, 2012). Due to concerns over increasing shrinkage and cracking at early stages, it is important to ensure proper curing during early stages, since the addition of silica fume has been found to contribute to autogenous shrinkage (Jensen and Hansen, 1996).

When simulating a concrete culvert wall, Kanstad et al. (2001) found that silica fume mixtures had only marginally lower risks of cracking over the control. Whiting et al. (2000) also found little difference in long term cracking between mixtures with and without silica fume, with slightly higher cracking observed during the early ages of the silica fume mixtures. They recommend a 6-8% silica fume content and also recommend moist curing bridge decks for at least seven days to mitigate cracking. The curing method is also important. Jahren (1983) found that when silica fume mixtures were wet cured this resulted in higher tensile and compressive strengths over those dry cured.

The primary purpose of silica fume is to increase durability which is achieved by reducing permeability (ACI, 2012). Silica fume improves compressive strength, particularly at 28 days (Siddique, 2011). The addition of silica fume has been found to decrease abrasion resistance but the use of coarse aggregates and the w/c ratio has been found to have a larger effect (Laplante et al., 1991). Regarding the dosage, Toutanji et al. (2004) determined the optimum dosage for silica fume to be 8% after investigating the compressive strength and F-T resistance of binary, ternary and quaternary mixtures containing 8-15% silica fume with fly ash and slag.

2.1.3.2. Fly ash. Fly ash is a byproduct of coal combustion. When coal is burned various byproducts are produced including fly ash, which is carried into the air during combustion and collected. There are two types of fly ash: Class C and F. Their class is determined by the sum of their SiO_2, Al_2O_3, and Fe_2O_3 oxides. For class C the sum of these oxides should be at least 50% of their chemical composition. For class F, a minimum of 70% content is required (ASTM, 2019). The optimum fly ash content for 28-day and 180-day compressive strength was determined to be 40%, which Oner et al. (2005) determined after testing class F fly ash samples at various w/c ratios of 0.50-0.94 and at replacement levels of 15-58%.

Harwalkar and Awanti (2014) tested the abrasion resistance and strength of samples with 60% class F fly ash and a 0.3 w/c versus two all-cement mixtures with w/c ratios of 0.3 and 0.35. The fly ash mixture had lower compressive and flexural strengths at seven, 28, and 90 days. Substantial strength gains occurred after seven days, due to the slow pozzolanic reaction. By 90 days the fly ash mixture's strength was almost identical to the 0.3 w/c all-cement mix.

When testing mixtures containing 40-50% class F fly ash at 28 days, the compressive, splitting tensile, flexural strength and abrasion resistance of the mixtures containing fly ash were all lower than the all-cement control (Siddique, 2004). The fly ash mixtures later age strength, which was measured over one year, did increase due to the late age pozzolanic reactions, but even after a year the strength of the fly ash mixture samples did not surpass the control. Due in part to their late age strength gains, using high volumes of fly ash in concrete can produce strong, durable concrete for use.

The calcium content of a fly ash best predicts its performance in concrete especially in respect to the heat of hydration and mitigating ASR and sulfate attack (Thomas et al., 2007). With each 10% fly ash cement replacement, water demand is reduced roughly 3% (Thomas et al., 2007). Due to the reduced water demand, bleeding is reduced. If water is not reduced when fly ash is added, bleeding will increase. If properly proportioned, drying shrinkage is reduced due to the lower w/c.

The addition of fly ash retards the initial and final set times of fresh concrete, which could be detrimental in cold climates. The use of fly ash also reduces the heat of hydration, and improves the long term flexural and tensile strength due to the pozzolanic reactions. If cured properly, the addition of fly ash helps reduce permeability. If fly ash mixes are exposed to F-T cycles and deicers, its replacement levels should be limited (Thomas et al., 2007).

2.1.3.3. Slag. Using slag as an SCM helps improve durability, increases resistance to chlorides and sulfates, and reduces ASR (Hooton, 2000). Slag replacement levels usually do not exceed 50%, but one study which used 78% slag content with a 0.28 w/c ratio found the strength to be only 1900 after one day, but 13000 psi at 28 days, with high resistance to salt scaling and F-T cycles (Lang and Geisler, 1996). When using up to 60% slag replacement, initial and final setting times are increased (Özbay et al., 2016). In addition to later set times, the use of slag has been shown to increase the amount and rate of bleeding, which is primarily due to delays in the hydration and formation of hydration products. Researchers have also found that using GGBFS may lead to increased thermal expansion and autogenous shrinking, higher flexural strengths after seven days, and reduced permeability due to the reactions with CH and alkalis (Özbay et al., 2016).

Fernandez and Malhotra (1990) tested the abrasion resistance of AE samples containing 0, 25 and 50% slag at w/c ratios of 0.45, 0.55, and 0.70. Mixtures containing slag had lower seven-day compressive strengths due to slag's slow rate of hydration. By 28 and 91 days mixtures containing 25% slag had similar compressive strengths to the control, but when replacing cement with 50% slag the compressive strength dropped lower than the control. When testing abrasion resistance, the slag mixtures, no matter their w/c ratio or slag content, had lower abrasion resistance than the control mixtures, which the authors attributed to the low compressive strength of the slag mixtures. When testing chloride ion penetration it was found that at higher slag contents, permeability was reduced substantially, irrespective of the w/c ratio. This was attributed to the smaller pores in the slag concrete versus the all-cement control. The optimum replacement level for using slag in cement for maximum compressive strength was determined to be 55% (Oner et al., 2005). This being said, slag is usually added at 25-50% replacement levels to mitigate ASR expansion (Bleszynski et al., 2002).

2.1.3.4. Ternary mixtures. In part due to the small size of fly ash and silica fume particles, using these SCMs together can reduce concrete permeability by filling in open pores between cement particles (Shon et al., 2018). Mehta and Gjørv (1982) found that when replaced 30% of cement volume with fly ash, compressive strength was lower than the control at three, seven, and 28 days, but was similar at 90 days. When using a 30% silica fume replacement, and changing the aggregate proportions to improve workability, the compressive strength of the silica fume mixes were higher than the control at all ages. When replacing cement with both 15% silica fume and 15% fly ash, the early age compressive strength at three and seven days was similar to the control, but by 28 and 90 days, the compressive strength of the ternary mixture had exceeded the control's strength. Mehta and Gjørv (1982) concluded that using a ternary mixture of both fly ash and silica fume may provide superior results over a binary fly ash mix.

Shehata and Thomas (2002) tested various mixtures containing high and low alkali cements, silica fume, and different types of fly ash with varying amounts of calcium. Twenty mixes, all with a 0.5 w/c ratio, were tested including five all-cement control mixes, eight 15-60% binary fly ash mixtures, a binary 5% silica fume mix, and six ternary mixtures containing 10-30% fly ash with 5% silica fume. Expansion was measured over two years. The control mixtures had the highest expansion while the addition of 5% silica fume alone did not mitigate expansion. The addition of fly ash or both fly ash and silica fume did reduce expansion. For fly ash samples, irrespective of the silica fume content, two-year expansion decreased as fly ash content increased.

Langan et al. (2002) investigated the cement hydration of binary silica fume or fly ash mixtures, as well as a ternary mixture of 10% silica fume and 20% fly ash. They found the addition of silica fume alone at high w/c ratios increased hydration, but at lower w/c ratios, hydration was retarded. The addition of silica fume increased the dormant period, reduced the acceleration period, and increased the deceleration period. As the w/c ratio increased, silica fume reactivity accelerated. In the binary fly ash mixtures as the w/c ratio increased, the retardation effect increased, whereas in the ternary mixtures hydration was found to be significantly retarded.

Khan (2003) investigated the permeability of binary and ternary mixtures containing up to 40% fly ash and up to 15% silica fume. Understanding this relationship is important for as a concrete's permeability increases, durability decreases. The addition of silica fume was found to decrease permeability at all ages up to 180 days regardless of fly ash content. Optimum silica fume content was determined to be 8-12%. The addition of fly ash minimally reduced permeability and porosity, while the addition of silica fume greatly reduced these characteristics. The largest effects were seen in silica fume replacement levels up to 10% after which effects leveled off. When considering strength and porosity, ternary mixtures containing both silica fume and fly ash performed better than either alone. These results corroborated with the ACI 234-06 Report (2012), which noted that using both silica fume and fly ash together works better than either alone.

When silica fume is combined with slag it has been found to have higher resistance to sulfate attack, and lower permeability and chloride diffusivity (ACI, 2012).

Scholz and Keshari (2010) looked into developing an abrasion-resistant concrete mix for the Oregon Department of Transportation using silica fume, fly ash, and slag. They found that a slag and silica fume combination had better durability, compressive strength and abrasion resistance over a fly ash and silica fume combination.

To investigate the effects of combining slag and fly ash in a ternary mix, Hale et al. (2008) tested 12 mixes using three different cements. For each cement type an all-cement control, binary mixes containing 15% fly ash or 25% slag, and a ternary mix of both 15% fly ash and 25% slag were tested. They found the fly ash mixtures had improved workability, higher air contents, and later set times over those containing slag. The slag mixes overall had improved compressive strength, modulus of rupture, and modulus of elasticity values over the fly ash mixes. The authors concluded the addition of slag had overall positive effects while the addition of fly ash had mixed effects.

2.1.3.5. Quaternary mixtures. Gesoğlu and Özbay (2007) tested 22 binary, ternary and quaternary SCC mixtures containing slag, fly ash, and silica fume at a 0.32 w/c ratio. Fly ash and slag was added at 20, 40, and 60% replacement levels while silica fume was dosed at 5, 10, and 15% replacement levels. The addition of SCMs were found to improve fresh properties by reducing slump flow time. Binary mixtures of fly ash or slag were found to have retarded set times while binary silica fume mixtures had earlier set times. Fly ash mixtures also had lower compressive strength while ternary slag and silica fume mixtures had strengths exceeding those of the control. The addition of SCMs also generally improved electrical resistance.

Gesoğlu et al. (2009) later studied the same 22 mixtures this time with a 0.32 w/c ratio. For 90-day compressive strength, mixtures containing fly ash generally had lower compressive strengths, while the binary and ternary mixes containing silica fume, slag, or both had higher compressive strengths similar to the control. The addition of silica fume was found to increase the superplasticizer demand which then increased viscosity. For both chloride and water permeability it appeared that the addition of SCMs reduced permeability. An optimum mix based on the experimental results was determined. Parameters required a low chloride permeability, electrical resistivity, sorptivity, water permeability, and shrinkage. This mix was determined to contain (by cementitious mass) approximately 1.2% fly ash, 43% slag, 14% silica fume, and 1.4% superplasticizer.

Li and Zhao (2003) tested an all cement-mix, a binary 40% fly ash mix, and a ternary 25% fly ash and 15% slag mix. At 28 days the binary fly ash mix had the lowest compressive strength, but after one year its compressive strength had exceeded both the ternary and all-cement mixtures with the binary fly ash mix having a one year strength of 107 MPa, the all-cement measuring 96 MPa, and the ternary mix measuring 99 MPa. When comparing the early-age hydration of the mixes at seven days, the ternary mixture had increased early age hydration and no un-hydrated particles visible, whereas for the binary fly ash mix, hydration was retarded and many un-hydrated particles were visible. After immersing samples in H_2SO_4 and measuring their compressive strength, the ternary mix performed the best, with higher relative strength over the all-cement control and binary fly ash mixtures.

Nehdi et al. (2004) investigated the durability properties of seven SCC mixtures including binary mixtures of 50% class F fly ash or 50% slag, ternary mixtures of 25% slag and 25% fly ash, and quaternary mixtures containing 20% slag, 24% fly, and either 6% silica fume or rice husk ash. Rice husk ash is a pozzolan formed from burning rice husks. The ternary slag and fly ash mixture had the highest compressive strength at 28 and 91 days at over 45 MPa, but lower early age strength than the control. The 50% fly ash mixture and the quaternary silica fume mixtures had the lowest 91 day compressive strength values of less than 30 MPa. When chloride ion penetration was measured at 28 days all of the mixtures had low to moderate penetration except the control which had a high penetration rating. By 91 days the control had moderate penetration while the SCM mixtures had very low penetration. Testing on the effect of deicing salt scaling after F-T cycles found that the binary fly ash mixture which didn’t contain a VMA performed the worst, while adding the VMA greatly improved its resistance. The visual rating of the scaling was the best for the all-cement mix and the ternary mixture containing a VMA. Expansion over nine months when submerged in a sulfate solution found the control had the highest expansion (0.13%) followed by the ternary mixtures (0.05%), with the quaternary mixtures the lowest (0.01%). The researchers concluded that replacing high volumes of cement caused decreased early age strength, but ternary and quaternary mixtures result in much lower chloride ion penetrability.

Kim et al. (2016) tested binary, ternary, and quaternary mixtures containing slag, fly ash, and silica fume with contents ranging from 25-65%, 15-30%, and 5%, respectively. Of all the samples, the binary mixture containing 5% silica fume had the highest compressive strength, splitting tensile strength, modulus of elasticity, and Poisson’s ratio. The authors attribute this to silica fume’s small particle size which can fill the voids between the larger cement, fly ash, and slag particles.

Rashad et al. (2014) tested binary mixtures with 70% fly ash, ternary mixtures with 50-60% fly ash and either 10-20% silica fume or slag, and quaternary mixtures with fly ash, silica fume, and slag. Supplementing concrete with 70% fly ash reduced compressive strength by 66% at 28 days and 38% at 180 days. At all ages, from seven to 180 days, the control mixture had the highest 180-day compressive strength at over 60 MPa. Following this, binary fly ash mixtures, and quaternary and ternary mixtures containing silica fume all had similar strength at 180 days of 35 to 40 MPa. Samples containing 10-20% slag had the lowest compressive strength at 180 days.

2.2. SURVEY

Alaska DOT&PF material engineers and lab technicians, a bridge engineer, researchers, private contractors, concrete suppliers, and public work directors in Alaska were surveyed about their experience regarding concrete pavements in Alaska and efforts made to combat abrasion resistance in concrete pavements. Because there are few concrete pavements in Alaska, to gain perspective from a state which regularly installs concrete pavements, two Wisconsin Department of Transportation (WisDOT) pavement engineers were also surveyed.

2.2.1. Concrete Pavements in Alaska. There are few concrete pavements in Alaska. In Alaska's central region (Figure 2.1), there are some concrete intersections in Anchorage including the high-traffic intersections at 5th street and E street, and 6th street and F street (Johnson, 2019), as well as some low traffic intersections in residential areas (Schlee, 2019). The Anchorage International Airport, at one point had concrete pavement, but is being repaved with asphalt, but there are some concrete hardstands at the Anchorage airport where planes park (San Angelo, 2019). In the northern region of Alaska the only places where concrete and vehicle tire wheels intersect is on bridge decks and some weigh in motion slabs (Currey, 2018). There are some concrete pavements at both the Ft. Wainwright Airport (Mappa, Inc., 2018), and at the Fairbanks International Airport, where there are also concrete hardstands for planes (San Angelo, 2019). The Eielson Airport was also concrete but has been paved over (Connor, 2019).

