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Designs of Abrasion Resistant and Durable Concrete Pavements Made with SCMs for Cold Climates

Diane Murph

Jun Liu

Jenny Liu
Missouri University of Science and Technology, jennyliu@mst.edu

Designs of Abrasion Resistant and Durable Concrete Pavements Made with SCMs for Cold Climates

Diane Murph[1]; Jun Liu[2]; and Jenny Liu, M.ASCE[3]

Abstract: Rutting from studded tire wear is a typical pavement distress in cold climates such as that of Alaska and other northern states. Current state-of-the-art advancements in material technology and concrete pavement design have allowed for implementation of improved materials and concrete pavement sections that are more resistant to rutting. The addition of supplementary cementitious materials (SCMs) has been identified as one effective way to produce concrete pavements with better abrasion resistance. The objective of this study was to identify and develop concrete pavement mix designs containing SCMs that can provide excellent abrasion resistance and durability to address rutting from studded tire wear and accommodate extreme climate conditions in cold regions. This study involved two phases of work. During Phase I, a series of ternary mixes containing silica fume with either slag or class F fly ash were produced and tested. The results were statistically analyzed using Minitab version 19.2.0 to identify mix designs with good performance in terms of workability, compressive strength, and flexural strength requirements for pavement applications. In Phase II, the mechanical properties and durability of concrete specimens with selected mix designs from Phase I were further evaluated to identify the optimum mix design with SCMs. This included tests for compressive strength, drying shrinkage, abrasion resistance, and other dualities such as scaling resistance to deicer salts, freeze-thaw resistance, and chloride ion penetration resistance. In terms of the properties evaluated within this study along with a cost analysis, five mixes, including four optimal mixes and the control, all provided good performance, but a quaternary mix design containing primarily silica fume and slag (SL12 SF4 FA1 mix) appeared to provide the overall best performance considering strength, durability, abrasion resistance, and cost. **DOI: 10.1061/JPEODX.0000360.**

Author keywords: Concrete pavement; Abrasion resistance; Durability; Supplementary cementitious materials (SCMs).

Introduction

Rutting that causes a progressive loss of material from pavement surfaces is a typical pavement distress that occurs in the central region of Alaska and other cold regions 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 which also tend to wear away the pavement surface in the wheel path and create safety issues such as depressions (Anderson et al. 2007; Cotter and Muench 2010). Millions of dollars in road maintenance costs are expended annually to address surface course wear and deformation of existing pavements (Malik 2000; Zubeck et al. 2004). According to Cotter and Muench (2010), the average concrete pavement in Washington State wears at about 0.254 mm per one million studded tire passes. The highest wear rate was near 0.5 mm/year on interstate 90 in the Spokane area, while the lowest wear rate was in the range of 0.04–0.09 mm/year in other locations. A wide range of wear rates has also been found in various sections of concrete and asphalt pavements in Oregon (Malik 2000), with an average wear rate of 0.236 mm per 100,000 studded tire passes for concrete pavement and a rate of 0.980 mm per 100,000 studded tire passes for asphalt pavement.

The primary current practice highway agencies are adopting to reduce studded tire damage is to limit its use. Thirty-six states have opted to only allow studded tire use during certain seasons or locations, while seven states prohibit their use with no exceptions (Zubeck et al. 2004). In addition, due to improvements in both pavement and studded tire designs, the damage studded tires have on pavement is likely decreasing; it is estimated that studs used in the 1960s were four to ten times more damaging to pavement compared to the present day (Tremblay and Fitch 2011).

In addition, the current state of the art in material technology and pavement design has allowed for the implementation of improved materials and pavement sections that are resistant to rutting. For example, the Alaska Department of Transportation and Public Facilities (ADOT&PF) requires some regions to test their aggregate samples using the Prall test to determine their resistance to wear to minimize the damage from studded tires (Gartin and Saboundjian 2005). The Washington State Department of Transportation (WSDOT) has conducted a series of studies to address the tire wear resistance of concrete pavements. Their efforts have included the use of combined aggregate gradations, ultrathin and thin whitetopping, experimental finishing methods such as longitudinal tining and carpet drag texturing, higher flexural strength mix designs, high cement content mix designs, and special additives (Anderson et al. 2011). In recent years, the addition of supplementary cementitious materials (SCMs) (i.e., silica fume, fly ash, and slag) have been found to produce concrete with more resistance to rutting (Ramana et al. 2014). In addition, the process of adding two SCMs to the binder fraction of ordinary portland cement (OPC)

[1]Graduate Research Assistant, Dept. of Civil, Architectural and Environmental Engineering, Missouri Univ. of Science and Technology, Rolla, MO 65409. Email: dmmctc@mst.edu

[2]Research Associate, Louisiana Transportation Research Center, 4101 Gourrier Ave., Baton Rouge, LA 70808. Email: junliu@lsu.edu

[3]Professor, Dept. of Civil, Architectural and Environmental Engineering, Missouri Univ. of Science and Technology, Rolla, MO 65409 (corresponding author). ORCID: https://orcid.org/0000-0002-3840-1438. Email: jennyliu@mst.edu

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is becoming increasingly prevalent, because the added benefits, such as enhanced performance and cost reduction, are gradually becoming more apparent (Schlorholtz 2004). Generally, ternary mixes show overall better performance, because the negative properties of any one SCM can be offset by the positive properties of another carefully selected material (FHWA 2015). For example, blending an ultrafine pozzolan such as silica fume with slag or fly ash can prevent excessive bleeding problems by offsetting the increased water demand typically associated with the use of silica fume (Thomas et al. 1999; Bleszynski et al. 2002). Higher compressive strength at 28 days was reported when comparing a ternary concrete mix containing 20%–25% slag and 3%–5% silica fume to the control mix (Thomas et al. 2007). Many improved durability characteristics have been reported for ternary mixes when proportioned accurately, including better chloride resistance (Wongkeo et al. 2014), higher resistance to alkali-silica reactions (Shehata and Thomas 2002), better scaling resistance (Radlinski et al. 2008), and less deterioration after freeze-thaw cycles (Rupnow 2012). Another important benefit provided by ternary concrete mixes is an increase in abrasion resistance. Scholz and Keshari (2010) found that, when compared with a control mix, a mix with 4% silica fume and slag showed significantly higher abrasion resistance, but increasing silica fume beyond 4% did not add further benefits. Rashad et al. (2014) indicated that high-volume fly ash (HVFA) concrete blended with either silica fume or equal combinations of silica fume and granulated blast-furnace slag (GGBS) showed higher abrasion resistance, while lower abrasion resistance was found in HVFA blended with GGBS. Another study of ternary concrete mixes with different proportions of low-calcium class F fly ash (20%, 30%, or 40%) and silica fume (5% or 10%) found that a ternary mix containing fly ash up to 30% and 5% silica fume showed better performance against abrasion erosion (Ramana et al. 2014).

