

**ALTERNATIVE HIGH EARLY
STRENGTH CONCRETE (HESC)
STRUCTURAL OVERLAYS**

Final Report

SPR 847



Oregon Department of Transportation

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16. Abstract: The objective of this research project was to identify the obstacles or impediments to successfully and reliably using HESC for structural bridge deck overlays on Oregon bridges and to develop the standards, specifications, processes, and practices necessary to using them on ODOT bridges. Rapid strength gain is a desired property for fast and efficient bridge deck overlay reconstruction. Three calcium sulfoaluminate (CSA) cement systems, an ASTM C 150 Type III ordinary portland cement (OPC), and a CSA-OPC blended system were evaluated in this research project for potential use as high early strength structural overlay materials for Oregon Department of Transportation (ODOT) concrete bridge decks. A high-performance concrete (HPC) mixture that is representative of ODOT bridge deck mixture designs from the early 2000s to present was also analyzed in this project. CSA cement systems performed satisfactorily when subjected to compressive strength tests, workability tests, curing tests, bond strength tests, early age cracking tests, freeze-thaw cracking tests, and thermal expansion tests when compared with HPC. Resistivity and formation factors of CSA concretes were measured and compared poorly to HPC resistivity and formation factors. While these values would normally suggest rapid rates of corrosion, the collected data is insufficient to conclusively indicate high corrosion rates since these tests were developed for OPC systems and may not be applicable to CSA systems. Additional work to elucidate the role of curing and conditioning of CSA systems is merited before definitive conclusions can be drawn regarding chloride-based corrosion in these systems.			
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SI* (Modern Metric) Conversion Factors

Approximate Conversions to SI Units

Physical Quantity	Symbol	When You Know	Multiply By	To Find	Symbol
Length	n	inches	25.4	millimeters	mm
Length	ft	feet	0.305	meters	m
Length	yd	yards	0.914	meters	m
Length	mi	miles	1.61	kilometers	km
Area	in ²	square inches	645.2	square millimeters	mm ²
Area	ft ²	square feet	0.093	square meters	m ²
Area	yd ²	square yard	0.836	square meters	m ²
Area	ac	acres	0.405	hectares	ha
Area	mi ²	square miles	2.59	square kilometers	km ²
Volume	fl oz	fluid ounces	29.57	milliliters	mL
Volume	gal	gallons	3.785	liters **	L
Volume	ft ³	cubic feet	0.028	cubic meters	m ³
Volume	yd ³	cubic yards	0.765	cubic meters	m ³
Mass	oz	ounces	28.35	grams	g
Mass	lb	pounds	0.454	kilograms	kg
Mass	T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
Temperature (exact degrees)	oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	oC
Illumination	fc	foot-candles	10.76	lux	lx
Illumination	fl	foot-Lamberts	3.426	candela/m ²	cd/m ²
Force and Pressure or Stress	lbf	poundforce	4.45	newtons	N
Force and Pressure or Stress	lbf/in ²	poundforce per square inch	6.89	kilopascals	kPa

*SI is the symbol for the International System of Measurement

** Volumes greater than 1000 L shall be shown in m³

SI* (Modern Metric) Conversion Factors
Approximate Conversions from SI Units

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

For More Information see: <https://www.fhwa.dot.gov/publications/convtabl.cfm>

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1.0 INTRODUCTION

ODOT's bridge inventory is continuing to age, and with that, ODOT needs to be proactive in its approach to bridge maintenance. With increasing traffic volumes and public/political involvement, bridge projects have been put into tighter time constraints while also minimizing disruption to traffic. ODOT cannot afford to construct diversion structures for every structural overlay project. Also, traffic volumes have gotten to the point where long-term closures or lane restrictions are nearly impossible.

As bridge decks continue to wear, there will be a need to replace the existing structural overlays. There is also a need to strengthen some bridges due to higher demands, including increasing deck thickness through a structural overlay. Considering the scheduling constraints, a high early strength overlay that can be performed in a single night could help solve this issue. Polyester polymer concrete (PPC) overlays are one option, but these do not address decks in poor condition. Mobility and safety issues have caused ODOT to reduce the time a bridge deck is not usable due to construction, leaving less time for traditional materials to be used. This has led to instances of using inappropriate materials instead of the preferred materials only due to rapid traffic turnaround.

When structural overlays wear out after 30-years, ODOT would like to replace them with another structural overlay to maintain structural capacity. Designers have been forced to place expensive PPC overlays over worn-out structural overlays simply because the PPC sets rapidly, allowing a fast traffic turnaround. High Early Strength Concrete (HESC) Overlays have the potential to be placed in a single overnight lane or bridge closure. Curing for HESC may require different curing techniques to minimize curing time while maintaining integrity of the overlay (Weiss, Zavattieri et al. 2019).

1.1 SIGNIFICANCE AND BACKGROUND

ODOT has implemented a number of HESC overlays up to the writing of this report. These were recent projects, and performance has not been thoroughly tracked. Construction methods were not tracked to the level necessary to create a standard specification. It is anticipated that change orders for HESC will continue in the coming years, providing additional full-scale overlays to observe, monitor, and document.

In July of 2020, a HESC structural overlay was constructed through the Maintenance Division that effectively minimized traffic disruptions and used a wet cure time of 3 hours. The two-lane bridge near milepost 47.2 on OR 18 (South Yamhill Crossing) was overlaid in July of 2020 using a calcium-sulfoaluminate-based cement (CTS Rapid Set Low-P) HESC. The eastbound and westbound lanes were successfully overlaid in two successive weekends (one lane each weekend). Overnight hydro demolition occurred on the first night (Friday), and the overlay was placed the second night (Saturday). In both cases, travel was opened to the public by 1-2 pm the next day (Sunday). At the time of writing, neither lane showed visible signs of cracking. While

this is only an example of one type of material, it offers the potential time-savings and possibilities with HESC. It thus further supports the benefits of HESC structural overlays.

Many other states have used HESC overlays in recent years with varying degrees of success. There are known issues with HESC overlays. The rapid hydration of specific cementitious systems can cause excessive shrinkage cracking, affecting the new overlay's service life (Mehta and Monteiro 2014). However, specific cementitious systems, especially those based on early-age ettringite formation, can provide early-age expansion characteristics that reduce shrinkage-related cracking risk (Ideker, Scrivener et al. 2019). Shrinkage reducing admixtures (SRAs), fiber reinforced concrete (FRC), internal curing (IC) with saturated fine lightweight aggregates (FLWA), and expansive cementitious systems may also be beneficial in this regard (Ideker, Deboodt et al. 2013, Ideker and Bañuelos 2014). Mixture designs and curing methods need to be evaluated to quantify the structural overlay's expected performance and increased service life. In this research, it will also be important to consider:

- materials availability and costs in Oregon,
- the impact of these materials on life cycle costs,
- the climatic conditions that can vary widely across the state,
- the need for minimal traffic interruptions and the target longevity of the overlay, which is targeted to be 25-30 years.

The final product of the research should be a standard specification that incorporates materials, mixture proportions, and construction processes to maximize the benefits of the high early strength materials while also minimizing cracking and therefore increasing service life of the overlay.

1.2 OBJECTIVE

The objective of this research project is to identify the obstacles or impediments to successfully and reliably using HESC for structural bridge deck overlays on Oregon bridges and to develop the standards, specifications, processes, and practices necessary to using them on ODOT bridges. To meet this objective the first task was to complete a literature review that documents HESC materials with respect to bridge overlays. After that, a research methodology was developed to select and evaluate the performance for high early strength material for structural bridge overlays. The following parameters were given to consideration while evaluating the performance of the selected high early strength materials- compressive strength gain, curing efficiency, activation energy, CTE (coefficient of thermal expansion), bond, freeze-thaw, resistivity, critical chloride threshold, volume change, and/or cracking risk.

2.0 LITERATURE REVIEW

The rapid strength gain of HESC materials is largely attributed to the cementitious components. The first section of this review places emphasis on understanding the chemistry associated with the rapid strength gain and durability of different cementitious systems. The mixture proportioning of aggregates and fibers in addition to the cementitious materials is discussed in section 2.2, along with the use of admixtures that can help control the workability and setting time of HESC for example.

Considerations for preparation and placing HESC overlays, like subsurface preparation to improve bond strength, are outlined in the section 2.3. Fittingly, background on the methods and importance of curing is included in the section following application practices (2.4). Finally, the findings of relevant studies from other state highway agencies on HESC overlays or rapid pavement repair materials is covered in section 2.5.

2.1 CEMENTITIOUS MATERIALS

This section covers the main types of cementitious materials that are used for HESC applications. It is important to note that often materials are blended (multi-component) systems to optimize the advantages of diverse types of cementitious systems. For example, very early-age strength (0-24 hours) may be provided by the calcium sulfoaluminate cement portion of a blended system with ordinary Portland cement where the ordinary Portland cement provides later age (> 24 hours) strength gain. In this literature review, the most prevalent types of these blended systems are highlighted. It should be noted that as the materials are selected and investigated for this project additional updates to the literature will be needed to reflect properties about those materials if they are not already included herein.

Further, many systems on the market are proprietary blends and only part, or in fact very little about the material may be known ahead of time. The researchers will also update the literature review and research results as more information is gathered about these materials. In Section 2.1.1 ordinary Portland cement (OPC) is covered first. This is done primarily as a reference to a system that is more familiar to the broader audience. OPC is often modified or a component of some early HESC systems to achieve desired rapid strength and durability. Each cementitious type is covered in a separate section that highlights chemical composition, hydration assemblages and performance with a focus on durability issues including corrosion, carbonation, volume change and freeze-thaw. Blended systems for a given cement type are also covered within that respective cementitious material section.

2.1.1 Ordinary Portland Cement (OPC)

In HESC applications, ordinary Portland cement (OPC) has been used because of its prevalence and cost-effectiveness over alternative cements. While traditionally thought of as a longer-term (e.g., 28 days) strength gaining binder, OPC can achieve high early strengths and providing predictable performance given proper binder and mixture proportion modifications (Mehta &

Monteiro, 2006). It is also a component in blended systems with other types of cements that will be discussed in subsequent sections. Although neat OPC may not be the ideal HESC material, it provides useful context for the durability and hydration mechanism of other cement systems to be discussed.

The primary phases in portland cement clinker are alite (C_3S) and belite (C_2S) with lesser proportions of aluminate (C_3A) and ferrite (C_4AF) phases. Small amounts of gypsum ($C\$,$) are interground with the clinker as a lubricant during grinding and to prevent flash setting upon contact with water. The percentage of phases specified for each type of portland cement outlined in ASTM C150 is summarized in Table 2-1 alongside the approximate phase composition of other cement types discussed in this review.

Table 2-1: Phase compositions (% wt.) of different cement types (Kosmatka & Wilson, 2016; Scrivener & Capmas, 1998; Thomas et al., 2018). Note that some minor phases of CSA and CAC may be assimilated with phase comprised of similar elements for brevity.

Phase	ASTM C150 Types I	ASTM C150 Types II	ASTM C150 Types III	ASTM C150 Types IV	ASTM C150 Types V	Calcium Sulfoaluminate Cement (CSA)	Calcium Aluminate Cement (CAC)
C_3S , Alite	55	50	60	30	35	-	-
C_2S , Belite	20	25	20	50	45	0 to 20	<10
C_3A , Aluminate	10	<8	10	5	5	-	-
C_4AF , Ferrite	7	10	7	12	10	5 to 15	10 to 40
$C_4A_3\$,$ Ye'elimite	-	-	-	-	-	50 to 60	-
$C\$H_2$, Gypsum or $C\$$, Anhydrite	2 to 3	2 to 3	2 to 3	2 to 3	2 to 3	15 to 25	-
CA, Monocalcium Aluminate	-	-	-	-	-	-	40 to 50

In comparison to ASTM C150 Type I portland cement, ASTM C150 Type III portland cement is usually selected for high early strengths and optimized to have:

- Increased alite (C_3S) content (46-71%), which reacts faster than other clinker compounds to form calcium silicate hydrates (C-S-H) and calcium hydroxide (CH).
- Increased Blaine fineness (390 –644 m²/kg), which promotes the reaction rate by providing a greater surface area.

The increased reaction rate in Type III cements is accompanied with a higher heat of hydration. This can be beneficial in cold/humid weather environments, but in warm/dry environments it could lead to greater evaporation rates and a risk of early-age cracking (Kosmatka & Wilson, 2016). DOTs that recommend the use of Type III cement for HESC include New York, Texas, Arkansas and Nebraska (Ghafoori et al., 2017).

It should be noted that “early strengths” in OPC are typically achieved in 6-8 hours, rather than under 4 hours, according to a review of numerous studies from highway agencies (Ghafoori et al., 2017). However, in some cases compressive strengths of 2,000 psi and flexural strengths of 300-450 psi have been achieved in 4 hours using OPC and several states continue to prefer even using Type I or I/II cements for high early strength applications (MD, MI, NJ, OH, IA and IL) (Ghafoori et al., 2017). Type I or I/II cements can be manipulated to achieve high early strengths using accelerating admixtures, higher cement content, lower water-to-cement ratio, heated mix water etc. These methods can also be used with Type III cements. Expediting strength gain is a challenge with any portland cement, portland limestone cement (PLC), and blended systems and the hydration mechanism behind the slow reaction behavior is discussed in the following section.

2.1.1.1 Hydration and Strength gain

Hydration refers to the chemical reaction(s) between anhydrous cement phases and water. Figure 2-1 shows a basic diagram of portland cement hydration. The first step in the hydration of portland cement is the initial dissolution of cement. The reaction of C_3S ¹ and C_2S stalls for several hours after this while C_3A reacts to form ettringite ($C_6A_3H_{32}$, Aft) in solution (Jennings & Tennis, 1994).

The induction period allows a generous window of time for working and placing concrete. It ends when the hydration C_3S resumes, and calcium silicate hydrate (C-S-H) and calcium hydroxide (CH) begin to precipitate in larger quantities. With the existing ettringite in the matrix, the growth of C-S-H and CH products causes initial set (Kosmatka & Wilson, 2016). The continued hydration of enables final set when enough C-S-H forms to provide strength and rigidity.

Following final set, the formation of ettringite ceases as C_3A is depleted. The remaining C_3A can hydrate by reacting with ettringite to form monosulfoaluminate ($C_4A_3H_{12}$, aluminoferrite-mono, AFm). Meanwhile, the hydration of C_3S slows down significantly as it becomes diffusion controlled with a dense coating of C-S-H around particles. C_2S is less reactive than C_3S and its hydration becomes more prominent at later stages of hydration and curing. However, the reaction C_2S is also diffusion controlled since it primarily forms C-S-H (Jennings & Tennis, 1994). The diffusion-controlled nature of C_3S and C_2S hydration is a key limitation in the potential for OPC to gain strength rapidly.

Supplementary cementitious materials (SCMs) rich in silica can facilitate further production of C-S-H through the pozzolanic reaction of CH. Fine particles of SCMs such as silica fume can promote C-S-H growth throughout the microstructure of portland

¹ Note that cement chemistry notation: C: CaO, S: SiO₂, F: Fe₂O₃, A: Al₂O₃, \$: SO₃, K: K₂O, N: Na₂O, M: MgO, H: H₂O

cement and yield lower porosities. The pozzolanic reaction can be quite slow, however, finer and more reactive SCMs such as silica fume and metakaolin may be beneficial for high early strength (Mehta & Monteiro, 2006).

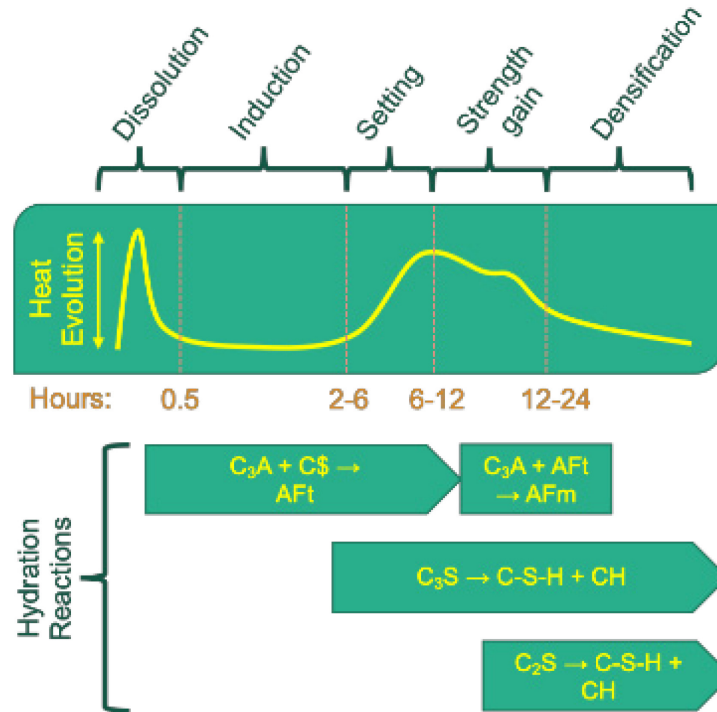


Figure 2-1 Diagram of OPC hydration process. Reactions are not stoichiometric and do not include the necessary water. Aluminate and ferrite reactions are also omitted for brevity

C-S-H makes up at least 50% of the mass in hardened portland cement and is mainly observed as a dense coating around cement grains in the microstructure (Kosmatka & Wilson, 2016). By using a lower water to cement ratio, the particles pack more densely and form a stronger microstructure as C-S-H and other hydrates form between them. The denser packing of cement grains also means that less hydration products are required to gain strength. Because of this, state DOTs typically use a relatively low w/c ratio between 0.33 and 0.43 for high early strength applications (Ghafoori et al., 2017).

2.1.1.2 Durability

The long-term durability concerns of OPC are often linked to the microstructure and chemistry resulting from the hydration process, and this is true for alternative cement systems as well. The most common and relevant concerns for bridge overlays are introduced here since the historical use of OPC provides useful background on the damage mechanisms.

Corrosion of Steel Reinforcement

Corrosion of reinforcing steel is widely regarded as the most prolific durability issue in concrete, accounting for upwards of \$100 billion USD worldwide in repairs each year (F. Li et al., 2011).

Steel rebar in portland cement is protected by an electrochemically passive layer due to the high pH of the pore solution. The cover depth of the steel helps to maintain this passive layer by: (1) withholding a decrease in pH due to carbonation and (2) preventing the ingress of chloride ions, which can lead to localized corrosion if they surpass a threshold concentration at the steel interface. If reinforcing steel is allowed to form rust within the concrete, expansive forces from the increased volume of iron oxide product will cause cracking and spalling. (F. Li et al., 2011)

The time to corrosion for steel in OPC can be extended by reducing the interconnected porosity in the microstructure (i.e. transport properties). This can be done by using a low w/c ratio, maximizing C-S-H formation from CH through pozzolanic reactions (e.g. including SCMs), or optimizing the aggregate gradation and interface. By increasing the diffusion coefficient, the propagation time for chlorides to reach reinforcing steel is increased. During propagation of chlorides, the binding of chlorides in the cement paste and formation of Friedel's salt can also increase the time to corrosion (Qiao et al., 2018).

The critical chloride threshold is another parameter in the time to corrosion. Figure 2-2 shows how a higher threshold can increase the service life of reinforced concrete. The threshold can be influenced by the type of steel, the pore solution chemistry and bond characteristics of the cement-steel interface. There are accelerated test methods for determining the threshold for different cement and rebar materials (Trejo & Pillai, 2003).

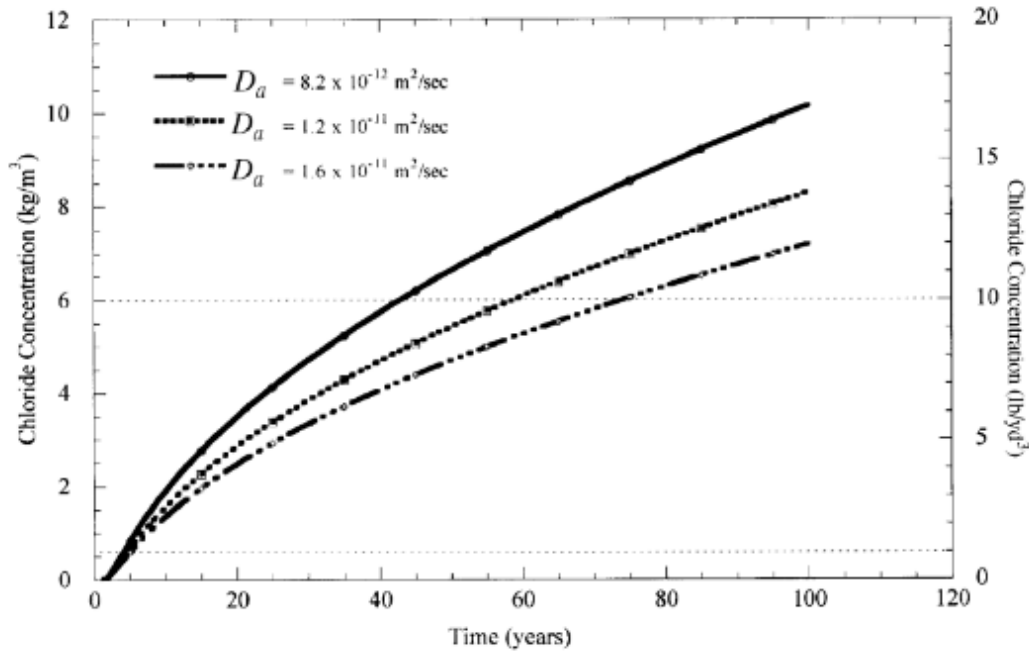


Figure 2-2 Critical chloride threshold versus time to corrosion in uncracked concrete (Trejo & Pillai, 2003).

With respect to preventing corrosion, it is also important to avoid any early age shrinkage cracking that would provide a “shortcut” for chlorides or carbonation to reach the steel. The mechanism of shrinkage will be discussed further in a following section.

Carbonation

Carbon dioxide from either the atmosphere or carbonated water will react with hydration products, mainly calcium hydroxide, to form carbonates such as CaCO_3 . While this does not negatively impact the strength directly, it lowers the pH of the pore solution. If the carbonation front reaches reinforcing steel, and the pH drops below a critical value, the passivation will be compromised, and corrosion can ensue (Kosmatka & Wilson, 2016).

The rate of carbonation can be reduced by maintaining a high relative humidity in the concrete since the solubility of CO_2 is highest at intermediate humidity (e.g. 50%). Decreasing the amount of CH hydration products can reduce the total amount of carbonation, but it also removes CH^2 from acting as a buffer in the carbonation front (Kosmatka & Wilson, 2016). The CH content is also relevant to the carbonation of OPC in blends with other cement types.

Volume Change

At early age, OPC is prone to several types of volume change including:

² In cement chemistry notation: CH = $\text{Ca}(\text{OH})_2$; C-S-H = $(\text{CaO})_3(\text{SiO}_2)_2(\text{H}_2\text{O})_8$; AFt or $\text{C}_6\text{A}\$3\text{H}_{32} = \text{Ca}_6\text{Al}_2(\text{SO}_4)_3(\text{OH})_{12}(\text{H}_2\text{O})_{26}$; and AFm or $\text{C}_4\text{A}\$H_{12} = (\text{CaO})_4(\text{Al}_2\text{O}_3)(\text{SO}_4)(\text{H}_2\text{O})_{12}$

- Autogenous deformation – when hydration products have a differing volume than the sum of their parts (water and clinker), overall expansion or shrinkage can occur in the system. This type of expansion is also referred to as “swelling.” However, inadequate curing often leads to negative deformation or autogenous shrinkage.
- Autogenous shrinkage – the volume reduction due to the chemical binding (i.e. consumption) of water by anhydrous phases to form hydration products.
- Plastic shrinkage – the combined effects of autogenous shrinkage and evaporation leading to drying **before** the concrete has set. This often leads to wide cracking resembling “rips” or “tears” in the concrete.
- Drying shrinkage – any decrease in length of a hardened concrete specimen that is not caused by external forces. The hydration of cement, evaporation rate, temperature, and relative humidity can influence the extent of drying shrinkage observed in concrete (ASTM C596).

Because the partially hydrated microstructure in the cement paste is unable to resist the stress generated by these volume changes, cracking can easily result from early age shrinkage mechanisms. The use of lower w/c can increase the potential for autogenous shrinkage as self desiccation in pores $\sim < 50\text{nm}$ leads to higher capillary stress that can collapse those pores.

Over 100,000 bridge decks in the U.S. exhibited transverse cracking, which is a known symptom of drying shrinkage (Krauss & Rogalla, 1996). Curing methods are an effective way to mitigate drying shrinkage and will be detailed in section 2.4.

Shrinkage attributed to drying or evaporation is also applicable to alternative cement systems to be discussed. The previously recommended free shrinkage limit for ODOT HPC bridge deck mixtures of 450 micro strain at 28 days of drying (Ideker et al., 2013).

Freeze-Thaw

The freezing and thawing of water in concrete can lead to deterioration through a complex set of mechanisms. Damage due to hydraulic pressures can occur in both aggregates and cement paste, because as water freezes in pores that are too small for ice crystals to form it will migrate towards larger pores or voids. This explanation provides a basic understanding of why entrained air voids can help mitigate freeze-thaw damage as they alleviate the buildup of hydraulic pressures that can cause cracking. Freeze-thaw damage is often observed as spalling on the concrete surface, particularly at exposed edges or corners or as aggregate “pop-outs.”

An alternative approach to quantifying freeze-thaw damage is gaining popularity. This approach, called the critical saturation method, states that freeze-thaw damage can only occur when the saturation level in concrete exceeds a critical saturation level (i.e., the sample absorbs water). The critical level of saturation has been measured to be between 78%-91% (AASHTO, 2017; Fagerlund, 1977; W. Li et al., 2012; Todak et al., 2015).

Recent work has suggested that the critical saturation can vary based on the air void spacing (W. Li et al., 2012).

Other factors and mechanisms influencing freeze-thaw damage include the role of alkali concentrations in the cement paste that can cause osmotic pressures (Kosmatka & Wilson, 2016).

De-icing Salts

Although they allow for safe winter driving, the addition of de-icing salts can enable other damage mechanisms to occur in freezing environments. Brine salt concentrations near the surface can create osmotic pressures that lead to salt scaling. Salt crystallization can also cause damage (Kosmatka & Wilson, 2016). Severe damage can occur when CaCl_2 reacts with CH to produce calcium oxychloride, which has a volume 303% greater than CH (C. Qiao et al., 2017).

Alkali-Silica Reaction (ASR)

Aggregates containing reactive silicates (i.e. non-crystalline) phases are susceptible to dissolution in the high pH pore solution of OPC. The silica can then form an expansive reaction product that may cause widespread cracking throughout the concrete (Kosmatka & Wilson, 2016). Methods for preventing ASR are well established in ASTM C1778 and AASHTO R80. Due to the high cementitious material content often used in HESC mixture designs (Ghafoori et al., 2017), there could be potential for ASR if a significant portion of OPC (e.g. > ~50% by mass) is used with reactive aggregates.

2.1.1.3 OPC Summary

The implications for OPC on the potential use in HESC are summarized in Table 2-2 The primary challenge highlighted is the likely inability of OPC systems to achieve adequate strength in less than 6 hours.

Table 2-2: Potential benefits and challenges associated with using portland cements for HESC bridge overlays.

Benefits	Challenges
Material availability	Achieving early strength <6hrs
Material cost	Proper curing within time constraints
Equipment availability	Cracking risk
Use of existing test methods	
Predictable performance	

2.1.2 Calcium sulfoaluminate cement (CSA)

Calcium sulfoaluminate cement (CSA) is a preferred option for HESC because of its ability to set within 15 min and easily surpass a compressive strength of 2,500 psi in under 3 hours. Its variations can be used alone or in conjunction with OPC and it is often an ingredient in proprietary rapid set mixtures (Juenger et al., 2011).