In southcoast Alaska there are concrete pavements in communities including Petersburg, Wrangell and Ketchikan (Harai, 2019; San Angelo, 2019). Ketchikan had concrete roads as early as the 1960s (Connor, 2019), and although some remain, many have been paved over with asphalt (Hilson, 2019). In Wrangell there are around a half

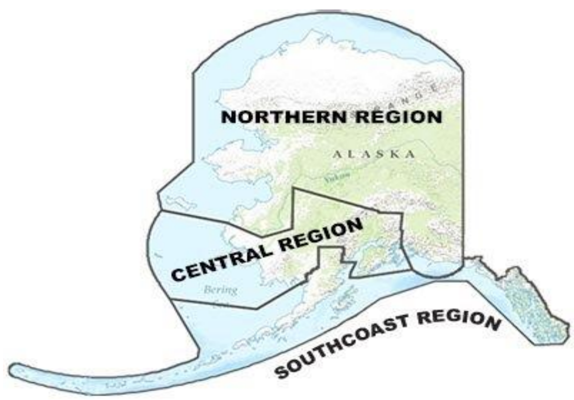


Figure 2.1 Alaska DOT&PF three regions (Alaska DOT&PF website

dozen streets paved with concrete with all but the main street around 20 years old (Howell, 2019). The only concrete road Wrangell has redone is the main street in 2011, which now contains fiberglass fibers and was redone after 37 years of service. Magnesium chloride deicers are applied each winter to the pavements there with no reported durability issues (Howell, 2019).

One concrete pavement many respondents mentioned is the 1600 foot long main street in Petersburg. The public works director during its construction, Hagerman (2019), cited longevity and cost as the reason concrete was chosen. Asphalt is expensive in Petersburg because there is no local hot mix asphalt (HMA) plant. In addition when the pavement needs patching, concrete can be drawn from a local concrete plant. The main street of Petersburg has been paved with concrete since the 1960s, and was first replaced in 1985 and later in 2012. The 2012 design consisted of a six inch class A-A concrete with a two day required compressive strength of 2500 psi and a 1½ pounds per cubic yard dosage of synthetic fiber reinforcement. A class A-A concrete is a “concrete where improved strength and durability is required” (Alaska DOT&PF, 2017). Sand was provided the first winter to mitigate use of deicers, but deicers have been used since with no major deterioration (Hagerman, 2019).

Although they are not highway pavements, there are eight weigh-in-motion (WIM) slabs located throughout Alaska near Anchorage, Fairbanks, Tok, and Soldotna. Many of the WIM slabs have a concrete surface. Gartin and Saboundjian (2005) measured the rut depth of two PCC WIM slabs in Anchorage and compared their rutting to nearby asphalt pavements of the same age and traffic. The PCC surfaces of WIM sites at Tudor Road and Minnesota Road had 29% and 38% less rut depth, respectively, than the nearby asphalt pavements measured. The mix designs of the WIMs was unavailable, but a 2010 mix design of the WIM slab near Tok found it to be a class A 6.5-sack mix with a 4500 psi design strength and a 0.36 w/c ratio (Mack, 2010). Rutting rates also vary by region, with minimal reported rutting problems in the northern region of Alaska (Currey, 2018).

Most concrete bridges in Alaska are paved with asphalt to protect the concrete (Marx, 2019). There are some bare concrete bridge decks including those on the Dalton Highway and in some low-traffic rural areas (Marx, 2019). One example of a bare deck would be the Atigun River No. 2 Bridge on the Dalton Highway which was built in 2000. Almost 20 years later tine marks are still visible (Figure 2.2). Many of the bridges built in the 1940s also have bare concrete decks. Typically though bridges are overlaid with asphalt so once the asphalt is damaged, decks can easily be repaired (Marx, 2019).

Alaska is one of eight states which have no reported concrete arterial or collector roads (FHWA, 2018). Therefore to better understand other state DOT's experiences with concrete pavements, pavement engineers at WisDOT were surveyed. In Wisconsin 11% of public arterial or collector roads are concrete (FHWA, 2018). At WisDOT when

Figure 2.2 Atigun River No. 2 Bridge (Alaska DOT&PF Bridge Section, 2018)

determining the appropriate pavement surface for a site, a 50-year LCCA is first performed (Harings, 2019). The lowest cost alternative is used, unless the results are within 5% at which point the engineer decides. Overall concrete typically has a higher initial cost, but at a certain depth of HMA, costs tend to equalize. In general in larger cities, where the AADT exceeds around 8,000, concrete is used (Harings, 2019), since concrete pavements also tend to have higher structural capacity (Kemp, 2019).

Although a project may initially use concrete pavement, by around the third rehabilitation the concrete is overlaid with asphalt typically due to joint failure (Harings, 2019). Wisconsin has not allowed studded tires since the 1970s (Kemp, 2019), except for postal, buses, out-of-state and emergency vehicles in the winter (Wisconsin State Legislature, 2017). WisDOT Pavement Engineer Harings noted he had never heard of rutting with concrete but longitudinal cracking does occur around the wheel path. There is also typically no premature rutting in their HMA. WisDOT concrete mix designs usually consist of a 6-sack concrete mix supplemented with fly ash, although silica fume and slag are allowed. Fly ash is usually added to decrease costs, with the added benefit of improved curing. The biggest problem reported regarding concrete pavements is the joints, which tend to deteriorate first. To limit panel cracking WisDOT has been reducing panel lengths from 18-22 feet to 15 feet. Overall Kemp noted they've had "pretty good success with concrete pavements."

2.2.2. Potential Benefits and Drawbacks Regarding Concrete Pavements. Most concrete mixes in Alaska do not contain silica fume, slag, or fly ash. However there are some cases where silica fume was used. A silica fume concrete mix used to be used on bridges decks, but this practice has been abandoned because the silica fume mixes were expensive, heavy, and tended to crack (Figure 2.3). Within the last decade this practice has been replaced by using polyester synthetic concretes, which do not shrink or crack (Marx, 2019). Other projects which used silica fume in their mixes

Figure 2.3 Cracks on silica fume deck at Troublesome Creek Bridge (Alaska DOT&PF Bridge Section, 2018)

include the downtown Anchorage intersections which were paved in the late 2000s with 7-sack 5% silica fume mixes (Johnson, 2019). One benefit to using silica fume over slag or fly ash would be that a 4-8% silica fume content can improve the concrete's properties, but higher contents, which incur higher shipping costs, are needed when using slag or fly ash (Schlee, 2019).

Outside of airports and some military sites, where fly ash mixes are used to adhere to either United States Army Corps of Engineers or Federal Aviation Administration requirements for ASR mitigation (Schlee, 2019; Schaefer, 2019), no one surveyed could recall a concrete pavement containing fly ash in Alaska. This may be because the cost of fly ash is roughly double that of cement and the benefits of its use do not typically outweigh the cost. If a project does require fly ash, it is imported with a high shipping cost.

There is one operating surface coal mine in Alaska, the Usibelli Coal mine, which supplies six coal plants ("Statewide Socioeconomic Impacts of Usibelli Coal Mine, Inc.", 2015) and produce fly ash as a byproduct. Unfortunately the fly ash produced at these plants cannot be used in concrete due to its high unburnt carbon content (Sonafrank, 2010). Marx (2019) noted there may be one coal-burning facility which could produce fly ash clean enough for use in concrete, but using it is likely not feasible. Although fly ash could be burned again for concrete use, doing so is likely not economical given the limited quantity of cement used in Alaska.

Similar challenges were cited when asked if slag was used. Because of shipping costs, slag is usually not used even if it is free (San Angelo, 2019). Schlee did note that slag typically costs less than fly ash, but is still more expensive than cement. For both fly ash and slag he said that when slag was used, it was used to mitigate ASR, and not to improve durability. The only reported location of a slag cement being used was at Fort Wainwright, which is near Fairbanks. These 5.5-sack mixes, used for airport paving, had a 0.40 w/c ratio and a 40% slag content. A recent 2018 visual inspection on four of these, aged 2-10 years, found no durability issues related to F-T cycles (Mappa Inc., 2018).

3. SCREENING TESTS AND ANALYSIS

Initial screening tests to determine the fresh properties, compressive strength, and flexural strength of 10 mixes were conducted. Based on the results, four optimal mixtures were determined. These mixtures were then used for further performance testing.

3.1. MATERIALS AND SPECIMEN PREPARATION

Screening test mixes included binary silica fume mixes and ternary silica fume mixes with slag or fly ash. Specimens were mixed and fabricated using ASTM standards.

3.1.1. Materials. Cementitious materials included type I/II cement, class F fly ash, GGBFS, and BASF MasterLife SF100 silica fume. The AEA used was BASF microair AE200, and the high range water reducer (HRWR) used was BASF Glenium 1466. Aggregate consisted of fine and intermediate-sized particles. Following ASTM C136, multiple sieve analyses were performed (Figures 3.1 and 3.2). The fineness moduli of the intermediate and fine aggregates were 6.0 and 3.0, respectively. Intermediate aggregate was cleaned with a #200 sieve and oven dried. The moisture content of the fine aggregate was measured regularly to maintain a consistent w/c ratio.

3.1.2. Mixtures. Using the initial mix design (Table 3.1) the w/c ratio, AEA dosage, and aggregate ratios remained constant, but the SCMs and their replacement levels were changed. The HRWR dosage was also altered depending on the batch to maintain workability. All mixes had a cement factor of 7.0 with a 0.331 w/c ratio. The original mix design was used in the field on the 2012 King Salmon Main Runway Rehabilitation project by Anchorage Sand and Gravel in King Salmon, Alaska. In total 10

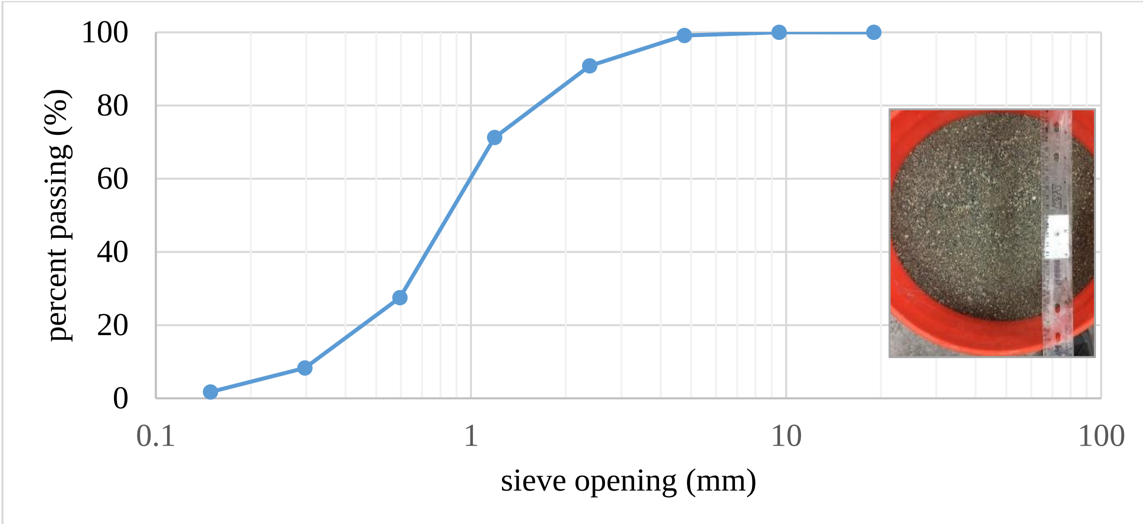


Figure 3.1 Alaska fine aggregate gradation chart

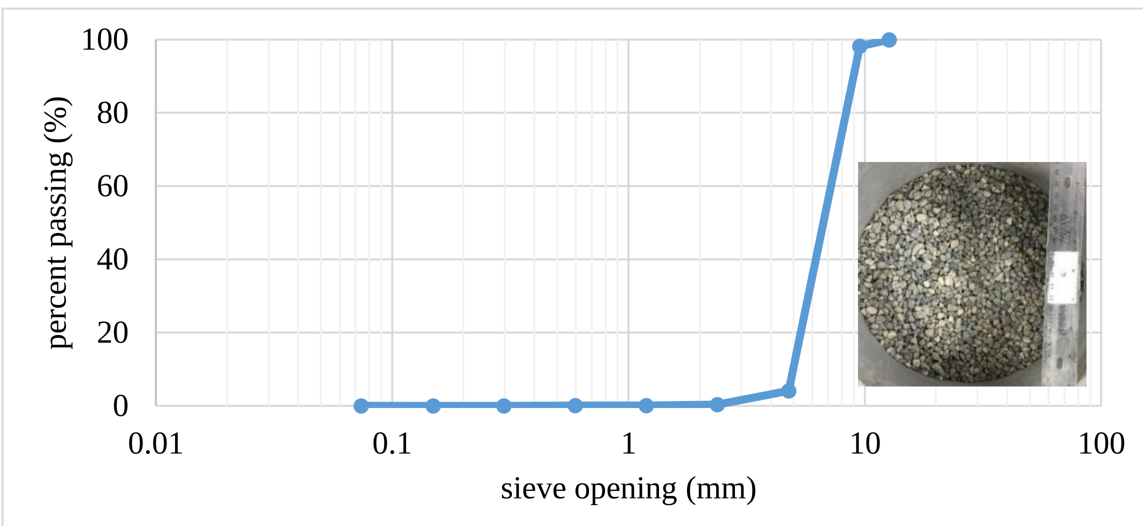


Figure 3.2 Alaska intermediate aggregate gradation chart

mixes were tested (Table 3.2). For silica fume, the equivalent dosage of either a full or half 50-lb bag of silica fume per cubic yard concrete was used, equivalent to 3.8% or

7.6% mass of cementitious material. The remaining cementitious material consisted of either 25% or 40% fly ash or GGBFS. These SCM replacement levels are commonly used and were recommended by professionals in the Alaska concrete industry.

Table 3.1 Base mix design

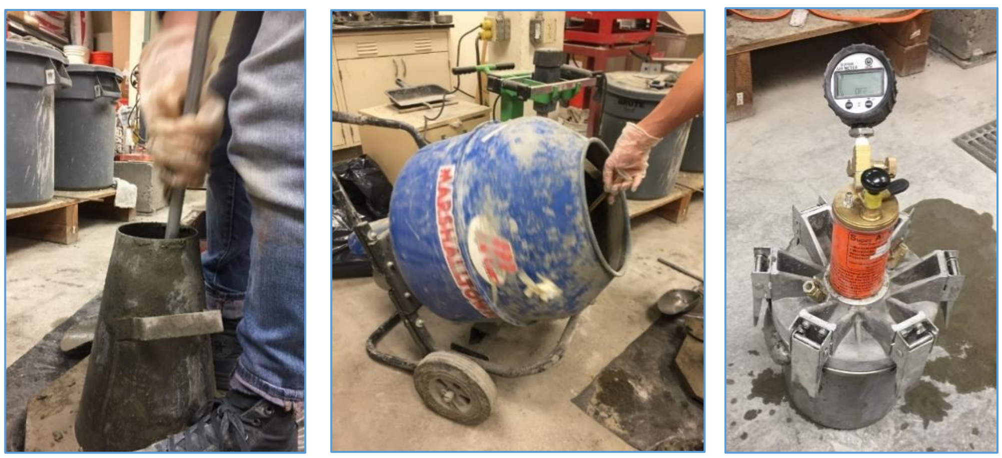
Constituent	**Quantity**	**Unit**	**Cementitious Material (% mass)**
Type I Cement	611	lb	92
Silica fume	50	lb	8
Intermediate aggregate	1826	lb	
Fine aggregate	1248	lb	
Water	252.5	lb	
AEA	14.8	mL	
HRWR	1956	mL	

AEA = air entraining admixture. HRWR = High range water reducer admixture.