According to those working in Alaska's concrete industry, there have been mix designs and applications of concrete with SCMs with demonstrated strength and rutting resistance. One example is the concrete mix design used in the main runway rehabilitation project in King Salmon, Alaska, in 2013. However, only one SCM, silica fume, was considered in this mix design. How other SCMs, such as fly ash or GGBS, could provide additional engineering and cost benefits has not yet been investigated. In addition to its fundamental strength properties, a comprehensive evaluation of the durability of concrete containing SCMs is crucial to arriving at a better understanding of whether mix designs can be successful for the extreme climate conditions in Alaska. Hence, the objective of this research was to identify and develop concrete mix designs containing SCMs to provide good workability, mechanical properties, and durability. The study included two phases (i.e., Phases I and II). In the phase of initial screening (Phase I), a series of ternary mixes containing silica fume with either slag or class F fly ash were produced and tested, and mix designs achieving good strength and workability were developed. In Phase II, the mechanical properties and durability of concrete specimens with selected mix designs from Phase I were further evaluated to identify the optimum mix design of durable concrete made with SCMs. This included tests for compressive strength, drying shrinkage, abrasion resistance, scaling resistance to deicer salts, freeze-thaw resistance, and chloride ion penetration resistance. In addition, a preliminary construction cost analysis for a hypothetical 1.6-km-long two-lane high-traffic stretch of highway in Anchorage, Alaska, was conducted to compare the different concrete mixes proposed.

Table 1. Mix proportions for 0.76 m^3 (1 yd^3) of the base line mix

Constituent	Quantity
Type I cement	277 kg (611 lb)
Silica fume	23 kg (50 lb)
Intermediate aggregate	828 kg (1,826 lb)
Fine aggregate	566 kg (1,248 lb)
Water	114.6 kg (252.5 lb)
AEA	14.8 mL
HRWR	1,956 mL

Phase I: Initial Screening

Materials

The cementitious materials used in this study included type I/II cement [ASTM C150 (ASTM 2017a)], class F fly ash [ASTM C618 (ASTM 2019a)], slag [ASTM C989 (ASTM 2009)], and silica fume [ASTM C1240 (ASTM 2020b)], which are typical and commercially available materials used in the Alaska concrete industry. Intermediate aggregates with a nominal maximum aggregate size of 9.5 mm were used. The fineness moduli of the intermediate and fine aggregates were 6.0 and 3.0, respectively. The mix design used by ADOT&PF in the main runway rehabilitation project in King Salmon, Alaska, in 2013 was used as a baseline mix design in this study; in that mix, the content of silica fume by weight of total cementitious materials was 8%. This mix was used for fixed-form paving. Table 1 presents the mix proportions. Using this base mix design, an additional nine mixes were created in which the SCMs and their respective contents were changed (Table 2). The water content, air entrainment admixture (AEA) dosage, and aggregate ratios remained the same. All mixes had a cement factor of 7.0 and a water to cementitious materials ratio (w:cm) of 0.331. For silica fume, the equivalent dosage of either a full or half 22.7-kg bag of silica fume per 0.76 cubic meters of concrete was used, equivalent to 3.8% or 7.6% of the cementitious materials by mass. The remaining cementitious materials consisted of either 25% or 40% fly ash or slag. These SCM dosages are commonly used and are recommended by the Alaska concrete industry. ADOT&PF (2020) limits SCM content in structural concrete to 35% for fly ash, 40% for slag, and 10% for silica fume.

Mixing Procedures and Screen Testing Details

The same mixing procedure was used for each batch. First, aggregates were mixed with 75% of the water for 5 min, then the silica fume was added and mixed for 5 min, and then the remaining cementitious materials were added. The high range water reducer (HRWR) was then added and mixed for 2 min; then, the AEA was added and mixed for 2 min. To measure slump, ASTM

Table 2. Cementitious materials used in mixes in Phase I study

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

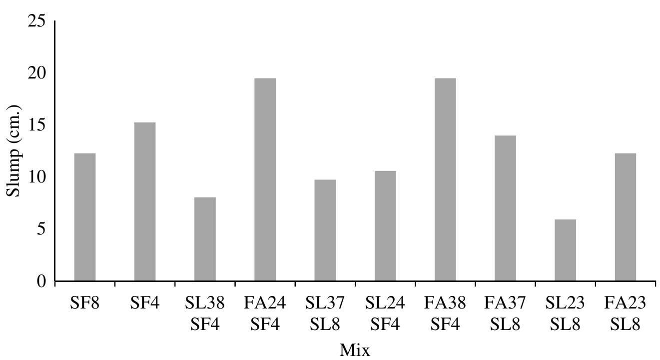
C143 (ASTM 2015) was followed. For compressive strength 10 × 20-cm cylinders were filled per ASTM C192 (ASTM 2020a). Samples were then wet cured in lime, and compressive strength was measured using an average of three samples following ASTM C39 (ASTM 2017c) at 1, 3, 7, 14, and 28 days. For flexural strength, two 15 × 15 × 53-cm beams were constructed for each mix, wet cured in lime, and tested at 14 and 28 days following a modified version of ASTM C39. The 14-day and 28-day beams for the control mix (SF8) were broken using a force method of 8,050 N/min. Due to safety concerns, the remaining beams were broken using a displacement method with a rate of 0.005 mm/s.