CSA is comprised of finer particles than OPC (Blaine fineness > 3000 m²/kg) with a chemistry that promotes rapid and significant ettringite formation. The clinker phases of a typical CSA in order of reactivity are:

1. mayenite ($12\text{CaO} \cdot 7\text{Al}_2\text{O}_3$ or C_{12}A_7)
2. ye'elimite ($\text{C}_4\text{A}_3\hat{\text{S}}$)
3. tetra calcium aluminoferrite (C_4AF)
4. belite (C_2S)

Calcium sulfate (C\$) is another compound in CSA systems. It is uncalcined, yet important since it facilitates ettringite formation and the rapid strength gain that is characteristic of CSA. Table 2-3 contains a general breakdown of the proportions of these anhydrous phases in CSA in comparison to OPC and CAC. It should be noted that CSA encompasses a variety of different compositions. Table 2-3 shows the different types of CSA proposed by Bescher and Kim. Belitic CSA (BCSA) is typically used as a standalone material and not blended with OPC since the content of belite (C_2S) has already been increased. (Bescher & Kim, 2019)

Table 2-3: Types of CSA proposed by (adapted from Bescher & Kim, 2019)

Types of CSA	Calcium Sulfoaluminate (%)	Dicalcium Silicate (%)	Calcium Sulfate (%)	Other (%)
Type A – Accelerating Additive	35-45	0-20	10-30	5-55
Type B – Belitic Calcium Sulfoaluminate Cement	20-30	40-60	5-25	0-35
Type C – Expansive Additive	10-20	10-30	40-60	0-40
Type K – Shrinkage Compensating Cement	1-10	30-50	1-20	20-70

The main incentive to using CSA is the rapid strength gain, but it is worth noting that the lower calcium oxide content leads to a 30-50% reduction in CO₂ emissions from calcination compared to OPC. Furthermore, the lower temperature needed to form the CSA compounds can reduce the energy demand of manufacturing by up to 60% (Thomas et al., 2018)

In CSA systems, as well as other non-portland systems, it is best practice to use quantitative x-ray diffraction (XRD) to perform quality control on the cement since even minor changes in the phase composition can dramatically influence the performance. The use of x-ray fluorescence (XRF) and bogue calculations do not encompass all the phases that may be present in these complex systems (ben Haha et al., 2019; Füllmann et al., 1999).

However, these measures could further add to the cost of CSA cement which was four times higher than that of type I/II cement in 2017 (Thomas et al., 2018). Other factors to consider in the economics of using CSA include the construction costs (likely reduced), and the life-cycle costs associated with durability (to be discussed). While a survey done in 2017 reported that California is the only state DOT recommending the use of CSA for HESC, another survey found that 9 other states, including Washington, have had success using CSA for repair applications (L. Burris et al., 2015; Ghafoori et al., 2017).

2.1.2.1 Hydration and Strength gain

Mayenite and ye'elimite phases are the first to dissolve when CSA first contacts water. The dissolution of these phases creates a saturation of alumina relative to the quantities of calcium and sulfate in solution. The inclusion of calcium sulfate in CSA is necessary for the alumina ions to be incorporated into ettringite ($C_3A \cdot C\bar{S}_3 \cdot H_{32}$). Ettringite and aluminum hydroxide (AH_3) are the main hydration products of CSA and form rapidly during the first hours of hydration (Telesca et al., 2014).

Winnfield and Lothenbach observed that when the $C\bar{S}$ source, i.e. gypsum, is depleted the formation of ettringite begins to slow. This occurred at around 10 hours for the two commercial CSAs investigated in their study. After the first 1-2 days of reaction, monosulfate, AH_3 and/or stratlingite begin to increase their rate of growth (Winnefeld & Lothenbach, 2009). The production of these minor hydrates is gradual throughout the hydration process, but at later ages it can have a significant role in reducing the overall porosity.

Contrary to other high early strength options, CSA presents the challenge of delaying the initial setting time beyond 1 hour (Thomas et al., 2018). Work by Zajac et al. reports that several retarders are able to do so, and these will be discussed with other admixtures in the section 2.2 (Zajac et al., 2016). Isothermal calorimetry by Burris and Kurtis also found that CSA hydration occurs most rapidly in the first 10 hours even with varying dosages of retarder. Without the use of a retarder, they found that the rate of heat evolution did peak before 1 hour in both CSA systems they investigated (L. E. Burris & Kurtis, 2017).

Roughly 50% of the ultimate strength is achieved within a few hours of set and by 3 days the strength can reach 90% (Thomas et al., 2018). The rapid strength gain is attributed mainly to ettringite growth, and this highlights another key difference from OPC. Because the stoichiometry of ettringite includes 32 molecules of water, the recommended w/c ratio of CSA systems is typically 0.7-0.8 whereas the w/c in high early strength OPC systems is preferred to be below 0.4. Using lower w/c ratios in CSA systems does not improve the strength and is liable to have other detrimental effects like shrinkage.

Ternary Blends (CSA-OPC- $C\bar{S}$)

Combining CSA cement (and $C\bar{S}$) with OPC can allow the properties to be fine-tuned for desired performance. For example, the study by Telesca et al. found that a blend of 40% CSA clinker, 20% natural gypsum and 40% OPC by mass yielded negligible expansion or shrinkage in air curing conditions. The blend also exhibited a finer porosity (Telesca et al., 2014).

The hydration kinetics of OPC:CSA:C\$ blends in the ranges of (6-8):3:(0.5-1.25) and a constant citric acid dose of 0.27% were investigated by Pelletier et al. The first stage of the hydration in this system is the reaction of ye'elimite (C_4A_3S) and C\$ with water to form ettringite and AH_3 . In all the blends studied, over three times as much ettringite was formed by 6 hours compared to the 100% OPC sample. This was often the maximum amount of ettringite and AH_3 present in the systems during hydration (Pelletier et al., 2010).

After 7 days, a more complex set of reactions takes precedence in which the OPC clinker begins to hydrate and consumes some AH_3 as well. This second phase of hydration forms C_2ASH_8 , CH, C-S-H and $C_4A\check{C}H_{11}$ (monocarbonate) products from AH_3 and C_3S . Simultaneously, AH_3 and C_3A also react with CH to form C_4AH_{13} . Between 7 and 28 days, the set of reactions that consume ettringite to form monosulfoaluminate also begin. The ettringite and AH_3 consumption in these later reactions is matched by the hydration of OPC clinker compounds so there is always a net gain in the volume of hydration products (Pelletier et al., 2010).

While ternary blends of CSA might be able to compensate for shrinkage without additional curing water, it is important to note that all the reactions taking place after the first phase of hydration still require water. These reactions contribute to reducing the overall porosity of the microstructure and sufficient humidity should be ensured for them to occur.

2.1.2.2 Durability

CSA pavements in southern California have proved to be durable over time, with almost 500,000 cubic yards of CSA consumed in total in the state (Bescher et al., 2018; L. Burris et al., 2015). However, the exposure conditions of these case studies are consistently mild compared to those found elsewhere, i.e. the wide range of climates in Oregon.

Corrosion of Steel Reinforcement

Contrary to other cement chemistries, Wang et al. found that $Al(OH)_4^-$ plays a significant role in CSA pore solution for passivating steel. The pH of CSA pore solution is typically around 12.5-12.9, though one study reported values just below 11.3. Although the typical pH levels are sufficient for passivating embedded steel, the alkalinity is lower than that of OPC (pH ~13.5). Furthermore, the presence of higher sulfate concentrations in CSA may contribute to instability of the passive layer for steel (Wang et al., 2019).

Mature CSA pastes can possess a finer pore structure than OPC samples of similar strength (Mobili et al., 2020) However, this metric does not correspond to improved resistance to chlorides. Jen et al. found that CSA binders with a larger proportion of ettringite (AFt) as the main hydrate bound fewer chlorides than those with more monosulfoaluminate (AFm, formed due to decreased C\$ content in raw material) (Jen et al., 2017). Moffat and Thomas found that a CSA- C_2S blend performed worse than a CSA-OPC ternary blend (Edward G. Moffatt & Thomas, 2018). The diverse assemblage of hydrate phases in OPC is evidently more adept to binding chlorides into phases like Friedel's salt.

The ingress of chlorides into concrete is based on two factors: the rate of chloride transport and the binding of these chlorides. Tests like RCPT or resistivity are rapid measures of transport. To account for chloride binding (or lack thereof), accelerated test methods such as the rapid chloride permeability test (RCPT, AASHTO T277) are not recommended as they do not allow enough time for chloride binding to occur (Jen et al., 2017). Findings from Moffat and Thomas corroborate further to show that electrical resistivity readings based on AASHTO T277 do not accurately represent the field performance of CSAs exposed to a harsh marine environment. The lack of correlation may be attributed to the differing pore solution composition in CSA and potentially limited chloride binding capacity (Edward G. Moffatt & Thomas, 2018). Further, the pore solution should be accounted for and then the formation factor can be calculated to accurately characterize transport. Then the binding potential of the cementitious system should be evaluated. Nevertheless, some manufacturers may still report RCPT values that suggest a low risk of chloride ingress.

Carbonation

The carbonation of ettringite is a decomposition to form calcite, gypsum, and alumina gel. These reactions can increase pore diameters in the microstructure and in turn promote further carbonation (Hargis et al., 2017). The carbonation rates of the CSA-C₂S and CSA-OPC blends evaluated by Moffat and Thomas indicated that in ettringite systems the carbonation front would only take 17 to 25 years to reach steel at a cover depth of 1 to 1.2 in (Edward G. Moffatt & Thomas, 2018). Further research on ternary blends with higher OPC content may provide better insight into reducing the rate of carbonation in CSA systems.

Volume Change

Because CSA binders typically form large amounts ettringite to rapidly gain strength, they are capable of expansive behavior associated with the growth of ettringite. Furthermore, the low silica content of CSA (as seen in) indicates that limited C-S-H will be able to form during hydration. Because of this the autogenous shrinkage associated with C-S-H formation in OPC binders will be less of a concern (Bescher et al., 2018).

Telesca et al. observed that increasing the C\$ content in CSA cements led to greater expansion in water curing conditions (Telesca et al., 2014). This could be attributed to increased ettringite formation (Jen et al., 2017) In fact, all CSA systems they investigated generated positive expansion when water cured and C\$ was prone to causing deleterious expansion in excessive doses. However, the CSAs with appropriate proportions of C\$ (<30% wt.) exhibited shrinkage in air curing conditions that was comparable or in excess of OPC . Other authors suggest that the optimum C\$ proportion for volume stability of CSA is around 25%.

Despite the expansive properties of ettringite, CSA concrete is still susceptible to autogenous shrinkage if insufficient mix water or curing water is supplied. For a range of proprietary and non-proprietary CSA cements, Maguire et al. performed drying shrinkage tests at 73°F and 50% RH in accordance with ASTM C157. They found that all the non-proprietary materials exceeded a positive 450 microstrain by 7 days. Restrained ring tests from the same study found that CSA mixtures cracked between 4-6 days even when

internal curing was implemented (Maguire et al., 2017). Work by other researchers found that proprietary rapid strength cements – which may or may not have contained CSA – exhibited slightly lower volume change (<500 microstrain), but some of the materials that possessed characteristic traits of CSA were determined to be at risk of cracking in restrained ring tests (Barde et al., 2006). Because of the expansive capabilities of CSA, it is necessary to use a dual-ring setup for restrained testing, which was not done by Maguire et al. and consequently prevented precise measurement of stress development and the time to cracking.

The possible shrinkage behavior of CSA can be explained by the high-water demand of ettringite hydration (and expansion in turn). As seen in the literature, the use of water curing, higher w/cm ratios or internal curing can effectively mitigate shrinkage (Telesca et al., 2014; Thomas et al., 2018). Anecdotal evidence of the water demand in CSA can be seen in a lack of bleed water. Appropriate curing practices are necessary for CSA and these will be discussed further in section 2.4, and remarks from other agencies on the need for proper curing in very-early strength materials like CSA are included section 2.5.

Freeze-Thaw

Work by de Bruyn et al. demonstrated that CSA exhibits comparable freeze-thaw durability to OPC. However, when poor quality (more porous) aggregates were used, the CSA binder was able to last three times as many freeze-thaw cycles than OPC before failure (following ASTM C666) (de Bruyn et al., 2017). Freeze-thaw performance alone is not a particular concern for CSA, and manufacturers are likely to provide satisfactory ASTM C666 results for any formulation. However, Moffat and Thomas did find that salt scaling lead to greater deterioration in CSA blends than other OPC or CAC materials (Edward G. Moffatt & Thomas, 2018).

Alkali-Silica Reaction (ASR)

The decreased pH levels of CSA pore solution are not conducive to ASR (Juenger et al., 2011). Dornak et al. performed ASTM C1293 tests and found that multiple CSA materials did not produce expansion with reactive aggregates. However, when latex was included in a CSA mixture the expansion dramatically increased (Dornak et al., 2015).

CSA Summary

The implications for HESC gleaned from this review of CSA behavior are summarized in Table 2-4. The main benefit noted with these materials is rapid strength gain, however some durability concerns may exist, especially in relation to corrosion of reinforcing steel.

Table 2-4: Potential benefits and challenges associated with using CSA for HESC bridge overlays.

Benefits	Challenges
Rapid strength gain	Inconsistent information due to variable compositions and use in proprietary products
Low CO ₂ footprint	Potential durability concerns
Low shrinkage/shorter curing duration	Limited working time, batch plant mixing may not be feasible
Successfully used by many agencies (see section 2.5)	Equipment availability and finding experienced contractors

2.1.3 Calcium aluminate cements (CAC)

The unique chemistry of calcium aluminate cements (CACs) makes them suitable for applications where OPC is not comparable option. These applications include those requiring rapid strength gain at low temperatures, high temperature exposure, aggressive chemical environments, and substantial mechanical abrasion (Scrivener & Capmas, 1998).

CAC is produced from limestone and bauxite since these are rich in calcium and aluminum, relatively. Standard grades of CAC typically include 38-40% alumina with 20% iron oxide also contributed by the bauxite. Higher grades of CAC require low iron bauxites. The Blaine fineness of CAC is relatively close to that of ASTM C150 Type I/II OPC (250-400 m²/kg). The omission of silicates is a clear departure from OPC that can lead to swifter reaction processes. The main clinker phase of CAC is monocalcium aluminate (CA), followed by C₁₂A₇. Other phases that may be present are pleochroite, ferrite solid solution, and gehlenite. The ferrous phases often appear in fibral geometries in the clinker morphology (Scrivener & Capmas, 1998). The generalized composition of CSA is included in Table 2-1.

Despite the performance advantages and existence of CACs for over 100 years, as of 2015 only four state DOTs reported having used CAC for repair applications with varying degrees of success (L. Burris et al., 2015). Understanding the factors influencing the hydration, strength gain, and durability of CAC systems is crucial for successful implementation and these are described in the following sections.

Hydration and Strength gain

In most cement systems, unstable anhydrous phases react with water to form stable hydrates. However, in standalone CAC systems metastable intermediate phases can form, which eventually convert to stable phases. The dominant stable phases in hydrated CAC are C₃AH₆ (hydrogarnet) and AH₃. The metastable phases are CAH₁₀ and C₂AH₈, which contain high amounts of water (50% and 40% respectively) (Scrivener & Capmas, 1998).

Because the metastable phases consume so much water, their production can be reduced by using a low w/cm. The resulting phase assemblage and setting behavior resulting from CAC hydration can also vary according to the temperature and proportions of C₁₂A₇ included with CA. In general, temperatures below 30°C will promote metastable phases such as CAH₁₀ and C₂AH₈ (though they are only metastable above 20°C). Temperatures above 30°C predominantly create stable phases of C₃AH₆ and AH₃ (Lothenbach et al., 2012).

When $C_{12}A_7$ is not present (at 20°C), CA dissolution will be followed by a lengthy induction period comparable to that of OPC. During this time, CAH_{10} slowly nucleates before precipitating collectively. Including $C_{12}A_7$ raises the overall calcium to alumina ratio and enables C_3AH_6 and C_2AH_8 to form. C_2AH_8 nucleates more easily and can reduce the setting time to less than 1 hour (Ideker et al., 2019).

Contrary to OPC, CAC hydration products precipitate throughout the matrix rather than forming around grains of clinker. This causes a greater heat to be released during the precipitation and strength gain phase and expedites its completion relative to OPC hydration that is more gradual and continuing. The feedback of self-heating can lend itself to the formation of greater quantities of stable hydrates and more rapid strength gain. However, as with other cement systems, the heat can also lead to increased evaporation rates if curing conditions are not controlled (Ideker et al., 2019).

In 100% CAC systems, conversion naturally occurs if the temperature remains above 20°C and the relative humidity is sufficient (Ideker et al., 2019). Fryda et al. found that conversion occurs around 20 days when samples were stored in a water bath at 30°C, but when the temperature was 100°C the conversion occurred in 15 minutes (Matthew Peter Adams, 2015; Fryda et al., 2001). Strength monitoring across the conversion period should be performed to determine the minimum converted strength to be used for design. Because water is released from the metastable products during the conversion, there is sometimes a slight rebound in strength attributed with the hydration of relict anhydrous phases. Therefore, strength measured at a single point after conversion – e.g. 28 days – may not correspond to the minimum strength caused by conversion.

Ternary Blends (CAC-OPC -C\$)

Reasons for using CAC in conjunction with OPC and C\$ include a reduction in cost, elimination of metastable phases and different performance advantages. There are three main zones in the ternary diagram that have been produced and studied (Figure 2-3).

- Zone 1: Blends in this range are comprised of over 50% OPC. When 10-30% CAC is used, the set time is a matter of minutes. OPC supplies all the C\$ in zone 1, which in turn forms ettringite early in the hydration process instead of controlling C_3A dissolution. The saturation of alumina hinders C_3S hydration and ultimately limits the long-term strength of the system relative to OPC independently.
- Zone 2: Compared to zone 1, zone 2 uses slightly less OPC, has an increased CAC content, and uses an additional source of C\$. Ettringite and AH_3 are the main hydration phases responsible for early strength gain in this system. The proportion of AH_3 is determined by the quantity of calcium supplied by the OPC. In this system, anhydrite forms of C\$ are preferred as hemihydrate C\$ can delay hydration and possibly react incompletely to have negative side effects. After 1 day of reaction, hydration contributes to further strength gain and reduction of porosity with phases of C-S-H, stratlingite, and/or hydrogarnet.
- Zone 3: This range contains <50% OPC with CAC and C\$ dominating the blend in comparable proportions. Ettringite formation is most abundant in this zone and thereby the improved properties of associated with ettringite are also more

pronounced. The enhanced properties include higher early strength, quicker drying and better shrinkage compensation. In this context, drying is attributed to the absorption of water into ettringite rather than the loss of water to evaporation. The source of C\$ for zone 3 blends should be hemihydrate or gypsum since the CAC content is higher and the C\$ can fully react readily. The higher content of CAC in zone 3 raises the cost compared to other blends. However, the improved performance of zone 3 blends – particularly the fast-drying time to reach a relative humidity <3% – is worth the increased cost for niche applications like floor leveling mixtures.

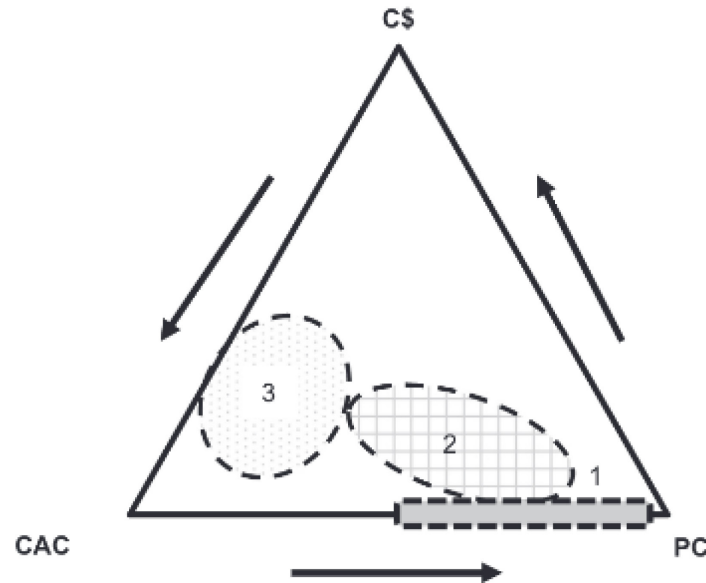


Figure 2-3 Three main zones of composition for CAC blended systems (Ideker et al., 2019).

The abundance of ettringite in zone 2 and 3 blends can have shrinkage compensating effects. The expansive behavior takes place during the hydration process and helps combat undesired drying shrinkage. Later expansion of ettringite in the system that might lead to damage is only possible if there is unreacted C\$ available. It is important for manufacturers to monitor the proportion and type of C\$ in the system to eliminate the possibility of later expansion in ternary CAC blends (Ideker et al., 2019).

2.1.3.1 Durability

CAC is known for its resistance to high temperatures in refractory applications. It can also provide advantages performance in sulfate and acid environments. However, this section will focus on the durability properties relevant to bridge overlays. A recurring theme for CAC durability is the need to use a low w/c ratio. Alkali hydrolysis is not discussed here as it is easily avoided by using a w/c below 0.7 (Ideker et al., 2019).

Corrosion of Steel Reinforcement

The pH of CAC concrete remains around 12 before and after conversion, though it is slightly higher prior to conversion. These pH levels are sufficient for passivation steel

reinforcement. In systems blended with OPC, the pore solution can exceed a pH of 13 after 1 day of hydration (Ideker et al., 2019).

Regarding the potential for chloride ingress and transport properties, CAC concrete benefits from have a w/c ratio of 0.40 or lower and minimal past content. Work done at the University of New Brunswick found that CACs and CAC blends can have a much lower diffusion coefficient than that of OPC. However, the CAC mixtures in their studies were cured at 73°F so the expected level of conversion is low (Ideker et al., 2019; Edward G. Moffatt & Thomas, 2018). Curing temperature can have a significant impact on the conversion process and resistance to chloride ingress. For 100% CAC concrete, the researchers found that samples cured at 73°F performed better than samples cured at 50°C when placed at an aggressive marine exposure site in Maine for 7 years. Both mixtures exhibited corrosion damage while the OPC control samples remained intact at 5 years. Work by Moffatt et al. suggests that ternary blends can show improved chloride resistance, however further research is warranted for a broader spectrum of blended materials (E G Moffatt & Thomas, 2016; Edward G. Moffatt & Thomas, 2018).

Carbonation

Like OPC, carbonation of hydration products in CAC can also form CaCO_3 and have a positive effect on strength. However, CAH_{10} and C_3AH_6 are affected rather than CH (in 100% CAC) and AH_3 and H_2O are also formed. Studies of indoor CAC concrete beams in the U.K. suggest that the rate of carbonation is comparable to that of OPC after 20-30 years. Limited research on ternary blends suggests that the blends do not perform as well as OPC or CAC do alone, and a general trend is that below 75% CAC increasing the OPC content improves the resistance to carbonation (Ideker et al., 2019; Lamberet, 2004). For ternary blends forming large amounts of ettringite, carbonation reactions like those described for CSA in section 2.1.2.2 could be expected. However, research is needed to confirm this correlation.

Volume Change

The rapid precipitation of hydration products in CAC can influence shrinkage mechanisms by consuming water more rapidly and exhibiting greater self-heating and possible thermal gradients. Because of the rapid reaction, the period where curing is required to mitigate shrinkage is shortened. Higher temperatures may promote evaporation during a time when water is most need, curing practice is still necessary. Further research is needed on the volumetric stability of ternary blends (Lamberet, 2004).

Freeze-Thaw

In laboratory (ASTM C666) and field exposure conditions, CAC concrete has remained intact after 300 and 600 freeze-thaw cycles, respectively. Similar results were obtained for a ternary blend of CAC (Ideker et al., 2019; Edward G. Moffatt & Thomas, 2018). To attain such freeze-thaw durability, it is recommended that the porosity of the concrete be below 13%. This is true for OPC as well, yet air entrainment is often prescribed for cold weather regions (Ideker et al., 2019).

Alkali-Silica Reaction

ASR is not a concern in 100% CAC because calcium hydroxide is absent, and the pore solution pH is not high enough to initiate dissolution of silicates in most aggregates. Laboratory tests support this theory for 100% CAC, but this does not extend to blends with OPC. The inclusion of OPC hydration products in ternary blends may lead to conditions favorable for ASR (Ideker et al., 2019).

2.1.3.2 CAC Summary

The implications for HESC gleaned from this review of CAC behavior and ternary blends are summarized in Table 2-5.

Table 2-5: Potential benefits and challenges associated with using CAC for HESC bridge overlays.

Benefits	Challenges
Rapid strength gain	Inconsistent information due to variable compositions
Cold weather application	Sensitive to different aggregate types
Durability	Limited working time, batch plant mixing may not be feasible
Used in existing structures >100 years old.	Equipment availability and finding experienced contractors
Ternary blends can offer rapid strength gain and long-term strength stability	Some information not well characterized for ternary blends, namely volumetric stability

2.1.4 Ultra-High-Performance Concrete (UHPC)

There is extensive literature on the use of UHPC for bridge overlays and other repairs for existing concrete structures. In terms of cementitious content, UHPC is technically a subcategory to OPC in which an optimized contents of silica fume, other SCMs, high quality aggregate – *and often finer coarse and finer fine aggregates*, and steel fibers are added to OPC to create concrete with compressive strengths exceeding 20 ksi. UHPC also has impressive tensile strength, workability and cracking resistance. Setting times under 4 hours can be achieved with a range of SCM types/doses in UHPC (Yalçinkaya & Yazıcı, 2017).

However, UHPC is a complete concrete product that comes at a price as much as 20 times that of normal OPC concrete. Using a 1.5% steel fiber content of UHPC increases the cost by roughly \$470/yd³ (Graybeal, 2013). The potential for rapid strength gain in UHPC is confined to similar limitations as OPC systems, except with the advantage of already possessing a low w/cm and dense particle packing. Many strategies of UHPC mixture design, such as fiber dosage and the use of water reducing admixtures, which enable its performance advantages can be extended to benefit other cement systems as well and these will be discussed in section 2.2.

Because the chemistry is broadly the same as OPC, details regarding the hydration processes and durability of UHPC can be interpreted from sections 2.1.1.1 and 2.1.1.2 respectively. Overall,

very high cementitious contents and low w/cm may lead to shrinkage concerns in these systems. Generally long-term durability is improved due to significantly improved transport properties.

2.1.5 Magnesium phosphate cement (MPC)

Magnesium phosphate cements (MPC) are a highly capable HESC binder with potential durability concerns for wet climates. Because of the freeze-thaw resistance, MPC has been used in Alaska for joints in between concrete deck panels. After 20 years of service, deterioration in these joints is mainly attributed to structural loading rather than material properties. MPCs offer a range of benefits as rapid repair materials including improved bond strength to OPC, low shrinkage and freeze thaw durability.

2.1.5.1 Hydration mechanisms and strength gain

Magnesium phosphate cements are comprised of MgO and various forms of phosphates. The phosphates dissolve rapidly and create an acidic solution that is counterbalanced by the subsequent dissolution of MgO to release hydroxide. As magnesium phosphate phases (predominantly struvite) precipitate, the pH approaches neutrality. The majority of heat from these exothermic reactions can be released in under 3 hours. Without additives, MPC can set in under 10 minutes (Haque & Chen, 2019; N. Yang et al., 2014).

2.1.5.2 Durability

In a study of 23 proprietary rapid-set products, products based on magnesium phosphate chemistries observed negligible shrinkage (Z. Yang et al., 2016). However, MPC concrete has been found to lose 44% of its strength when cured in water for 28 days (N. Yang et al., 2014). Because of the early age sensitivity of un-hydrated phases in MPC to dissolution, MPC may not be suitable for application in wet or moist environments. Another drawback to MPC is the limited working time before setting. Additives like borax or fly ash in moderate doses have been shown to delay setting without affecting other properties (J. Li et al., 2014).

Regarding corrosion resistance, MPC may not have a high enough pH for passivating embedded steel, but it does have low permeability. Furthermore, corrosion is inhibited by an impermeable iron phosphate film that forms around the steel in MPC (J. Li et al., 2014).

2.1.6 Alkali activated materials (AAMs)

Interest in AAMs has largely been sparked by their resistance to high temperatures (Belena & Zhu, 2009; J Davidovits, 1991; Kong & Sanjayan, 2008; Rivera et al., 2016) (e.g. up to 1000°F), and the potential to reduce the CO₂ footprint by 80% compared to that of OPC (Joseph Davidovits, 1994). However, more recent studies suggest that the maximum CO₂ reduction is more realistically in the range of 40-50% by using either fly ash or calcined clay (Heath et al., 2014; Mclellan et al., 2011).

While AAMs can set rapidly (Arnoult et al., 2018; Siyal et al., 2016; Suraneni et al., 2014), a case study in Georgia suggests that AAMs may not reach 2,500 psi within 4 hours (L. Burris et al., 2015). Despite extensive research on AAMs, AAM performance is highly dependent on formulation and precursor materials. The performance desired for HESC applications would likely need to be achieved through further research and development. Commercial geopolymer products are available, but these are typically intended for rapid repair pipe linings.

2.1.6.1 *Hydration mechanisms and strength gain*

Unlike most other cements, which are hydraulic, alkali-activated materials (AAMs) rely on an alkaline solution to enable (i.e. “activate”) the dissolution of solid aluminosilicate precursors. AAMs with precursors low in calcium will form an amorphous reacted phase of cross-linked alumina and silica tetrahedra that has frequently been dubbed “geopolymer” (Bernal & Provis, 2014; J Davidovits, 1991; Joseph Davidovits, 2015; Fernández-Jiménez et al., 2005; Juenger et al., 2011; Lloyd et al., 2009; Provis et al., 2005; White et al., 2011, 2017; Xu & Deventer, n.d.). Several researchers suggest that there is very little to no water chemically bound in amorphous aluminosilicate phases (Park & Pour-Ghaz, 2018; Rowles et al., 2007; Tuinukuafe et al., 2019)

In the presence of greater calcium precursors, reaction via alkaline solution will yield calcium alumino-silicate hydrate (C-(A)-S-H) products more like that of C-S-H in OPC products but with greater substitution of alumina in the cross-linked tobermorite structure and more chemically bound water (Walkley et al., 2016, 2017; White et al., 2015). With precursors with high calcium content, such as slag, the short- and long-term performance of AAMs can also be comparable to that of OPC (Bernal et al., 2013; Wardhono et al., 2017; Xu et al., 2008).