Table 3.2 Total cementitious material percent composition for each screening test mixture

Mix (No.)	**Cement** (%)	**Silica fume (SF)** (%)	**Slag (SL)** (%)	**Class F fly ash (FA)** (%)
1. SF8 (base)	92	8	0	0
2. SF4	96	4	0	0
3. SF4 SL38	58	4	38	0
4. SF4 FA24	72	4	0	24
5. SF8 SL37	55	8	37	0
6. SF4 SL24	72	4	24	0
7. SF4 FA38	58	4	0	38
8. SF8 FA37	55	8	0	37
9. SF8 SL23	69	8	23	0
10. SF8 FA23	69	8	0	23

3.1.3. Mixing. The same mixing procedure was used for each batch. First aggregate was mixed with 75% of the water for five minutes. Then silica fume was added and mixed for five minutes, followed by the remaining cementitious material. The HRWR was then added and mixed for two minutes, followed by the AEA for two minutes. Workability (slump) was then measured (Figure 3.3a). If workability was poor, additional HRWR was added to improve it. Batches, with the exception of a few smaller ones, were all made in the same mixer (Figure 3.3b). Once an appropriate slump was achieved, air content was measured (Figure 3.3c).



(a) Slump (b) Drum mixer (c) Super air meter

Figure 3.3 Mixing and testing concrete

3.1.4. Specimen Fabrications. After mixing and testing fresh properties of mixes, molds were filled per ASTM C192. Four by eight inch cylindrical molds were filled in two equal layers, rodded 25 times, and hit with an open palm 10-15 times after each layer. Excess cement was struck off. The concrete surface was then smoothed over and covered. To fill the molds for measuring flexural strength, six by six by 21 inch beam molds were filled in two equal layers. After each layer the concrete was rodded 60 times, and each side tapped 15 times with a mallet. After filling, excess concrete was struck off and the surface was smoothed over (Figure 3.4a). Samples were then covered. The following day samples were removed from their molds (Figure 3.4b), labeled, and cured in lime saturated water (Figure 3.4c).

(a) Finishing samples (b) Covering samples (c) Samples curing

Figure 3.4 Preparing samples

3.2. TESTING PROCEDURES

The testing procedures used for measuring the workability, air content, compressive strength, and flexural strength are summarized below.

3.2.1. Workability and Air Content. To measure workability, ASTM C143 was followed. The mold was filled in three equal layers. After each layer the mold was tamped 25 times. Excess cement was struck off, the mold removed, and slump was measured.

To measure the air voids of the fresh cement, AASHTO method TP 118-17 (2017) was followed using a Super Air Meter (Figure 3.3c). The mold and instruments were wetted beforehand. Cement was then added in three equal layers. After each layer the chamber was rodded 25 times and tapped 10-15 times with a mallet. Excess cement was then struck off, the lid secured, and water was added through the petcocks. The pressure was then increased to 14.5, 30, and 45 psi before releasing the pressure and repeating. Afterwards concrete was disposed of.

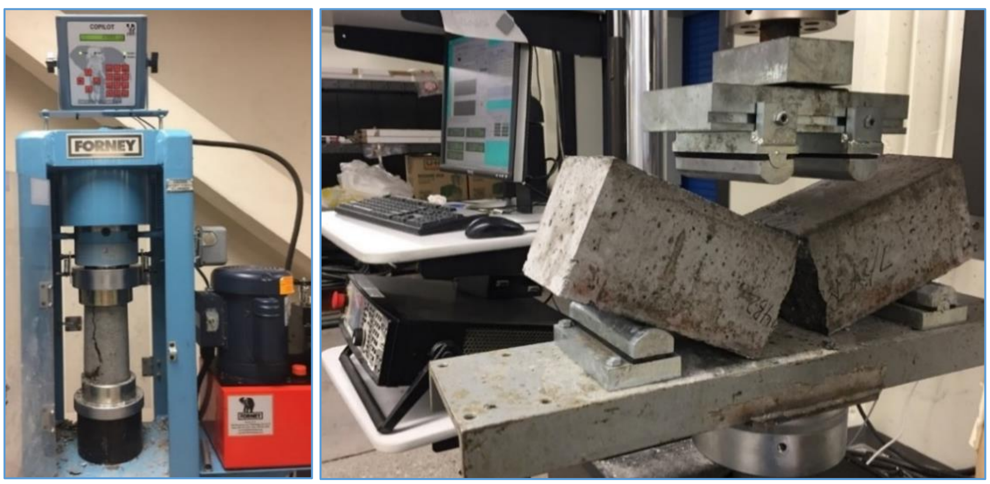
3.2.2. Compressive and Flexural Testing. To measure compressive strength ASTM C39 was followed. Cylinders were loaded at 35 psi per second until failure (Figure 3.5a). For the flexural test a modified version of ASTM C78 was used. The 14 day and 28 day beams for the control mix (SF8) were broken using a force method of 1800 pounds per minute. Due to safety concerns the remaining beams were broken using a displacement method with a rate of 0.0002 inches per second (Figure 3.5b).

3.3. RESULTS

The results of workability, air content, compressive strength, and flexural strength for all ten mixtures were collected.

3.3.1. Workability. Despite adding additional HRWR to some mixes to maintain workability, workability still varied. As shown in Figure 3.6, workability decreased as the silica fume content increased. This is not surprising given the high surface area of silica fume particles which in turn increases water demand (ACI, 2012). Al-Amoudi et al. (2011) also found the addition of silica fume, when compared to an all-cement mix, required an increase in water to maintain similar workability, while Mazloom et al.

(2004) found that as silica fume dosages increased to 15%, additional superplasticizer was needed to maintain workability. Research by El-Chabib and Syed (2012) on binary, ternary, and quaternary mixtures containing fly ash, silica fume and slag also found that when mixtures contained silica fume contents up to 10% the compressive strength increased, but workability decreased. Wang and Li (2012) had similar results, finding that a 12% silica fume content caused a 14% decrease in workability, but only minimal effects on workability when contents were limited to less than 6%.



(a) Compressive (b) Flexural

Figure 3.5 Compressive and flexural strength testing

As shown in Figure 3.6, the addition of fly ash appears to improve workability while the addition of slag reduced workability, which aligns with the findings of other researchers (Berndt, 2009; Hale et al., 2008). The incorporation of fly ash reduces both the water demand (Naik and Ramme, 1989; Ravina and Mehta, 1986), and the

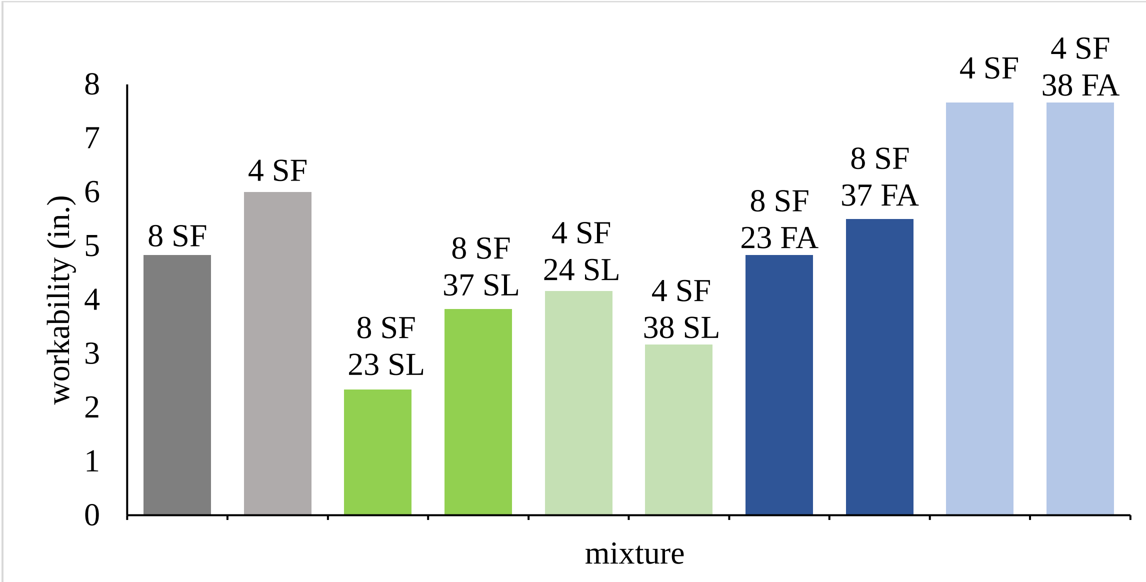


Figure 3.6 Workability of each screening mixture

workability due to the spherical shape of fly ash particles (ACAA, 2003). Since the w/c ratio was consistent between mixes, despite fly ash mixes having a lower water demand, this equivalent w/c ratio between mixes contributed to a higher workability in fly ash mixes. This may partially explain why the fly ash mixes would have a higher workability than the control. Regarding slag, Sivasundaram and Malhotra (1992) found slag cement had reduced workability when compared to plain cement, while other researchers found slag improved workability (Meusel and Rose, 1983; Oner et al., 2005). The low workability of slag mixes in this study can be partially attributed to the angular shape of their particles which results in a higher surface area to volume ratio, which subsequently requires additional water to coat each particle's surface (Kashani et al., 2014).

3.3.2. Air Content. Overall fly ash mixtures had the highest air content, while the slag mixes had the lowest air contents (Figure 3.7), which was consistent with the findings of Hale et al. (2008). The air content values of the SF8 SL23 and SF8 FA23 were not measured and were not included. The high air content in the fly ash mixtures can be partially attributed to its higher surface area over that of cement particles (Du and Folliard, 2005), as well as the high workability of the fly ash mixtures, which helps to distribute air bubbles during mixing (Hale et al., 2008).

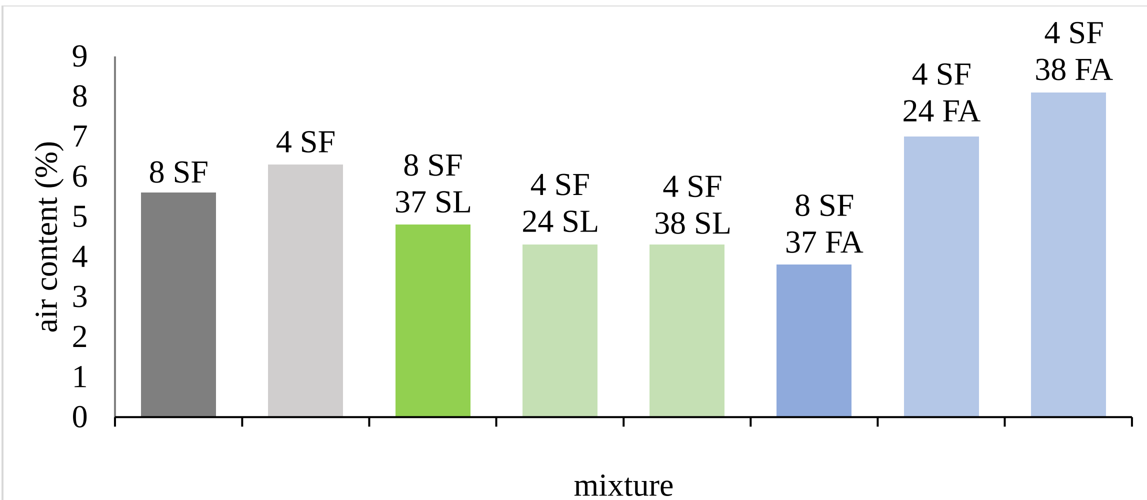


Figure 3.7 Air content of each screening mixture

3.3.3. Compressive Strength. When averaging the compressive strength of mixes containing either 4% or 8% silica fume, samples containing 8% silica fume had higher compressive strength than those with 4% at all ages from one to 28 days (Figure 3.8). This can be explained in part by the increased packing density in the silica fume mixes due to the size of the silica fume particles, which are approximately 1/100th the size of cement particles, and can fill the pores between cement particles. In addition the

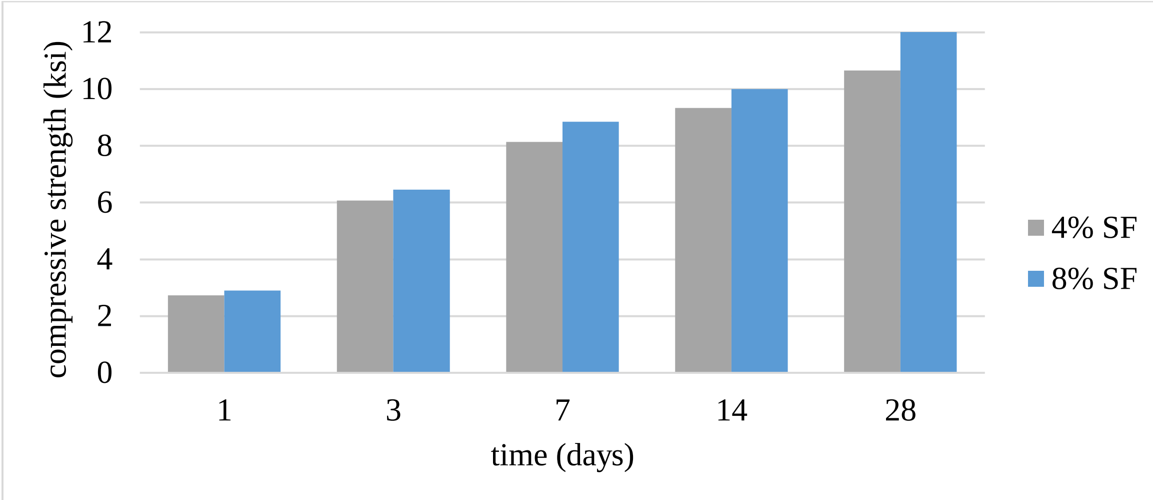


Figure 3.8 Compressive strength (ksi) vs. time (days) at 4% and 8% silica fume

addition of silica fume increases strength by reacting with CH to produce additional calcium-silicate-hydrates (CSH), which subsequently contribute to the concrete's strength (Erdem and Kırca, 2008). The high surface area of silica fume particles also provide additional nucleation sites for hydration products to form on (Erdem and Kırca, 2008). Shannag (2000) tested compressive strength up to 56 days and also found a positive correlation between silica fume contents up to 15% and compressive strength. Bhanja and Sengupta (2005) found that optimum 28 day compressive strength could be achieved with a 15-25% silica fume content.