Screening Tests Results

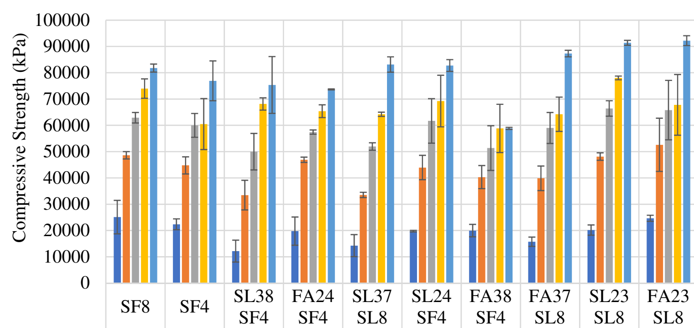
Fig. 1 illustrates the initial screening test results, which include properties of the workability of the fresh concrete (i.e., slump) and the strength of hardened concrete (i.e., compressive and flexural strengths at various ages) for all mixes. The purpose of selecting these properties for initial screening was to ensure that these ternary mixes were workable and easy to use in construction and placement in the field and to have good fundamental engineering properties.

For structural concrete, ADOT&PF allows for a maximum slump of 15 cm when using a type A, D, or E water reducer and a 23-cm slump when using HRWR type F or G (ADOT&PF 2020). The HRWR used in this research met the requirements for both types A and F, and the slumps of all mixtures were below the maximum slump of 23 cm for a type F HRWR. In addition, as shown in Fig. 1(a), the slump decreased as silica fume content increased. This was not surprising, given the high surface area of silica fume particles, which, in turn, increases water demand (ACI 2012). Overall, the slump varied from about 5–18 cm. Although the HRWR was adjusted slightly to improve workability, this was still a large variance; however, other research has found large variances in workability when working with different combinations of SCMs. In particular, Toutanji et al. (2004), when working with binary, ternary, and quaternary mixtures of silica fume, slag, and fly ash, found workability ranging from a 0-cm slump for a mix of silica fume, fly ash, and slag to a 15-cm slump for a 30% fly ash mix.

ADOT&PF requires a minimum compressive strength at 28 days of 36 MPa (5,200 psi) for general use structural concrete and 43 MPa (6,200 psi) when improved strength and durability are required (ADOT&PF 2020). As Fig. 1(b) shows, all mixes involved in this study showed 28-day compressive strengths higher than 43 MPa. Regardless of silica fume content, the control mix had the highest compressive strength at 1 day [Fig. 1(b)], which may have been due to the higher pozzolanic activity of silica fume over that of fly ash and slag. Fly ash mixes had higher 3-day compressive strengths than slag mixes with the same SCM content. The 7-day compressive strengths of mixes containing fly ash or slag were similar. At 14 days, mixes containing slag had higher compressive strengths than fly ash mixes with the same SCM contents. In addition, in this study for almost all SCM mixes at ages up to 28 days, the mixes containing lower dosages of fly ash or slag had higher compressive strengths than those containing higher replacement

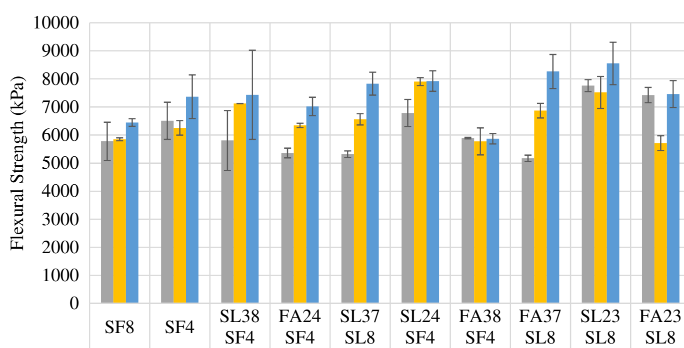


(a)



	SF8	SF4	SL38 SF4	FA24 SF4	SL37 SL8	SL24 SF4	FA38 SF4	FA37 SL8	SL23 SL8	FA23 SL8
1-day	25106	22366	12160	19776	14261	19742	19970	15736	20185	24648
3-days	48601	44777	33457	46909	33536	43950	40310	39863	48125	52579
7-days	62897	59982	49990	57423	51952	61692	51358	59013	66411	65800
14-days	73972	60474	68122	65369	64196	69235	58831	64207	78022	67778
28-days	81782	76920	75342	73641	83124	82739	58849	87297	91386	92191

(b)



	SF8	SF4	SL38 SF4	FA24 SF4	SL37 SL8	SL24 SF4	FA38 SF4	FA37 SL8	SL23 SL8	FA23 SL8
7-days	5776	6510	5806	5360	5317	6788	5894	5170	7761	7424
14-days	5846	6256	7122	6344	6559	7905	5773	6870	7520	5709
28-days	6446	7366	7436	7018	7831	7923	5871	8265	8550	7460

(c)

Fig. 1. Screening tests results: (a) workability; (b) compressive strength; and (c) flexural strength.

levels. Regarding the effect of SCM content on flexural strength at 7, 14, and 28 days, there were no obvious trends [Fig. 1(c)].

Selecting Mix Designs of Concrete with SCMs

Using the results obtained, an optimum mix design of concrete made with SCMs for each property (e.g., 1-day compressive strength, 3-day compressive strength, etc.) was determined. This was done using the Minitab version 19.2.0 statistical software response optimization tool. Using Minitab response optimization, slag, fly ash, silica fume, and cement contents were limited to the maximum and minimum quantities tested, and the responses were then modeled. These 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, quadratic, and special cubic models [Eqs. (1)–(3), respectively]. Linear models describe how each individual component affects the response. Quadratic models describe how two different components may affect each other and the response, and a special cubic model describes how the combination of three components may affect an outcome. Other models were not used because modeling the effects of, say, cement × cement, is unrealistic and redundant. In these three selected models, some relationships were not included. These include silica fume × fly ash, slag × fly ash, cement × slag × fly ash, and silica fume × slag × fly ash. Silica fume × fly ash was not investigated due to multicollinearity. In the case of the latter three relationships, the combination of slag and fly ash together was not tested, and, therefore, an appropriate coefficient for representing this relationship was not determined. Detailed calculations, which can be found in the reference Liu and Murph (2019), are not included in this section due to space constraints. Overall, it appeared that the special cubic model resulted 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 is marginally statistically significant. Therefore, a special cubic model was used to model the data

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

$$Response = A(cem) + B(sf) + C(fa) + D(sl) + E(cem)(sf) + F(cem)(sl) + G(cem)(fa) + H(sf)(sl) \tag{2}$$

$$Response = A(cem) + B(sf) + C(fa) + D(sl) + E(cem)(sf) + F(cem)(sl) + G(cem)(fa) + H(sf)(sl) + I(cem)(sf)(sl) + J(cem)(sf)(fa) \tag{3}$$

where cem = cement percentage; sf = silica fume percentage; fa = fly ash percentage; sl = slag percentage; and A, B, C ... = coefficients.