2.1.6.2 *Durability*

While AAMs rich in calcium (alkali activated slags), can exhibit similar durability to OPC, low-Ca AAMs can have highly interconnected porosity and fleeting pore solution alkalinity that may lead to poor protection for reinforcing steel (Babae & Castel, 2018; Lloyd et al., 2010; Moradillo & Ley, 2017; Osio-Norgaard et al., 2018). Therefore the permeability of these materials are high and it would not be expected to be durable in a corrosive environment.

As mentioned earlier, the composition and performance of AAMs is widely variable and based on source materials. Moreover, the environmental footprint and cost of AAMs is minimized by using local materials. The development of sustainable AAMs for HESC overlays, or other paving applications, could be done using precursors native to Oregon such as Mt. Mazama ash (Sleep & Masley, 2019) but significant research would need to be done to ensure desired performance.

2.1.7 *Proprietary Materials*

Rapid repair materials that are well suited for HESC are often sold as proprietary products. These may include any type of cement or a combination of multiple cements, e.g., OPC + CSA. Proprietary blends may also include dry admixtures required to meet their marketed

performance. For example, set retarders and/or accelerators can be included to optimize setting time or corrosion inhibitors can be included to reduce the risk of corrosion. Manufacturers sometimes also include aggregates to eliminate the possibility of undesired cement-aggregate reactions and ensure reliable performance of their product. Analytical techniques such as XRF and XRD can determine the general chemical composition and compounds in cements, should one desire to troubleshoot the performance of a proprietary material.

2.2 MIXTURE PROPORTIONING

There are several other important factors to HESC materials besides the cementitious material composition. The water content (w/c and total paste volume) and aggregate properties can have a strong influence on the concrete behavior (Mehta & Monteiro, 2006). Also, the use of fibers and admixtures can be essential for HESC overlay performance.

2.2.1 Water Content

As discussed in the hydration sections for each cement, the water demands for different types of cement can vary. Concrete mix design following ACI 211 selects an initial water content that is used to determine the cement content from the w/c ratio. The recommended water to binder ratios for the main cement types discussed previously are summarized here:

- OPC, w/c 0.20 – 0.45 (Ghafoori et al., 2017; Kosmatka & Wilson, 2016)
- CSA, w/c 0.35 – 0.50 (Thomas et al., 2018)
- CAC, w/c < 0.40 (Ideker et al., 2019)

Considering proprietary materials, the recommended w/c from the manufacturer should be followed. The total cement content is often increased for HESC (675 – 1000 lb/yd³) (Kosmatka & Wilson, 2016). This increase may or may not be necessary when alternative cements are employed. Furthermore, an increase paste content can increase the potential for shrinkage (Ideker et al., 2013).

2.2.2 Aggregates

Economical HESC mixtures are largely comprised of aggregates by volume. Therefore, the properties of the material, such as the coefficient of thermal expansion (CTE), are influenced by the aggregates. Furthermore, the angularity and surface texture of aggregates may influence the shrinkage behavior of the concrete as previous work has observed greater shrinkage in mixtures containing rounded river gravel compared to crushed aggregates (Ideker et al., 2013).

2.2.2.1 Gradation

With respect to bridge overlay applications, aggregate gradation can have a significant impact on the following factors:

- Workability for timely placement and finishing

- Shrinkage reduction primarily attributed to minimizing the paste content
- Transport properties associated with paste volume reduction and a more tortuous microstructure

The nominal maximum aggregate size should generally be $\frac{1}{2}$ " or $\frac{3}{4}$ " – falling well within limits for thin applications or narrow clear spacing of rebar and substrate. High strength concretes generally limit the aggregate size to $\frac{3}{4}$ " (Kosmatka & Wilson, 2016), and this approach can be extended to improve the performance of HESC mixtures. Proprietary products including aggregate are likely to take this approach as well.

2.2.2.2 *Lightweight aggregates*

Durable and long-lasting concrete is important, especially for bridge decks, which are susceptible to premature cracking and corrosion of the reinforcing steel. To increase the durability of the concrete and its service life, many transportation agencies use high-strength concretes. However, high-strength concretes often have problems with early age cracking due to shrinkage. These cracks can then open a path for chloride ions (found in road salts) to reach the reinforcing steel. The results of prior research, which compared bridge installations on two structures in Monroe County, Indiana in 2012, confirmed that internally cured concrete presents a better alternative to traditional plain concrete for durable bridge decks. With internally cured concrete, a portion of the fine aggregate in the concrete is replaced with the same volume of prewetted lightweight aggregate. As the concrete cures, water from the prewetted aggregate provides the hydration necessary for curing and enables curing from the inside. This internal curing process results in concrete with less initial cracking, less shrinkage, lower thermal stress, lower strain, and greater resistance to chloride ion penetration, with similar or slightly higher strength, relative to plain concrete (Barrett et al., 2015; di Bella et al., 2012; Henkensiefken et al., 2009).

Aside from internal curing, FLWA can produce and improve interfacial transition zone (ITZ) that reduces the permeability of the concrete (Dale P. Bentz, 2009). Maguire et al. observed that using lightweight aggregates for internal curing can increase creep in a variety of cementitious systems (Maguire et al., 2017). Raoufi et al. demonstrated that mortars with increasing FLWA showed greater stress relaxation in restrained ring tests when temperature drops occurred, effectively broadening the range of temperature change required to cause cracking (Raoufi et al., 2011). In addition to internal curing, these properties of FLWA could improve the robustness and durability of HESC bridge overlays.

2.2.2.3 *Interaction with Cement*

Separate from deleterious alkali-aggregate reactions, aggregates can still have important chemical and physical interactions with cement that influence the overall concrete properties. The interfacial transition zone between aggregates and cement paste is a weak point in concrete microstructures. This is most often attributed to lower particle packing densities against the aggregate which inhibit C-S-H growth in OPC binders. However, alternative cements can have different relationships with aggregates.

In the case of CSA cements, the finer cement particles can create a denser interface with aggregates. Adams and Ideker found that high purity limestone aggregates performed far better than siliceous aggregates in 100% CAC systems in terms of overall strength gain (and impacts from conversion). Certain siliceous or aluminosiliceous aggregates may result in unacceptable strength loss during conversion and this should be accounted for in mixture design and aggregate selection. (Adams & Ideker, 2017)

2.2.3 Admixtures

Starting with mixtures based on OPC, HESC has a precedence of relying on admixtures to attain a desired setting time. Admixtures are also used to improve workability and air entrainment as they are with normal concrete. Finally, background on the use latex admixtures for HESC bridge overlays is included in this section.

2.2.3.1 *Water Reducing*

Water reducing admixtures are often necessary to achieve workable HESC mixtures due to the low w/c ratios used. High-range water reducers, or superplasticizers, are based on polycarboxylate technology where polar chain molecules are attracted to the charge of cement particles. The polycarboxylate chains surround cement grains and create a barrier of electrostatic repulsion that keeps cement particles from interlocking and impeding flow (Kosmatka & Wilson, 2016).

Polycarboxylate ether (PCE) compounds are also compatible with CAC and CAC blends showing fluidity and no reduction in setting time (Ideker et al., 2019). For CSA, retarders such as tartaric acid can also have plasticizing qualities (Coppola et al., 2018). Conversely for CSA, polycarboxylates may have retarding effects in CSA blends (Ma et al., 2014) though they are still effective for improving workability (G. Zhang et al., 2016).

2.2.3.2 *Set-Retarding*

Set retarding admixtures are sometime used in OPC mixtures to delay setting in hot-weather or as particular placement conditions demand. In the context of HESC materials, there is no foreseen need for set retarders in OPC. Retarders are, however, necessary for rapid setting cements like CSA for adequate working time. Citric and tartaric acids are commonly used as retarders for CSA, and CAC blended systems (L. E. Burris & Kurtis, 2017; Ideker et al., 2019; Thomas et al., 2018). While CaCl_2 can be used as an accelerator in OPC, it has also been observed to retard CSA – though the alternatives are more effective (Long, 2019).

2.2.3.3 *Set-Accelerating*

For OPC concretes, CaCl_2 is a highly effective accelerator that functions by weakening the initial layer of hydration product that forms around cement grains. This enables the cement grains to reach higher levels of hydration at an earlier age. Because a major concern with using CaCl_2 is the potential for rebar corrosion, the CaCl_2 dose is typically

limited to 2% or less. In the case of HESC overlays, the reason for repair could be due to existing corrosion damage, which could be exacerbated by applying an overlay containing CaCl_2 (Kosmatka & Wilson, 2016). Non-chloride alternatives, such as triethanolamine (TEA) can also be used with lesser effect.

The effects of a non-chloride-based accelerator on the hydration of OPC was investigated by Todd et al. (Todd et al., 2017). When cured at 50°C , the accelerator was found to be less effective in shifting the reaction enthalpy forward and there was a decrease in the resulting flexural strengths. The impact of such admixtures and/or environmental conditions should be considered before the maturity method is used to estimate strength (Todd et al., 2017; Weiss et al., 2019).

In 100% CAC concrete, lithium-based salts can be used to accelerate setting by facilitating earlier and more thorough nucleation of hydration products. For CAC blends with high OPC content, sodium carbonate may be used as an accelerator (Ideker et al., 2019).

2.2.3.4 *Shrinkage Reducing*

Shrinkage reducing admixtures (SRAs) are primarily composed of surfactant molecules that accumulate at the air-solution interface when added to an aqueous medium. This can significantly reduce meniscus forces in capillary pores of concrete, and thereby decreasing capillary stresses associated with drying fronts the concrete. Studies have shown that SRAs can be effective in reducing the effects of autogenous shrinkage in high performance concrete (i.e. low w/c OPC) (D. P. Bentz & Jensen, 2004; Kosmatka & Wilson, 2016; Tazawa & Miyazawa, 1995) and can be used synergistically with pre-soaked fine lightweight aggregates for further reduction (Deboodt et al., 2015; Ideker et al., 2013). Additionally, neutron radiography has been used to observe a decreased rate of evaporation and total water loss in high performance concrete (Villani et al., 2014; Wyrzykowski et al., 2015)

One disadvantage for high-early strength concrete is that SRAs can interact with the pore solution during OPC hydration to delay setting and slow strength development (Rajabipour et al., 2008).

This can also influence the bleeding behavior of the concrete and proper planning is recommended for finishing (Kosmatka & Wilson, 2016).

Mehdipour and Khayat found that OPC-CSA blends can also benefit from SRAs in the absence of moist curing, with a 20% higher non-evaporable water content (Mehdipour & Khayat, 2018a, 2018b). Because of reaction inhibiting side effects with SRA, the OPC-CSA blend they studied observed a peak in reaction enthalpy close to the time of the OPC control sample when SRA was included.

2.2.3.5 *Air Entraining*

Air entraining admixtures (AEAs) are composed of surfactants that are attracted to air-water interfaces and help form microscopic bubbles by concentrating around interfaces

(Kosmatka & Wilson, 2016). Structural provisions for concrete impose minimum air content requirements to mitigate freeze-thaw damage in cold weather regions (ACI 301R-16, 2016).

AEA can show similar performance in CAC systems to OPC, though some proprietary products may be less effective (Ideker et al., 2019). Air entrained CSA concrete has demonstrated similar freeze thaw durability to air entrained OPC, suggesting that AEAs are compatible (de Bruyn et al., 2017). Air entrainment is a feasible and effective method for mitigating freeze-thaw damage in HESC overlays.

2.2.3.6 *Latex*

Latex modified concrete (LMC) is produced by adding polymer latex, typically styrene-butadiene rubber (SBR), to concrete in liquid form. The solution is usually 47-50% solids with a pH around 11 and specific gravity of 1. The latex solids fill pores in the hydrated cement paste to create a denser microstructure (Shaker et al., 1997). This can be advantageous for HESC overlays where a denser microstructure can improve transport properties and relevant mechanical properties like bond strength and flexural strength (Sprinkel, 1999).

While there is a long history of OPC based LMC for overlays, its best use may be with cements like CSA that have higher early strength. Latex admixtures have a retardation effect on OPC that also applies to CSA – in the latter case the retardation effect is more desirable for HESC (L. Li et al., 2018). Conversely, latex may accelerate some CAC blended systems (Ideker et al., 2019).

Latex can be especially beneficial for ettringite accelerated binders (i.e. CSA or CAC blends) that may have higher permeability at younger ages (<28 days) (Thomas et al., 2018; Winnefeld & Lothenbach, 2009). In one study, latex addition to CSA improves permeability, carbonation and sulfate resistance. SBR was found to improve freeze-thaw durability of CSA while other forms of latex did not (L. Li et al., 2018).

The addition of latex can reduce the compressive and tensile strength of concrete with OPC, CSA, or CAC (L. Li et al., 2018; Shaker et al., 1997; Ukrainczyk & Rogina, 2013). Nevertheless, very early strength (VES) LMC has been used by VDOT for over 20 years in overnight overlay repairs. Map cracking has been reported in many cases in Virginia (Sprinkel, 1999; Yun & Choi, 2014). Researchers have postulated that cracking initiates at early ages, and cracks grow and become visible due to drying shrinkage – a combination of mechanisms originally proposed by Krauss and Rogalla (Krauss & Rogalla, 1996; Yun & Choi, 2014).

Researchers from the University of Texas (Dornak et al., 2015) also found that the use of latex with CSA or CAC led to reductions in engineering properties such as compressive strength, modulus, tensile and bond strength, but the flexural strength increased. In a related field study, CAC overlays containing latex exhibited more signs of deterioration than neat CAC. Moreover, the addition of styrene-butadiene latex was found to cause deleterious expansion due to ASR in CAC mixtures (following ASTM C1293), and

increased ASR expansion in CSA. The benefits of latex for the durability of HESC materials are not absolute and many aspects of the use of latex in concrete have yet to be elucidated.

2.2.4 Fibers

Data analysis of over 22,000 bridges in Pennsylvania revealed that tensile forces from load reflection were a statistically significant factor in overlay durability (Manafpour et al., 2018). Macrofibers can be added to concrete to improve flexural toughness. Harrington and Fick proposed the following mixture proportioning advice for fiber-reinforced concrete overlays (Harrington et al., 2014):

- w/c 0.38 – 0.45
- air content 5 – 7 %
- well graded aggregates
- 15 – 35 % SCM replacement for OPC

Their study also suggested that using macro fibers can decrease slump by 2 – 4 inches. However, others have found that moderate doses (5 – 7 lb./yd³) have a limited impact on workability. Moreover, water reducing admixtures are an effective way to improve the workability of fiber reinforced concrete (Ideker & Bañuelos, 2014). Bundling of fibers may be a concern if the aspect ratio of the fibers is greater than 60, the coarse aggregate is too large (e.g. greater than ASTM C33 #67), or insufficient shear is present during mixing (Roesler et al., 2019).

While macro- and micro- fibers do not address the direct cause of shrinkage, they can effectively reduce crack widths and the time to crack initiation (Ideker & Bañuelos, 2014). Synthetic microfibers were developed in the 1980's to combat early age shrinkage and do not provide structural benefits (Roesler et al., 2019). Fibers can play an important role in mitigating cracking from shrinkage in HESC overlays, and in the case of macrofibres the resistance to cracking from structural loads can also be improved.

2.3 APPLICATION

The construction of overlays presents a unique problem set where sufficient bonding to the subsurface is required and the surface needs to be suitable for riding. The use of HESC materials in this application presents another layer of challenges where conventional construction practice may not work. This section explores these practical considerations for application of HESC bridge overlays.

2.3.1 Subsurface Preparation

The cleanliness and texture of substrates are predominant factors in the strength of concrete overlays (Beushausen, 2010). Large scale UHPC deck overlays experiments have demonstrated

that interface failure does not occur if the substrate surface roughness, as measure by the texture depth, is greater than 3 mm (Aaleti & Sritharan, 2019).

The main types of surface preparation techniques for HESC overlays include surface milling, abrasive blasting, hydro demolition and mechanical concrete removal (e.g. saws, hand chipping, power driven chipping tools). A prepared concrete surface for an HESC overlay by hydro demolition is shown in Figure 2-4.



Figure 2-4 Hydro demolition of an Oregon DOT bridge deck prior to HESC placement

Plastic shrinkage can also lead to poor bond performance (Espeche & León, 2011). This might suggest that a subsurface should be in a saturated-surface dry condition before casting an overlay, but there is evidence that this does not improve the bond strength (Beushausen, 2010). The logic behind this theory is that a dry substrate will facilitate greater sorption of new paste into the pre-existing microstructure.

In addition to subsurface preparation, the bond strength is affected by the porosity of the overlay transition zone (OTZ) on a meso- and micro- structural level. Therefore, the role cement composition should also be considered in evaluating bond strength. Fibers reinforced concrete can also improve interface strength for repairs, but the effects of wetting and drying cycles are not mitigated by this (Zanotti et al., 2014).

ASTM C1583–20, Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method), can be used to assess the bond strength of HESC overlay materials and a concrete base layer. The test can provide data to compare bond strength of various materials. This test is performed on a concrete repair or overlay material that has been “bonded” to a prepared concrete surface. The test can be performed in-situ by core drilling around a cylinder that remains in place with a base in the substrate material. A steel disk is bonded to the top

surface of the test specimen and a tensile load is applied to the steel disk until failure. The failure load and the failure mode are recorded and the nominal tensile stress at failure is determined. A typical sample size is 2 inches (50 mm) in diameter.

This method has been adapted for laboratory use with a 3 in. diameter comparable to that of current ODOT specifications. ASTM C1857 presents a similar method for measuring “pull-off” strength in which a typical HESC overlay sample size is 2 inches (50 mm) in diameter placed on a wider base panel of substrate concrete. Various conditions can be assessed with either of these approaches, including surface preparation, HESC material type, HESC curing regime, etc.

The S. Yamhill project showed high bond strengths for CSA-based material with values above 450 psi, but these values did vary between core locations. The variation may be attributed to surface preparation. Techniques such as LIDAR scanning could provide a means of quantitative measurement of surface roughness for research as well as quality control in the field (Sharma et al., 2021).

2.3.2 Batching and Mixing

Because of the rapid setting behavior of HESC materials, on-site mobile mixers are a convenient mode of delivery/placement, however it is possible to produce rapid setting mixes at a plant if it is in proximity and this can increase the output for a project (Ramseyer & Perez, 2009). There may be other concerns for batching HESC materials as well.

Special care should be taken in batching CSA concretes, as the fine particle size is susceptible to “pre-hydration” in moist/humid environments. Ramanathan et al. found that the partial hydration of OPC-CSA blends before mixing reduced the compressive strength by 29% (Ramanathan et al., 2020). Alkali-activated materials require further attention since the liquid component is a reagent grade alkali or alkali-silicate solution (Provis, 2016).

2.3.3 Finishing

Caltrans experience suggests that it may be difficult to properly finish CSA systems in the working/setting time allotted, resulting in a need for surface grinding (L. Burris et al., 2015). Finishing equipment and crews should be in place immediately following the placement of concrete. For many HESC materials it is imperative that measures be taken to limit evaporation after placement and begin curing as soon as possible after finishing (Yun & Choi, 2014). This will be discussed in the following section.

2.4 CURING METHODS

In addition to being subjected to temperature effects that can accelerate or slow the development of material properties, concrete bridge overlays have a large surface area exposed to evaporation. Rapid evaporation, suction of water to the substrate, and self-desiccation result in a volume change that can render bridge deck overlays prone to cracking. This is often exacerbated by insufficient or inappropriate curing (Barrett et al., 2015).

State highway agencies (SHAs) specify a series of steps to be taken after concrete is placed. Curing consists of maintaining “satisfactory moisture content and temperature of concrete for a sufficient period of time during and immediately following placing so that desired properties may develop (Kosmatka & Wilson, 2016)”. Proper curing is vital to develop long-lasting concrete since it promotes hydration of the cement leading to strength and durability and can minimize volume change and cracking propensity. The curing needs will vary when other binder systems are used. The following section discusses curing requirements in greater detail.

Curing in practice primarily consists of either:

- supplying additional water through water ponding, wet fabrics (e.g., burlap), or internal curing or
- providing a curing compound that minimizes water loss from the system through a plastic membrane or a membrane-forming curing compound (ASTM C309 - 11, 2015).

As described by Weiss et al. (Weiss et al., 2019), there are fundamental differences between these methods, specifically in mixtures that self-desiccate. Water curing supplies additional water that is absorbed by the concrete whereas membrane curing only minimizes water loss. This is of particular importance for low w/c, HES materials that self-desiccate easily. This is also of particular interest in systems that rely on the formation of ‘products’ that expand as they can be sensitive to water activity.

2.4.1 Wet Curing

Despite being able to gain strength rapidly, overlay systems may not be conducive to quick turnaround times for construction if external wet curing methods with long durations are specified as this is incompatible with opening to traffic (Todd et al., 2017). As such, if one is going to alter the specifications for wet curing, one needs to understand where the specified wet curing times originate and what is their original intention. (ASTM C309 - 11, 2015)

Many specifications were originally written to provide the contractor with instruction on the duration of time for which wet curing should be provided. This is often based on anecdotal evidence and on conventional concrete mixtures, as such questions arise as to how these specifications impact materials that are fundamentally different like HESC overlays. These curing specifications are often intentionally simplistic to enable them to be implemented easily in the field easily.

For example, AASTHO recommends three days of curing for pavements and seven days for bridge decks, which extends to ten days when supplementary cementitious materials are used (AASHTO T23, 2017, Mordallo et al., 2018). Clearly, these specifications cannot be followed if the pavement needs to be opened to traffic at early ages (within hours). This has prompted questions as to what type of curing should be used and what the duration of this curing should be. This has also led to suggestions for curing membranes or internal curing as described in section 2.4.3 especially for rapid repairs.

Standards for wet curing are often based on criteria that are not consistent with HESC needs. For example, Poole (Poole, 2006) found that the duration of curing required was frequently based on attaining some fraction of overall strength. For example, ACI 301 recommends a minimum curing duration for the concrete to attain 70 % of the specified compressive strength which often corresponds to 7-day curing (ACI 301R-16, 2016). This however is for standard concrete mixtures and hails from use primarily in OPC mixtures.

Australian specifications (Standard, 2009) relate curing requirements with the environment to which the concrete will be exposed. While completely logical, this is often overlooked in standards. As such a marine exposure with air with a high moisture content is often treated like a location with high winds in a high desert which is not logical. Interestingly this uniquely describes the challenges that ODOT faces.

Carino and Clifton (Carino & Clifton, 1990) identified that the required curing depends on the type of concrete being used and performed research to examine the curing needed for high-performance concrete. Fundamental research is needed to better understand how wet curing influences the performance of HESC materials. However, this does not apply only to wet curing but to all the curing methods that can be used.

2.4.2 Curing Compounds

The intention of using HESC for bridge overlays is to reduce closure times and curing compounds provide a means for minimizing moisture loss in the concrete after traffic resumes. Curing compounds or membranes are sprayed on after placing concrete to help control evaporation. They are often comprised of resin or wax suspended in mineral solution with surfactants. When water from the curing membrane solution evaporates, a layer of resin or wax is deposited on the surface of the concrete. Poly-alpha-methylstyrene compounds are a higher cost alternative that also limits evaporation. It is important, however, to recognize that the costs of the curing compound may be lower in general nature than the life cycle costs of the bridge or costs associated with higher performance binders.

Hajibabae et al. measured weight loss and curling for paste samples using different curing compounds (Hajibabae et al., 2018). They found that curing compounds reduced water loss and curling further than wet or sealed curing. Poly-alpha-methylstyrene was the most effective, even in smaller doses, potentially justifying their higher cost. Applying a wax-based curing compound after 1-day of curing reduces weight loss by 50%. Vandenbossche reviewed curing compounds and application techniques used by Minnesota Department of Transportation and noted that nozzle calibration and maintenance is important for successful application of curing compounds (Vandenbossche, 1999). The need and effectiveness of curing compounds will vary for cement systems including not-only OPC, but the concepts discussed here are likely to be applicable.

2.4.3 Internal Curing

Internal curing is being increasingly used as a method to improve the performance of concrete (Barrett et al., 2015). In North America internally cured concrete is typically made by replacing a portion of the volume of fine aggregate with prewetted fine lightweight aggregate (FLWA). This has been particularly well suited for paving repairs, especially when they are opened to traffic

early as they provide additional water to the system to combat self-desiccation (Wilson, 2018). While Philleo outlined the need for internal curing in 1991 (Philleo, 1991), Figure 2-5 illustrates the fundamental difference between external curing where water is only able to penetrate several mm into low water to cement ratio concrete, whereas internal curing enables the water to be distributed more equally across the cross section. This minimal water penetration with wet curing is particularly problematic for HESC materials and as mentioned earlier no water is added when curing compounds are used.

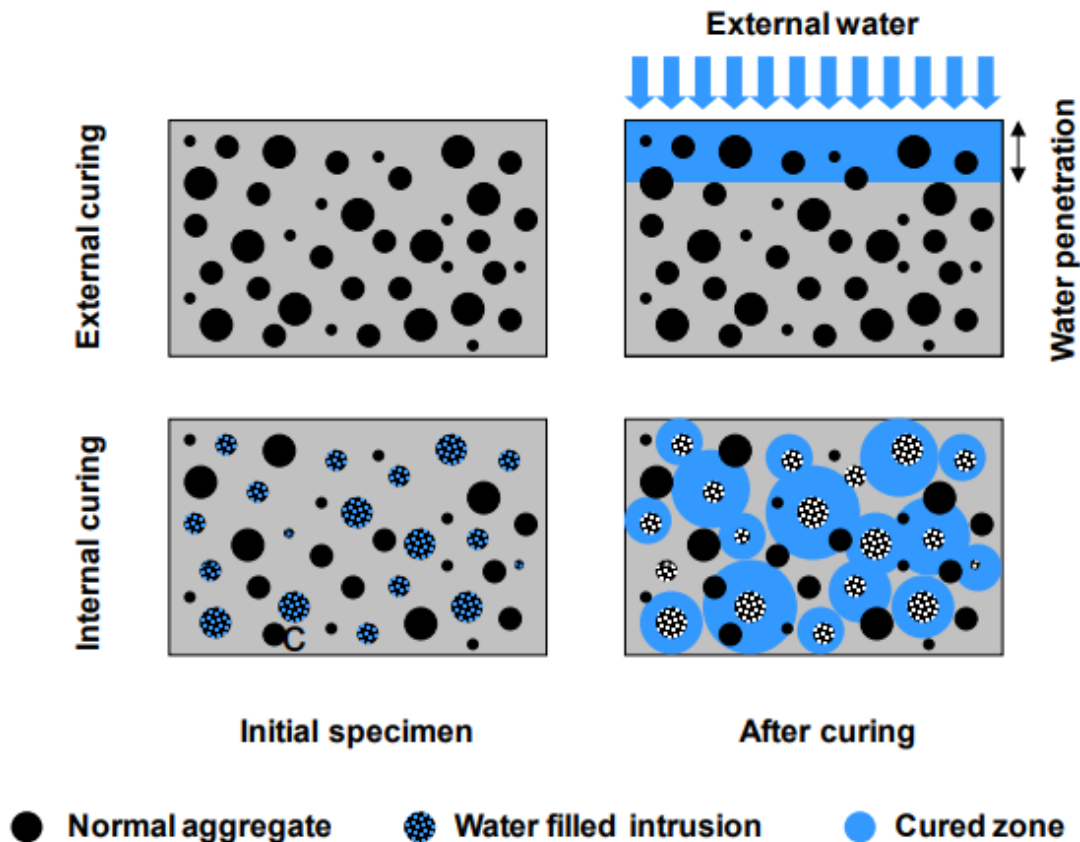


Figure 2-5 Illustration of the difference between external and internal curing. The waterfilled inclusions should be distributed uniformly and spaced close enough to provide

Prewetted FLWA releases water to the paste after set which helps to reduce autogenous shrinkage (Dale P Bentz & Weiss, 2011; Henkensiefken et al., 2009; Jensen & Hansen, 1995), reduce restrained shrinkage cracking (Radlinska et al., 2008), increase strength (Dale P Bentz, 2007), reduce fluid transport (Dale P Bentz & Snyder, 1999; Castro et al., 2012; Golias et al., 2012; Henkensiefken et al., 2009; Peled et al., 2010), and improve robustness to thermal cracking during construction (Barrett et al., 2015; Byard et al., 2012; Raoufi et al., 2011; Schlitter et al., 2010). The majority of benefits have been reported in low w/c mixtures; however, the use of internal curing in more moderate w/c systems (0.39 – 0.45) had shown benefits also (Castro et al., 2012; Espinoza-Hijazin & Lopez, 2011; Miller et al., 2014; Paul & Lopez, 2011). Internal curing supplies additional water that can increase hydration of the cement and densify

the microstructure at the surface of the lightweight aggregate (Dale P. Bentz, 2009; Henkensiefken et al., 2009; Trtik et al., 2011b). This can also be used effectively in repair materials to mitigate subsurface suction (Dale P Bentz & Weiss, 2011; Henkensiefken et al., 2009).