Regardless of silica fume content, the control mix had the highest compressive strength at one day (Figures 3.9 and 3.10), which may be due to the higher pozzolanic activity of silica fume over that of fly ash and slag. By three days fly ash mixtures had higher compressive strength than slag mixtures with the same SCM content. By seven days compressive strength of fly ash and slag were similar. By 14 days mixes containing

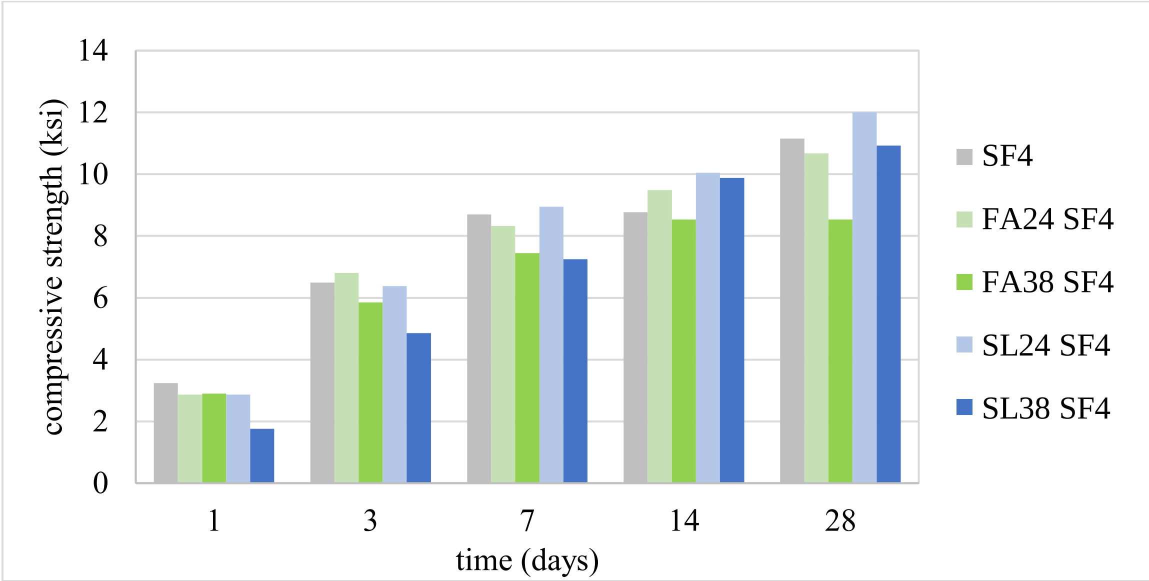


Figure 3.9 Compressive strength (ksi) of mixtures with 4% silica fume vs. time (days)

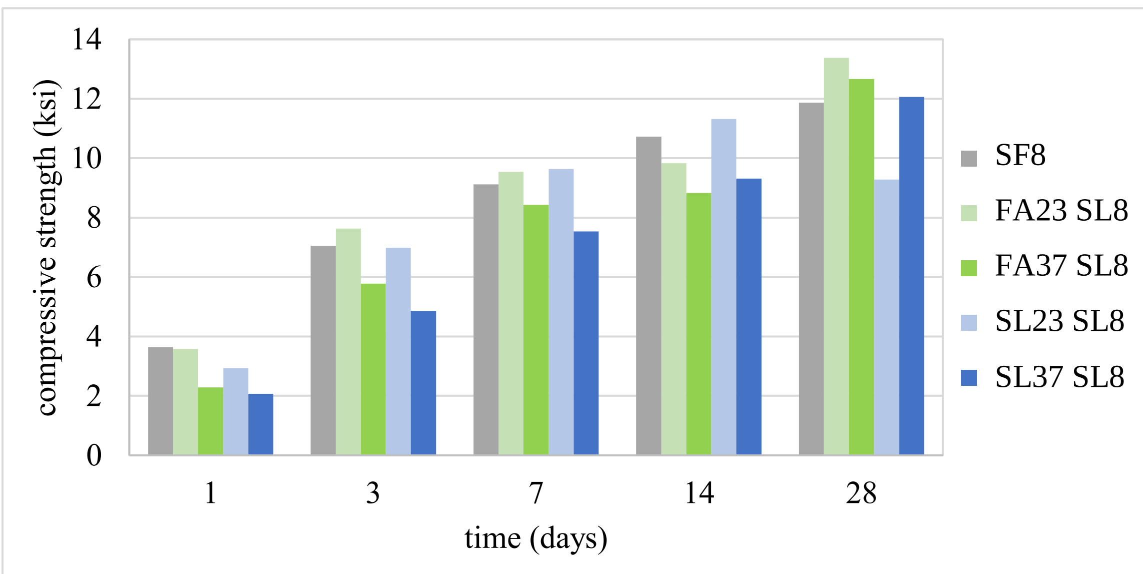


Figure 3.10 Compressive strength (ksi) of mixtures with 8% silica fume vs. time (days)

slag had higher compressive strength than fly ash mixes with the same SCM contents. This was inconsistent with Erdem and Kırca's research (2008) on ternary blended concretes with silica fume and either class F fly ash, class C fly ash, or slag which found that for compressive strength from three to 28 days, class C fly ash performed the best, followed by slag, and class F fly ash. One day strength was not investigated.

Research by Gesoğlu (2009) on SCC measured the 28 day compressive strength of binary silica fume mixes and ternary silica fume mixes containing fly ash or slag. They found that mixes containing fly ash generally had lower compressive strength. Hale et al. (2008) investigated the compressive strength of four mixes: a PCC cement, a 25% slag mix, a 15% type C fly ash mix, and a 25% slag with 15% fly ash mix, and found the slag mix had the highest compressive strength at all ages from three to 90 days.

Additionally in this study for almost all SCM mixtures at ages up to 28 days, the mixture containing the lower dosage of fly ash or slag had higher compressive strengths than those containing higher replacement levels. This does not align with Oner et al. (2005) which found that 28 day compressive strength increased as fly ash content increased up to 40%, but their mixtures did not contain silica fume. On the other hand, Yen et al. (2007) tested fly ash mixes with a 0.33 w/c ratio and found that samples containing 15% fly ash had higher compressive strength at all ages from 28-364 days over those containing 0, 20, 25, and 30% fly ash.

3.3.4. Flexural Strength. Regarding the effect of silica fume content on flexural strength at seven, 14, and 28 days, there were no obvious trends (Figure 3.11), which is surprising given that generally as compressive strength increases, flexural strength increases, and the mixtures containing higher dosages of silica fume had higher compressive strength so mixtures with higher silica fume contents would likely have higher flexural strength. This is corroborated by other researchers (e.g. Bhanja and

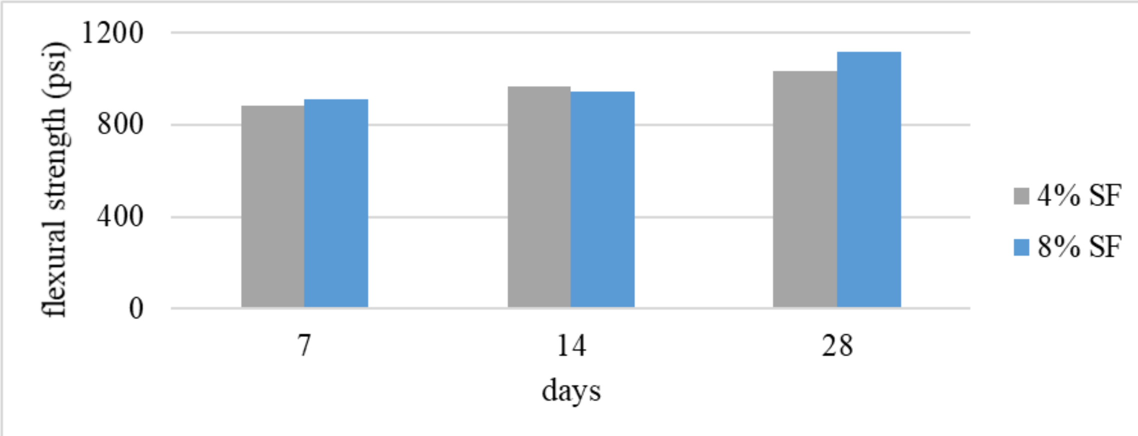


Figure 3.11 Average flexural strength of mixtures and their silica fume contents

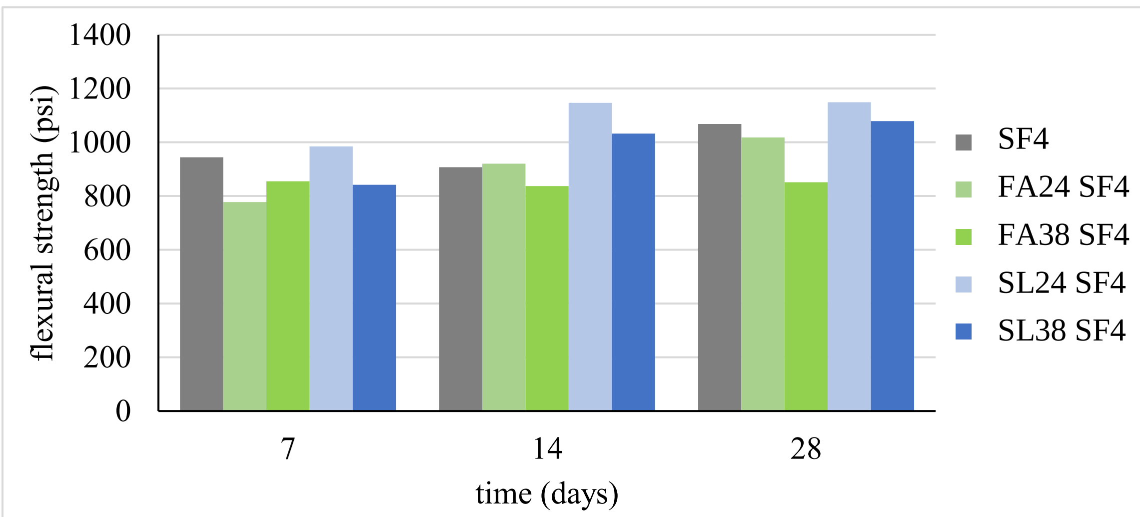


Figure 3.12 Flexural strength (psi) of mixtures with 4% silica fume vs. time (days)

Sengupta, 2005; Yogendran et al., 1987) which found that as the silica fume content increased up to 10% the 28 day flexural strength increased. Results presented here found that mixes containing 23-24% slag had the highest flexural strength at both silica fume contents (Figures 3.12 and 3.13). Research by Lee and Yoon (2015) on binary and

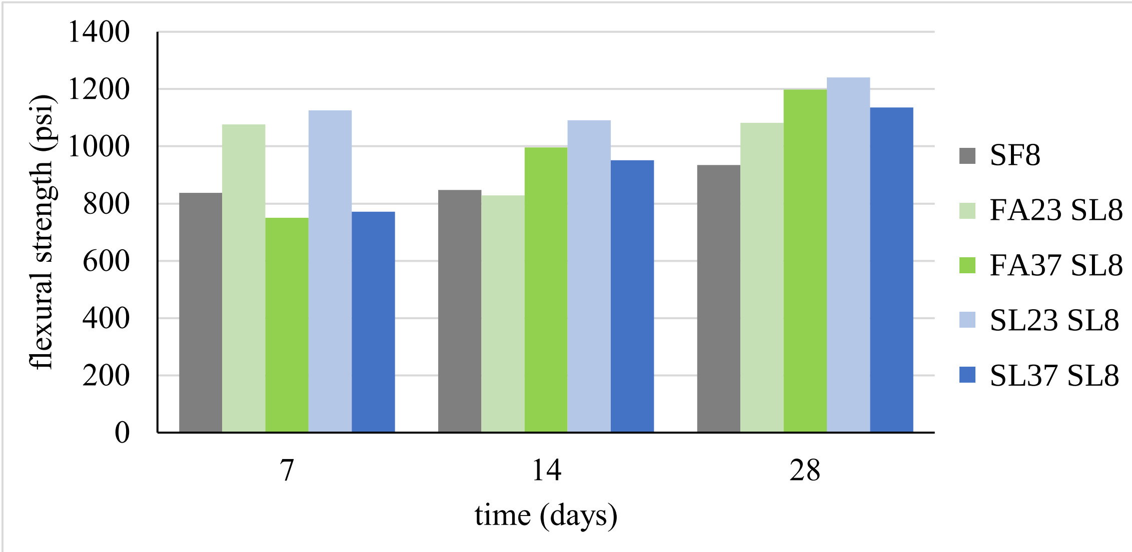


Figure 3.13 Flexural strength (psi) of mixtures with 8% silica fume vs. time (days)

ternary fly ash and slag mixes had different results, concluding that the SCM type had no significant effect on flexural strength. Bharatkumar et al. (2001) also found the addition of fly ash or slag did not significantly affect flexural strength.

3.4. DETERMINING THE OPTIMUM MIXTURE

Using the results obtained, an optimum mix for each parameter (e.g. 1 day compressive strength, 3 day compressive strength, etc.) was determined. This was first done using Minitab® Statistical Software Response Optimization tool (Minitab 2019), and later verified in Excel using special cubic models and desirability functions. Minitab is a statistical analysis program which has a function available to optimize mixtures.

3.4.1. Minitab Method. Using Minitab response optimization, slag, fly ash, silica fume, and cement contents were limited to the maximum and minimum contents tested

(Table 3.3). Then responses were modeled. Responses included workability; 1, 3, 7, 14, and 28 day compressive strength; and 7, 14, and 28 day flexural strength. Three models were investigated including linear (Eq. 1), quadratic (Eq. 2) and special cubic models (Eq. 3). Linear models describe how each individual component affects the response,

$$\mathrm{response} = \mathrm{A(cem)} + \mathrm{B(sf)} + \mathrm{C(fa)} + \mathrm{D(sl)} \tag{1}$$

$$\begin{aligned}\mathrm{response} = {} & \mathrm{A(cem)} + \mathrm{B(sf)} + \mathrm{C(fa)} + \mathrm{D(sl)} + \mathrm{E(cem)(sf)} \\ & + \mathrm{F(cem)(sl)} + \mathrm{G(cem)(fa)} + \mathrm{H(sf)(sl)}\end{aligned} \tag{2}$$

$$\begin{aligned}\mathrm{response} = {} & \mathrm{A(cem)} + \mathrm{B(sf)} + \mathrm{C(fa)} + \mathrm{D(sl)} + \mathrm{E(cem)(sf)} \\ & + \mathrm{F(cem)(sl)} + \mathrm{G(cem)(fa)} + \mathrm{H(sf)(sl)} \\ & + \mathrm{I(cem)(sf)(sl)} + \mathrm{J(cem)(sf)(fa)}\end{aligned} \tag{3}$$

cem = cement, sf = silica fume, fa = fly ash, sl = slag

quadratic models describe how two different components affect each other and the response, and special cubic models describes how the combination of three components affect each other and the response. Other models were not investigated in part due to redundancy (e.g. cement × cement). In addition, of the models investigated some relationships were not included. These include silica fume × fly ash, slag × fly ash, cement × slag × fly ash, and silica fume × slag × fly ash. In the first case, silica fume × fly ash, this is due to multicollinearity. In the case of the latter three, the combination of slag and fly ash were not tested, and therefore a coefficient for representing their relationship could not be determined. The sum of squares (S), r-squared value (R^2) and probability value (P) for each model and response are summarized in Table 3.4. Overall it appears using the special cubic models results in the highest R^2 values. The association between the estimated and actual data was significant at the 0.05 level for all responses except 28 day flexural strength (P=0.06), which was marginally statistically significant. Therefore special cubic models were used to model the data.