After determining the appropriate model, targets were set to maximize each response. These targets were set at 10% larger than the highest average mix measurement. For example, mix SF8 (the control) had the highest average 1-day compressive strength, so 110% of its compressive strength was used as the target. The minimum value used was the lowest average measurement in the laboratory. Each response was set to maximize at these set targets except workability, which was set at 15 cm. All the responses were weighed equally at 1.0, but the importance factor, k, varied. Workability, flexural strength, and compressive strength were each considered of equal importance at 3.33. Therefore, for each compressive strength response (1-day, 3-day, etc.) importance k was 0.67, and for each flexural strength response the importance was 1.11. Subsequently, Minitab determined the optimum mix design to contain 12% slag, 4% silica fume, and 1% fly ash.

Table 3. Selected mix designs for future testing

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

This method was repeated for determining additional mix designs: (1) for optimal workability, (2) for optimal compressive strength from 1 to 28 days, and (3) for optimal flexural strength from 7 to 28 days. In Case (1), the only response used was workability. In Case (2), all five compressive strength responses were used with equivalent importance assigned, while in Case (3), all three flexural strength responses were used with equivalent importance assigned. These results, along with the overall optimum mix and the original control mix [mix SF8 with 8% silica fume (SF)], were then used for further performance testing (Table 3).

The SL12 SF4 FA1 mix, with optimal overall properties considering compressive strength, flexural strength, and workability, contained primarily slag and silica fume (only 1% fly ash). Gesoğlu et al. (2009) had similar results; their optimum mix contained 44% slag, 1% fly ash, and 14% silica fume. Although the slag and silica fume ternary mixes generally improved hardened properties, only the ternary mix of fly ash and slag satisfied the V-funnel flow time requirements, which may explain the additional 1% fly ash contribution.

Phase II: Performance Tests to Identify Durable Mixes

Testing Details

The methods used in Phase I for mixing and measuring compressive strength were followed with three samples tested at 1, 3, 7, 14, and 28 days for each mix. To measure drying shrinkage, ASTM C157 (ASTM 2014) was followed. Three samples were used to measure shrinkage for each mix. Shrinkage samples were demolded 24 h after mixing, measured, and then cured for 28 days in a temperature-controlled water bath. After 28 days, the samples were measured again and left at 50% humidity at 23°C and measured daily for 28 days.

Alaska has used the Prall method to evaluate the rutting resistance due to studded tire wear of asphalt pavements, with good correlation between rutting rates and Prall values in previous studies (Gartin and Saboundjian 2005). However, when applied to concrete specimens, it has been found that the values of the volume loss of concrete specimens were much higher than common values for asphalt concrete specimens (not comparable at all); this is due to the significant difference in stiffness between asphalt and concrete materials (Liu and Murph 2019). Therefore, the Prall test is not feasible for evaluating concrete materials in comparison to asphalt materials. A modified ASTM C944 (ASTM 2017b) method was used instead for abrasion resistance measurements. For this test, three samples were tested for each mix to measure abrasion resistance by mass loss on samples aged 28 days. ASTM C944 requires

Fig. 2. Abrasion resistance measurement by mass loss.

Table 4. 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)

Table 5. Chloride ion penetrability based on charge passed

Charge passed (coulombs)	Chloride ion penetrability
>4,000	No scaling
2,000–4,000	Moderate
1,000–2,000	Low
100–1,000	Very low
<100	Negligible

Source: Data from ASTM C1202.

a rotating-cutter drill press to spin at a rate of 3.3 Hz (200 rpm). However, 2.5 Hz (150 rpm) was used in this study, because the press used only could rotate at 2.5 or 5.0 Hz (150 or 300 rpm). A 10-kg force was applied for 2 min in four sections of three separate samples (Fig. 2), and mass loss was measured after each 2 min period.

The scaling and deicing resistances of the samples were investigated following ASTM C672 (ASTM 2012). Two samples for each mix were first cured in a water bath for 14 days and then in air for 14 days. A 4% calcium chloride ($CaCl_2$) solution was then applied to the sample's surface at a 0.64-cm depth, and the samples were placed in deicing chambers. Every 5 days the solution was replaced. The samples were cast in a box, the bottom side of the box was tested to minimize the potential variations on finishing. The samples were photographed, and the condition of their surface was rated 0–5 as per ASTM C672 ratings (Table 4).

Using ASTM C1202 (ASTM 2019b), chloride ion penetration tests were conducted to evaluate the durability of the mixes. First, three samples for each mix were prepared and wet cured for 28 days. The conditioned samples were placed in the testing chamber, and each side was filled with either a 3.0% NaCl or a 0.3-N NaOH solution. A 60-volt electrical current was then applied across the sample for 6 h. Afterward, current versus time was plotted and a curve was drawn. The area under the curve was integrated to determine the coulombs passed. Based on this, the penetrability was determined (Table 5).