Internal curing (IC) can be achieved on a commercial scale by replacing the fine aggregates with an equivalent volume of pre-wetted fine lightweight aggregates (Dale P Bentz & Weiss, 2011; Kevern & Nowasell, 2018) or by using superabsorbent polymers (SAP) that absorb water (pore fluid) quickly during mixing and release it slowly into the hydrating cement paste which then maintains high relative humidities over time (Schroefl et al., 2015). Relatively little work has been performed to understand how the duration of curing may change when internally cured mixtures are used. The impact of IC for pervious mixtures (Kevern & Nowasell, 2018) and IC mixtures have shown great promise in field applications, especially rapid repairs (Barrett et al., 2015; di Bella et al., 2012).

Contrary to external curing, internal curing supplies extra water throughout the concrete. In the case of HESC materials that hydrate rapidly, this mechanism can be especially useful in ensuring water demands are met to reduce autogenous shrinkage (Maguire et al., 2017). Beuchausen and Gillmer showed that internal curing via SAPs can also reduce cracking in the case of bonded mortar overlays (Beushausen & Gillmer, 2014) Further background on the mechanisms of internal curing is discussed in the following sections.

2.4.4 Measuring Curing Efficiency

A variety of test methods have been used to assess curing efficiency by either measuring moisture/moisture loss or surface/near-surface physical properties. The simplest test appears to be to visually verify that the surface of the concrete remains wet (Carrier & Cady, 1970). Menzel (Menzel, 1954) provided early work to examine the influence of evaporation rates on concrete. Jensen and Hansen (Jensen & Hansen, 1995) proposed the use of a novel curing sensor that consisted of a capillary tube that is placed on the concrete surface. Hanson (Hanson, 1968) measured the relative humidity (RH) near the surface of concrete during drying, however, it should be noted that currently many of the RH sensors are relatively large (Villani, 2014) and have a larger variability at high RH levels. Powers et al. (Powers et al., 1959) studied how the duration of wet curing pastes of differing water-to-cement ratio (w/c) impacted the permeability of the paste. Surface absorption was used measure the effectiveness of water curing (Dhir et al., 1991). Others (Hilsdorf & Kropp, 1995; Meeks & Carino, 1999) used a variety of methods to measure permeability, water absorption, relative humidity, abrasion resistance, hardness of the surface concrete, and a notched cylindrical test specimen to measure the tensile strength of concrete at different depths. Poole (Poole, 2006) asked whether it may be a ‘better approach’ to ‘rely totally on performance testing’ citing that curing could be ceased once concrete achieved a specific strength level.

2.4.5 Using Neutron Radiography to Assess Curing Efficiency

It is important to note that while the entire cross-section of concrete being placed requires curing, the surface of the concrete is the most sensitive to moisture loss, and therefore is the most

sensitive to curing. Cather (Cather, 1992) defined a zone at the surface of concrete (i.e., the skin of the concrete), called the curing-affected zone (CAZ). While estimates of the size of the CAZ may vary, the CAZ can be thought of as extending from the surface to a depth of approximately a half inch to an inch and a half (12.5 to 37.5 mm). Moradillo et al. (Moradillo, Montanari, et al., 2018) showed that neutron radiography can be used to quantify curing as a function of the distance from the surface of the concrete. This is particularly useful in determining the ‘curing affected zone’ (CAZ). The following section will describe the use of NR and provide an example of a typical measurement quantifying the impact of curing.

Neutron radiography (NR) can be used to study curing efficiency with high spatial and temporal resolution (Berger, 1971; Hussey et al., 2012; Vontobel et al., 2006). NR consists of passing a neutron beam through a material, in this case a cementitious material. A portion of this neutron beam passes through the sample while a portion is absorbed or reflected. The natural logarithm of the ratio of the intensity of the neutron that is transmitted through the concrete (I_T) and the original beam (I_0) is related to an absorption coefficient for the sample (μ_s) and the thickness of the sample (x_s) as shown in (2-1).

$$\ln\left(\frac{I_T}{I_0}\right) = -\mu_s x_s \quad (2-1)$$

Unlike X-ray radiography, which is based on the density of a material, NR is primarily related to the neutron atomic cross section (a term used to express the likelihood that an incident neutron will interact with the target nucleus). The neutron atomic scatter and absorption cross sections are unique to each atom but are particularly large for the first isotope of hydrogen (H-1), which composes 99.9885% of all hydrogen found in nature (Sears, 1992). For example, H-1 has a total scattering cross section of 82.02 barns as compared with other materials frequently found in concrete that are an order of magnitude lower (e.g., sodium 3.28, calcium 2.90, magnesium 4.03, oxygen 4.23, and silicon 2.17) (Sears, 1992). As a result, NR is used to quantify water content in materials and has been widely used for fuel cell testing (Hickner et al., 2008).

NR has been used to study cementitious materials. Najjar et al. (Najjar et al., 1986) provided early use of NR in studies of microcracking in concrete. De Beer and co-workers studied the water movement during the drying process of concrete (de Beer et al., 2004). Villani et al. investigated drying in concrete containing shrinkage reducing admixtures (Villani et al., 2014). Wyrzykowski et al. used neutron tomography to examine the loss of water from fresh concrete (Wyrzykowski et al., 2015). NR has been used to investigate fluid absorption in cementitious materials, especially at saw cuts and cracks (W. Li et al., 2016; C. L. Lucero et al., 2017; Catherine L. Lucero et al., 2015). Zhang et al. (P. Zhang et al., 2010; Peng Zhang et al., 2011) and Kanematsu et al. (Kanematsu et al., 2009) examined fluid transport and the role of cracking on fluid transport. Jones et al. (Jones et al., 2017) employed neutron tomography to quantify the crack filling ability of polymers in cracked concrete. Trtik et al. used neutron tomography to examine the movement of water from lightweight aggregate (Trtik et al., 2011a) and SAP (Trtik et al., 2010) for IC applications. Livingston (Richard A. Livingston, 2015) has focused on using neutron tomography to measure deleterious reactions like delayed ettringite formation. While most of the aforementioned studies have focused on the qualitative movement of ‘free water’ in

concrete, Lucero et al. quantified the attenuation coefficients of both the constituent and hydrated materials (Catherine L. Lucero et al., 2015).

Moradllo et al.(Moradllo, Montanari, et al., 2018) used neutron radiography specifically to measure the water contained in hydrated cements and can be used to determine the degree of hydration in a cementitious system and the CAZ (Figure 2-6). In the mixture exposed to drying after 1 day, the top 12.5 mm (1/2 inch) of the mortar was dramatically impacted by the loss of water to evaporation. While the top 5 mm of the surface of the sample exposed to drying at 1 day had a degree of hydration that was 32% less than the 14-day moist cured sample, it should be noted that the overall degree of hydration in the core mortar was 11 % less than a water cured sample (3% less than a sealed sample). This clearly indicates transport of water to and from the core of the concrete (Villani, 2014; Villani, C. et al., 2015). The use of superabsorbent polymers demonstrated benefits in terms of both an increase in the overall degree of hydration (this can conservatively be assumed to be approximately 3.7-7.8%) and a reduction in the depth of the CAZ by 4 mm (1/6 in) when SAP is used. Further work would be necessary to develop the quantifiable correlation between the use of SAP and equivalent moist curing durations.

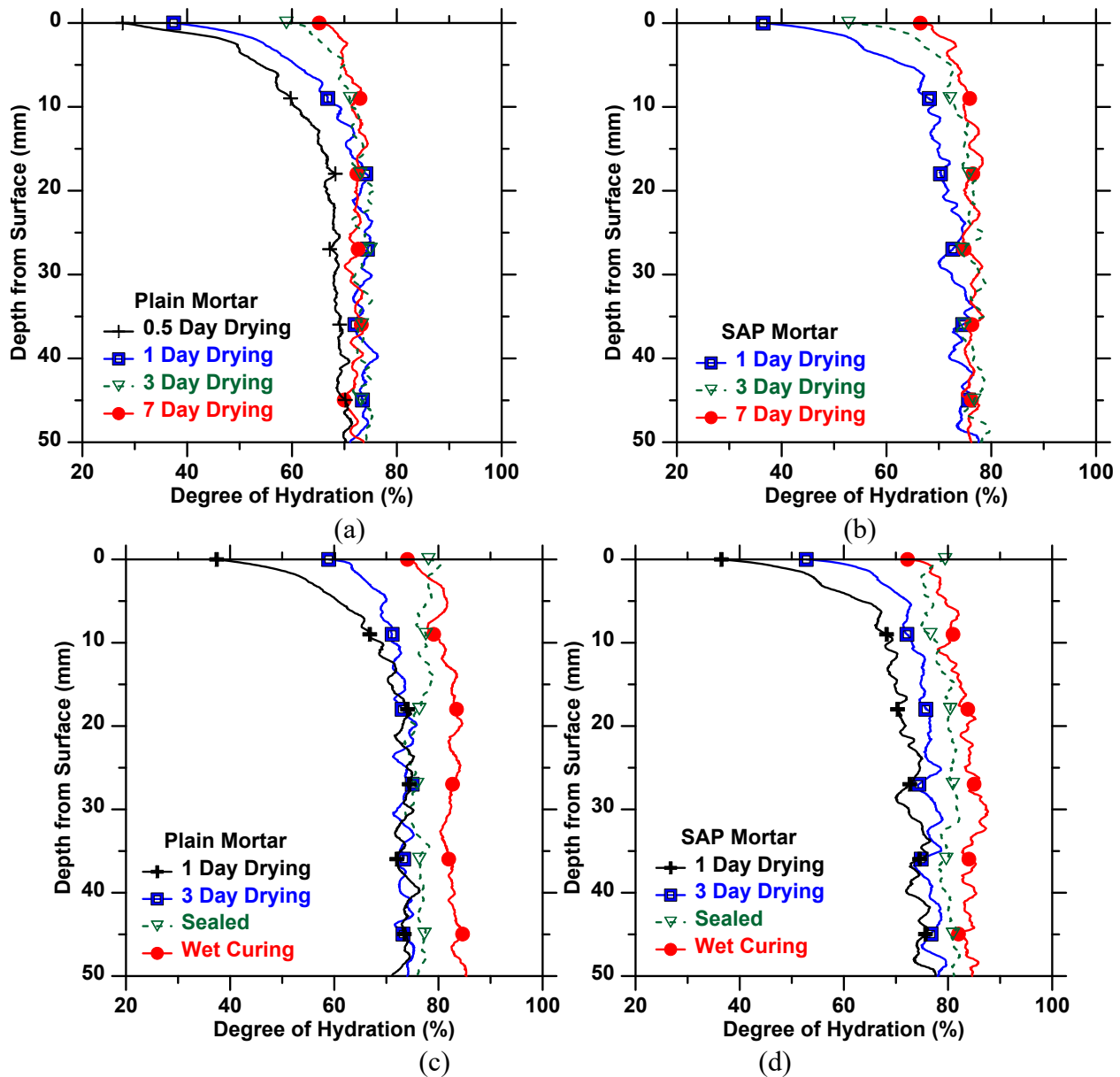


Figure 2-6 DOH over depth as measured by NR for (a) plain mortar exposed to drying (b) SAP mortar exposed to drying (c) comparison between various curing practices – plain mixture and (d) comparison between various curing practices – SAP mixture (symbols are at every 100th data point) (Note: “wet curing” represents water ponded curing in moist room and “drying” shows wet curing with saturated wet burlap for different periods of time and then drying at 50 ± 2 % RH and 23 ± 1 °C until age of 14 days.)

2.5 LESSONS FROM OTHER AGENCIES

Several highly relevant studies have been published based on work from other state highway agencies. Insights from these select studies that pertain the HESC bridge overlays are summarized in this section.

2.5.1 Indiana

Barde et al. (Barde et al., 2006) discussed how the repair of concrete pavements and bridge decks is increasingly requiring a rapid setting material that can be placed, cured, and opened to traffic in a relatively short period of time. Ensuring that these materials are compatible with the existing pavement, structures, and environment is of utmost importance. Research was performed to assess strength development, temperature sensitivity, bond, and early-age cracking in the first project and durability was assessed in follow up projects. Measurements were made for initial set, final set, long term values for compressive strength, flexural strength, elastic modulus, ultrasonic wave speed, unrestrained shrinkage tests, restrained ring tests, shear bond and tensile bond strength tests. While desirable material characteristics included rapid setting and strength development, volumetric stability and good bond properties (especially for sawed surfaces) and recommendations were made for specific repair materials it may be possible to utilize several of these evaluation techniques in their study in the present investigation.

The use of HES concrete to open repaired pavement to the travelling public was an attractive solution to reduce the delay to the traveling public (Barde et al., 2006). There are challenges due to strict requirements for opening strength and severe penalties for not achieving the target strength. Weiss et al. performed a study associated with the impact of temperature on sulfate balance, flexural strength prediction (maturity methods) considering the influence (i.e., lack of) of moisture (effects of self-desiccation), shrinkage mitigation techniques (internal curing), and improving the overall durability of HES concrete patching materials (Weiss et al., 2019). Antico et al. examined the current opening strength requirements for concrete pavements (typically a flexural strength from beams, and proposed a criterion based on flexural strength and pavement thickness. It was found that the early strength requirements could be reduced for thicker pavements (8-10 inches), and this was implemented in specifications. However, the required flexural strength for reopening thinner pavements (5 inches) was recommended to remain at 550 psi (Antico et al., 2015). A single strength metric is not sufficient for determining reopening times.

2.5.2 Texas

Research for TxDOT conducted at UT Austin examined a wide range of rapid repair materials including OPC, LMC, CAC, CSA, and alkali activated fly ash (AAFA) systems (Dornak et al., 2015). A thorough testing regime was performed for mixtures after an initial screening phase to investigate the following factors:

- Mechanical properties – compressive strength, flexural strength, modulus of elasticity, splitting tensile strength, bond strength.
- Thermal properties – robustness, coefficient of thermal expansion (CTE)

- Early-age volume change – drying shrinkage, restrained stress development, unrestrained deformation
- Calorimetry – isothermal, semi-adiabatic, temperature gradients in slabs
- Field testing – simulated bridge deck repairs, highway repair evaluation,
- Durability – freeze-thaw and salt scaling, alkali-silica reaction, sulfate attack, permeability and corrosion
- Latex was also part of their experimental investigations.

At standard temperatures, the OPC mixtures met most of the performance criteria outlined in their study. CAC mixtures were highlighted for their potential use in cold weather placements, with a peak heat flow observed in under 3 hours even at 10°C. At 10°C, significant retardation was observed for CSA with the peak heat flow in isothermal calorimetry occurring closer to 10 hours instead of less than 3 hours. However, three out of four CAC mixtures exceeded TxDOT shrinkage limits while CSA exhibited positive autogenous deformation (i.e. expansion).

Adequate resistance to chloride ingress was suggested for all materials by RCPT. However, as previously mentioned, this test may be inappropriate for alternative cements. Furthermore, one of the CSA mixtures were found to have moderate permeability after a 1-day curing period. The hydration of CSA continues to decrease porosity at later ages after the initial strength gain. The materials were all found to have sufficient salt scaling and freeze thaw durability according to ASTM C672 and C666, yet the requirements for air content and spacing factors were not met in all cases. This suggests that factors besides air content influence the durability in freezing and thawing conditions. One durability concern was that carbonation depths were significantly higher for CSA and CAC mixtures.

As part of the durability testing regime, pore solution extraction (Barneyback & Diamond, 1981), aka “pore press”, was outline in the methods, but these results were not reported. A possible explanation is that the alternative cement systems did not yield enough pore solution after a complete curing regime.

In their field study, one LMC and one CSA mixture performed the best with no signs of cracking or surface deflection after 8 months of heavy traffic. All the CSA mixtures performed well, although one exhibited transverse cracking that could have been attributed to incompatibility with the existing pavement.

Instead of selecting repair materials with the highest performance, the authors suggest aiming for a material like that of the existing substrate. For example, aggregates and the coefficient of thermal expansion could be important to consider. Different cement types each have unique advantages and disadvantages to consider in conjunction with construction requirements. From this research, no clear favorites are identified for cement types in HESC bridge overlays. However, TxDOT does provide an extensive list of commercial early-strength concrete products and their properties, which could be useful for material selection in the present investigation.

2.5.3 Virginia

VDOT has extensive experience and field data for epoxy concrete (EC), silica fume concrete (SF), latex modified concrete (LMC), and very early strength latex modified concrete (VESLMC) use in overlay applications.

The use of EC overlays is confined to applications with little to no deterioration and the expected service life is 18-22 years, as a preventative measure. Other overlay material performance was not influenced by the prior deck condition (assuming surface preparation was performed correctly). This finding corroborates with the objective of the current project seeking to establish longer lasting alternatives to PPC. VDOT experience with SF concrete includes cases of widespread cracking that are likely attributed to inadequate curing practices.

The most common overlay system employed by VDOT is LMC, which has a reported service life as high as 50 years. The main drawback of LMC is the traffic closure time of 3 days. Incorporating rapid setting cements for VESLMC is a promising alternative for repairs of high traffic volume regions. However, many cases of VESLMC use in overlays have exhibited cracking associated with poor curing timing or conditions. Weather sensitivity, restricted construction time, and the overall learning curve for contractors were cited as challenges in adopting VESLMC. Nevertheless, its use is increasing, and good performance has been seen in projects during 10+ years of service.

Early field experience with VESLMC in Virginia observed transverse cracking in overlays. The suspected cause of cracking was load reflection from the base concrete (Sprinkel, 1999) This finding could suggest that latex modification does not improve flexural strength enough for HESC overlay applications. Fibers or other reinforcement in the HESC material may be needed to achieve the required flexural strength. Map cracking from other sources have also been reported as previously discussed in section XYZ. Recommended methods for mitigating shrinkage cracking involved 1) applying a curing compound as soon as possible and 2) fogging immediately after finishing in dry conditions (Yun & Choi, 2014).

VESLMC samples from these pilot projects showed similar permeability to conventional LMC at 4 weeks and the permeability was lower at 6 weeks, 5 months, and 12 months age using the RCPT method (AASHTO T277). Cylinders were cured in molds for 24 hours then air until the time of testing. The composition of the VESLMC (from XRF) suggests that CSA is a main component (Sprinkel, 1999) This corroborates with other findings that CSA can have appreciable microstructural development after 28 days (Bescher & Kim, 2019) and demonstrates that latex can provide low permeability at earlier ages.

2.5.4 Utah

Laboratory work by Maguire and co-workers focused on developing low shrinkage mixtures with rapid strength gain, i.e. CSA. To reduce external curing requirements and expedite construction, they examined the use of lightweight aggregates to mitigate shrinkage. (Maguire et al., 2017) A relevant finding was that higher proportions of lightweight aggregate than those recommended by ESCSI may be necessary for mitigating shrinkage in HESC (for both OPC and

CSA binders used in their study) (*ESCSI Guide Specifications for Internally Cured Concrete*, 2012).

2.5.5 North Carolina

Cavalline and co-workers recently investigated internal curing for concrete pavements and included VESLMC mixtures in their study. Autogenous strain was reduced by 30 to 56 %. The mixtures also benefited from increased workability and working time due to the use of FLWA. The relatively low absorption FLWA used in the study warranted a replacement level of 50% according to ACI 308 guidelines. This was recommended over the dosage of 30-35% use by other SHAs. (Cavalline et al., 2020)

2.5.6 California

Caltrans has extensive experience using proprietary very-early strength (VES) products to replace entire highway panels. CSA products were first used by Caltrans in 1994 to repair damage from the Northridge earthquake and the successful implementation fueled continued interest (Bescher & Kim, 2019). CSA mixtures used in California gain structural strength in approximately an hour at standard placement temperatures and exhibit low shrinkage. The freeze thaw durability and chloride permeability are both satisfactory according to ASTM C666 and ASTM C1202, respectively.

“Caltrans has placed over 70,000 highway panels using CSA concrete, an equivalent of approximately 322 lane-km (200 lane-mi) since 1994. It is easy to work with and requires a minimum of equipment. It can be successfully produced at a batch plant or Volumetric mixer. It provides VES, which minimizes a project’s impact on traffic. CSA concrete provides a cost-effective solution for highway panel replacement.” (Ramseyer & Perez, 2009)

Rapid setting mixes are only permitted by Caltrans standard specifications if the environmental temperature is above 40°F. The standard specifications also state that the bridge substrate for overlays may be damp but not saturated, and in the case of magnesium phosphate cement the surface should be dry. Controlled conditions likely reduce the variability of rapid repair materials.

2.5.7 Sea-Tac

Although it is not a SHA, documented experience from Seattle-Tacoma International Airport (Sea-Tac) is include here because of geographic proximity and the use of CSA cement for high early strength. More specifically, belitic CSA (BCSA) cement – containing 30-60% C₂S – was implemented for overnight repairs starting in 1994. The concrete flexural strength was 500 psi at 4 hours and 715 psi after 28 days. After 15 years in service, the flexural strength was 1,160 psi. Because ettringite hydration is completed within the first 10 hours of hydration, it can be deduced that subsequent reactions forming strätlingite and AFm contribute to later age strength gain in BCSA cements (Bescher & Kim, 2019). The continued microstructural development of BCSA cement at later ages suggests that laboratory tests conducted in a shorter time span may underestimate durability performance.

3.0 RESEARCH METHODOLOGY

To evaluate the performance of the high early strength material, the project was performed in two phases as outlined in this section.

3.1 PHASE I

The research work done in this project followed a two-phase approach. In Phase I an initial screening was done for all materials that were selected/provided to the research team. Materials that performed well in Phase I (e.g. $f'_c > 3,000$ psi at 3 hours after casting, and with at least 15 minutes of working time for placement) were moved forward to Phase II. In Phase II more detailed investigations including curing, cracking risk, and durability were completed. An overview of the different types of materials (paste, mortar, or concrete) in each of the different Phases and tests is provided in Table 3-1.

Table 3-1 Overview of the material type used in each Phase and testing method in the research project

Phase, Task and Test	Mixture Type
Phase I Task 1a: Mixture Designs and Materials Optimization Setting Time	Paste
Phase I Task 1a: Mixture Designs and Materials Optimization Semi-adiabatic temperature rise	Paste
Phase I Task 1a: Mixture Designs and Materials Optimization Compressive Strength	Mortar
Phase I Task 1b: Concrete Materials Verification Compressive Strength	Concrete
Phase I Task 1b: Concrete Materials Verification Working Time - qualitative	Concrete
Phase I Task 1b: Concrete Materials Verification Air Entrainment	Concrete
Phase II Task 2: Curing Neutron Radiography	Mortar
Phase II Task 3: Property Development Mechanical property development	Concrete
Phase II Task 3: Property Development Activation energy	Paste
Phase II Task 3: Property Development Coefficient of thermal expansion	Concrete
Phase II Task 3: Property Development Bond	Concrete (plus a prior mortar scrub coat for some specimens indicated)
Phase II Task 4: Volume change and cracking risk Dual ring cracking risk	Mortar
Phase II Task 5: Durability Freeze-thaw	Concrete
Phase II Task 5: Durability Electrical properties and formation factor	Concrete
Phase II Task 5: Durability Critical chloride threshold	Mortar
Phase II Task 6: Field evaluation Yamhill Crossing Overlay	Concrete

In Phase I Task 1a the research team focused on pastes and mortars to provide a more expedient investigation that could take place while minimizing material waste. In Phase II most of the research was done on concrete with the exception of neutron radiography, activation energy, dual ring cracking risk, and critical chloride threshold determination. This was because each of these test methods was designed specifically to use the material type listed in Table 3-1.

3.1.1 Task 1a: Mixture Designs and Materials Optimization

The research team initially selected approximately ten (10) high early strength materials based on the literature, ODOT and other DOT experience that were of interest. After that, a survey was sent out by ODOT to potential manufacturers to determine interest in using their material as part of this study. It was found that most of the rapid set-high early strength material are formulated for rapid patch repair applications (e.g., small applications $\sim 1 \text{ ft}^3$ or less). A central question was also if certain preselected materials that were produced as “pre-bagged” products could be scaled up for use in ready-mix or mobile mix to be cast as structural bridge deck overlays. No companies expressed interest in scaling these products to ready-mix or mobile mix. As a result, the research team selected six out of the original ten materials for initial screening. The following evaluations were done on those six mixtures utilizing either paste, or mortar: setting time, semi adiabatic temperature, and compressive strength. Task 1b: Concrete Materials Verification

In Task 1b, the research team took the findings from Task 1a to inform trials done on full-scale concrete mixtures. The key decision-making data that was evaluated in Phase I are shown in Table 3-2. Note that the estimation of working time was qualitative due to how rapidly the workability of these mixtures deteriorates. The team focused on mixtures that had a working time of at least 15 minutes.

Table 3-2 Phase I Task 1b Initial materials screening test method list

Test	Test Time (after casting)	Test method	Number of samples	Criteria	Notes
Compressive strength	3, 6*, 24 hours	ASTM C39-20	3	$f'_c \geq 3000$ psi at 3 hrs	Pad caps were used
Working time	varies	-	-	-	qualitative
Air entrainment	<15 minutes	ASTM C231/C231M-17a	1	$5.5 \pm 1.5\%$	-

*6 hours used on samples with high early strength portland cement only

Materials not meeting certain performance expectations, e.g., $f'_c \geq 3,000$ psi at 3 or 6 hours after casting, may be eliminated from Phase II eligibility. The number of materials selected for Phase II will be determined following the outcome of this task.

3.2 PHASE II

In Phase II the following parameters were evaluated for the HESC materials selected from Phase I:

1. Task 2: Curing
2. Task 3: Property Development

- a. Mechanical property development
 - b. Activation energy or maturity
 - c. Coefficient of thermal expansion
 - d. Bond
- 3. Task 4: Volume change and cracking risk
- 4. Task 5: Durability
 - a. Freeze-thaw
 - b. Electrical properties and formation factor
 - c. Critical chloride threshold
- 5. Task 6: Field evaluation

Table 3-3. Summary of all testing done and mixture type (e.g. paste, mortar and/or concrete)

Phase, Task and Test	Mixture Type
Phase I Task 1a: Mixture Designs and Materials Optimization Setting Time	Paste
Phase I Task 1a: Mixture Designs and Materials Optimization Semi-adiabatic temperature rise	Paste
Phase I Task 1a: Mixture Designs and Materials Optimization Compressive Strength	Mortar
Phase I Task 1b: Concrete Materials Verification Compressive Strength	Concrete
Phase I Task 1b: Concrete Materials Verification Working Time - qualitative	Concrete
Phase I Task 1b: Concrete Materials Verification Air Entrainment	Concrete
Phase II Task 2: Curing Neutron Radiography	Mortar
Phase II Task 3: Property Development Mechanical property development	Concrete
Phase II Task 3: Property Development Activation energy	Paste
Phase II Task 3: Property Development Coefficient of thermal expansion	Concrete
Phase II Task 3: Property Development Bond	Concrete (plus a prior mortar scrub coat for some specimens indicated)
Phase II Task 4: Volume change and cracking risk Dual ring cracking risk	Mortar
Phase II Task 5: Durability Freeze-thaw	Concrete
Phase II Task 5: Durability Electrical properties and formation factor	Concrete
Phase II Task 5: Durability Critical chloride threshold	Mortar
Phase II Task 6: Field evaluation Yamhill Crossing Overlay	Concrete

3.3 MATERIALS

3.3.1 Cementitious Materials

The cementitious materials used in this research project are listed in Table 3-4 with their chemical compositions determined from X-ray fluorescence (XRF). Calcium Sulfoaluminate cements- CSA1, CSA2, PrCSA1 and PrCSA2 were manufactured by CTS. CSA1 and CSA2 did not contain any admixtures while PrCSA1 and PrCSA2 contained built-in chemical admixtures such as air entrainment agents, superplasticizer, and/or polymer modifiers. Type I and Type III OPC were manufactured provided by a regional cement supplier.