Table 3.3 Constraints used (% cementitious material)

Constituent	Cement (%)	Silica fume (%)	GGBFS (%)	Fly ash (%)
lower limit	55	3.8	0	0
upper limit	96	7.6	3.8	3.8

After determining the appropriate model, targets were used to maximize each response. Targets were set at 10% higher than the highest average mix measurement. For example the SF8 mix had the highest average one day compressive strength so 110% of its compressive strength was used as the target. A target of 10% higher than the average was used to improve reliability to better represent the highest value measured in the lab. The lower limit was the lowest average measurement. Each response, except workability, was set to maximize at these targets. Workability was set at six inches. The upper limit (target), lower limit, weight, and importance for each response is summarized in Table 3.5. All responses were weighed at 1.0, but the importance factor, k, varied. Workability, flexural strength and compressive strength were considered of equal importance at 3.33. Therefore, for each compressive strength response (1 day, 3 day, etc.) the importance factor was 0.67, and for each flexural strength response the importance factor was 1.11. Using all these inputs and limits, Minitab determined the optimum mix design to contain 12% slag, 4% silica fume and 1% fly ash (Figure 3.13).

Table 3.4 Models fit for each response

Response	**Model**	**S**	**R^2**	**P**
1 day compressive strength	linear	541	50.59	0.000
	quadratic	529	59.96	0.002
	special cubic	507	66.60	0.003
3 day compressive strength	linear	817	44.36	0.001
	quadratic	633	71.74	0.000
	special cubic	637	73.98	0.000
7 day compressive strength	linear	959	31.64	0.018
	quadratic	839	55.71	0.006
	special cubic	869	56.86	0.022
14 day compressive strength	linear	1050	27.69	0.035
	quadratic	940	51.02	0.015
	special cubic	957	53.85	0.036
28 day compressive strength	linear	1050	54.08	0.000
	quadratic	629	86.14	0.000
	special cubic	656	86.38	0.000
7 day flexural strength	linear	141	11.38	0.574
	quadratic	126	47.34	0.243
	special cubic	72	85.72	0.003
14 day flexural strength	linear	96	39.87	0.039
	quadratic	56	83.68	0.001
	special cubic	42	92.52	0.000
28 day flexural strength	linear	128	25.36	0.186
	quadratic	99	66.61	0.030
	special cubic	100	71.40	0.064
workability	linear	2	47.32	0.001
	quadratic	2	59.29	0.006
	special cubic	2	65.70	0.007

3.4.2. Excel Method. Using the Minitab constraints (Table 3.3) and a special cubic model, coefficients were determined for each parameter (Table 3.6). A constraint was also added requiring the sum of cement, silica fume, slag and fly ash to equal 100%. The Minitab targets (Table 3.5) were also used to maximize the desirability of each response. This can be done by using equations to either reach a certain target (Eq. 4), maximize the response (Eq. 5), or minimize the response (Eq. 6) (Derringer and Suich, 1980). Since the aim of the optimum mixture was to maximize the strength at all ages, Eq. 5, was used for all responses except slump (workability). For workability, Eq. 4 was used to reach a target of six inches.

$$d(x) = \begin{cases} 0 & \text{if } x<L \\ \left(\dfrac{x-L}{T-L}\right)^{w} & \text{if } L\leq x\leq T \\ \left(\dfrac{x-U}{T-U}\right)^{w} & \text{if } T\leq x\leq U \\ 0 & \text{if } x>U \end{cases} \qquad (4)$$

$$d(x) = \begin{cases} 0 & \text{if } x<L \\ \left(\dfrac{x-L}{T-L}\right)^{w} & \text{if } L\leq x\leq T \\ 1 & \text{if } x>T \end{cases} \qquad (5)$$

$$d(x) = \begin{cases} 1 & \text{if } x<T \\ \left(\dfrac{x-U}{T-U}\right)^{w} & \text{if } T\leq x\leq U \\ 0 & \text{if } x>U \end{cases} \qquad (6)$$

L = lower bound U = upper bound T = target W = weight

Table 3.5 Response limits and importance

Response	**goal**	**lower limit**	**upper limit**	**target**	**weight**	**importance**
1 day compressive	maximize	1764	-	3641	1.0	0.67
3 day compressive	maximize	4853	-	7626	1.0	0.67
7 day compressive	maximize	7250	-	9632	1.0	0.67
14 day compressive	maximize	8533	-	11316	1.0	0.67
28 day compressive	maximize	8535	-	13371	1.0	0.67
7 day flexural strength	maximize	750	-	1126	1.0	1.11
14 day flexural strength	maximize	828	-	1147	1.0	1.11
28 day flexural strength	maximize	852	-	1240	1.0	1.11
workability	target	2.3	7.7	6.0	1.0	3.33

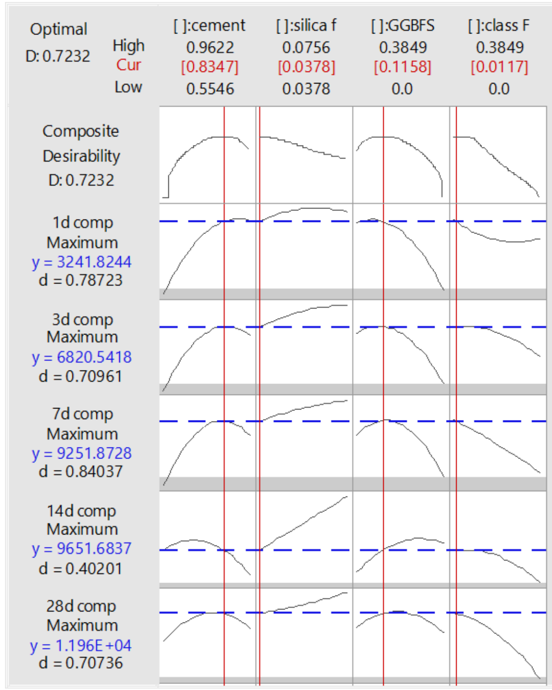


Figure 3.14 Optimum mixture as determined by Response Optimizer for Mixture on Minitab

Table 3.6 Special cubic model coefficients for each response

Term	**Compressive strength (days)**					**Flexural strength (days)**			**Slump**
	1	**3**	**7**	**14**	**28**	**7**	**14**	**28**	
C	1412	4914	7782	6938	11270	526	1229.4	1429	0.49
SF	-432495	-298347	-134331	97290	283651	-164925	81067	68220	1009
SL	-15473	-5407	-7835	22839	-1266	2417	-658	1572	-31.9
F	22104	4917	4245	-6534	-25697	9077	-3366	-3549	-39.5
C × SF	501296	358616	172912	-43531	-286215	183452	-91826	-79318	-1161
C × SL	22963	11744	19600	-12725	17182	-4287	3698	-1318	54.9
C × F	-33563	-1580	-2946	29747	35454	-15566	6772	5457	101
SF × SL	606619	120262	-21029	-641665	-404447	80667	-103951	-115140	-208
C × SF × SL	-192406	379658	360476	776260	297396	192641	24627	91538	-1500
C × SF × F	766740	585718	390542	-273278	-52043	327056	-135994	-79493	-2195

C = cement, SF = silica fume, SL = Slag, Fly ash = F

Following this, each response was assigned an importance value and each response's desirability was used to determine the overall desirability the mix would provide (Eq. 7) (Derringer and Suich, 1980; Aksezer, 2008). Excel solver was then used

$$D = \left(d_1(x_1)^{k_1} \times d_2(x_2)^{k_2} \times \ldots \times d_n(x_n)^{k_n}\right)^{\frac{1}{\sum_i k_i}} \tag{7}$$

to maximize the desirability within the limits. The optimum mixture determined using Excel was almost identical to the mixture determined using Minitab (Tables 3.7 and 3.8).

Table 3.7 Excel versus Minitab optimum mixture

Method	**Desirability**	**Cement**	**Silica fume**	**GGBFS**	**Class F fly ash**
Excel	0.7233	0.8355	0.0378	0.1159	0.0109
Minitab	0.7232	0.8347	0.0378	0.1158	0.0117

Table 3.8 Predicted value and desirability for each response in the optimum mixture

Response	**Excel**			**Minitab**		
	Predicted		Desirability	Predicted		Desirability
1 day compressive strength	3245	psi	0.79	3242	psi	0.79
3 day compressive strength	6820	psi	0.71	6821	psi	0.71
7 day compressive strength	9255	psi	0.84	9252	psi	0.84
14 day compressive strength	9650	psi	0.40	9652	psi	0.40
28 day compressive strength	11958	psi	0.71	11956	psi	0.71
7 day flexural strength	992	psi	0.64	991	psi	0.64
14 day flexural strength	1086	psi	0.81	1086	psi	0.81
28 day flexural strength	1137	psi	0.73	1136	psi	0.73
workability	5.2	in.	0.78	5.2	in.	0.78

3.4.3. Results. This method was repeated in determining three other mixes: (1) for optimal workability, (2) for optimal one to 28 day compressive strength, and (3) for seven to 28 day optimal flexural strength. For (1), workability was the only response used. In (2) all compressive strength responses were used with equivalent importance, and in (3) all flexural strength responses with equivalent importance were used. These mixes, along with the overall optimum and control mixes (Table 3.9), were then used for further testing.

Table 3.9 Mixtures determined for performance testing

Name	**Silica Fume** (%)	**Slag** (%)	**Fly Ash** (%)	**Cement** (%)
Control (SF8)	8	0	0	92
Optimal flexural strength (SL22 SF8)	8	2	0	70
Optimal workability, flexural strength, and compressive strength (SL12 SF4 FA1)	4	2	1	83
Optimal compressive strength (SL8 SF8 FA3)	8	8	3	81
Optimal workability (FA31 SF4)	4	0	31	65

Other researchers also found that a primarily slag and silica fume mixture would provide an optimal mix for concrete pavements. For example, Scholz and Keshari (2010) developed an abrasion-resistant mix using silica fume, fly ash, and slag. They found a slag and silica fume mix had better durability, compressive strength, and abrasion resistance over that of a fly ash and silica fume mix. Gesoğlu et al. (2009) tested 22 SCC binary, ternary, and quaternary mixes containing silica fume, slag and fly ash and concluded an optimum mix would contain primarily silica fume and slag. The optimum mix they determined contained 44% slag, 1% fly ash, and 14% silica fume. Although the slag and silica fume ternary mixtures generally improved hardened properties, only the ternary mixture of fly ash and slag satisfied the V-funnel flow time requirements, which may explain the additional 1% fly ash contribution.

4. PERFORMANCE TESTS AND RESULTS

After determining the optimum mix designs, subsequent performance tests were conducted on these mixtures to ascertain their mechanical and durability properties. Tests included measuring free shrinkage, abrasion resistance, compressive strength, F-T resistance, deicer scaling resistance, and chloride ion penetration.

4.1. MATERIALS AND SPECIMEN PREPARATION

Regarding materials, mixing, and fabrication of specimens, care was taken to use the same or similar materials and methods to those used during the screening tests.

4.1.1. Materials. For cementitious materials, Type I cement from Missouri was used. The same silica fume, fly ash, and slag used during the screening tests were used. The same AEA was used, but the HRWR used was Glenium 7500. Similar aggregates to those used for the screening tests were used. The fineness moduli of the fine aggregate and intermediate aggregate were 3.0 and 5.8, respectively (Figures 4.1 and 4.2).

4.1.2. Mixtures. As mentioned previously, five mixtures were tested (Table 4.1). Similar to the screening mixtures, the w/c ratio, air entrainment dosage, and aggregate ratios remained the same for all mixes, but the cementitious material dosages changed. The HRWR dosage was also altered depending on the batch to improve workability.

4.1.3. Mixing and Specimen Fabrication. For the performance tests the mixing method was identical to the screening test mixing method, with a few exceptions. Instead of adding silica fume before the other cementitious materials, silica fume was added at the same time as the other cementitious materials. In addition, the AEA was added

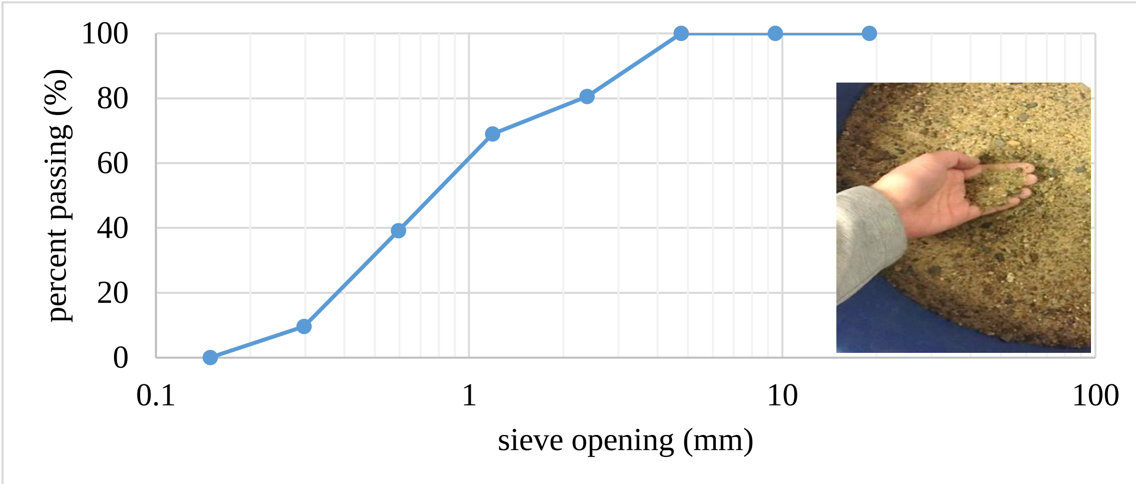


Figure 4.1 Missouri fine aggregate gradation chart

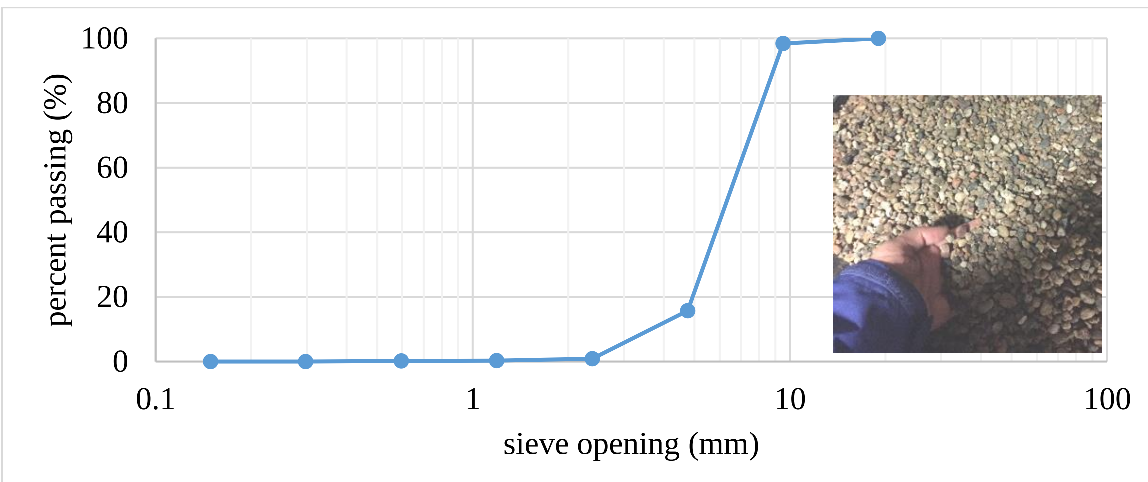


Figure 4.2 Missouri intermediate aggregate gradation chart

at the same time as the water and aggregate at the beginning of mixing, instead of adding the AEA near the end of mixing. Similar to the screening test specimen fabrications, after the fresh concrete was prepared the air content and workability were measured (Figure 4.3). Subsequently, molds were filled in two layers and vibrated. After filling, molds were covered. The following day samples were demolded and placed in lime saturated water in temperature-controlled curing baths.