Using ASTM C666 (ASTM 2010), the resistance of each mix to rapidly repeated cycles of freezing and thawing was measured (Fig. 3). Two samples were prepared for each mix, and after curing in a water bath for 14 days, samples were kept in a temperature-controlled cabinet that exposed the samples to freezing temperatures for 4 h, followed by 2 h of thawing. Each sample was measured every 18 cycles using an ultrasonic test device that used transducers to determine the ultrasonic pulse velocity [Ultrasonic Pulse Velocity (UPV) Instrument, Lewis Center, Ohio]. The parameter durability factor (DF), which was used to indicate how many freeze-thaw (F-T) cycles a sample could withstand before deteriorating, was calculated [Eqs. (4) and (5)]. A higher durability factor suggests that a sample has better resistance to F-T cycles. A lower durability factor suggests that a sample's durability is low and degrades quickly after several F-T cycles. In particular, a DF of 100% after hundreds of cycles would mean the ultrasonic pulse velocity measured did not decrease over time, and, therefore, the sample has high durability with regard to exposure to freeze-thaw cycles. A lower DF demonstrates a decrease in the ultrasonic pulse velocity and, subsequently, the lower quality and durability of the sample

$$\text{RDME}(\%) = \frac{v_n^2}{v_0^2} \times 100 \tag{4}$$

$$\text{DF} = \text{RDME}_f \times n_f/(180 \text{ cycles}) \tag{5}$$

where v_0 = initial ultrasonic pulse velocity; v_n = ultrasonic pulse velocity at n number of cycles; RDME = relative dynamic modulus of elasticity; and n_f = number of cycles the RDME_f represents. RDME_f represents either the RDME once it reaches 60% or lower or the RDME after 180 cycles, whichever occurs sooner.

Performance Testing Results

Compressive Strength

Fig. 4 illustrates the compressive strength values of the tested mixes. As shown, the 28-day compressive strength of the control

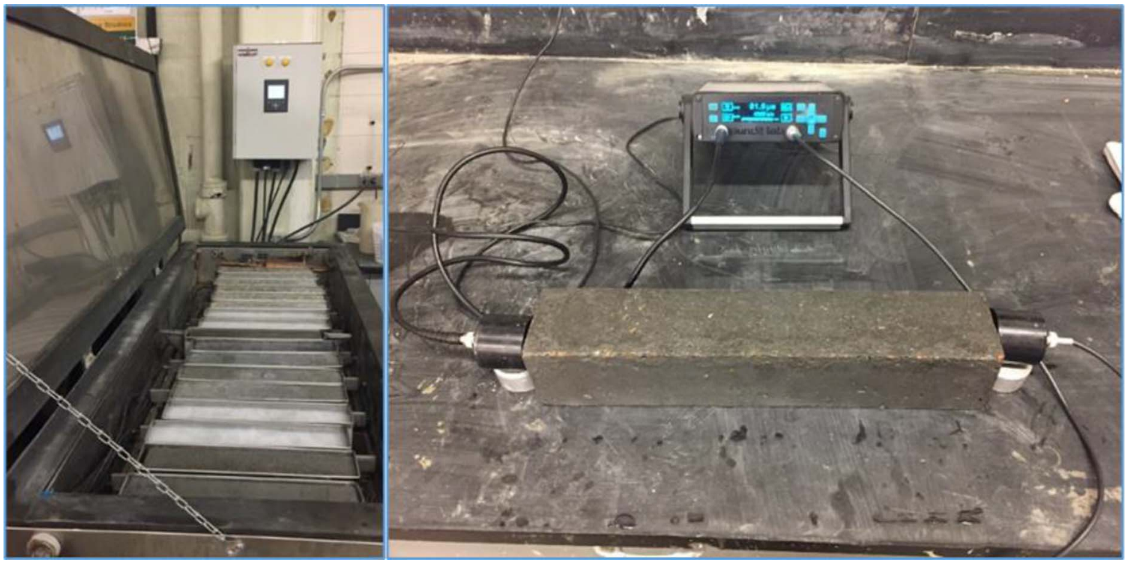
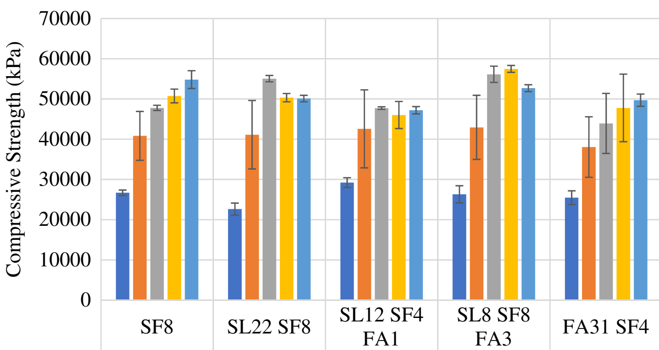


Fig. 3. Freeze-thaw testing of samples.



	SF8	SL22 SF8	SL12 SF4 FA1	SL8 SF8 FA3	FA31 SF4
1-day	26685	22633	29211	26296	25476
3-days	40830	41117	42573	42946	38054
7-days	47787	55050	47732	56129	43932
14-days	50753	50327	46000	57469	47773
28-days	54814	50108	47191	52689	49702

Fig. 4. Compressive strength of tested mixes.

mix had the highest value, followed by the SL8 SF8 FA3, SL22 SF8, FA31 SF4, and SL12 SF4 FA1 mixes. Nonetheless, all mixes had strengths higher than 41,000 kPa by 28 days, which is the compressive strength value for high-strength concrete defined by the American Concrete Institute (ACI) (Mehta 1991). In addition, the results indicated that all mixes met the ADOT&PF structural concrete strength requirement of 43,000 kPa at 28 days for concrete placed where improved strength and durability is required (ADOT&PF 2020). Therefore, any of these mixes would be adequate for use in high-strength concrete applications.