Table 3-4 Chemical composition analysis by X-ray fluorescence

Chemical composition (%/wt)	CSA1	CSA2	PrCSA1	PrCSA2	Type III OPC	Type I OPC
Na ₂ O	0.108	0.109	0.256	0.243	0.210	0.271
MgO	0.841	0.954	0.828	0.828	1.199	0.928
Al ₂ O ₃	13.927	12.456	13.623	13.711	3.434	4.287
SiO ₂	13.567	14.504	13.451	13.395	20.014	17.837
P ₂ O ₅	0.075	0.081	0.078	0.078	0.116	0.039
SO ₃	14.777	12.979	14.799	14.911	2.87	2.64
Cl	0.008	0.013	0.012	0.015	0.006	0.014
K ₂ O	0.403	0.407	0.393	0.389	0.434	0.253
CaO	47.56	49.73	46.555	46.804	64.893	62.601
TiO ₂	0.707	0.641	0.696	0.699	0.307	0.266
Mn ₂ O ₃	0.033	0.047	0.033	0.032	0.054	0.03
Fe ₂ O ₃	1.06	1.481	1.039	1.048	3.148	3.056
ZnO	0.005	0.006	0.005	0.004	0.013	0.03

Two blended cement compositions prepared with Type I and Type III Portland cement with an accelerator made of calcium aluminate were also included in the study. The calcium aluminate-based accelerator was manufactured by IMERYS. The chemical composition of the calcium aluminate cement was as follows: Al₂O₃= 30.0-37.0%, CaO= 35-37%, SO₃= 14.0-20.0%, SiO₂ ≤ 6.0%, and Fe₂O₃ ≤ 4.0%. The specific gravity values of the selected cements were as follows: CSA1= 3.00, CSA2= 3.00, PrCSA1= 3.00, PrCSA2= 3.00, CAC= 2.94, Type III OPC = 3.16, Type I OPC = 3.15.

3.3.2 Admixtures

The chemical admixtures used in this project are listed in Table 3-5. It is important to note the recommended dosage of the admixture was documented in the producer's product sheet which also highlighted that the dosage can be adjusted based on the specific application.

Table 3-5 Chemical admixtures and their recommended dosage based on their product datasheet.

Chemical Admixture	Type	Manufacturer	Recommended dosage by manufacturer
MasterGlenium 7925	High-range water reducer	Masters Builders Solution	2-15 fl oz/100 lbs of cementitious materials
ADVA® Cast 600		GCP	3 to 6 fl oz/100 lbs of cementitious
MasterAir AE200	Air-entraining admixture	Masters Builders Solution	No standard dosage
DARAVAIR® 1000		GCP	No standard dosage, typical ½ to 3 fl oz/100 lbs
DARASET® 400	Set-accelerating admixture ASTM C494 Type C	GCP	Typical addition levels range from 10 to 60 fl oz/100 lbs of cement
Citric acid	Retarder	Fisher Scientific	No standard dosage
Gypsum	Setting control	Fisher Scientific	No standard dosage

3.3.3 Aggregates

Table 3-6 shows the properties of coarse and fine aggregate used in this project. The sourced aggregates were local aluminosiliceous river gravel and sand.

Table 3-6 Properties of coarse and fine aggregate (as received). Here, SSD= Saturated Surface Dry

Aggregate type	Specific Gravity Dry	Specific Gravity SSD	Specific Gravity Apparent	Absorption Capacity (%)	Fineness modulus	Dry rodded unit weight (lb/ft ³)
Local fine aggregate	2.51	2.48	2.71	3.20	2.96	NA
Local coarse aggregate (3/4 in. MSA)	2.54	2.62	2.75	2.95	NA	104

Figure 3-1 shows the particle size distribution of the coarse and fine aggregate. The particle size distribution data was obtained from the supplier.

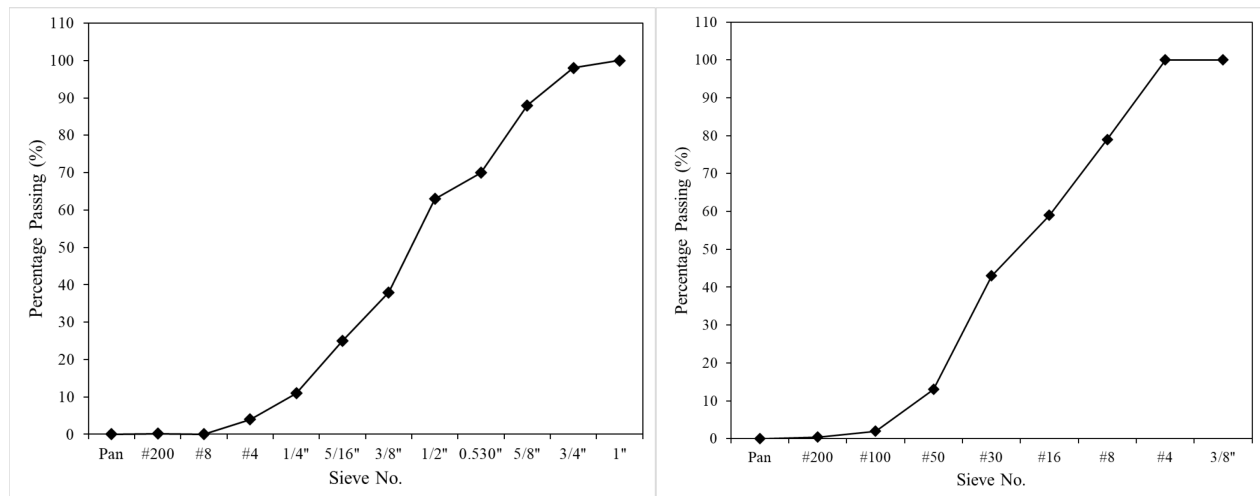


Figure 3-1 Particle size distribution of coarse aggregate (top) and fine aggregate (bottom) provided by the supplier

3.4 METHODS

3.4.1 Phase I

3.4.1.1 Task 1a: Mixture Designs and Materials Optimization

Setting Time

ASTM C191-21 “Standard Test Methods for Time of Setting of Hydraulic Cement by Vicat Needle” was used to measure the setting time of cement paste samples (ASTM C191, 2021). An automatic Vicatronic apparatus was used to measure the data.



Figure 3-2 Setting time for cement paste by an automatic Vicatronic apparatus.

Table 3-7- shows the test matrix for setting time measurements of initially screened high early strength cement. A water to cement ratio of 0.46 was used. In the case for CAC-Accelerated OPC and T3A mixtures, the water to cement ratios were selected based on manufacturer recommendation and previous literature (Todd et al. 2017). The cement paste was mixed following the procedure described in ASTM C305. The penetration reading was collected at two-minute intervals beginning immediately after molding the specimens and data were collected until final set.

Table 3-7 Test matrix for setting time, semi-adiabatic temperature, and mortar compressive strength measurement.

Samples	Type I/II OPC Replacement %	w/cm	Retarder/Accelerator
CSA1	N/A	0.46	0.00-0.35% by mass of cement ³
CSA2	N/A	0.46	0.00-0.35% by mass of cement ³
PrCSA1	N/A	0.46	0.00-0.35% by mass of cement ³
PrCSA2	N/A	0.46	0.00-0.35% by mass of cement ³
CAC-Accelerated OPC	70-90%	0.35	0.00-0.35% by mass of cement ³
CSA-OPC	60-85%	0.42~0.46	0.00-0.35% by mass of cement ³
T3A	N/A	0.35	1.82-3.65 % mass of cement (Daraset 400)

N/A = not applicable.

Semi Adiabatic Temperature

Figure 3-3 shows the testing procedure used to measure the temperature rise in select high early strength materials. the same test matrix as shown in Table 3-7 was used.

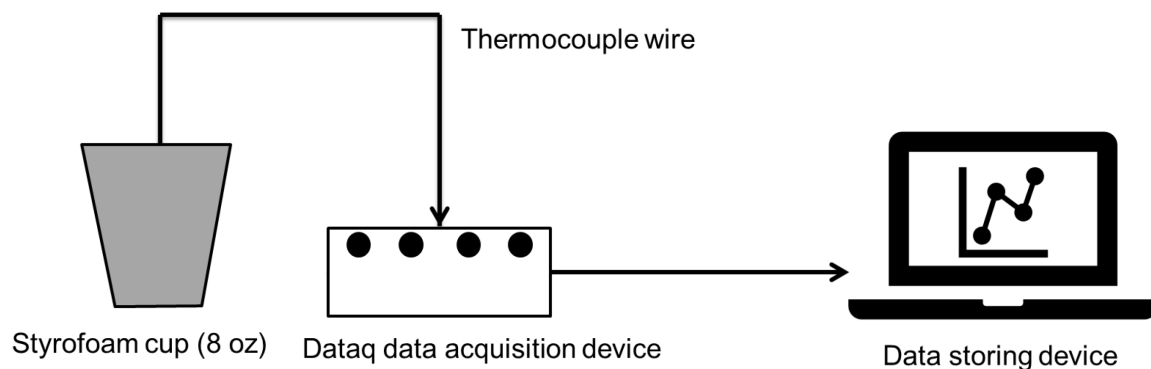


Figure 3-3 Schematic diagram of semi adiabatic temperature determination of high early strength cement.

After mixing the cement paste specimens following ASTM C305, the paste was separated and used to measure the setting time and the temperature rise. One end of a K-type Teflon coated thermocouple wire was attached to a data acquisition software and the other end was inserted into the cement paste. The data were collected for 6 to 12 hours. The total time to reach the maximum temperature is defined as the time the cement first contacts water until the time the maximum temperature occurs.

Compressive Strength

The compressive strength of the mortar specimens was tested according to ASTM C109/C109M-21. ASTM C305 was used to mix mortar specimens. The specimens were tested at 3, 6 and 24 hours after introducing water to the cement. Six-hour testing was

³ Depending on workability and setting time

only done for the samples made with high early strength ASTM C150 Type III cement (T3A is the sample identifier). Local river sand was used as the fine aggregate. Initially 0.10% citric acid was used as a retarder in all mixtures except T3A.

3.4.1.2 Task 1b: Concrete Materials Screening

In Task 1b, the research team took the findings from Task 1a to perform assessments of concrete mixtures.

Mixture Design

A typical ODOT HPC (high performance concrete) mixture design for bridge decks was used as a starting point to design the concrete mixture for HESC for this project. The target compressive strength for the HESC mixture was ≥ 3000 psi (20 MPa) in three to six hours (Table 3-2). The total cementitious material content was 658 lbs/yd³ and the water to cementitious materials ratio was adjusted in the trial mix to achieve the target compressive strength. Table 3-8 shows the mixture proportioning for trials. The volume of total material was kept constant (1.1 ft³) during all trials.

Table 3-8 Concrete mixture proportioning (trial mixtures)

Mix ID	Cement (lb/yd ³)	w/c [‡]	Coarse aggregate (lb/yd ³)	Fine aggregate (lb/yd ³)	Admixture*
CSA	658	0.46, 0.38	1601	1267	AE, HRWR, R, and LLPL
PrCSA1	658	0.46, 0.38	1601	1267	AE
PrCSA2	658	0.46, 0.38	1601	1267	-
CSA- OPC*	526-132	0.41, 0.37	1601	1288	AE, HRWR, R
T3A	658	0.35	1601	1343	AE, HRWR, Gypsum, SAA

*Admixture dosage was adjusted to control setting, workability, target air content, and compressive strength requirement

[‡] w/c was adjusted to satisfy the target compressive strength.

* 80% OPC was replaced by CSA cement

Here, AE= Air entraining admixture, HRWR= High Water Reducing Admixture, LLP= Liquid low-P admixture, R= Retarder, SAA= Set Accelerating Admixture.

All HESC mixtures tended to set very quickly. As a result of this early set the following modified concrete mixing process was used:

1. Add 75% of aggregates into the drum mixer.
2. Add 80% of batch water with admixture in the drum and mix for 30 seconds.

3. Add cement and the remaining batch water.
4. Mix for 90 seconds (lower or adjust drum for optimal mixing).
5. Stop mixer and add remaining aggregates within 30 seconds.
6. Begin mixing and after 1 minute of mixing, stop mixer and scrape any buildup inside the drum mixer within 15 to 30 seconds.
7. Start mixer and mix for an additional 2 minutes.
8. Discharge the mixture.

For adding or mixing with different admixtures the following precautions were maintained-

- Where AE and HRWR were both used, it was ensured that the admixtures were sourced from the same manufacturer.
- Air entraining admixture was directly added to the aggregate prior to mixing.
- HRWR, R, and LLP were added to the mixing water.
- Gypsum was added to the cement and mixed for 30 seconds before adding in the mixer.
- The set accelerating admixture was added in the last minute of mixing before discharge.

Fresh properties

Slump, workability, air content, and unit weight were measured to determine the fresh characteristics of all trial mixtures. There were no specified values for target slump or unit weight. However, the target air content was 5.5 ± 1.5 %. A pressure air meter was used to determine the air content by following ASTM C231-09. Slump was measured according to ASTM C143-20, and unit weight was measured according to ASTM C138-17.

Compressive strength

Compressive strength was measured according to ASTM C39-20. For each mixture, nine 4x8 inch cylindrical samples were cast then cured in sealed conditions in the mixing room (temperature 73°F and relative humidity $\geq 50\%$) until the age of testing. All samples were tested at 3, 6, and 24 hours. The target average compressive strength requirement criterion was ≥ 3000 psi in 3 hours. Mixtures comprised primarily of ASTM C150 Type III cement were also evaluated at 6 hours as there was interest in having this as an option for ODOT if 3,000 psi could be met by 6 hours.

3.4.2 Phase II

3.4.2.1 Task 2: Curing

The HESC mixture compositions approved from phase I task 1b were used to prepare mortar specimens with local fine aggregate to evaluate the effects of different curing practices and identify the minimum curing duration for each mix. In addition, the effect of substrate on overall bound water profile was also evaluated using Neutron Radiography (NR). For each mixture, four samples with the following substrate conditions were prepared- i) no substrate ii) substrate conditions at 73°F-50%RH for ≥ 6 months iii) substrate conditions at 73°F-80%RH for ≥ 6 months, and iv) substrate condition with lime solution. Mortar specimens for substrate were prepared with Type I Portland cement with a w/c of 0.50. For the substrate, six prismatic specimens (3x3x12 inch) were prepared and kept in 73°F and 50%RH climatic chamber for 28 days. After 28 days, the specimens were sliced into 3x3x1-inch dimensions and stored in respective different RH conditions until the day of casting samples with overlay. Table 3-9 shows the mixture proportions used for mortar specimens of HES cement. All mixture proportion was scaled based on ODOT's HPC mixture proportions used for bridge design.

Table 3-9 Mixture proportion for mortar specimens (in SSD condition)

Mixture	w/c	Cement, kg/m ³ (v*)	Sand, kg/m ³ (v)	Water, kg/m ³ (v)	Admixture ^a Retarder (%)	Admixture ^a Superplasticizer (%)	Admixture ^a Set accelerating admixture and gypsum (%)
CSA	0.38	1119 (0.22)	2200 (0.53)	425 (0.25)	0.20	0.40	N/A
PrCSA1	0.38	1119 (0.22)	2200 (0.53)	425 (0.25)	N/A	N/A	N/A
PrCSA2	0.38	1119 (0.22)	2200 (0.53)	425 (0.25)	N/A	N/A	N/A
CSA- OPC	0.37	1119 (0.22)	2235 (0.53)	415 (0.25)	0.20	0.40	N/A
T3A	0.35	1197 (0.22)	2200 (0.53)	420 (0.25)	N/A	0.50	3.65 and 1.00

^a Based-on cement weight; Citric acid was used as a retarder and a high range water reducing admixture was used as superplasticizer.

*Volume proportion

NA= Not Applicable

The water to cement ratio was adapted from the finalized mixture design obtained from the results of phase I task 1b. ASTM C305-20 was used for mechanical mixing and a

planetary mixer with a capacity of mixing 0.67 ft³ was used to mix the material. All admixtures, except the set accelerating admixture, were added with the mixing water before adding any dry material. The set accelerating admixture in T3A mix was added before the final 60 seconds of mixing. After mixing, specimens were cast in 3x3x3 inch molds to evaluate curing efficiency. The thickness of the overlay HES material with substrate was also 3-inch where top 2-inch was overlay and bottom 1-inch was substrate. After casting, the specimens were subjected to different curing practices. Figure 3-4 shows the experimental setup used in this project to evaluate the curing efficiency of HES cements by NR. All the specimens, except T3A mixture, were demolded after 3 hours. All T3A mixture specimens were demolded after 6 hours, since the T3A mixture gained >3000 psi compressive strength in 6 hours. The demolding time was selected based on the target compressive strength criteria. After demolding, an aluminum tape was used to seal all the sides of the specimens except the top surface and then kept in 73°F-50%RH climatic chamber for 28 days to ensure one directional drying. After 28 days, samples were taken out from the climatic chamber and a 1-inch-thick slice from the middle of the specimen was cut using a wet blue saw. For each mixture, a total of 16 samples were sliced. The sliced specimens of CSA, PrCSA1, PrCSA2, CSA-OPC mixture were then dried at 104°F for 24 hours, to minimize the decomposition of ettringite, before taking images by neutron radiography. However, the T3A specimens were dried at 105°C for 48 hours before taking NR images. The NR images were collected by the Neutron Radiography Facility (NRF) located at the Radiation Center in Oregon State University (OSU).

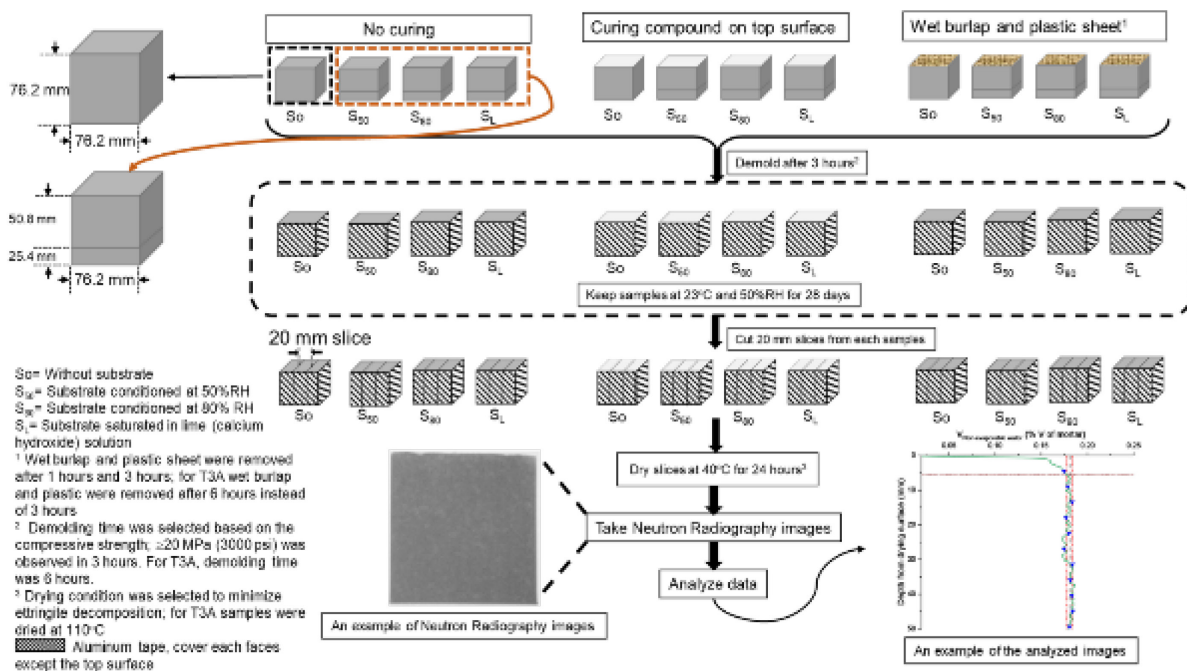


Figure 3-4 Experimental setup to evaluate the effect of external curing on high early strength cement by neutron radiography.

Determination of CAZ

The neutron radiography images were collected by following the procedure developed by Moradillo et al. (Moradillo, 2018). ImageJ software was used to analyze the images to determine the volume of non-evaporable water. The CAZ was calculated from the top surface where the coefficient of variation in average volume of non-evaporable water was $\leq 2\%$. The following Equation (3-1) was used to calculate the volume of non-evaporable water from the NR images)-

$$V_{non-evaporable\ water} = - \left[\frac{1}{\mu_w} \right] \left[\frac{\ln \frac{I_o}{I_T}}{\chi_s} - \mu_a V_a - \mu_c V_c \right] \quad (3-1)$$

Where, $V_{non-evaporable\ water}$ is the volume of non-evaporable water at different depth of the specimen and:

$\mu_w = 0.0048\ \text{inch}^{-1}$, attenuation coefficient of water.

I_o = Intensity of the original beam

I_T = Intensity of the beam transmitted through the sample with thickness T,

$\mu_a = 0.0006\ \text{inch}^{-1}$, attenuation coefficient of the fine aggregate,

V_a = volume fraction of fine aggregate,

μ_c = attenuation coefficient of the cement. Here the attenuation coefficient of the cement was as follows- CSA = $0.00055\ \text{inch}^{-1}$, PrCSA1 = $0.00072\ \text{inch}^{-1}$, PrCSA2 = $0.00079\ \text{inch}^{-1}$, CSA-OPC = $0.00057\ \text{inch}^{-1}$, and OPC type III = $0.00058\ \text{inch}^{-1}$

V_c = volume of the unhydrated cement.

Determination of porosity

To determine the porosity, after taking the NR images of the dry specimens the samples were vacuum saturated by following AASTHO TP135-20. After that, NR images of the saturated specimens were collected. The porosity was determined by subtracting the beam intensity collected from the saturated specimens to the intensity collected from the dry specimens. The following equation (3-2) was used to determine the porosity from the NR images-

$$Porosity\ (\%) = \frac{\left[\frac{\ln \frac{I_{OD}}{I_{Sat}}}{\mu_w} \right]}{t} \times 100 \quad (3-2)$$

Where:

I_{OD} = intensity of NR beam in oven-dried specimens.

I_{sat} = intensity of NR beam in vacuumed saturated specimens.

μ_w = attenuation coefficient of water in inch¹.

t = sample thickness in inch.

3.4.2.2 Task 3: Mechanical Property Development

Activation Energy

A CalMetrix isothermal calorimetry was used to measure the heat flow and cumulative heat release of high early strength cement pastes at four different temperatures: 50°F, 73°F, 100°F, and 122°F. An internal mixer designed for mixing cementitious materials inside the calorimeter was used. Unlike Portland cement, the rate of heat liberation during the first 30 minutes to 1-hour was significantly higher in these high early strength cementitious systems. As a result, internal mixers were used to capture the heat release during the first 30 minutes to 1-hour of hydration. At first, dry cement powder was placed in a plastic vial and the internal mixer was attached subsequently, which contained water inside a syringe system. After that, the internal mixer with all the raw material was kept inside the calorimeter for at least 3 hours to equilibrate with the calorimeter temperature. No data was collected during this equilibration time. After 3 hours, the samples were mixed for approximately one minute and the heat release was recorded at one-minute intervals up to 48 hours for (73, 100, and 122°F) and 138 hours (50°F). Details of isothermal calorimeter testing are shown in Table 3-10.

Table 3-10 Test matrix for isothermal calorimeter testing. Here, N/A means not applicable.

Mixture ID	Tested temperatures at IC (°F)	Cement (g)	w/c	External Admixture
CSA	50 ^a , 73, 100, 122	3.5	0.5	N/A
PrCSA1	50 ^a , 73, 100, 122	3.5	0.5	N/A
PrCSA2	50 ^a , 73, 100, 122	3.5	0.5	N/A
CSA-OPC	50 ^a , 73, 100, 122	3.5	0.5	N/A
T3A	50 ^a , 73, 100, 122	3.4	0.5	1.00% gypsum and 3.65% set accelerating admixture was used

^aThe testing was aborted after 138 hours due to the fluctuations observed in the temperature in calorimeter 50~54°F. Except 50°F, the testing duration for other temperatures was 48 hours

Modified ASTM C1074-17 was used to determine the activation energy (E_a) using heat evolution data from isothermal calorimetry rather than strength data of mortar specimens (Poole et al. 2007). The following procedure was used to calculate the E_a values-

- I. Cumulative heat evolution data was collected from the isothermal calorimeter at 50°F, 73°F, 100°F, and 122°F.
- II. The cumulative heat data was then fit to the following (3-3) three-parameter exponential function (Liao, 2021)-

$$H = H_u \cdot e^{-\left(\frac{\tau}{t}\right)^\beta} \quad (3-3)$$

here H= cumulative heat (J/g of cement), H_u = ultimate cumulative heat (J/g of cement), τ = hydration time parameter in hours, β = hydration shape parameter. The value of H_u , τ , and β was calculated from the fitted exponential function.

- III. After that, the reaction rate, $k = \frac{1}{\tau}$ was calculated in hours.
- IV. Finally, $\ln \frac{k_c}{k_{ref}}$ versus $\frac{1}{T_c} - \frac{1}{T_{ref}}$ was plotted for the tested temperature. The slope of the best fitted line of the plotted figure was multiplied with the gas constant (R) to determine the activation energy E_a as shown in the following equation (3-4 (Jonathan L. Poole, 2007; Liao, 2021)-

$$E_a = - \frac{\ln \frac{k_c}{k_{ref}}}{\left(\frac{1}{T_c} - \frac{1}{T_{ref}}\right)} R \quad (3-4)$$

Where:

E_a = Activation energy (J/mol)

k_{ref} = reaction rate at reference temperature

T_{ref} = reference temperature

k_c = reaction rate at temperature of concrete T_c

R= the gas constant is 8.31 J/(K·mol)

Coefficient of Thermal Expansion (CTE)

The CTE for each of the concrete mixtures in the study was determined according to AASHTO T336. A frame was constructed using a stainless-steel plate, two threaded stainless-steel rods, and stainless-steel lug nuts to fix the top plate of the frame in place. A moisture resistant, high resolution linear voltage difference transducer (LVDT) was used to measure the displacement. A digital, temperature controllable water bath was used to expose the concrete samples to temperature cycles from 50°F to 122°F. LVDT measurements were recorded when the difference between values was not more than 500 μ inch different at 10-minute intervals. Three samples for each concrete mixture were tested.

A stainless-steel calibration specimen, certified by an independent testing laboratory, with a 7-inch length, 4-inch diameter, and CTE of 0.5 μ strain/°F was used in determining the correction factor for calculation. The 7-inch verification specimen was cut from a 4-inch diameter piece of bulk aluminum and sent to a certified lab for calibration and had a

CTE of 0.7 $\mu\text{strain}/^{\circ}\text{F}$. See Table 3-11 for calibration values and specimen details for calibration test specimens.

Table 3-11. Length, Diameter, and CTE of reference and calibration specimens

	Stainless Steel	Aluminum
Length (inch)	6.96	6.98
Diameter (inch)	3.99	4.00
CTE ($\mu\text{strain}/^{\circ}\text{C}$)	0.50	0.70

All samples were placed in a saturated lime bath 24 hours after casting and remained there for 28 days until testing. Two days prior to testing the ends of the cylinders were saw cut to achieve a sample length of 7-inch as required in AASHTO T336. It is known that the degree of saturation (DoS) has a large impact on the value of a concrete's CTE expansion (Jeong and Zollinger, 2012). The DoS was determined for one of the samples after measuring the CTE of each concrete and the other 2 samples were used to determine the concrete strength at 28 days.

Along with the laboratory samples, field samples were cored from a three-year-old bridge deck constructed with the PrCSA1 cement used in this study. Due to field coring, depth of the sample lengths varies from 3 to 6-inches after saw cutting for flat ends. An independent correction factor was used for these shorter samples which use an aluminum cylinder cut from the same bulk steel that the calibrated verification sample was from. An assumption was made that because it is from the same bulk metal that the shorter specimen has the same CTE value as the taller specimen.

Bond Testing

The tensile bond strength of the overlay concrete bonded to the HPC substrate was determined by following ASTM C1583-20. Previously cast concrete slabs, measuring 3.3x3.3x0.40 ft, were reused from a previous project. The mixture designs for these repurposed slabs were representative of the HPC mixture that also used in this current project and outlined in the previous section. A bond fracture occurring either at the bond interface or in the substrate must reach or exceed 174 psi to be deemed passing according to Oregon DOT standards (ODOT, 2021).

To prepare the slabs for overlay the following process was used. First, the slab was cut in both directions of the vertical surface at a 6-inch spacing and to an approximate depth of 5-inch. Next, a 25-kipf demolition hammer and 10-inch chisel was used to chip out the squares from saw cutting. A final working space of 2.5 x 2.5 ft² was achieved and used as the substrate for the bond performance testing. The saw cut slab and roughened and cleaned surface are in Figure 3-5. This was done as a surrogate approach to provide a surface preparation similar to hydrodemolition in the field. The use of a demolition hammer may have introduced micro cracks in the concrete and does not directly replicate hydrodemolition. A quote was obtained to have a company come and do hydrodemolition at the university but the cost was prohibitive given the research budget.



Figure 3-5. Slab saw cut at a six-inch spacing (left) and removed with demolition hammer (right)

Prior to casting, the roughened surface was brushed with a wire brush to loosen dust and remove as much micro cracking as possible. A vacuum was then used to remove the dust and any remaining concrete pieces. Wood formwork was constructed of 2-inch thick by 4-inch-wide boards were used as the mold for casting (Figure 3-6).