Table 4.1 Cementitious material percent composition for the optimal and control mixtures

Mix	Cement (%)	Silica Fume (%)	Slag (%)	Class F Fly Ash (%)
Control (SF8)	92	8	0	0
Optimal flexural strength (SL22 SF8)	70	8	22	0
Optimal flexural, compressive, and workability (SL12 SF4 FA1)	83	4	12	1
Optimal workability (FA31 SF4)	65	4	0	31
Optimal compressive (SL8 SF8 FA3)	81	8	8	3

4.2.TESTING PROCEDURES

All of the performance testing procedures used followed ASTM standards, unless noted otherwise.

4.2.1. Workability and Air Content. For workability ASTM C143 (2015a) was followed. To measure the air voids of the fresh cement, an air meter and the ASTM C231 (2017b) method was used (Figure 4.4). For this test, the air meter and lid were first wetted. Then the meter was filled by thirds with concrete. After each third concrete was rodded 25 times and the sides tapped 10-15 times with a mallet. Excess concrete was struck off, edges wiped down, and the lid attached and sealed shut. Water was added through one petcock until clear water and no bubbles emerged from the opposite petcock. The air meter was then pressurized by pumping the knob to the designated pressure.

Petcocks were then closed and the lever pressed. The vessel was hit once with a hammer and then the air content was read from the gauge.

4.2.2. Compressive Strength and Shrinkage. For compressive strength ASTM C39 (2018) was followed. Samples were crushed at a rate of 35 psi per second. For shrinkage ASTM C157 (2017a) was followed. Shrinkage samples were demolded

(a) Slump and air meter equipment

(b) Molds being finished

(c) Unmolding cylinders

(d) Samples wet curing

Figure 4.3 Sample preparation

Figure 4.4 Air meter

24 hours after mixing, measured, and then cured for 28 days in a temperature-controlled water bath. After 28 days samples were measured again and left at 50% humidity at 23°C and measured daily for 28 days (Figure 4.5).

(a) Shrinkage molds (b) Measuring sample (c) Samples

Figure 4.5 Measuring shrinkage

4.2.3. Durability. The durability properties investigated included chloride ion penetration, F-T resistance, abrasion resistance, and scaling resistance from deicing salts.

4.2.3.1. Abrasion resistance. Two methods were used to measure abrasion resistance. The first, ASTM C944, measured abrasion resistance through mass loss while the second, the Prall Method, measured abrasion resistance through volume loss.

ASTM C944 Test: For measuring abrasion resistance by mass loss, a modified ASTM C944 method (2012b) was followed. ASTM C944 requires the rotating-cutter drill press to spin at a rate of 200 revolutions per minute (rpms), but the press used only could rotate at 150 or 300 rpms, so 150 rpms was used. A 22 pound force was applied for two minutes in four sections of the samples (Figure 4.6a). Mass loss was measured after each two minute period (Figure 4.6b).

Abrasion by Studs, Method A: Prall Method: Abrasion resistance was also measured through volume loss using the Nordic Prall testing apparatus (Figure 4.7) and

(a) Applying force to sample (b) Measuring mass loss

Figure 4.6 Testing abrasion by mass loss the Abrasion by Studs, Method A: Prall Method standard (CEN WG1 Bituminous Materials, 1997). To prepare samples for this test, four by eight inch cylindrical concrete samples were cured for 28 days and sent to the Alaska DOT&PF Southcoast Materials Lab. Upon arrival samples were cut into 100 mm diameter by 30 mm long disks and brought to a temperature of 5°C. Samples were then weighed and placed in the Prall machine**.** In the machine samples were exposed to cooling water at a rate of two liters per minute, and worn for 15 minutes by 40 steel spheres at a rate of 950 rpms. The loss in

(a) Prall test setup (b) Temperature controls

(c) Setting asphalt sample in chamber (d) Adding steel spheres

Figure 4.7 Nordic Prall Test

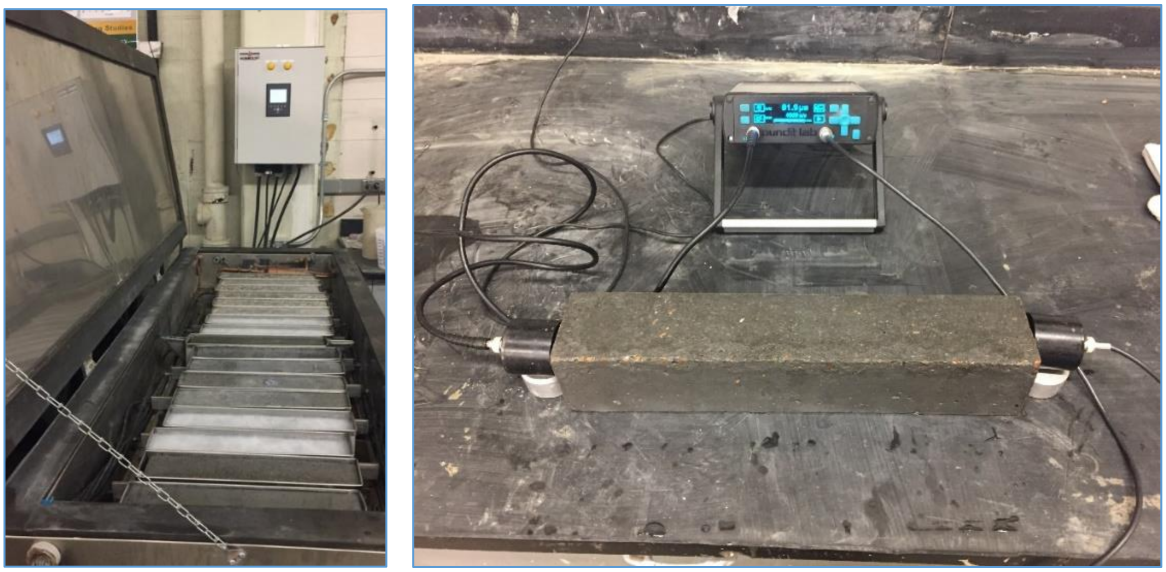
volume before and after testing, referred to as the abrasion value, was measured. Two samples were tested for each mixture. The volume loss per sample was then used to determine the wear resistance (Table 4.2).

Table 4.2 Prall results interpretation

Volume loss (cm^3)	**Wear resistance**
<20	Very good
20-29	Good
30-39	Satisfactory
40-50	Less satisfactory
>50	Poor

4.2.3.2. Freeze-thaw resistance. To measure the F-T resistance of samples, ASTM C666 (2015b) was followed. After wet curing samples for 14 days each sample's length and mass was measured, as well as the ultrasonic pulse velocity. This velocity was measured using a PROCEQ ultrasound with a frequency of 54 Hz (Figure 4.8b). Samples were kept in a temperature-controlled cabinet (Figure 4.8a) which exposed samples to freezing temperatures for four hours, followed by two hours of thawing. After every 18 cycles each sample's mass, length and ultrasonic pulse velocity was measured again. Originally it was planned to expose the samples to 300 cycles, but due to time constraints samples were only exposed to 180 cycles. To calculate the relative dynamic modulus of elasticity (RDME), Eq. 8 was used.

$$RDME\ (\%) = \frac{v_0{}^2}{v_n{}^2} \tag{8}$$



(a) Freeze-thaw cabinet (b) Measuring frequency

Figure 4.8 Freeze-thaw testing

In this, v_0 is the initial ultrasonic pulse velocity and v_n is the ultrasonic pulse velocity at *n* cycles. The durability factor (DF) for each mixture was also determined using Eq. 9.

$$DF = RDME_f \times n_f/n_t \tag{9}$$

In this equation n_f is the cycles the $RDME_f$ represents while n_t is the cycles at which all testing was terminated, which in this case was 180 cycles. The $RDME_f$ represents either the RDME once it reaches 60% or lower, or the RDME after 180 cycles, whichever occurs sooner. The durability factor ranges from 0% to 100%. A higher durability factor suggests the sample has high resistance to F-T cycles. A lower durability factor suggests the sample's durability is low, and degraded quickly after many F-T cycles.

4.2.3.3. Scaling resistance of samples exposed to deicing chemicals. To measure the effect deicer salts have on the scaling resistance of samples, ASTM C672

(2012a) was followed. Samples were cured in a water bath for 14 days and then in air for 14 days. Then the top edges were taped and caulked using waterproof silicone to provide a waterproof boundary (Figure 4.9a). A 4% calcium chloride ($CaCl_2$) solution was then applied to the sample's surface at a ¼ inch depth (Figure 4.9b) and samples were placed

(a) Preparing samples (b) Replacing salt solution on samples

Figure 4.9 Preparing and testing deicing samples

in the deicing chambers. The chamber was calibrated to expose samples to freezing temperatures for 16 hours and then 23°C for eight hours daily. Every five days the solution was replaced, samples were photographed, and the condition of their surface was rated 0-5 as per ASTM C672 ratings (Table 4.3).

4.2.3.4. Chloride ion penetration resistance. For chloride ion penetration ASTM C1202 (2019) was used. First four by eight inch cylindrical samples were wet cured for 28 days. Following this samples were cut into 50 mm disks using a water saw (Figure 4.10a), grinded smooth, and placed in a desiccator for three hours at a 50 mm Hg pressure (Figure 4.10b). With the vacuum pump still on water was added through a stopcock until samples were covered. Samples were then left submerged under pressure for an hour. Following this the pump was turned off and samples were soaked for 18 hours. Samples were then placed in the testing chamber (Figure 4.10c) and each side was filled with either a 3.0% NaCl or 0.3 N NaOH solution. A 60 Volt electrical current was then applied for six hours (Figure 4.10d). The current versus time was then plotted and a curve drawn. The area under the curve was then integrated to determine coulombs passed. Based on this, the penetrability was determined (Table 4.4).

Table 4.3 ASTM C672 sample degradation ratings

Rating	**Condition of Surface**
0	No scaling
1	Very slight scaling (3 mm [1/8 in.] depth, max, no coarse aggregate visible)
2	Slight to moderate scaling
3	Moderate scaling (some coarse aggregate visible)
4	Moderate to severe scaling
5	Severe scaling (coarse aggregate visible over entire surface)

4.3. RESULTS

Data was collected on each mixture's workability, air content, compressive strength, drying shrinkage, and resistance to deicing salts, abrasion, and F-T cycles.

4.3.1. Workability and Air Content. As shown below in Table 4.5, workability varied widely from 1½ to 9¾ inches while the air content varied between 3-5%. As

(a) Cutting samples

(b) Samples in dessicator

(c) Sample in chamber

(d) Testing sample

Figure 4.10 Testing chloride ion penetration

predicted, the optimum workability mixture, which contained 31% fly ash, had the highest workability and air content of the mixes. As mentioned earlier the high workability of the fly ash mixtures can be partially attributed to the spherical shape of the fly ash particles. These results corroborate with research by Hale et al. (2008) which found that mixtures containing fly ash had higher slump and air content than those containing slag cement.

Table 4.4 Chloride ion penetrability based on charge passed (ASTM 1202, 2019)

Charge passed (Coulombs)	Chloride Ion Penetrability
>4,000	High
2,000-4,000	Moderate
1,000-2,000	Low
100-1,000	Very Low
<100	Negligible

Table 4.5 Workability and air content of optimum and control mixtures

Mix	Workability (in.)	Air content (%)
Control (SF8)	8.00	5.5
Optimal flexural strength (SL22 SF8)	9.50	3.4
Optimal all (SL12 SF4 FA1)	1.50	4.5
Optimal compressive (SL8 SF8 FA3)	3.00	5.3
Optimal workability (FA31 SF4)	9.75	5.6

4.3.2. Mechanical Properties. Mechanical properties measured include compressive strength and drying shrinkage.

4.3.2.1. Compressive strength. By 28 days the compressive strength of the control was the highest, followed by the SL8 SF8 FA3, the SL22 SF8, the FA31 SF4 and the SL12 SF4 FA1 mixes (Table 4.6). Nonetheless, all had strengths higher than 6,000 psi by 28 days, which ACI defines as high-strength concrete (ACI, 1992; Mehta, 1999). Therefore any of them could potentially be used for high-strength concrete applications.