Drying Shrinkage

Regarding drying shrinkage, as shown in Fig. 5, the FA31 SF4 mix had almost no change in length for the first 28 days and then had minimal shrinkage of 0.005% after 56 days. The other mixes had shrinkage rates of 0.03% to 0.04% after 56 days, with the SL12 SF4 FA1 mix shrinking the most (Fig. 5). Typical drying shrinkage requirements are 0.05% or less at 28 days (NRMCA 2020), which all mixes met. Research by Akkaya et al. (2007) found that when

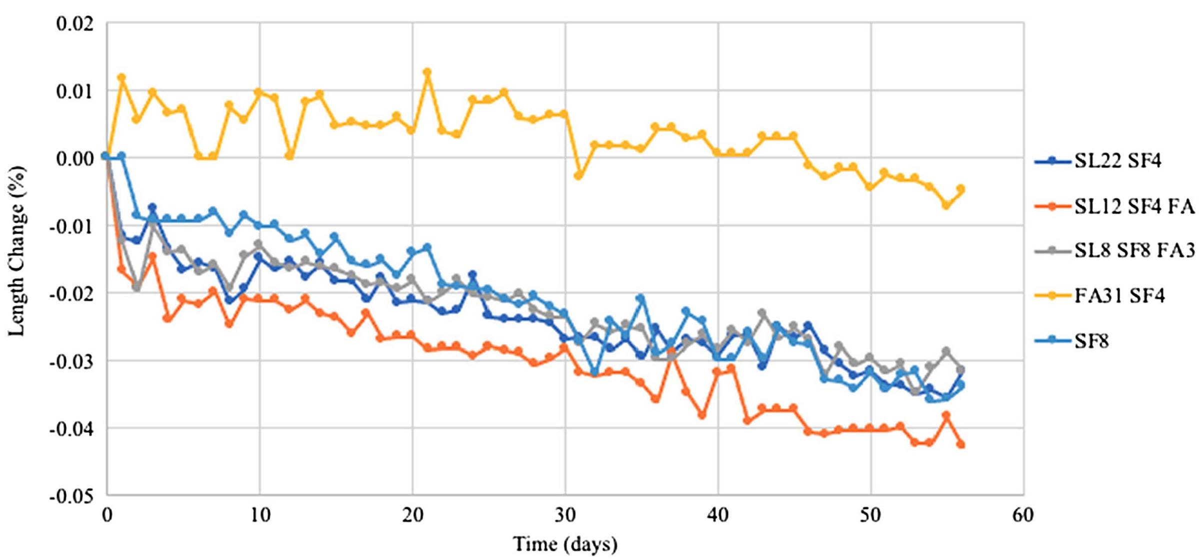


Fig. 5. Time (days) versus length change (%).

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comparing an all-cement mix to a mix containing 20% class F fly ash and 8% silica fume, the fly ash mix had higher drying shrinkage and lower autogenous shrinkage than the control. The drying shrinkage results presented here are similar when comparing the control and fly ash mixes.

Abrasion Resistance

As shown in Fig. 6, as SCM content increased, mass loss decreased. In particular, the SF4 mix (4% SCMs) had the highest mass loss, while the SL22 SF8 and FA31 SF4 mixes (30% and 35% SCMs, respectively) had the lowest mass loss. Each mix's mass loss can also be partially attributed to the higher packing density in mixes containing SCMs as well as the late-age strength-contributing pozzolanic reactions between the silica in the SCMs and the available calcium hydroxide. 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. However, from the data presented here, the fly ash mix actually had the lowest mass loss. The data presented here do align with the findings in Atiş (2002), in which cement content was replaced with 50% and 70% fly ash and abrasion resistance was measured in samples aged 3 days to 3 months; it was found that the fly ash mixes had improved abrasion resistance over the all-cement mixes. Regarding the effects of slag, Fernandez and Malhotra (1990) measured the wear depth at 120 days of binary mixes containing up to 50% slag replacement and found that the addition of slag reduced abrasion resistance; this does not align with this study's findings. One challenge would be the minimal amount of mass loss compared to the size of the sample. Each sample weighed 18–19 kg (40–43 lb) and lost only 0.5 g (0.001 lb) after each application of the drill press.

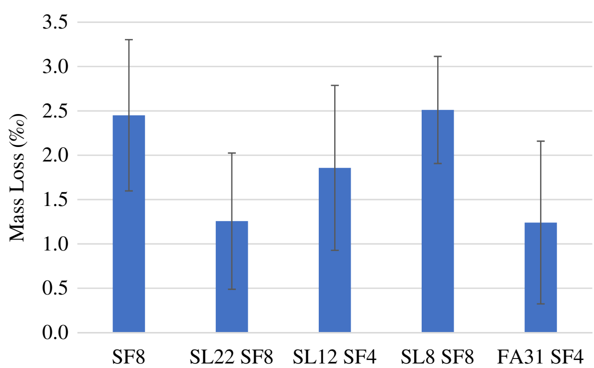

Fig. 6. Mass loss of mixes due to abrasion testing.

Scaling and Deicing Resistances

For scaling resistance after exposure to deicing chemicals, all the mixes performed poorly, with visual ratings of 4 to 5 after 50 days of exposure to a $CaCl_2$ solution and daily freeze-thaw cycles (Fig. 7). The SF8 and SL12 SF4 FA1 mixes performed the worst, with severe surface scaling and a visual rating of 5 at 50 days. The remaining mixes performed marginally better, with moderate to severe scaling at 50 days with visual ratings of 4. Taylor et al. (2004) tested the scaling resistance of samples containing either all cement, 50% slag, or 25% fly ash. They also compared the effect of different finishing techniques. They found that for samples which were finished soon after the molds were filled, as was done in this study, by 50 days the all-cement samples had an average rating of 5, the 50% slag mixes had a rating of 3, and the 25% fly ash samples had a rating of 0.5. Bouzoubaâ et al. (2008) tested seven mixes, including an all-cement control, binary fly ash and slag mixes, and ternary mixes consisting of silica fume with either slag or fly ash. Similar to this study, after 50 days, all mixes, excluding the all-cement mix, had ratings ranging from 3 to 5. Sidewalks were placed in Canada that were cast from the same mixes studied; it was found that after four winters, all mixes but the ternary fly ash silica fume mix had visual ratings ranging from 0 to 3. The ternary fly ash silica fume mix had a rating exceeding 4. The authors concluded that the ASTM C672 method may be too severe, because the same mixes that generally performed poorly during the ASTM C672 tests performed well in the field. Therefore, although our results found visual scaling ratings of 4–5 at 50 days, this does not necessarily mean that they will perform poorly in the field.