The substrate surface was conditioned to a saturated surface dry (SSD) state before the overlay material was placed. Half of the slab was treated with a mortar scrub coat made with a 0.45 w/c ratio and using the same cement as the overlay material being tested. The scrub coat had a sand to cement ratio of 2.3 and was made with the same local sand as the concrete. The other half of the slab did not receive the scrub coat and was only cleaned and conditioned to SSD. Samples were sawcut through the entire thickness of the overlay material and at least 0.2-inch into the substrate material. Overlays were poured to 1.2 to 1.6-inch and a standard core depth of 2.0-inch was made to ensure an appropriate depth was cut, with an additional 0.2-inch into the substrate. The diameter of the coring bit was 1.6-inch which resulted in a nominal sample area of 2.5-in² used for calculating the force of the bond strength.



Figure 3-6. Slab in saturated surface dry (SSD) condition and frame for pouring in place.

The bond strength was evaluated at 24 hours, 72 hours, and 14 days with a goal of 174 psi. Overlays with and without a scrub coat were cored for four total samples at 24 and 72 hours with a spacing of 4-inch from edge to edge. Three samples were taken at 14 days in a portion of the slab with adequate space to support the footprint of the pull off tester.

A commercially available bond pull-off tester was used to evaluate the tensile strength of the overlay to the substrate. The 2-inch steel disks were fixed to the concrete using a 5-minute epoxy with a cured strength of 3626 psi. Testing occurred in a room that was 73°F and 40% RH. Epoxy was allowed to cure for at least 12 hours prior to testing. The overlay surface was cleaned, roughened to increase adhesion to the steel disk, and any water remaining from coring was dried with a heat gun to remove moisture that could impact the epoxy curing.

The most aggressive field curing conditions were trying to be replicated. To allow for the fastest construction, no curing condition was implemented in bond testing of all materials. After casting, the concrete was left uncovered and in the wood framing for six hours before being demolded. Once the frame was removed, the material was cured in ambient air conditions (73°F and 40% RH) until testing. Coring was done approximately seven hours after casting.

The strength difference at each interval of time, along with surface preparation was evaluated for statistical significance following a two-sample t-test with unequal variances. This was to show if there is any benefit in applying a scrub coat or if it is more advantageous to simply cast directly onto an SSD substrate.

3.4.2.3 Task 4: Volume Change and Cracking Risk

The volume change and cracking risk of HESC mixtures and the HPC mixture were evaluated following AASHTO T363-17. Four (4) days after casting the mixtures they were artificially cooled (to further induce shrinkage stresses) to 32 °F. Specimens were continuously monitored for temperature and strain. The data was analyzed following AASHTO T363-17 and a cracking temperature was identified for any mixtures that cracked either prior to, or during the artificial cooling period. Typically, a lower cracking temperature (or the lack of any cracking) indicates a mixture with greater resistance to cracking risk in the field. Figure 3-7 shows a schematic of the dual ring test geometry.

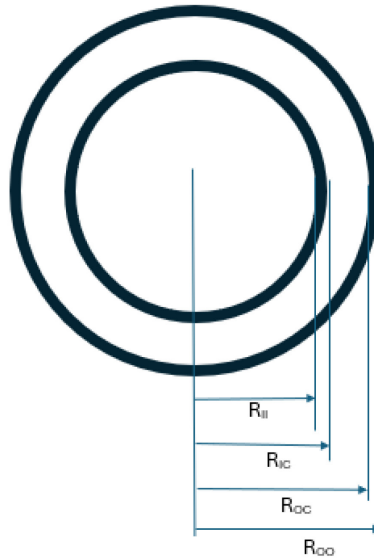


Figure 3-7. Dual ring geometry of dual ring test setup

The dimensions of the ring used R_{OC} measures nominally (converted from SI units) 7.99 in., R_{IC} measures nominally 6.5 in., R_{II} measures nominally 5.75 in. and R_{OO} measures nominally 8.74 in. This provides a nominal 1.5 in. thick specimen.

3.4.2.4 Task 5: Durability

Freeze-Thaw

Freeze-thaw performance was evaluated according to ASTM C666 Procedure B. Three 3x4x16 inch prisms were cast in accordance with ASTM C309-19. At the time of mixing, the air content was recorded for each concrete following ASTM C231. Due to the rapid setting behavior of the materials (≤ 30 minutes) the slump of the concretes could not be determined. Additionally, because of this behavior, the slump at the time of discharge would not be a good comparison to the workability of the material 10 minutes later.

A set of three 4x8 inch cylinders were cast at the same time for compressive strength at 28 days. Samples containing CSA cement were demolded at 24 hours and cured in ambient laboratory conditions for 25 days, at which time they were submerged in tap

water for the remaining period until 28 days, based on manufacturer recommendations. OPC samples were demolded at 24 hours and cured in a saturated lime bath at 73°F for 28 days. Prior to exposure to freeze-thaw cycles, after 28 days the concrete prisms were conditioned in the freeze-thaw machine's water bath at 37.4°F for 2 hours. All freeze-thaw samples and cylinders had a targeted air of 7%.

A commercially available chamber-style (Scientemp) freeze-thaw machine was used to expose the samples to 6-hour cycles with termination at 300 cycles of freezing and thawing. The transverse frequency was measured before exposure and at intervals not more than 36 cycles between each measurement. To prevent drying and reduce temperature change during measurements, samples were transported wrapped in plastic sheeting. Three samples were cast from each concrete mixture and each sample had the transverse frequency measured three times at each measurement interval.

Electrical Properties and Formation Factor

Three concrete cylinders were cast for each concrete mixture in accordance with ASTM C305-20. In the case of mixing the T3A samples, accelerator was added in the last 60 seconds of mixing. They remained in a sealed condition for 28 days in a room maintained at standard laboratory temperature (73°F) and humidity ($\geq 50\%$). After 24 hours, the cylinders were removed from the molds and three surface resistivity measurements were recorded equidistant apart for each sample and the temperature recorded. The samples were then placed back in their respective molds and carefully resealed. At 28 days the surface resistivity was once again measured, and the samples were also used to determine bulk resistivity following procedures in literature.

After the measurement in sealed condition, the average degree of saturation (DoS) was determined for each sample to determine the saturated formation factor of different concrete mixtures. The initial sample mass was recorded before placing samples in a 110°C oven until 2 consecutive mass measurements 24 hours apart were identical. Once dried, the samples were vacuum saturated with saturated calcium hydroxide to match the pH of the CSA pore solution, and the saturated mass was recorded. With DoS, the saturated formation factor is calculated by performing moisture corrections.

The determination of the pore solution resistivity for the calculation of the formation factor is done on paste samples. Pore solution extraction was done in 3 hours, 24 hours, and 28 days on paste samples that were stored in a sealed condition for the HESC cements. Pore solution from T3A and HPC pastes were extracted at 24 hours, 72 hours, and 28 days. The solution was filtered, and the ion compositions were determined using X-ray fluorescence (XRF). Following XRF, the pH was measured for each duration of curing. Lastly, the pore solution conductivity, which is the inverse of its resistivity, was measured.

Critical Chloride Threshold

The Critical Chloride Threshold (OC_{crit}) test, developed at Oregon State University, was used to determine the critical chloride threshold (Cl_{crit}) for each cement in the study (Trejo et al., 2021). These investigations were performed: (i) assessing the influence of cement type on critical chloride threshold and (ii) assessing the influence of different

cement types as anodic and cathodic environments on the corrosion rate. Figure 3-8 shows the test matrix. Samples were prepared as mortars using graded standard sand combined with each of the cements in the study.

All mortars were prepared with a w/cm of 0.38 and a sand to cementitious ratio of 2.3. This allowed for only one variable in the testing: the amount of superplasticizer used to achieve the desired flow. A modified flow test reported by Vaddey et. al. (2021) was used in this study. All steel used in the specimens was cleaned of residual oil and debris using xylene. The 5.5-inch steel samples (for the anodes) were cleaned in an ultrasonic bath and the 12-inch steel samples for the cathodes were cleaned by scrubbing with a soft bristled brush as these pieces were too large for the bath. Clean nitrile gloves were used to place the prepared steel into the molds prior to mixing.

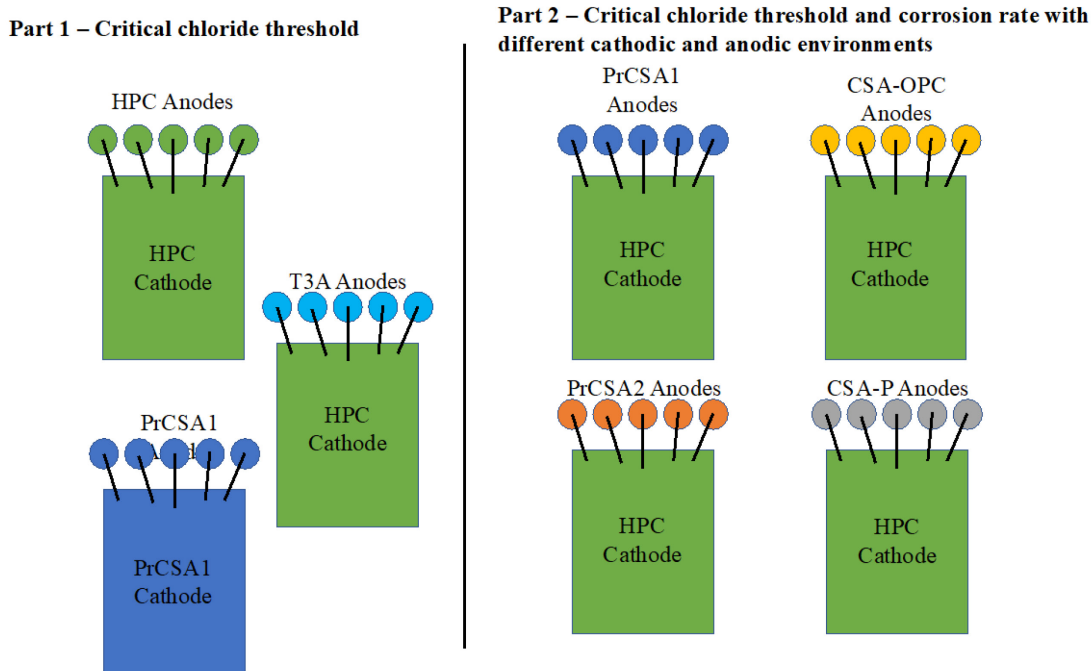


Figure 3-8. Test matrix for critical chloride threshold and corrosion rate

As shown in Figure 3-8, the PrCSA1 cement was investigated in part one. This involved both the anodic and cathodic environment being made of a mortar using the PrCSA1 cement. The reference HPC and the T3A were also evaluated in part one. In part two all cements in the study were cast as the anodic environment and the cathodic environment was always made of the HPC mortar. This setup using the HPC mortar represents a bridge deck where the lower substrate would have existing HESC materials, and the overlay would have the HESC. Both anodes and cathode specimens were cured for 28 days. Samples made with CSA cement were cured for 26 days at 73°F and ambient humidity. After 26 days, the specimens were soaked in tap water for two days before beginning the exposure for the critical chloride testing. The HPC and T3A accelerated samples were submerged in saturated lime water for 28 days.

Figure 3-9 shows the reinforcing bar set up in the mold.

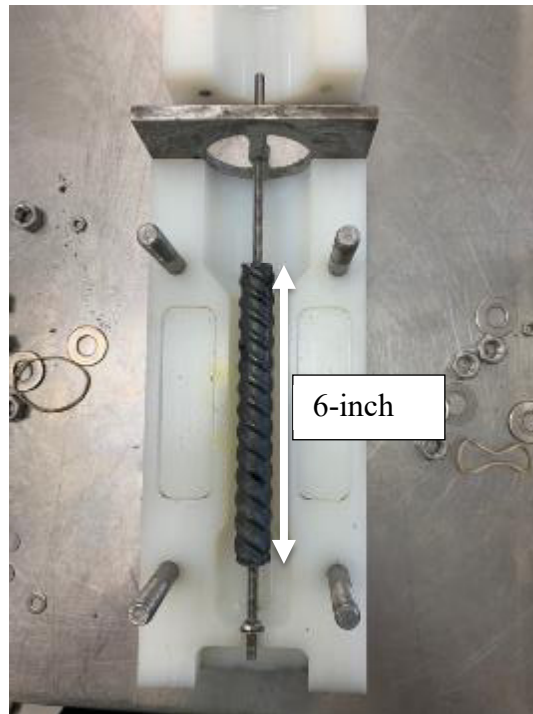


Figure 3-9. Anode specimen prepared in the mold

The mortar sample for each anode was cast in 3 layers. Each layer was vibrated for not more than 15 seconds at 150 Hz to allow consolidation into the mold and compacted with a wire rod 25 times prior to placement of the next layer. Once filled, the anodes were covered with a 0.0005-inch plastic sheeting and set aside to cure for 24 hours in the mold.

The cathode samples were cast in two layers, rodded 25 times, and each layer was vibrated to ensure adequate consolidation. These were also set aside and covered with plastic to cure in their molds for 24 hours. In addition to the anode and cathode specimens, additional 3 x 6-inch cylindrical specimens were cast for bulk resistivity testing of the mortar. Along with this, three 2-inch cubes were cast to be tested at the end of the 28-day curing time and at the time of chloride exposure. After the indicated curing period, the initial open circuit potentials (OCP) were recorded following ASTM C867.

Immediately after initial OCP measurements were taken, anodes were fully submerged in a saturated lime solution (0.03 lb/gal of calcium hydroxide and 0.28 lb/gal of sodium chloride). Samples were stored upright in PVC cylinders for the duration of the test until activation. Cathodes were placed in a saturated lime solution to half the height of the cathode or just below the embedded reinforcement in a container. The height of the cathode solution was constantly maintained to be higher than the solution of the anodes to prevent transport of chlorides into the solution surrounding the cathodes. The anode solution and cathode solution were replaced every 3 weeks. Two connections are made between the anodes and cathodes: 1) the electrical connection between the embedded steels, and 2) a salt bridge, which allows for the movement of ionic species. Figure 3.9

shows the test set up. Full details on the testing set up can be found in (Trejo and Vaddey, 2021).

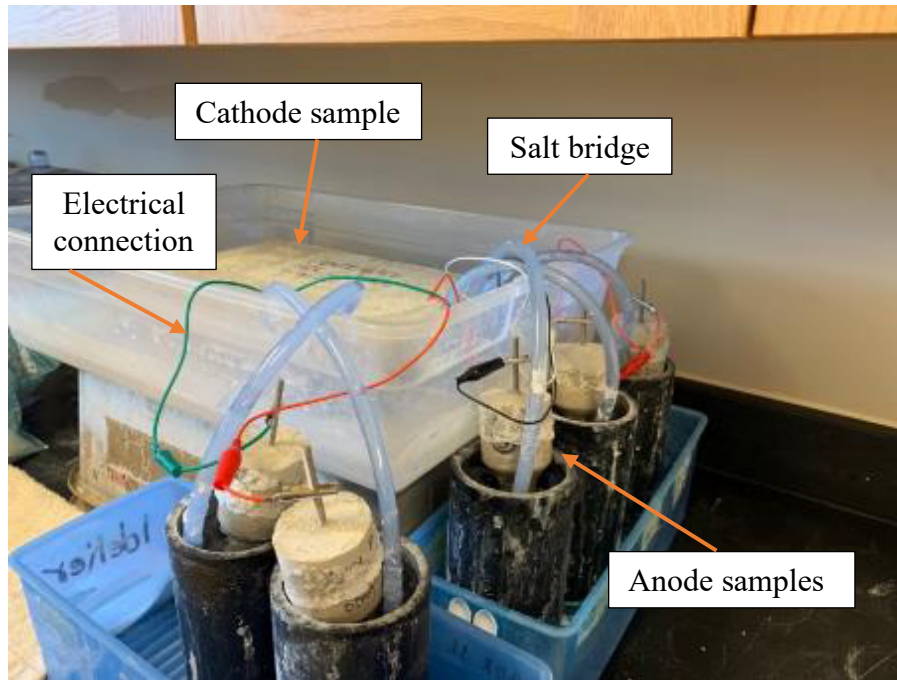


Figure 3-10. $O_{C_{crit}}$ testing set up

The test set up in Figure 3-10 was used to assess the critical chloride threshold of conventional black reinforcement embedded in each cement used in the study. OCP measurements were recorded daily for each sample using a copper-copper sulfate ($Cu-CuSO_4$) electrode and a high impedance multimeter. These measurements allow the user to identify the time to corrosion activation, defined as the duration from initial exposure to the time of corrosion initiation resulting from chloride ingress. After a sample has activated (i.e., when corrosion has initiated), it was removed from the solution and stored at ambient laboratory conditions until being prepared for the determination of chlorides. Samples were considered activated when the half-cell potential was more negative than -350 mV for two consecutive days. Table 3-12 summarizes the interpretation of the potential differences using the ASTM C876 half-cell test.

Table 3-12. Overview of interpreting half-cell potential measurements when using a copper-copper sulfate electrode (CSE)

Potential difference versus CSE	Interpretation
-200 mV	Greater than 90% probability of no corrosion
-200 mV > X > -350mV	Probability of corrosion is uncertain
<-350 mV	Greater than 90% probability of corrosion in the area of measurement

To prepare a sample for chloride determination, the mortar surrounding the embedded steel was cleaned of any surface contamination and then the mortar chipped away from the steel. Acid and water-soluble chlorides were determined using ASTM C1152/C1152M-20 and ASTM C1218/C1218M-20, respectively. Sample were collected from the mortar covering the steel and ground to pass a No. 20 US sieve as described in the standard.

3.4.2.5 *Field Evaluation*

Field cores were taken from the Yamhill Crossing (highway 39, OR-18, MP:45.76, Br# 08492) overlay from three separate locations. From each location two cores were removed. Compressive strength was measured on two samples in field condition. Another set of two was conditioned and evaluated for CTE following the methods reported in the previous section. The last set was used for indirect tension and carbonation testing. As part of the inspection a crack survey was performed. All damage for the eastbound lane was observed and reported on a set of drawings. The westbound lane was not closed for inspection.

4.0 RESULTS AND DISCUSSION

4.1 PHASE I

4.1.1 Phase I: Task 1A

4.1.1.1 *Setting time*

Table 4-1 shows the initial and final set measurement of the initially screened high early strength material for this project. It was found that the initial setting time of most of the materials was ≤ 1 - hour, except T3A mixture. The difference between initial and final set time was found to be significantly lower compared to OPC concrete (Zhang H, 2011). Most of the high early strength materials achieved the final set in 6 to 30 minutes after initial set. The lowest initial and final setting time was observed in OPC-CAC mixture where OPC replacement was 20%. The highest initial and final setting time was observed in T3A mixture. It was assumed that the blended composition of CSA2-OPC will exhibit similar measurement as CSA1-OPC, as a result, further investigation was excluded.

Table 4-1 Setting time measurement of high early strength materials

Material	Type I/II OPC Replacement (%)	w/c	Accelerator/ Retarder	Initial set (min)	Final set (min)
CSA1	0	0.46	0.10% retarder	50	75
CSA2	0	0.46	0.10% retarder	39	59
PrCSA1	0	0.46	0	65	71
PrCSA2	0	0.46	0	62	88
OPC-CAC	10	0.35	0.10% retarder	62	N/A*
	20	0.35	0.10% retarder	22	30
	30	0.35	0.10% retarder	36	52
CSA1-OPC	80	0.44	0.10% retarder	50	82
	70	0.43	0.10% retarder	47	73
	60	0.42	0.10% retarder	39	63
T3A [‡]	0	0.35	3.65% set accelerator	93	169

*N/A (not applicable) means final set did not achieve within test duration. Here the test duration was 82 minutes.

[‡] Penetration reading was collected at 5 minutes interval.

The initial set of all mixtures provided enough working time to place, consolidate, and finish the concrete cylinders that were then cured, and measured for compressive strength at subsequent ages. The admixture dosages should be sufficient in this case to provided adequate working time for placement, consolidation, and finishing for the increased concrete mixture volumes in Phase II.

4.1.1.2 *Semi-adiabatic temperature*

Table 4-2 shows the semi-adiabatic temperature measurement of the high early strength materials. Based on the observation, the materials can be divided into two categories- i) materials achieving maximum temperature in between 86-122°F, and ii) materials achieving maximum temperature in between 140-185°F.

Table 4-2 Semi-adiabatic temperature of high early strength materials

Material	Type I/II OPC Replacement (%)	w/c	Accelerator/ Retarder	Maximum temperature (°F)	Time to reach maximum temperature (min)
CSA1	0	0.46	0.10% retarder	170	71
CSA2	0	0.46	0.10% retarder	165	69
PrCSA1	0	0.46	0	178	73
PrCSA2	0	0.46	0	185	70
OPC-CAC	10	0.35	0.10% retarder	86	55
	20	0.35	0.10% retarder	117	52
	30	0.35	0.10% retarder	111	59
CSA1-OPC	80	0.44	0.10% retarder	151	85
	70	0.43	0.10% retarder	142	83
	60	0.42	0.10% retarder	142	69
T3A [†]	0	0.35	3.65% set accelerator	91	300

The highest temperature was achieved by PrCSA2 cement within 70 minutes of cement contracting with water. The lowest temperature development was observed in OPC-CAC cement where OPC was replaced by 10%.

4.1.1.3 *Compressive strength*

Figure 4-1 shows the compressive strength of mortar specimens.

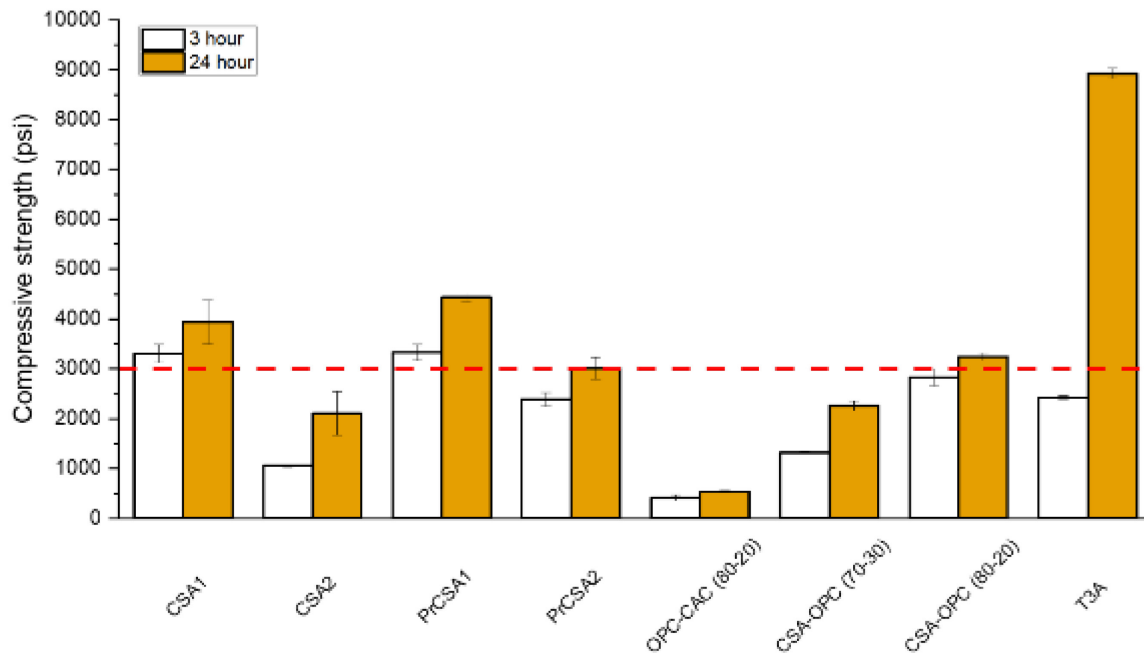


Figure 4-1 Compressive strength of mortars. Here, the dashed line indicated the target requirement for compressive strength (≥ 3000 psi)

It was found that CSA1, PRCSA1, and CSA-OPC (80%OPC was replaced by CSA1) met the target criteria of compressive strength at 3 hours. PRCSA2 achieved the target compressive strength after 24 hours which might be caused by the addition of 0.1% citric acid as a retarder. T3A mixture also did not satisfy the 3-hour strength requirement criteria, however, the 24-hour compressive strength was significantly higher compared to all mixtures. CSA2, OPC-CAC (20% OPC was replaced by CAC), and CSA-OPC (70% OPC was replaced by CSA1) did not satisfy the strength requirement criteria even after 24 hours. It is important to note that CSA2-OPC mixture were not further investigated for compressive strength since CSA2 did not satisfy the compressive strength requirement criteria after 24 hours.

4.1.1.4 Summary from Phase I- Task 1a

Table 4-3 summarizes the findings from Phase I- Task 1a.

Table 4-3 Summary of findings from Phase I- Task 1a. Here “✓” = value observed in a particular range or satisfy a particular criterion, “_”= not applicable, and “x”= did not satisfy the criteria

Mixture ID	Set time (min) ≤ 30	Set time (min) $30 \leq x \leq 60$	Set time (min) $60 \leq x \leq 120$	Set time (min) ≥ 120	Max Temp (°F) 86-122	Max Temp (°F) 140-185	$f'c \geq 3000$ psi at 3 hour or 24 hours	Qualify for phase I- task 1b (Y/N)
CSA1	-	-	✓	-	-	✓	✓	Y
CSA2	-	✓	-	-	-	✓	x	N
PrCSA1	-	-	✓	-	-	✓	✓	Y
PrCSA2	-	-	✓	-	-	✓	✓	Y
OPC-CAC (80-20)	✓	-	-	-	✓	-	x	N
CSA1- OPC (80-20)	-	-	✓	-	-	✓	✓	Y
T3A	-	-	-	✓	✓	-	✓	Y

It is important to note that there were not any specified target criteria for final setting time or maximum temperature. As a result, the decision whether a particular material was moved to Phase I -Task 1b was determined primarily by compressive strength. All mixtures had a final setting time (paste) of at least 15 minutes. The fastest setting mixture was the OPC-CAC (80-20) blend. There was still enough working time in this mixture to cast specimens and this could be further extended with minor modifications using a set retarder or superplasticizer. From Phase I- Task 1a the following materials were qualified to test for Task 1b- CSA1, PrCSA1, PrCSA2, CSA-OPC (80-20), and T3A. In the next following sections CSA1 and CSA-OPC (80-20) will be refereed as CSA (without permeability modifier), CSAP (with permeability modifier), and CSA-OPC for clarity and brevity.

4.1.2 Phase I: Task 1A

4.1.2.1 Finalized concrete mixture proportioning

Table 4-6 show the finalize concrete mixture proportions for HESC developed in this study. It must be noted that the PrCSA1 and PrCSA2 cements had premixed air entraining admixture. For each mixture, three different subsets with different target air content levels were evaluated. In phase I, the target air content was $5.5 \pm 1.5\%$ to determine the air entrainment capability of the designed mixtures. After that, an increased target air content of 7% was set to access the freeze-thaw performance of the mixtures. In both cases, some mixtures achieve the target content and some not. As a result, mixture designs for formation factor and critical chloride threshold did not include any additional air entrainment (Table 4-5).

The water to cement ratio was adjusted in the trial mixes to get the finalized water to cement ratio to achieve the target compressive strength. The local coarse and fine aggregate were used as their receiving condition. Both AE (air entraining admixture) and superplasticizer were adjusted to achieve the target/increased air content and similar working time. However, priority for finalizing the mixture proportion was given to achieve the target compressive strength where target air content was difficult to achieve after 3 trials. Advacast 600 was used as a superplasticizer and DARA-VAIR 200 was used as air entraining admixture for only T3A mixture unless mentioned otherwise.