Table 4.6 Compressive strength of optimum mixtures

	Compressive strength (psi)				
Mix	**1 day**	**3 days**	**7 days**	**14 days**	**28 days**
Control (SF8)	3870	5920	6930	7360	7950
Optimal flexural strength (SL22 SF8)	3280	5960	7980	7300	7270
Optimal all (SL12 SF4 FA1)	4240	6180	6920	6670	6840
Optimal compressive (SL8 SF8 FA3)	3810	6230	8140	8340	7640
Optimal workability (FA31 SF4)	3700	5520	6370	6930	7210

4.3.2.2. Drying shrinkage. As shown below in Figure 4.11, the FA31 SF4 mixture had almost no change in length, expanding 0.006% instead of shrinking. The other mixtures had shrinkage rates ranging from 0.02% to 0.03% (Table 4.7). The ability of fly ash to reduce shrinkage in concrete is well-known (Chindaprasirt et al., 2004) and is also demonstrated here with minimal change in the FA31 SF4 mixture shrinkage compared to the other mixtures. Research into the effects of adding silica fume to concrete on drying shrinkage found similar shrinkage rates to that of all-cement control concrete (Carette and Malhotra, 1983), while other research found the addition of 5% and 15% silica fume reduced drying shrinkage by 29% and 35% (Güneyisi et al., 2012). Hale et al. (2008) measured shrinkage over 90 days and found the addition of slag reduced shrinkage while fly ash mixtures had similar shrinkage to the all-cement control mixture. Similarly, Mokarem et al. (2005) tested binary mixtures and found fly ash mixtures had higher drying shrinkage over those of silica fume or slag. They suggested the 28 day length change for concrete mixtures containing SCMs should be limited to 0.04%, which all the mixtures presented here adhere to (Table 4.7)

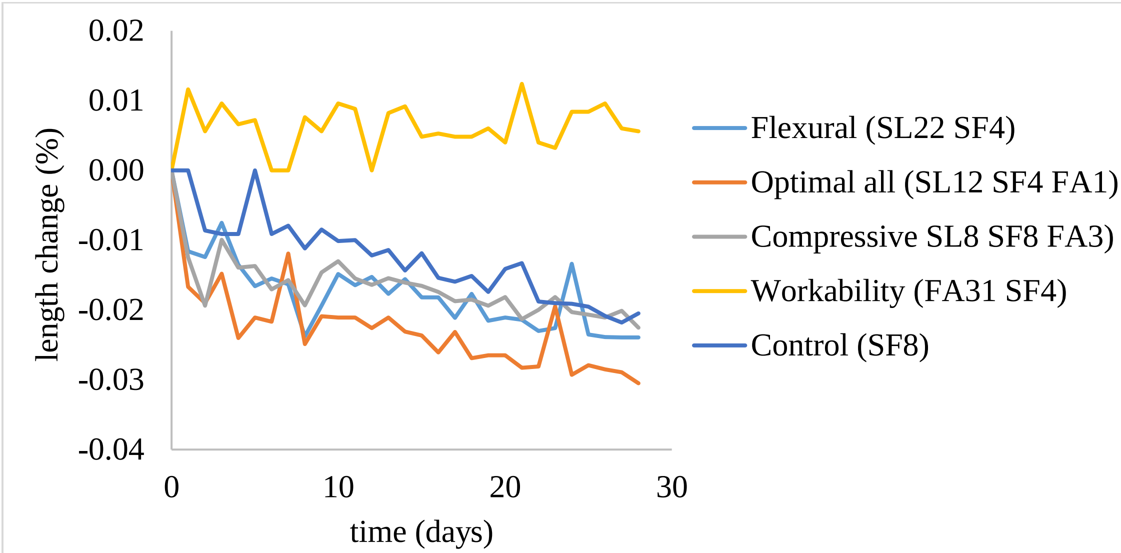


Figure 4.11 Time (days) vs. length change (%)

Table 4.7 28-day shrinkage per mixture

Mixture	28-day length change (%)
Control (SF8)	-0.020
Flexural (SL22 SF8)	-.0.024
Optimal all (SL12 SF4 FA1)	-0.031
Compressive (SL8 SF8 FA3)	-0.023
Workability (FA31 SF4)	0.006

4.3.3. Durability of Hardened Concrete. Durability parameters measured include abrasion resistance, F-T resistance, scaling resistance from deicer exposure, and chloride ion penetration.

4.3.3.1. Abrasion resistance. Abrasion resistance was measured through both the Nordic Prall and the ASTM C944 mass loss test.

ASTM C944 Test: For abrasion resistance, generally as the SCM content increased, the mass loss decreased (Figure 4.12). In particular, the SF4 mixture (4%

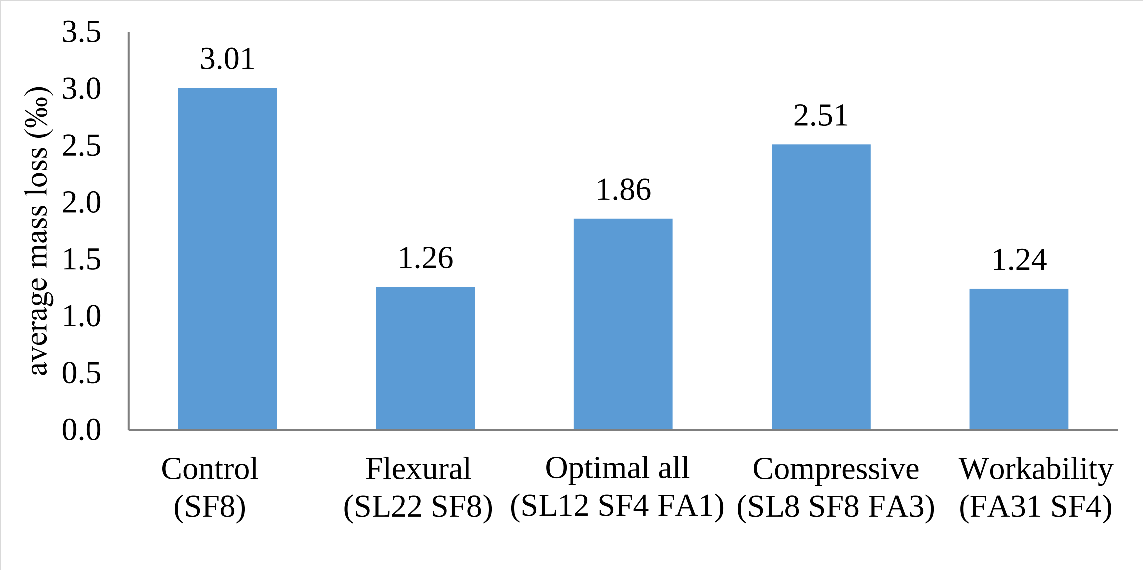


Figure 4.12 Mass loss of mixtures due to abrasion testing

SCMs) had the highest mass loss while the SL22 SF8 and FA31 SF4 mixtures (30% and 35% SCMs, respectively) had the lowest mass loss. Each mixture's mass loss can also be partially attributed to the higher packing density in mixtures containing SCMs as well as the late-age strength-contributing pozzolanic reactions between the silica in the SCMs and the available CH. Langan et al. (1990); Rashad et al. (2014); and Atiş (2002) all found that generally when adding SCMs to mixes, as the compressive strength increased, abrasion resistance increased. Rashad et al. (2014) measured abrasion resistance in wear loss and found that as fly ash content increased to 70% in samples aged 28 to 180 days, abrasion resistance was reduced. On the converse in the data presented here, the fly ash mix had the lowest mass loss. The data presented here does align with the findings of Atiş (2002). Atiş (2002) replaced cement with 50% and 70% fly ash and measured abrasion resistance in samples aged three days to three months and found that fly ash mixtures had improved abrasion resistance over the all-cement mixtures.

Regarding slag, Fernandez and Malhotra (1990) measured the wear depth at 120 days of mixes containing up to 50% slag and found the addition of slag reduced abrasion resistance, which does not align with the findings here. One challenge would be the minimal mass loss compared to the sample size. Each sample weighed 40-43 pounds and lost only 0.001 pounds (approximately one gram) after each application of the drill press. Langan et al. (1990) also found their SCM-containing samples had minimal mass losses.

Abrasion by Studs, Method A: Prall Method: Of the five mixes, only the quaternary SL12 SF4 FA1 and SL8 SF8 FA3 mixes performed satisfactory, according to the Nordic Classification (Table 4.2). The other three mixes performed less satisfactorily (Table 4.8 and Figure 4.13). These classifications are dependent on the volume lost, so if there is a high Prall value of over 40 or 50 cm^3, then the sample will be designated as less satisfactory or poor, respectively. Although none of the samples had Prall values which were good or very good, one contributing factor may have been the aggregate hardness, which has found to be a large contributor to pavement performance (Frith et al., 2004).

SF8 SL22 SF8 SL12 SF4 FA1 SL8 SF8 FA3 FA31 SF4

Figure 4.13 Prall samples after testing (Bowthorpe, 2019)

Table 4.8 Prall test results

Mixture	**Prall-loss (cm^3)**	**Nordic Classification**
SL8	45.3	Less satisfactory
SL22 SF8	43.2	Less satisfactory
SL12 SF4 FA1	37.3	Satisfactory
SL8 SF8 FA3	39.4	Satisfactory
FA31 SF4	49.5	Less satisfactory

Data was not collected in this study regarding the aggregate hardness, although care was taken to use consistent aggregate during the screening and performance tests. Research by Snilsberg et al. (2016) investigated the effect of aggregate size on the abrasion resistance of asphalt pavements using the Prall test and a road simulator and found the coarse aggregate content in asphalt concrete was an important contributing factor to abrasion resistance, with smaller aggregate resulting in lower abrasion resistance. The concrete in this study contained no coarse aggregates, which likely contributed to the low Prall results as well. The materials technicians who performed the Prall tests for this study also noted the small aggregate size, noting that although a skid resistant calcined bauxite aggregate was used, the aggregate was very small which resulted in a high paste surface area, which eroded, released the aggregate particles, and may have contributed to the low test results (Bowthorpe, 2019).

Gartin and Saboundjian (2005) correlated four Alaskan asphalt pavement rutting rates in inches per million traffic passes and their respective Prall values and found an R^2 value of 0.933, suggesting a Prall test value is indicative of field performance. As mentioned previously, they also measured the rut depth of two PCC WIM slabs in Anchorage and compared their rutting to nearby asphalt pavements of the same age and experiencing the same traffic. The PCC surfaces of WIM sites at Tudor Road and Minnesota Road had 29% and 38% less rut depth, respectively, than those of the nearby asphalt pavements measured.

Limited research was found on the Prall test results of other concrete pavements, but Scholz and Keshari (2010) did conduct Prall tests on high strength concrete mixes. Their results varied from 18.0 (very good) for a mixture with a 13,600 psi 28 day compressive strength to 49.1 (less satisfactory) for their control mix which had a 7,900 psi 28 day compressive strength. In this study mixes had 28 day compressive strengths of 6,000 to 8,000 psi, which if 28 day compressive strength is indicative of abrasion resistance, is reasonable given that some of the mixtures' resistances were also classified as less satisfactory, similar to Scholz and Keshari's (2010) control mix. Although the samples had results which varied from satisfactory to less satisfactory, according to their Nordic Classification, it is important to keep in mind that this test is usually used in Alaska to analyze the performance of flexible pavements, and therefore may not be as predicative of concrete pavement's field performance as it is for asphalt pavement.

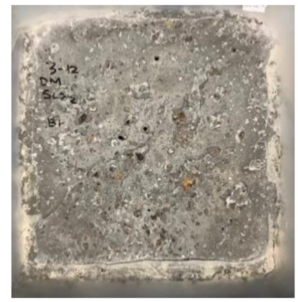
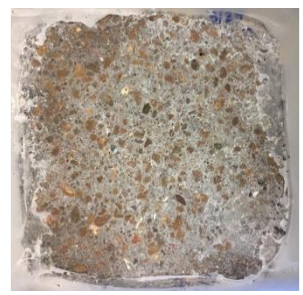
4.3.3.2. Scaling resistance after exposure to deicing chemicals. Overall all the mixtures performed poorly with visual ratings of four to five after 50 days of exposure to a $CaCl_2$ solution and daily F-T cycles. These visual ratings were based on the ASTM C672 standard (Table 4.3). The SF4 and SL12 SF4 FA1 mixes performed the worst with severe surface scaling and a visual rating of five at 50 days. The remaining mixes performed marginally better with moderate to severe scaling at 50 days with visual ratings of four (Tables 4.9 and 4.10).

Taylor et al. (2004) tested the scaling resistance of samples containing either all cement, 50% slag, or 25% fly ash, and compared different finishing techniques. They found that for samples which were finished soon after molds were filled, as was done in this study, by 50 days the all-cement samples had an average rating of five, the 50% slag mixtures had a rating of three, and the 25% fly ash samples had a 0.5 rating. Interestingly, Bouzoubaâ et al. (2008) had different findings. In their study seven mixes were investigated including an all-cement control, binary fly ash and slag mixtures, and ternary mixtures of silica fume with either slag or fly ash. Their 25-35% fly ash mixes had a 50 day rating of five, while their 25-35% slag mixes having a rating of 3-4, and their all-cement control mixture had a rating of zero. Similar to this study, after 50 days all mixes, excluding the control, had ratings ranging from three to five. Sidewalks placed in Canada, which were cast from the same mixes studied, found that after four winters, all mixes, but the ternary fly ash silica fume mixture, had visual ratings of zero to three. The fly ash and silica fume mix had a rating exceeding four. The authors concluded ASTM C672 may be too severe since most of the mixes which perform poorly during the ASTM C672 tests performed well in the field. Therefore although the mixes in this study had visual scaling ratings of 4-5 at 50 days, this does necessarily mean they will perform poorly in the field.

Table 4.9 Visual rating at 50 days

Mixture	**Visual rating at 50 days**
Control (SF8)	5
Flexural (SL22 SF8)	4
Optimal all (SL12 SF4 FA1)	5
Compressive (SL8 SF8 FA3)	4
Workability (FA31 SF4)	4

Table 4.10 Deicer scaling samples before and after 50 cycles

Mixture	Before (0 days)	After (50 days)
Control SF8		
Optimal flexural strength SL22 SF8		
Optimal compressive strength, flexural strength and workability SL12 SF4 FA1		
Optimal compressive strength SL8 SF8 FA3		
Optimal workability FA31 SF4		

4.3.3.3. Freeze-thaw resistance. Concerning F-T resistance, SL8 performed the best with a durability factor of 98.9% while the SL12 SF4 FA1 and SL22 SF8 mixes performed the worst with factors of 25.1% and 30.7%, respectively (Table 4.10). The durability factor is determined using Equation 4.2 and is indicative of the F-T cycles a sample can withstand before deteriorating (Toutanji et al., 2004). A 100% durability factor after 300 cycles would signify no decrease in the ultrasonic pulse velocity measured over time, and therefore high durability in regard to exposure to F-T cycles. A lower durability factor would demonstrate a decrease in the ultrasonic pulse velocity and subsequently lower quality and durability of the sample. It is important to keep in mind these factors are based on 180 cycles and not the ASTM 666 standard of 300 cycles. Therefore all the mixtures may have different durability factors than those in Table 4.10.

Table 4.11 Durability factor of each mix

Mixture	**Durability factor (%)**
Control (SF8)	98.9
Optimal Flexural (SL22 SF8)	25.1
Optimal all (SL12 SF4 FA1)	30.7
Optimal compressive (SL8 SF8 FA3)	70.1
Optimal workability (FA31 SF4)	74.2

Toutanji et al. (2004) found similar results to those presented here when they tested 17 mixtures including an all-cement mix and 16 binary and ternary mixes of silica fume, class C fly ash, and slag. The all-cement mix performed the best with a durability factor of 89.7% after 300 cycles, followed by the 8% silica fume mixture at 34.9%.