Chloride Ion Penetration

All mixes had chloride ion permeability ratings of low (<2,000 coulombs) or very low (<1,000 coulombs), with the SL12 SF4 FA1 mix having the highest charge passed and the fly ash mix having the lowest (Fig. 8). Because a low chloride ion penetration is indicative of low porosity, which, in turn, is related to improved durability, all the mixes would likely have good durability performance in the field.

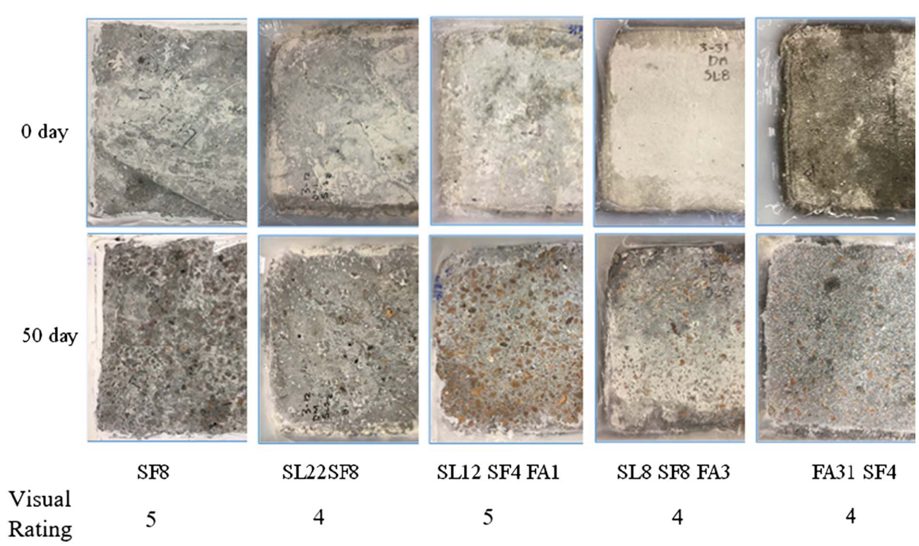

Fig. 7. Deicer samples before and after 50 cycles.

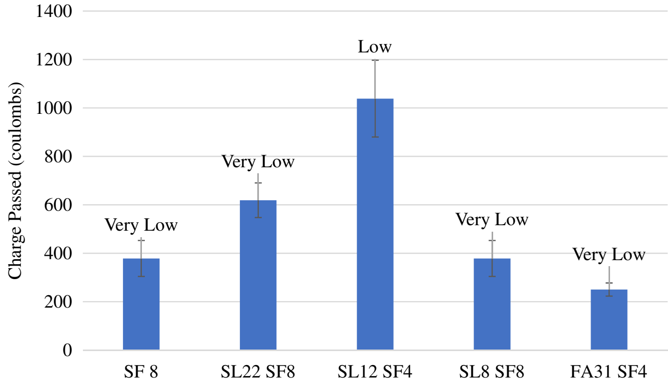

Fig. 8. Chloride permeability results.

Table 6. Freeze-thaw results

Mix	Durability factor	RDME (%) after 184 cycles
SF8	99 ± 3	99 ± 3
SL22 SF8	25 ± 36	97 ± 4
SL12 SF4 FA1	31 ± 43	45 ± 24
SL8 SF8 FA3	70 ± 7	71 ± 7
FA31 SF4	74 ± 11	75 ± 10

Freeze-Thaw Resistance

As shown in Table 6, which presents the DF results of the studied mixes, the SF8 mix performed the best in terms of F-T resistance with a DF of 98.9%, while the SL12 SF4 FA1 and SL22 SF8 mixes performed the worst with DFs of 25.1% and 30.7%, respectively. Note the huge variances in standard deviation. Data were collected from two samples for each mixture, and the RDME of samples could drop from 100 to 0 between the 18 cycles in which readings took place. Therefore, this large variance (particularly seen in the SL22 SF8 and SL12 SF4 FA1 mixes) was due to one sample dropping to almost 0 while the other maintained a reading of 100. It is also important to keep in mind that F-T laboratory cycles are more extreme than what normally occurs in the field, and samples that perform poorly in the laboratory may not always perform poorly in the field (Mehta 1991).

Preliminary Cost Analysis

A preliminary construction cost analysis for a hypothetical 1.6-km (1-mi.) long two-lane high-traffic stretch of highway in Anchorage, Alaska, was conducted to compare the different concrete mixes proposed. This analysis was conducted to further evaluate the economic efficiency of the selected mix designs. Based on material costs from Alaska Basic Industries in Anchorage from June 2019 (X. Schlee, personal communication, 2019), the following raw material costs were assumed: silica fume \$30/11 kg (25 lb); fly ash \$295/t; cement \$165/t; and slag \$250/t. These costs depend on availability and the market; for large-scale pavement construction, these costs would likely be reduced due to purchasing materials in bulk. Using construction cost data obtained from the RS Means heavy construction costs book (RSMeans 2019), the remaining construction costs were calculated. All costs were based on an assumed 2-lane 7.3-m (24-ft) wide pavement with a 0.61-m (24-in.) thick subbase. Communications with ADOT&PF professionals found that a high-traffic [approximately 40,000 annual average daily traffic (AADT)] pavement in central Alaska would generally have a 0.46–0.91 m (18–36 in.) deep subbase, depending on whether permafrost was present. Therefore a 0.61-m (24-in.) thick subbase was assumed. The concrete layer was assumed to be 15 cm (6 in.) thick with 176 kg/m^2 (18 lb/yd^2) of reinforcing steel. In addition, transverse joint dowels were assumed to be spaced at 0.3 m (1 ft) with contraction joints spaced at 3.7 m (12 ft). All RS Means costs were increased by the Anchorage, Alaska, rate of 115.8% the national average. In addition, following WSDOT example calculations (WSDOT 2018), cost increases of 5%, 15%, and 10% were added to represent mobilization, engineering, and contingencies costs, respectively (Table 7). The combined costs sourced from Alaska prices of cementitious materials and RS Means cost estimations resulted in the assumed costs per two-lane 1.6-km (1-mi) stretch of pavement for each mix design (Table 8). These values were calculated using the assumed quantities and costs summarized in Table 7, but the SF8 cementitious value was changed to represent each mix's respective cementitious materials costs, which are also summarized in Table 8 as the cost per 18-cm-thick square meter (6-in.-thick square yard) of pavement. Based on the results, the SL12 SF4 FA1 mix proved to be the most inexpensive design at around \$1.6 million. However, the cost between the five options varied only by about 2%, with a standard deviation of \$30,000. With such a minimal difference between the construction costs of using any of the mix designs, any of them would likely be a good choice.