Table 4-4 Finalized concrete mixture proportions for target air content

Constituents	CSAP	PrCSA1	PrCSA2	CSA-OPC	T3A
Cement, lb/yd ³	658	658	658	526/132	658
Coarse aggregate (SSD), lb/yd ³	1601	1601	1601	1601	1601
Fine aggregate (SSD), lb/yd ³	1267	1267	1267	1288	1343
Water, lb/yd ³ ↓	250	250	250	243	230
Superplasticizer, oz/cwt	12	0	0	12	15
Air entrainer, oz/cwt	30	12	0	30	30
Permeability modifier, oz/cwt	10.6	0	0	0	0
Accelerator, % cement wt.	0	0	0	0	4
Citric Acid, % CSA cement wt.	0.2	0	0	0.2	0
Gypsum, % of cement wt.	0	0	0	0	1.0
Slump, inch	3	10	10	1	8
Air content, %	4	6	10	6	6
Unit weight, lb/ft ³	146	145	132	143	144
Working time (minutes)	15~20	25~30	25~30	10~15	20~25

Table 4-5 Finalized non-air entrained concrete mixture proportions

Constituents	CSAP	PrCSA1	PrCSA2	CSA-OPC	T3A	HPC
Cement, lb/yd ³	658	658	658	658	658	658
Coarse aggregate (SSD), lb/yd ³	1601	1601	1601	1601	1601	1824
Fine aggregate (SSD), lb/yd ³	1333	1266	1266	1271	1327	1136
Water, lb/yd ³ ↓	249	249	249	249	236	236
Superplasticizer, oz/cwt	12	0	0	12	17	17
Air entrainer, oz/cwt	0	0	0	0	0	0
Permeability modifier, oz/cwt	12	0	0	0	0	0
Accelerator, % cement wt.	0	0	0	0	4	0
Citric Acid, % CSA cement wt.	0.2	0	0	0.2	0	0
Gypsum, % of cement wt.	0	0	0	0	1	0
Air content, %	3	2	10	3	2	2

Table 4-6 Finalized concrete mixture proportions with increased air content

Constituents	CSAP	PrCSA1	PrCSA2	CSA-OPC	T3A	HPC
Cement, lb/yd ³	658	658	658	658	658	658
Coarse aggregate (SSD), lb/yd ³	1601	1601	1601	1601	1601	1824
Fine aggregate (SSD), lb/yd ³	1225	1225	1225	1225	1225	1136
Water, lb/yd ³	249	249	249	249	238	238
Superplasticizer, oz/cwt	12	0	0	12	17	17
Air entrainer, oz/cwt	34	14	0	3	40	17
Permeability modifier, oz/cwt	12	0	0	0	0	0
Accelerator, % cement wt.	0	0	0	0	7	0
Citric Acid, % CSA cement wt.	0	0	0	0	0	0
Gypsum, % of cement wt.	0	0	0	0	1	0
Air content, %	5	5	10	10	9	7

For CSA, PrCSA1, and PrCSA2 a total of six trials were conducted to satisfy either or both target compressive strength and air content criteria in Phase I. For CSA-OPC and T3A, three trials were conducted before finalizing the mixture proportion. PrCSA2 contained an integral air entrainer. Many attempts were made during trial mixing to reduce the air content. None were successful and ultimately the research team decided to proceed with this mixture and evaluate mechanical and durability performance.

4.1.2.2 Compressive Strength

Figure 4-2 shows the summary of the compressive strength for all concrete mixtures.

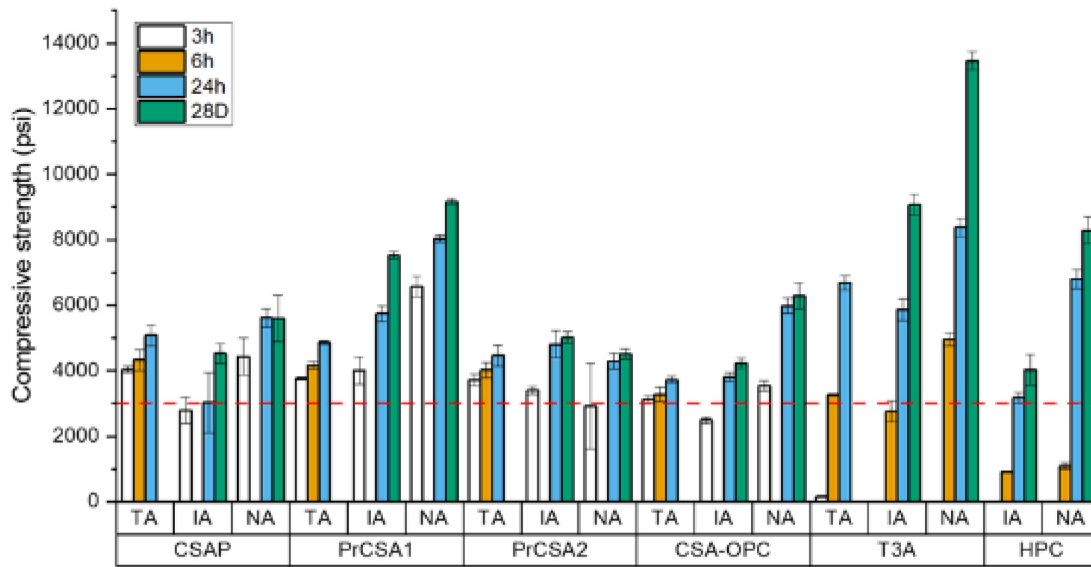


Figure 4-2 Compressive strength of all mixtures. Here, a dashed line was used to indicate the target compressive strength. TA= air entraining admixture was added to achieve target air content $5.5 \pm 1.5\%$, IA= increased air entraining admixture with target air content of 7%, and NA= mixture with no air entraining admixture

In Figure 4-2 Mixtures T3A and HPC do not have 3 hour results (except for the TA) as the strength was too low at that time to measure. By 6 hours these mixtures did have measureable strength. The TA mixtures were part of Phase I where the target strength development was only monitored for the first 24 hours. The 28-day results were not obtained until later mixtures in Phase II (for the IA and NA mixtures). Mixtures with CSA were not measured for 6 hour strength in Phase I and II since the 3,000 psi target was met at 3 hours with one exception. Mixture CSA-OPC did not meet the 3,000 psi target with the increased air entraining admixture; it did meet this strength by 24 hr and 28 day and the NA mixture also achieved 3,000 psi in 3 hours.

With the exception of T3A and CSA-OPC with IA (mixture with an increased target air content of 7%), all HESC mixtures (not including HPC) reached the target compressive strength (≥ 3000 psi) within 3 hours. The target compressive strength was achieved by T3A mixture at 6 hours. As a result, for Phase II, the compressive strength of the T3A mixture was tested at 6 hours, 24 hours, and 28 days. No data is shown for 3 hours for this mixture or for the HPC mixture as the strength would have been too low to properly evaluate. In CSAP mixture, no significant difference was observed in 3-hour compressive strength in TA (mixture with target air content of $5.5 \pm 1.5\%$) and NA (mixture with no air entraining admixture). However, the compressive strength was significantly lower in IA mixture, though, it was close to the target strength. The difference in 24 hour compressive strength was also negligible. In PRCSA1 mixture, the 3-hour compressive strength was not significantly different in TA and IA mixture, though it was significantly higher in the NA mixture. PrCSA1 mixture NA showed significantly higher compressive

strength at all tested ages compared to TA and IA mixture. In PrCSA2 mixture, a gradual increase in compressive strength was observed in the TA mixture at 3 hours, 6 hours, and 24 hours with marginal differences between them. No significant difference in strength values was observed when samples were tested at 6 hours, 24 hours and 28 days for all conditions (TA, IA, and NA). In CSA-OPC, an increased dosage of air entraining admixture adversely affected the target compressive strength criteria at 3 hours. However, at 24 hours the difference between compressive strength in TA and IA mixture was insignificant. Similar to CSA and PrCSA1 mixture, CSA-OPC mixture also showed significantly higher compressive strength in NA mixture compared to the TA and IA mixture. While T3A mixture did not achieve the target compressive strength at 3 hours, significantly higher compressive strength was observed in this mixture at 24 hours and 28 days compared to the mixture based on CSA cement. It was also observed that the increased dosage of air content affected the early strength gain at 6-hours. Compressive strength at 6 hours in IA-T3A mixture was significantly lower than TA-T3A mixture, however, at 24 hours the difference was marginal. It was also evident that the rate of strength development with time was significantly higher in T3A mixture compared to the other high early strength mixture. Compared to the baseline HPC mixture, the finalized high early strength mixtures showed improved compressive strength over time.

4.2 PHASE II

4.2.1 Task 2: Curing

4.2.1.1 Evaluation of CAZ

Figure 4-3 shows the effect of different external curing practices and substrate conditions on the CAZ profile of CSA mortar mixture. Based on the bound water profile, the CAZ was determined (as described in section 3.4.2.1) in CSA mixture. It was evident that the CAZ in CSA mixture was significantly lower compared to OPC mortar mixture (Bouchelil, 2022). It was observed that the CAZ values were marginally affected by the external curing practices. An increase in CAZ value was observed in “no-curing 50RH” specimens compared to the other tested conditions where CAZ values were in the range of 0.2-0.4 inch. Figure 4-4 shows the CAZ of PrCSA1 mixture.

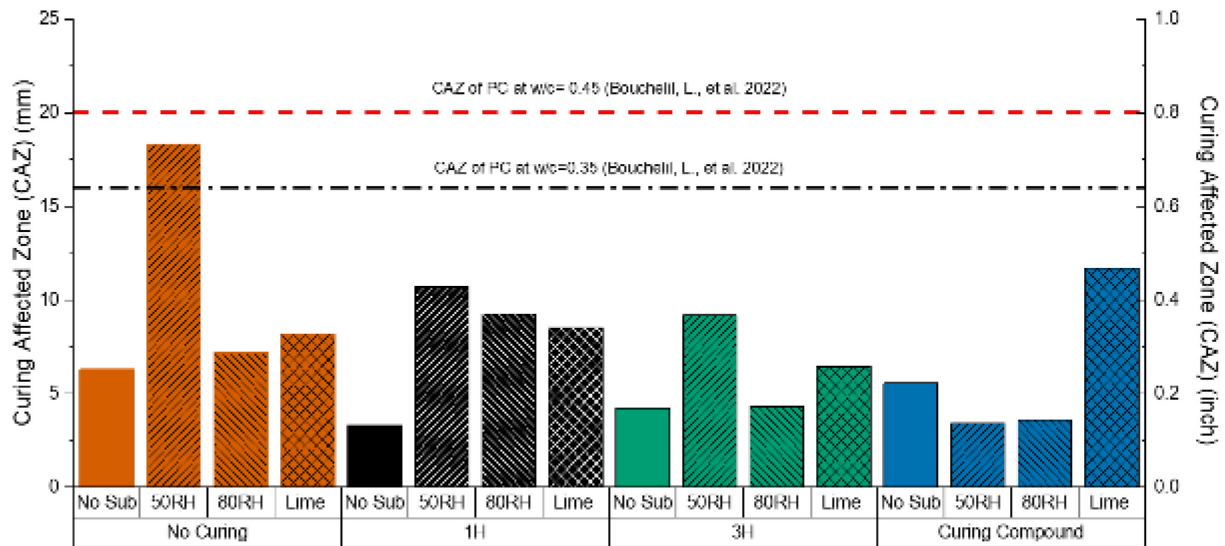


Figure 4-3 CAZ of CSA mixture

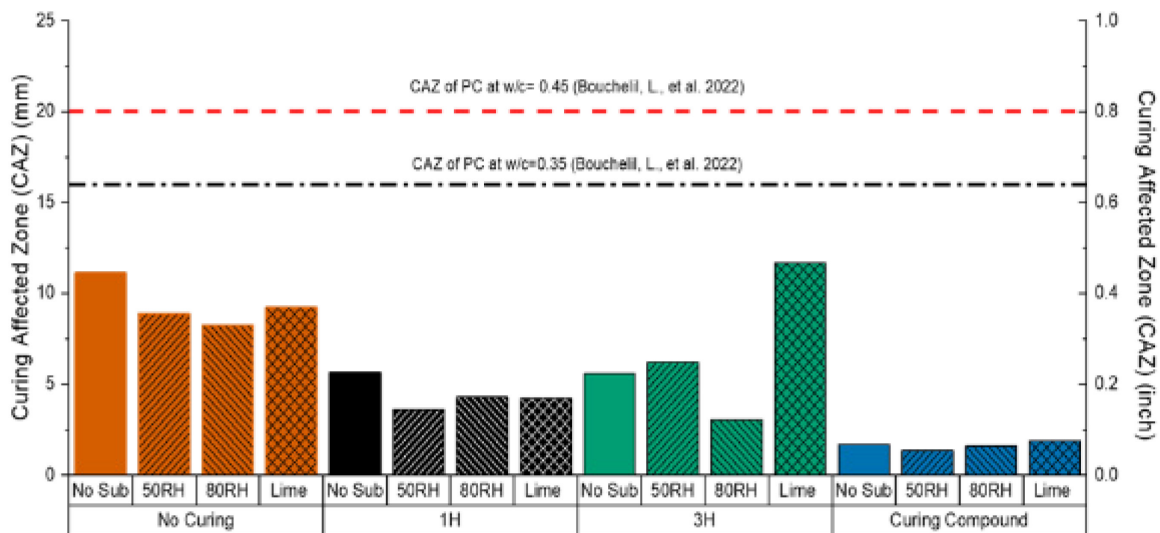


Figure 4-4 CAZ of PrCSA1 mixture

In the PrCSA1 mixture, significantly lower (< 0.1 inch from top surface) CAZ was observed in samples treated with curing compound compared to the other curing practices. In “no curing” samples, the CAZ was in the range of 0.3-0.4 inch. Curing with wet burlap and plastic sheet showed CAZ in between 0.1-0.2 inch for 1 hour curing and 0.1-0.5 inch for 3-hour curing. A much smaller CAZ was observed in the PrCSA1 mixtures compared to OPC mortars, particularly for curing with a curing compound and 1 hour curing conditions. Figure 4-5 shows the CAZ for different curing approaches for the PrCSA2 mixture. Like PrCSA1 mix, lower CAZ was observed in PrCSA2 mixture when

curing compound was used as external curing practice. From the top surface, only 0.04-0.20 inch was affected by the drying process when a curing compound was used. The lowest CAZ was observed when the substrate conditioned at 80%RH and lime. Except for curing compound, no difference was observed in “no curing” and curing with “wet burlap and plastic sheet” cured specimens. However, comparatively lower CAZ (0.3-0.4 inch) was observed when samples were cured for 3 hours with wet burlap and plastic sheeting. In the remaining cases, the CAZ ranged from 0.3-0.5 inch.

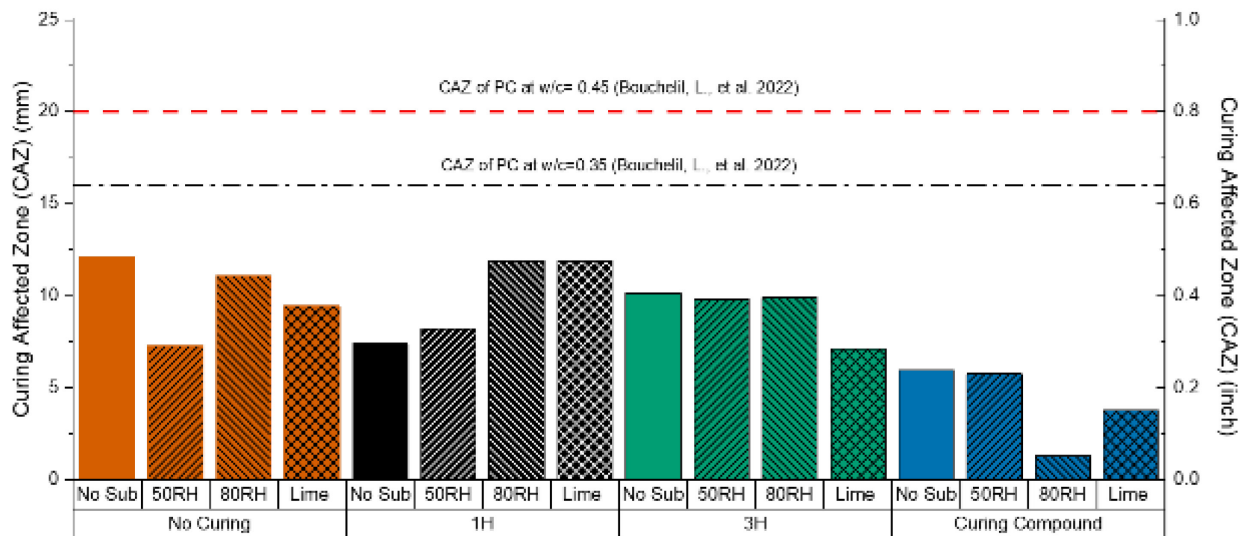


Figure 4-5 CAZ of PrCSA2 mixture

Figure 4-6 shows the CAZ of the CSA-OPC mixture.

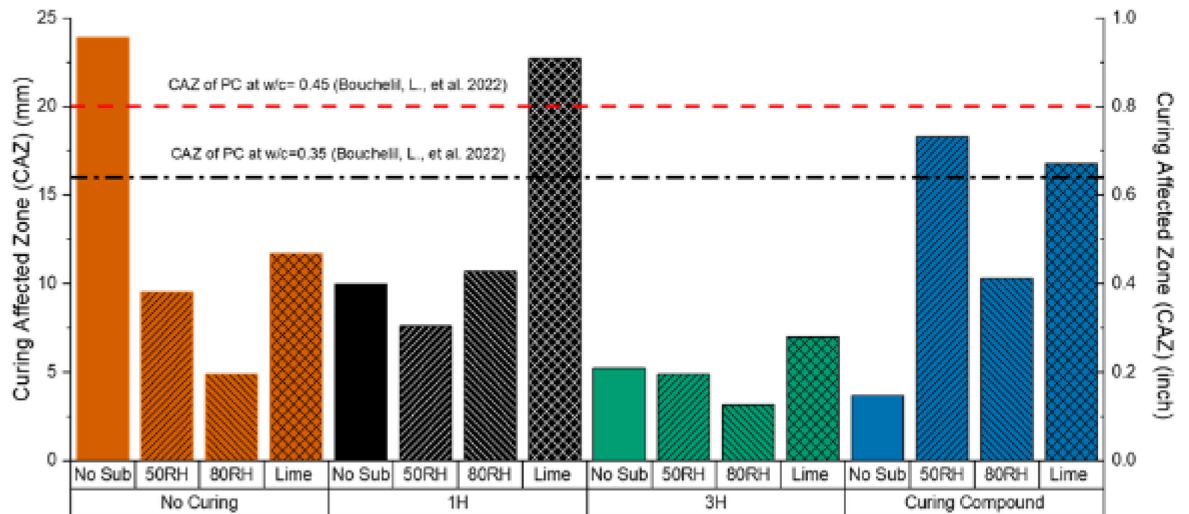


Figure 4-6 CAZ of CSA-OPC mixture

In the CSA-OPC mixture, comparatively lower CAZ was observed when specimens were cured with wet burlap and plastic sheeting for 3 hours (Figure 4-6). The CAZ ranged from 0.1-0.3 inch. Samples cured with curing compound showed CAZ in the range of 0.2-0.7 inch, where 0.2 inch was observed in samples with no substrate and 0.7 inch was observed when substrates were conditioned at 50%RH. The CAZ ranged between 0.2-0.9 inch “no curing” samples based on the substrate conditions. A CAZ of 0.2 inch was observed when the substrate was conditioned at 80%RH where 0.9 inch was observed in samples with no substrate. Samples cured with wet burlap and plastic sheet for 1 hour showed CAZ in the range of 0.2-0.9 inch, where 0.2 inch was observed when substrate was conditioned at 50%RH and 0.9 inch was observed when substrate conditioned with lime solution. The range of CAZ was observed between 0.2-0.7 inch when curing compound was used as the curing practice. Only 0.2 inch from top surface was affected in samples with no substrate whereas 0.7 inch was affected when substrate conditioned at 50%RH.

Figure 4-7 shows the CAZ of the T3A mixture. In the T3A mixture, use of a curing compound showed lower CAZ compared to the other curing practices. The range of CAZ was between 0.4-0.9 inch, where 0.4 inch was observed in the sample with no substrate and 0.9 inch was observed when substrate was conditioned with lime solution. The T3A samples with no external curing showed CAZ ranging from 0.8-1.3 inch. The substrate conditioned at 80%RH showed significantly lower CAZ compared to the other substrate conditions. Marginal differences were observed in CAZs when samples were cured with wet burlap and plastic sheet for 1 hour and 6 hours. The CAZs in 1 hour cured samples ranged between 1.0-1.1 inch where 1.0-inch CAZ was observed in samples with no substrate and 1.1-inch CAZ was observed in samples where substrate was conditioned at 50%RH. It was observed that the 6-hour curing with wet burlap and plastic sheet did not improve the CAZ compared to 1 hour curing. The CAZ ranged from 1.1- 1.5 inch where lower CAZ (1.1 inch) was observed in samples with no substrate, and higher CAZ was observed in samples with substrate (1.5 inch) conditioned at 50%RH.

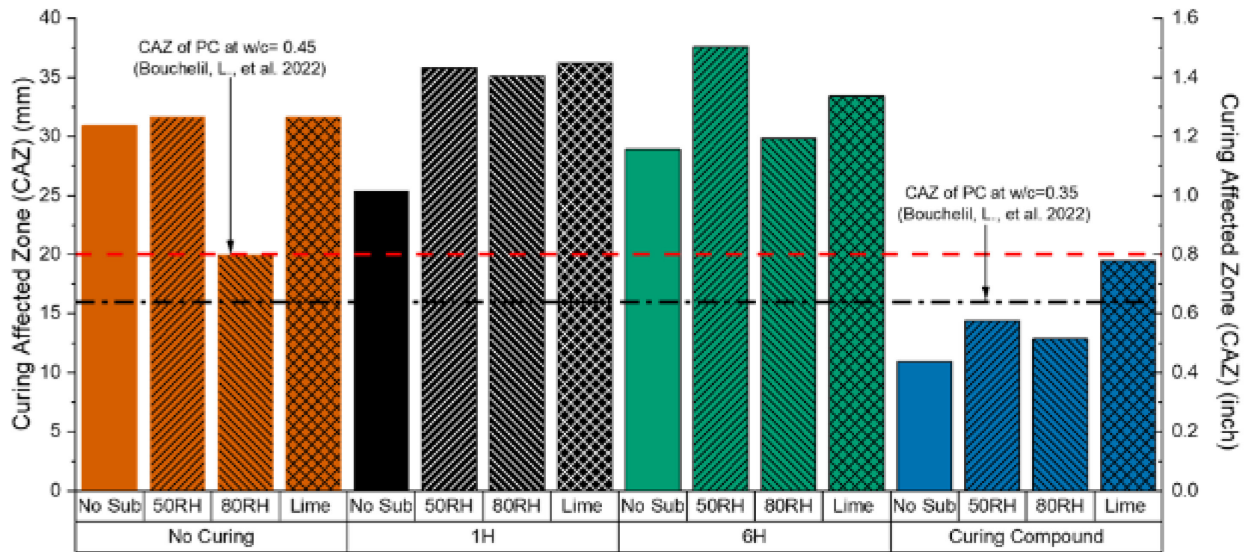


Figure 4-7 CAZ of T3A mixture

4.2.1.2 Porosity

Powers demonstrated that in Portland cement paste, the permeability coefficient significantly decreased when capillary porosity decreased from 40 to 30%. As a result, lower porosity as well as permeability in concrete samples can be inferred as an indicator of higher resistance to different forms of chemical attack such as chloride, sulfate, or freeze-thaw (Powers 1958, Kosmatka and Wilson 2011).

Figure 4-8 showed the porosity profile (over the depth of the sample from top surface) of the PrCSA1 mixture determined from the NR image analysis. The porosity in CSA samples within the CAZ ranged from 4.5% to 5% when curing compound was used as external curing. The average porosity was calculated from the CAZ to the bottom of the samples with no substrate. For samples where substrate was used, the porosity was calculated from the CAZ to 1.5-inch depth to exclude the region of the overlay that might be affected by the substrate.

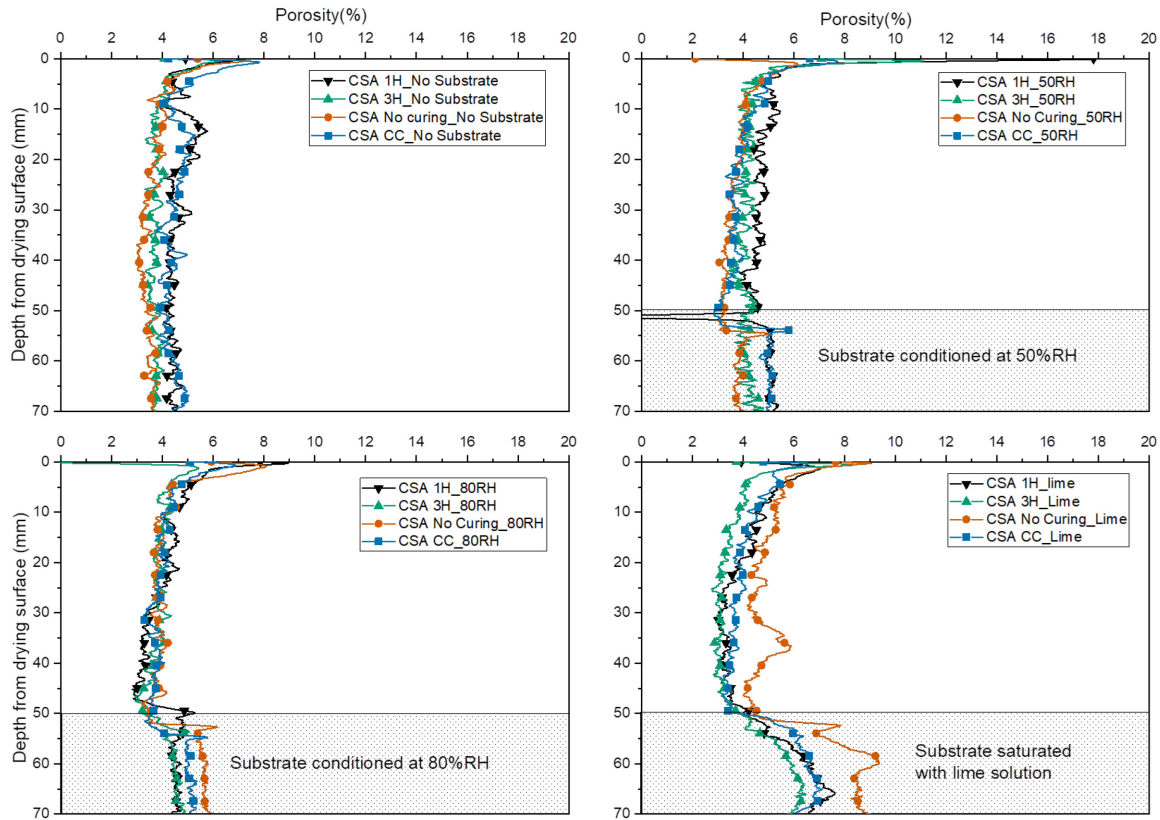


Figure 4-8 Porosity of CSA mixture

It was observed that the average porosity of the CSA mixture with no substrate and treated with curing compound was 4.5%, whereas a lower porosity (3.8%) was observed when the substrate was conditioned with saturated lime solution. CSA samples with no external curing showed average porosity in the range of 3.8-6.0%, where lower average porosity (3.8%) was observed in samples with no substrate and higher average porosity (6%) was observed in samples with substrate conditioned with lime. The average porosity in samples cured with wet burlap and plastic sheet ranged from 3.8-5%. Overall, this porosity is (low/very low) compared to OPC systems hydrated at sealed condition for 28 days with 0.37 w/cm (Rößler, 1985). Figure 4-9 shows the porosity profile of PrCSA1 samples.

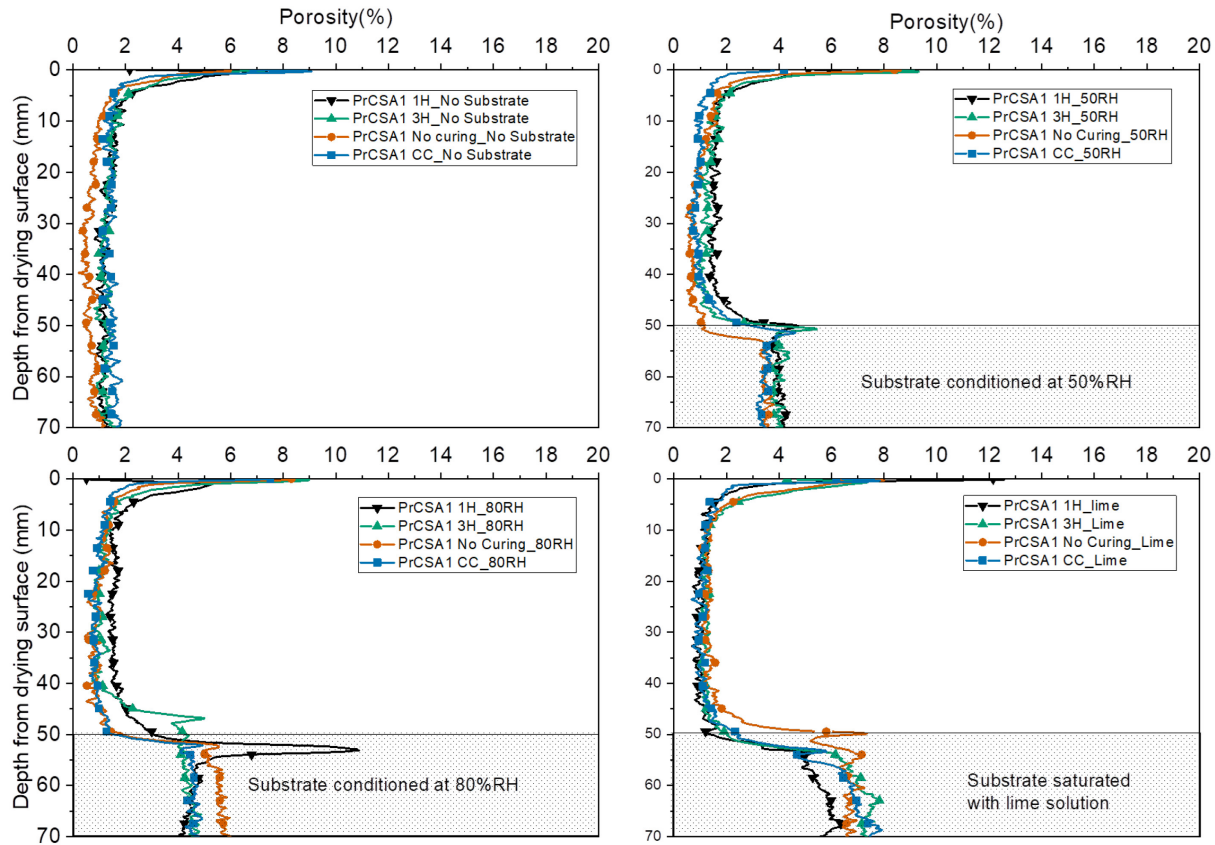


Figure 4-9 Porosity profile of PrCSA1 mixture

The lowest CAZ was observed in PrCSA1 specimens when a curing compound was used as external curing. Supporting this is a very low porosity calculated from neutron radiography with an observed range between 1.0-1.6% depending on the substrate conditions. An average porosity of 1% was observed when substrate was conditioned at 50%RH and 1.6% was observed in samples with no substrate. PrCSA1 samples with no external curing showed an average porosity in the range of 1.0-1.7%. Higher average porosity was observed in samples with substrate conditioned in lime and lower porosity was observed in samples with no substrate. The average porosity in 1 hour cured with wet burlap and plastic sheet was lied in the range of 1.3-1.8% and in 3 hours cured samples were lied in the range of 1.4-1.7%. Overall, the average porosity in PrCSA1 samples irrespective of curing or substrate conditions were found to be less than 2% indicating a very low porosity compared to other samples in this study as well as literature reported values for OPC-based systems at similar w/cm.