Overall the binary fly ash mixtures performed poorly during F-T testing while the ternary fly ash and slag mixtures and the binary slag mixtures had better resistance. Other research by Chung et al. (2010) on all-cement, binary 10% silica fume and binary 20% fly ash mixtures on varying w/c ratios and air contents found all mixtures to have durability factors over 95%. It is important to keep in mind that F-T laboratory cycles are more extreme than what would normally occur in the field and samples which perform poorly in the lab may not always perform poorly in the field (Mehta, 1991).

4.3.3.4. Chloride ion penetration resistance. All mixtures had chloride ion permeability ratings of low (<2000 coulombs) or very low (<1000 coulombs) with the SL12 SF4 FA1 mixture having the highest charge passed and the fly ash mixture having the lowest (Table 4.12). Since a low chloride ion penetration is indicative of low porosity, which in turn is related to improved durability, all the mixtures would likely have good durability performance in the field.

Table 4.12 Chloride permeability results

Mixture	**Coulombs**	**Rating**
Control (SF8)	429	Very low
Optimal Flexural (SL22 SF8)	619	Very low
Optimal all (SL12 SF4 FA1)	1038	Low
Optimal compressive (SL8 SF8 FA3)	378	Very low
Optimal workability (FA31 SF4)	250	Very low

Other researchers also found the addition of SCMs reduces permeability. In a similar study, Gesoğlu et al. (2009) tested the chloride permeability of binary, ternary and quaternary mixtures containing slag, fly ash and silica fume. While the all-cement mixture had a moderate permeability rating, all the SCM mixtures had chloride permeability ratings of either low or very low. They also tested binary and ternary mixtures similar to those tested in this study. Their data ranged from 410-800 coulombs with very low ratings, similar to the findings presented here. Nehdi et al. (2004) also investigated the chloride permeability of SCM mixtures including binary mixtures containing 50% fly ash or 50% slag, ternary mixtures of 25% fly ash and 25% slag, and a mixture containing 20% slag, 24% fly ash and 6% silica fume. The control had a high chloride ion permeability rating, the binary mixtures had moderate ratings and the ternary and quaternary mixtures had low permeability ratings. Yang et al. (2017) also measured the chloride permeability of all-cement mixtures as well as those containing either 40% fly ash or slag. After wet curing for five days, samples were dry cured for 360 days. The permeability ratings of the all-cement mixture was moderate, the fly ash mixture was low and the slag mixture was very low. Although an all-cement mixture was not tested in this study, all the SCM mixtures had low or very low chloride permeability ratings, suggesting a high resistance to chloride ion penetration and subsequently good durability. The lower chloride permeability of SCM mixtures can be partially due to the increased particle packing of the concretes due to the typically smaller size of SCM particles in comparison to cement particles.

5. PRELIMINARY COST ANALYSIS

A preliminary construction cost analysis for a hypothetical mile-long two-lane high-traffic stretch of highway in Anchorage, Alaska was conducted to compare the different concrete mixtures proposed. This analysis was conducted to evaluate the economic efficiency of the fix mix designs investigated. Based on material costs from Alaska Basic Industries in Anchorage from June 2019 (Schlee, 2019), the following raw material costs were assumed (Table 5.1). The cost of ground granulated blast furnace slag (GGBFS) in Fairbanks was assumed to be the same in Anchorage. These costs depend on availability and the market, and that if constructing a pavement on a large scale, these costs would likely be reduced due to purchasing materials in bulk.

Table 5.1 Cost of materials in Anchorage, AK in June 2019

Material	Cost per unit
silica fume	$30 / 25-lb
fly ash	$295 / ton
cement	$165 /ton
GGBFS (slag)	$250 /ton (in Fairbanks)

Using construction cost data obtained from the RS Means Heavy Construction Costs book (2019), the remaining construction costs were calculated. All costs were based on an assumed 2-lane 24-feet wide pavement with a 24-inch-thick subbase. Communications with Schaefer (2019) at Alaska DOT&PF found that a high traffic (approximately 40,000 AADT) pavement in Central Alaska would generally have an 18-

36 inch deep subbase, depending on whether permafrost was present. Therefore a 24-inch thick subbase was assumed. The concrete pavement was assumed to be six inches thick with 18 pounds per square yard of reinforcing steel. In addition, transverse joint dowels were assumed to be spaced at one foot with contraction joints spaced at 12 feet. All costs obtained from RS means were increased by the Anchorage, Alaska rate of 115.8% the national average. In addition, following the WSDOT (2018) example calculations, cost increases of 5, 15, and 10% were added to represent the mobilization, engineering, and contingencies costs, respectively (Table 5.2). The combined costs sourced from Alaska prices of cementitious materials and RS cost estimations resulted in the following assumed cost per two-lane one-mile stretch of pavement for each mix design (Table 5.3). These values were calculated using the assumed quantities and costs summarized in Table 5.2, but the SL8 cementitious value was changed to represent each mixture's respective cementitious materials cost, which are also summarized in Table 5.3 as the cost per 6-inch-thick square-yard of pavement ($/6"-thick/yd^2).

Based on the results the SL12 SF4 FA1 mix proves to be the most cost-effective mix design at around $1.6 million dollars. This being said, the cost between the five options varies only by about 2% with a standard deviation of $30,000. With such a minimal difference between the construction cost of using any of the different concrete mix designs in a pavement, any of the mixtures would likely be a good choice.

Since there are only a few concrete roads built and maintained by Alaska DOT&PF it is challenging to estimate and verify these costs using historical data. The cost of paving varies widely depending on the location, design, and traffic load, but for comparison, Sullivan and Moss (2014), in their report for the Portland Cement

Association, estimated paving an urban 2-lane mile with concrete to cost $770,000. Another estimate by the Arkansas Department of Transportation (ArDOT, 2016) estimates the total costs for a mile-long concrete lane in Arkansas to be $1.1 million, or around $2.2 per 2-lane mile. Although there appears to be a wide variance in these costs, construction costs in Alaska are likely even higher due to the geographical location and short construction season in Alaska.

Table 5.2 Assumed construction cost for 2-lane rigid pavement using Control SL8 Mix

Item	**Unit**	**Cost /unit**	**Quantity /24'-wide road mile**	**Total ($1000)**
non-cementitious materials	6" pavement/yd^2	15.09	14080	212
SL8 cementitious materials*	6" pavement/yd^2	18.31	14080	258
placement labor and equipment	6" pavement/yd^2	4.63	14080	65
18 lb./ yd^2 reinforcing steel	yd^2	15.86	14080	223
transverse joint dowels every 12'	ea.	13.32	10560	141
transverse contraction joints every 12'	l.f.	5.96	10560	63
24" deep subbase course	yd^2	31.27	14080	440
subtotal				1,403
mobilization (5% materials)			45,205	1,448
engineering and contingencies (15% mobilization and materials)			142,395	1,590
preliminary engineering (10% total)			109,169	1,699
total				1,699

*Cost/unit varies depending on mix. Cost is adjusted for Anchorage, AK prices from RS Means national average.

Table 5.3 Estimated cost of each alternative

Alternative (no.)	**Cementitious Materials** ($/6"-thick/$yd^2$)	($/$yd^3$)	**Total Cost** ($/2-lane mile)
1. SF8	18.31	110	1,699,000
2. SL22 SF4	19.26	116	1,713,000
3. SL12 SF4 FA1	14.31	86	1,643,000
4. SL8 SF8 FA3	18.86	113	1,707,000
5. FA31 SF4	15.84	95	1,665,000

6. CONCLUSION

The objective of this study was to identify and select concrete mix designs which would provide excellent abrasion resistance and durability. Following this a literature review of past and present studies regarding these topics was performed, as well as a survey of Alaskan engineers and Alaska DOT&PF material and pavement engineers to determine current practices and methods regarding concrete pavements in Alaska. Preliminary screening tests of ternary mixtures containing silica fume with either GGBFS or class F fly ash were conducted. Tests included workability, air content, compressive strength and flexural strength. Following this four optimal mixtures were determined using the statistical software Minitab. Results obtained were verified using special cubic models and desirability functions. These four mixtures, as well as the control binary mixture, were then subjected to further durability and mechanical testing. These mixtures included an 8% silica fume control mixture (SF8), a 22% slag with 8% silica fume mixture (SL22 SF8), 12% slag with 4% silica fume and 1% fly ash mixture (SL12 SF4 FA1), 8% slag with 8% silica fume and 3% fly ash mixture (SL8 SF8 FA3) and a mixture containing 31% fly ash with 4% silica fume (FA31 SF4). Testing included drying shrinkage, abrasion resistance, scaling resistance to deicer salts, F-T resistance and chloride ion penetration resistance. Regarding each test the following results were found:

- Regarding compressive strength and shrinkage, by 28 days the SF8 had the highest compressive strength while the FA31 SF4 mixture had the lowest drying shrinkage at 0.01% expansion. However, all mixtures have 28-day compressive strength greater than 6,000 psi, which fulfills the minimum strength requirement of 6,000 psi to be considered high-strength concrete (ACI, 1992). All mixtures are also within the SCM drying shrinkage limits of 0.04% suggested by Mokarem et al. (2005).

- For abrasion resistance the FA31 SF4 mixture had the highest resistance by mass loss and the SL12 SF4 FA1 mix had the lowest volumetric mass loss by Prall abrasion testing. Regarding the mass loss, an average of only one gram of material was lost after each application of the drill press, so overall there was almost negligible mass loss equivalent to 0.01-0.03% per sample, indicative of likely a high abrasion resistance to studded tires. For Prall abrasion testing, two mixtures had Nordic Classifications of satisfactory while the remaining three were classified as less satisfactory. Although the classification ratings are not all satisfactory, it is important to keep in mind this test is usually used for asphalt pavements, and other researchers (i.e. Scholz and Keshari, 2010) which used Prall testing to test their 8,000 psi concrete found their ratings to be classified as less satisfactory as well, similar to these findings.
- The SL22 SF8, SL8 SF8 FA3 and FA31 SF4 mixes had similar 50 day visual ratings of four, equivalent to moderate to severe scaling, when measuring their respective deicer salt scaling resistance. The SL8 SF8 FA3 and SL12 SF4 FA1 mixes performed worse with visual ratings of five, equivalent to severe scaling. Although these ratings indicate the samples performed poorly, this may not be indicative of field performance. For example, Bouzoubaâ et al. (2008) found that SCM mixes which performed poorly during ASTM C672 did not have as severe scaling in the field.
- After testing chloride ion penetration, all mixtures but SL12 SF4 FA1 had very low ratings of less than 1,000 coulombs. SL12 SF4 FA1 had a low rating of 1,038

coulombs. FA31 SF4 had the lowest rating of 250 coulombs. Therefore all the mixtures likely have low permeability and subsequently high durability.

- For F-T resistance after 180 cycles the SF8 mixture performed the best with a durability factor of 99% while the SL22 SF8 and SL12 SF4 FA1 mixtures performed the worst with durability factors of 25% and 31%, respectively. The other two had factors of 70% and 74%. A durability factor below 60% is considered failure, at which point testing can end, and by 180 cycles two of the five mixtures had failed. A preliminary cost analysis comparing the construction costs in Alaska associated with each of the five performance testing mixtures found that the SL12 SF4 FA1 mixture would have the lowest construction cost of $1.6 million per 2-lane highway. The variance in cost though was minimal with the construction costs of the five mixtures ranging from $1.6 to $1.7 million.

In terms of the properties evaluated within this study (i.e. strength, shrinkage, chloride ion penetration, F-T resistance, deicer scaling resistance, and abrasion resistance), the five mixtures, including the four optimal mixtures and control, all provided overall good performance. Therefore of the five mixtures, the quaternary SL12 SF4 FA1 provided the overall best performance due to its good strength and abrasion resistance, favorable fresh and durability properties, and low construction cost. Subsequently, within the scope of this study, a quaternary mix design, containing primarily silica fume and slag, appears to provide the overall best performance in terms of strength, durability, abrasion resistance, and cost.

The next recommended step in this research would be constructing and monitoring test sections in the field using the optimal mixtures determined to verify and validate results generated from the laboratory tests. Long-term performance data could be collected and analyzed for an in-depth life cycle cost analysis. In addition, this study focused on silica fume, slag, and fly ash, but further research could investigate other types and dosages of SCMs using additional tests and more extensive F-T testing. Additionally, these tests primarily focused on properties measured over 28 days and longer term strength and durability properties were not investigated. Further research into the long term durability characteristics of abrasion resistant concrete pavements would also be beneficial.

APPENDIX

Table A-1 Alaska fine aggregate gradation

Sieve No.	% Total	% Passing
1/2"	0.02	99.98
3/8"	0.01	99.97
#4	0.88	99.09
#8	8.23	90.86
#16	19.63	71.23
#30	43.71	27.52
#50	19.25	8.26
#100	6.53	1.73

Table A-2 Alaska intermediate aggregate gradation

Sieve No.	% Total	% Passing
1/2"	0.03	99.97
3/8"	1.74	98.23
#4	94.09	4.14
#8	3.81	0.33
#16	0.27	0.06
#30	0.01	0.05
#50	0.02	0.03
#100	0.00	0.03

Table A-3 Missouri fine aggregate gradation

Sieve No.	% Total	% Passing
3/4"	0.00	100.00
3/8"	0.06	100.00
#4	-0.01	100.00
#8	19.48	80.52
#16	11.53	68.99
#30	29.82	39.16
#50	29.52	9.65
#100	9.59	0.05

Table A-4 Missouri intermediate aggregate gradation

Sieve No.	% Total	% Passing
3/4"	0.00	100.00
3/8"	1.56	98.44
#4	82.67	15.77
#8	14.91	0.86
#16	0.56	0.29
#30	0.15	0.15
#50	0.15	0.00
#100	0.00	0.00

Table A-5 Prall test results

DEPARTMENT OF TRANSPORTATION
SOUTHEAST MATERIALS LAB
PRALL WORKSHEET
PROJECT: Diane Murph/Jenny Liu Research project
SAMPLE DATE: 6/7/2019
TECH: TB

Sample Number	Mass in Air (A)	Weight in Water (B)	Mass SSD (C)	Bulk Specific Gravity A/(C-B)	Mass Cold before Abrasion SSD	Mass after Abrasion SSD	Abrasion Value
1a	599.0	349.0	600.5	2.382	582.8	475.6	45.0
1b	571.8	333.9	573.0	2.391	574.6	465.7	45.5
						Average	45.3
2a	585.1	343.8	586.5	2.411	587.7	471.9	48.0
2b	604.8	352.9	606.4	2.386	583.8	492.5	38.3
						Average	43.2
3a	550.6	313.8	552.8	2.304	555.7	460.6	41.3
3b	569.1	322.9	571.5	2.289	545.0	468.9	33.2
						Average	37.3
4a	557.3	315.9	559.9	2.284	532.7	449.3	36.5
4b	545.2	311.1	548.0	2.301	551.1	453.6	42.4
						Average	39.4
5a	517.3	288.9	520.1	2.237	524.6	419.5	47.0
5b	569.3	321.0	572.4	2.265	551.5	433.7	52.0
						Average	49.5

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