Because there are only a few concrete roads built and maintained by ADOT&PF, it is challenging to estimate and verify these costs using historical data. The cost of paving varies widely

Table 7. Assumed construction cost for two-lane rigid pavement using the control SL8 mix

Item	Unit	Cost/unit	Quantity/two-lane 7.3-m-wide, 1.6-km-long road (24 ft by 1 mi)	Total (\$1,000)
Noncementitious materials	18 cm pavement/m^2 (6 in pavement/yd^2)	15.09	14,080	212
SF8 cementitious materials[a]	18 cm pavement/m^2 (6 in pavement/yd^2)	18.31	14,080	258
Placement labor and equipment	18 cm pavement/m^2 (6 in pavement/yd^2)	4.63	14,080	65
176 kg/m^2 (18 lb/yd^2) reinforcing steel	0.84 m^2 (yd^2)	15.86	14,080	223
Transverse joint dowels every 3.7 m (12 ft)	Each	13.32	10,560	141
Transverse contraction joints every 3.7 m (12 ft)	0.30 m (l.f.)	5.96	10,560	63
0.61-m (24-in.) deep subbase course	0.84 m^2 (yd^2)	31.27	14,080	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

Note: l.f. = linear foot.
[a]Cost/unit varies depending on mix. Cost is adjusted for Anchorage, Alaska, prices from RS Means national average.

Table 8. Estimated cost of each alternative

Alternative (No.)	Cementitious materials cost $/18-cm-thick/m^2 ($/6-in.-thick/yd^2)	$/0.76 m^3 ($/yd^3)	Total cost($)/ 2-lane 1.6-km ($/2-lane 1-mi) road
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

depending on 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 1.6-km (1-mi) road with concrete to cost $770,000. Another estimate by the Arkansas Department of Transportation (ArDOT 2016) estimated the total costs for a 1.6-km (1-mi) long concrete lane in Arkansas to be $1.1 million, or around $2.2 million per 2-lane 1.6-km (1-mi) road. 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.

Conclusions and Recommendations

The objective of this study was to identify and develop mix designs of concrete made with SCMs that would provide excellent abrasion resistance and durability. Preliminary screening tests of ternary mixes containing silica fume with either slag or class F fly ash were conducted. Workability, compressive strength, and flexural strength tests were conducted to select a list of mix designs with SCMs to meet strength and workability requirements. The selected mixes as well as the control binary mix were then subjected to further performances tests to identify the optimum concrete mix design that would provide excellent abrasion resistance and durability. These tests focused primarily on the SCM contribution to short-term durability. It is important to keep in mind that a long durability study may find increased durability, because the SCMs continue contributing strength after 28 days. Based on the results, the following conclusions can be drawn:

- The mixes selected from the screening tests included an 8% silica fume control mix (SF8), a 22% slag with 8% silica fume mix (SL22 SF8), a 12% slag with 4% silica fume and 1% fly ash mix (SL12 SF4 FA1), an 8% slag with 8% silica fume and 3% fly ash mix (SL8 SF8 FA3) and a mix containing 31% fly ash with 4% silica fume (FA31 SF4).
- Regarding compressive strength and shrinkage, the SF8 mix had the highest 28-day compressive strength, while the FA31 SF4 mix had the lowest drying shrinkage at 0.01% expansion. However, all mixes had 28-day compressive strength greater than 43,000 kPa (6,200 psi), which fulfilled the minimum strength requirement of 43,000 kPa (6,200 psi) to be considered high-strength concrete (ADOT&PF 2020).
- The FA31 SF4 mix had the highest abrasion resistance. Regarding mass loss, an average of only 1 g 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, likely indicative of a high abrasion resistance to studded tires.
- Concerning durability, the SL22 SF8, SL8 SF8 FA3, and FA31 SF4 mixes had similar 50-day visual ratings of 4, equivalent to moderate to severe scaling, when measuring their deicer salt scaling resistances. Although these ratings indicated that the samples performed poorly, this may not be indicative of field performance. After testing chloride ion penetration, all mixes but SL12 SF4 FA1 had very low ratings of less than 1,000 coulombs. The SL12 SF4 FA1 mix had a low rating of 1,038 coulombs. The FA31 SF4 mix had the lowest rating of 250 coulombs. Therefore, all the mixes likely have low permeability and, subsequently, high durability. For F-T resistance after 180 cycles, the SF8 mix performed the best with a durability factor of 99%, while the SL22 SF8 and SL12 SF4 FA1 mixes performed the worst with durability factors of 25% and 31%, respectively. The other two mixes had factors of 70% and 74%.
- 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 mix would have the lowest construction cost of $1.6 million per 2-lane highway. The variance in cost, however, was minimal, with the construction costs of the five mixes ranging from $1.6 to $1.7 million.
- In terms of the properties evaluated in this study (i.e., strength, shrinkage, chloride ion penetration, freeze-thaw resistance, deicer scaling resistance, and abrasion resistance), the five mixes, including the four optimal mixes and the control, all provided good performance, but a quaternary mix design containing primarily silica fume and slag (i.e., SL12 SF4 FA1), appeared to provide the overall best performance considering strength, durability, abrasion resistance, and cost.

The next recommended step in this research would be to construct and monitor test sections in the field using the optimal mixes determined to verify and validate the 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. In addition, the focus of this study was on the characterization and comparison of the mechanical properties and durability of concrete mixes. A better understanding of underlying mechanisms, including pozzolanic reactions and the compatibility of SCMs and their effects on the structure and performance of concrete at micro- and meso-levels, would be greatly beneficial.

Data Availability Statement

All data used during the study appear in the published article.

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