Figure 4-10 shows the porosity profile of PrCSA2 samples. It was found that the average porosity in PrCSA2 samples was higher than PrCSA1 samples. PrCSA2 samples treated with curing compound showed an average porosity in the range of 6.5-7.8% (compared to 3.8-6.0% for PrCSA1). Wet burlap and plastic sheet cured samples showed a slightly

higher average porosity of 6.8-7.9% for 3 hours curing and 6.3- 6.6% for 1 hour curing. Samples where no external curing practices were applied showed an average porosity in the range of 7.1-7.9%. Overall, the range of average porosity of PrCSA2 samples were very close to each other and did not significantly vary by the external curing practices or substrate conditions.

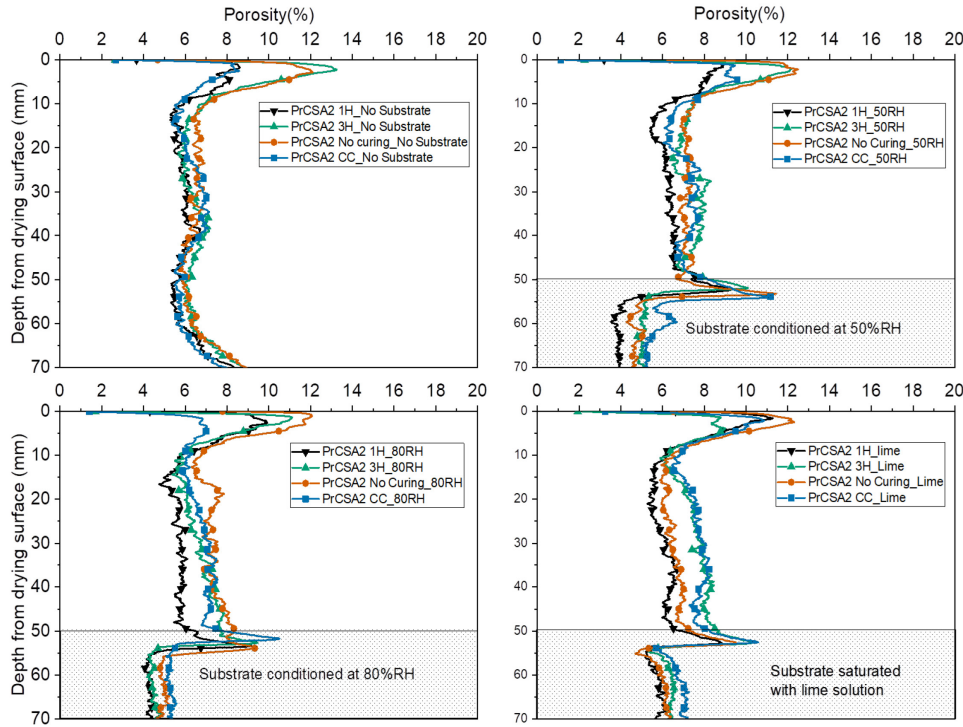


Figure 4-10 Porosity profile of PrCSA2 mixture

Figure 4-11 shows the porosity profile of CSA-OPC samples. The average porosity of the samples treated with curing compound ranged from 5.3-6.3%. The average porosity of the samples with no external curing practices ranged from 5.7-7.0%. The average porosity of 1 hour cured samples with wet burlap and plastic sheet was ranged from 6.6-6.8%. The average porosity of 3 hour cured samples with wet burlap and plastic sheet ranged from 6.3-6.7%. Overall, the average porosity in samples with curing compound was lower, though the difference in porosity values in all curing conditions are marginal irrespective of different substrate conditions. Figure 4-12 shows the porosity profiles of T3A mixtures.

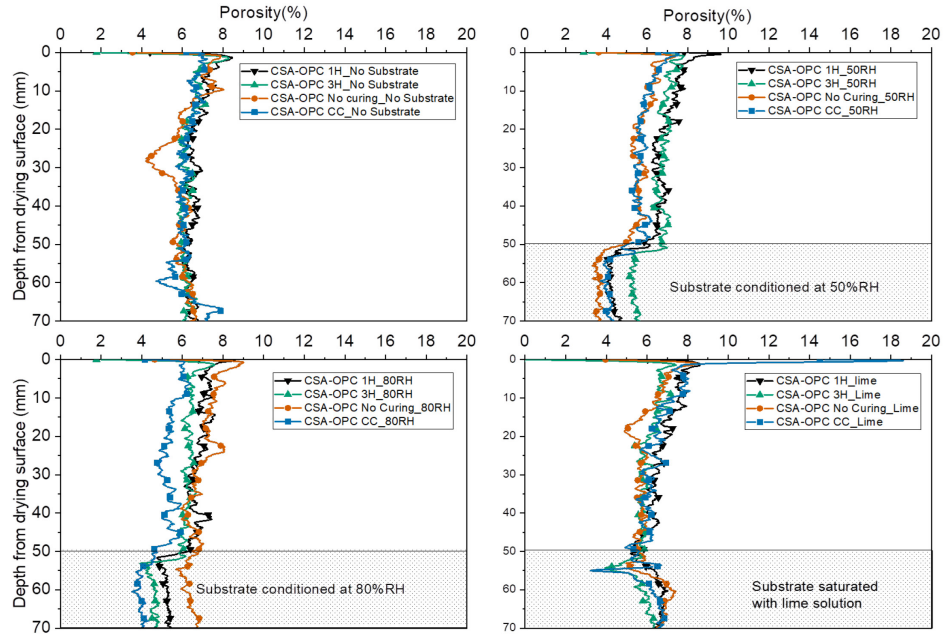


Figure 4-11 Porosity profile of CSA-OPC mixture

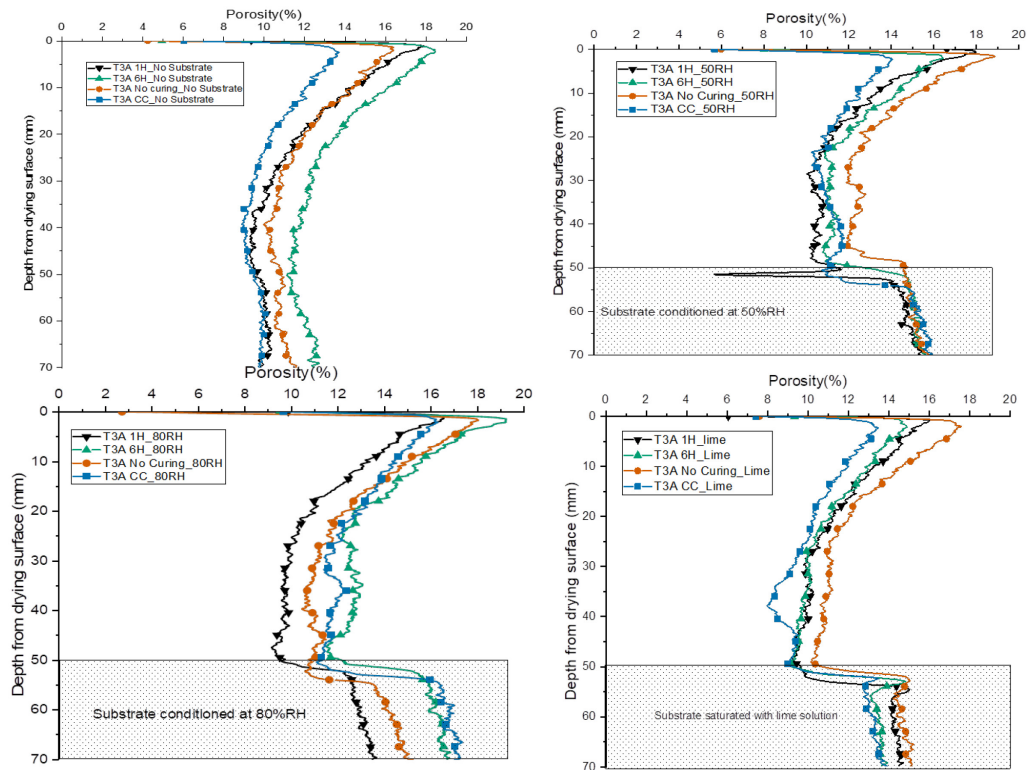


Figure 4-12 Porosity of T3A mixture

The porosity values were significantly higher in the T3A (Portland cement-based) compared to the other CSA based HES mixtures in this study. A gradual decrease in

porosity values in all T3A specimens was observed from the top surface to 0.8 inch depth. After that, the porosity values plateaued. For this reason, the average porosity was calculated in T3A specimens from 0.8-1.6 inch depth of the samples. The average porosity in curing compound treated samples were observed in the range of 9.2-12.0%. Lower porosity (9.2%) was observed in samples where substrate was saturated with lime solution. Higher porosity (12%) observed when substrate was conditioned at 80%RH. The average porosity in “no curing” samples were ranged from 11.0-12.3%. In 1 hour cured samples with wet burlap and plastic sheet the average porosity was ranged from 10.0-10.6%. On the other hand, the T3A specimens cured with wet burlap and plastic sheet showed an average porosity in the range of 10.1-12.6%

Overall, the use of a curing compound was the most effective for all but one mixture in this study. The mixture with 80% CSA and 20% OPC was most effectively cured with a 3-hour application of wet burlap and plastic sheeting. Using no curing at all still provided a smaller curing affected zone in any mixtures containing CSA compared to the ordinary Portland cement mixtures referenced in the literature. For the one system that was predominantly ordinary Portland cement (T3A), the most effective curing approach (e.g. a smaller CAZ) was the use of a curing compound.

4.2.2 Task 3: Mechanical Property Development

4.2.2.1 *Activation energy*

Figure 4-13 shows the cumulative heat evolution of all high early strength cementitious systems tested in this project and

Table 4-7 shows the calculated Ea values of all five mixtures and corresponding curve fit parameters that were used to calculate the Ea values.

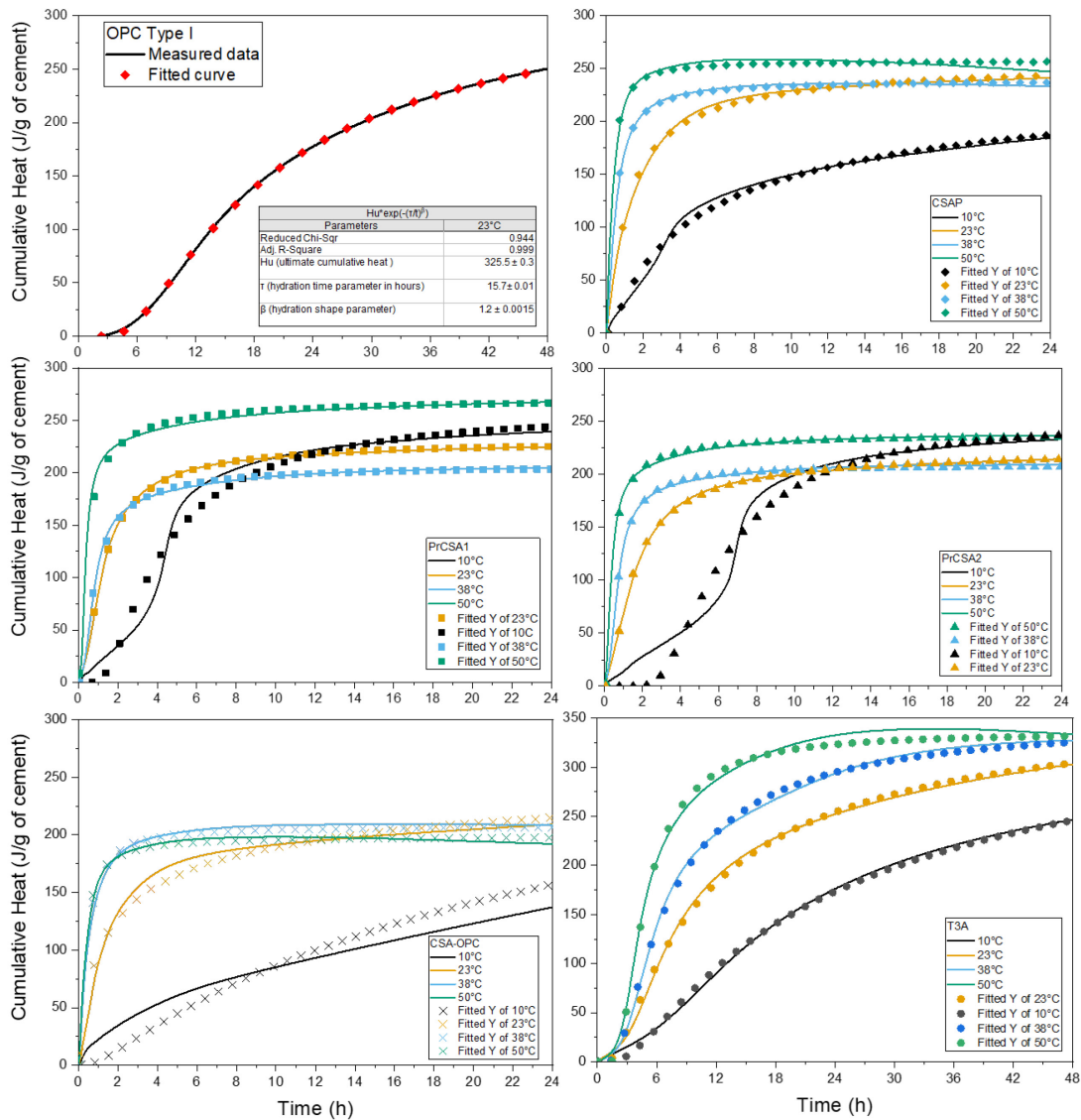


Figure 4-13 Cumulative heat curves of the high early strength cement mixtures

Table 4-7 Ea calculation using cumulative heat curve from isothermal calorimeter

Mixture	Temp. (°C)	Hu*	τ *	β *	R ² *	Slope of ln(kc/kref) vs. (1/Tc-1/Tref)	Ea (J/mol)	R ² ↓	Comparison with literature
CSA	10	280	4.34	0.53	0.99	6502.9	54039	0.95	42240 ~80220 J/mol
	23	259	0.85	0.83	0.99	6502.9	54039	0.95	42240 ~80220 J/mol
	38	242	0.38	1.09	0.99	6502.9	54039	0.95	42240 ~80220 J/mol
	50	267	0.25	1.06	0.99	6502.9	54039	0.95	42240 ~80220 J/mol
PrCSA1	10	263	3.44	1.35	0.98	5193.3	43156	0.95	42240 ~80220 J/mol
	23	232	0.96	1.12	0.99	5193.3	43156	0.95	42240 ~80220 J/mol
	38	207	0.67	1.13	0.99	5193.3	43156	0.95	42240 ~80220 J/mol
	50	273	0.30	0.87	0.98	5193.3	43156	0.95	42240 ~80220 J/mol
PrCSA2	10	249	5.32	1.99	0.98	6335.8	52650	0.96	42240 ~80220 J/mol
	23	223	1.14	1.04	0.99	6335.8	52650	0.96	42240 ~80220 J/mol
	38	208	0.58	1.35	0.99	6335.8	52650	0.96	42240 ~80220 J/mol
	50	238	0.30	1.01	0.99	6335.8	52650	0.96	42240 ~80220 J/mol
CSA-OPC	10	465	27.80	0.56	0.97	10456	86889	0.87	N/A
	23	284	1.19	0.43	0.98	10456	86889	0.87	N/A
	38	207	0.34	1.23	0.97	10456	86889	0.87	N/A
	50	198	0.27	1.18	0.98	10456	86889	0.87	N/A
T3A	10	400	19.10	0.78	0.99	3394.8	28210	0.96	N/A
	23	375	8.30	0.89	0.99	3394.8	28210	0.96	N/A
	38	351	5.80	1.22	0.99	3394.8	28210	0.96	N/A
	50	335	4.10	1.88	0.98	3394.8	28210	0.96	N/A

*Hu, τ , β , and R² are the parameters from fitted cumulative heat release curves. R² values representing the proportion of variance between fitted and measured cumulative heat curves.

↓R² values representing the proportion of variance between fitted and measured Ea values

N/A= not available

The R^2 values represented the proportion of variance between measured and fitted values. For E_a determination, the R^2 values ranged from 0.87 to 0.96. Here, the R^2 value for E_a determination of CSA-OPC mix was 0.87, which was slightly lower compared to the other mixture. This indicated that the predicted E_a value of CSA-OPC mix was either overestimated or underestimated. The possible explanation of this phenomenon can be attributed from the prediction of H_u at lower temperature (10°C) due to lower signal to noise ratio at this temperature.

The E_a values calculated in this project were also compared with the existing literature. Liao et al. showed that the average E_a values of CSA cement ranged from 42240 ~80220 J/mol (Liao, Wei et al. 2011). In this project, the predicted E_a value of CSA, PrCSA1, and PrCSA2 were 54039 J/mol, 43156 J/mol, and 52650 J/mol which fell in the range of reported values by Liao et al. Not enough literature was found to report the activation energy of the blended cement of CSA-OPC. In literature, values E_a values of type I Portland cement ranges from 34500~40540 J/mol (Jonathan L. Poole). However, a lower E_a value of 28210 J/mol was observed for T3A cement which indicated that a faster hydration reaction happened in T3A mix compared to Portland cement which is likely due to the addition of accelerator and gypsum. No previous literature was found that directly reported the activation energy value of ASTM C150 Type III portland cement with accelerating admixture and gypsum. However, Poole et al. showed that the addition of accelerating admixture reduced the E_a value of Type I Portland cement from 40550 J/mol to 35750 J/mol. Poole et al. (2011) also reported that the activation energy of Type III Portland cement without any admixture was 37350 J/mol (Poole Jonathan, Riding Kyle et al. 2011). As a result, it is possible that a higher dosage (~4%) of accelerating admixture in addition to gypsum might cause this higher reduction in E_a value. Further study on the effect of different dosage of accelerating admixture on the value of activation energy will help to understand the hydration mechanism of type III Portland cement with the addition of accelerating admixture.

4.2.2.2 *Coefficient of thermal expansion*

Figure 4-14 shows the CTE values of HESC used in this project. The dashed line indicates the CTE value of HPC concrete based on the ODOT mix design of structural bridge overlay.

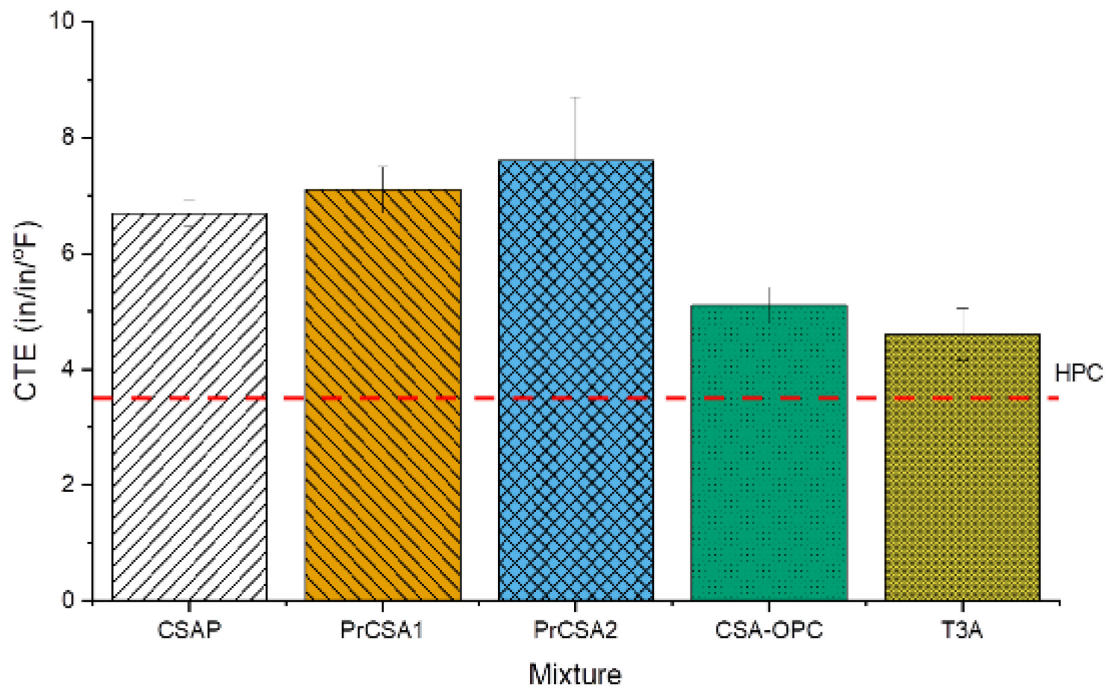


Figure 4-14 Coefficient of thermal expansion of concrete without air entrainment following AASHTO T336. Error bars are one standard deviation

All HES concrete showed higher CTE values compared to HPC. The values of CTE in CSA, PrCSA1, and PrCSA2 did not significantly vary. On the other hand, the CTE value in CSA-OPC and T3A mixtures was not significantly different from each other, but were lower than CSA, PrCSA1, and PrCSA2 mixtures.

A low CTE is desired to limit the amount of thermal cracking developed in service as well as keeping stresses developed at the bond interface low to prevent debonding. A direct comparison of the substrate and overlay CTE is not all encompassing, as to accurately determine a pavement's performance the temperature gradient through the layers needs to be considered (O. Selezneva, 2004). Most work evaluates CTE for pavement structures.

One author ran mechanistic empirical models for bonding concrete pavement to an asphalt surface. They found that in a mild climate if the CTE of the pavement is < 9 microstrain/°C the pavement would just be reaching the failure criteria for cracking after 30 years (Gauhar Sabih, 2018). Considering bridge deck overlays contain reinforcing steel, the time to reach severe thermal cracking would likely be greater. For continuously reinforced concrete structures, thermal cracking is controlled by the tensile capacity of the rebar. Sabih also found that thicker overlays will better control cracking related to the CTE of the material, with lower CTE materials needing thinner overlays to achieve the same performance as higher coefficients with thicker applications.

4.2.2.3 Bond

Due to the difference in surface preparation (demolition hammer in the laboratory versus hydrodemolition in the field) as described in Section 3.4.2.2, the results of bond strength are not representative of values obtained in the field. According to ODOT bond strength obtained in the field typically meets or exceeds the 175 psi minimum. Figure 4-15 shows the bond strength of HESC used in this project as measured in the laboratory setting.

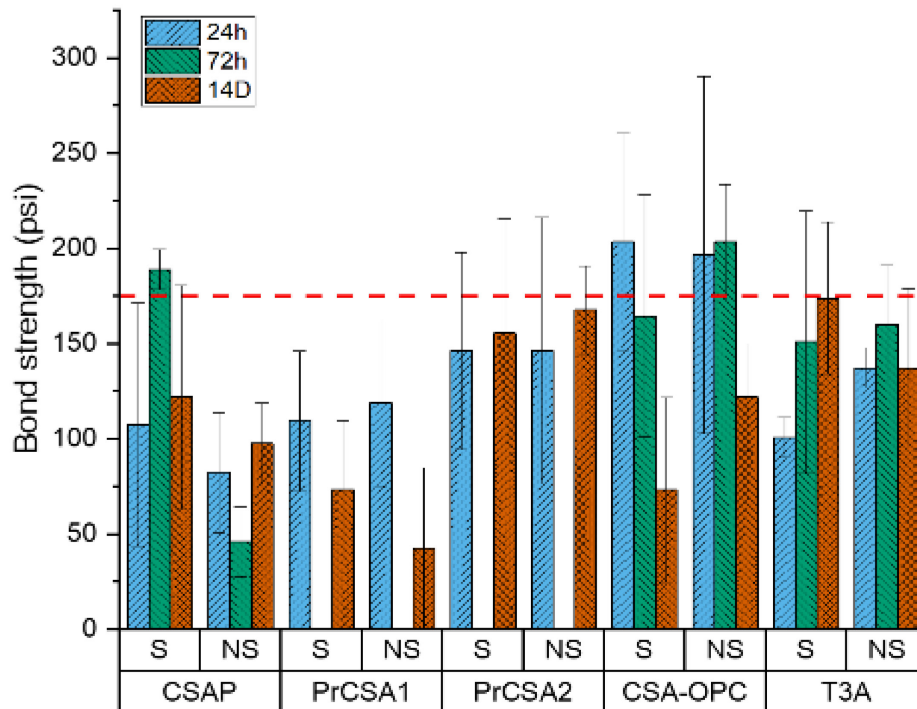


Figure 4-15 Tensile bond pull of strength for each concrete with (S) and without (NS) scrub coat subgrade preparation. Error bars are one standard deviation

The target bond strength according to ODOT specification was 175 psi within 28 days of placement of the overlay (ODOT, 2021). Reduction in average values of bond strength after 14 days was observed in CSA-LPA, PrCSA1, PrCSA2, and CSA-OPC mixture compared to value at 24 hours. However, in the T3A mix, the bond strength values either gradually increased or remained the same after 14 days depending on the scrubbing condition. It was also observed that the T3A mix with a scrub coat applied to the surface was able to satisfy the bond strength requirement.

A statistical analysis was performed on the results of the pull off testing. A two-sample t-test with unequal variances was done to compare the strength difference at 24 hours and 14 days for each of the overlay materials. The effect of a scrub coat treatment and no scrub coat treatment was also compared. The null for testing was that there is no

difference in the mean pull off value for different subgrade preparations and time of testing. The summary of statistical values can be seen in Table 4-8 below, with a significance value of 0.05 for comparison. The null hypotheses for comparison are as follows:

- Null 1: The mean pull off is not statistically different regardless of scrub coat or no scrub coat
- Null 2: The mean pull off is not statistically different for testing at 24 hours or 14 days.

Table 4-8 Summary of two-tail P statistics comparing pull off strength at different ages and subgrade preparation.

Pairwise comparison	Null	P ($T > t$) two-tail CSAP	P ($T > t$) two-tail PrCSA1	P ($T > t$) two-tail PrCSA2	P ($T > t$) two-tail CSA-OPC	P ($T > t$) two-tail T3A
24hr S vs 24hr NS	1	0.52	0.78	1.00	0.91	0.02
14d S vs 14d NS	1	0.55	0.40	0.76	0.23	0.33
24hr vs 14d S	2	0.77	0.39	0.84	0.02	0.09
24hr vs 14d NS	2	0.48	0.07	0.60	0.23	1.00

From the analysis, there is generally little statistically significant difference between the pull-off strength for different subgrade preparation and length of time before testing. The T3A at 24 hours is the only system that rejects Null 1 at a significance value of 0.05. There are some values in a “gray-area” close to the significance level. The sample size would need to be increased to evaluate for significance further. For most systems, the time of testing is also not significantly different except for the CSA-OPC system with a scrub coat where the 24-hour and 14-day pull off values are significantly different. This system shows a large decrease in strength at 14 days, which could be due to the large variation in values during testing. The ASTM standard does not have a criterion for variation of this testing method and reports one study that recorded values from 60 – 500 psi for three replicate tests (ASTM C1583).

To be considered passing ODOT specifications, a pull off strength of 175 psi should be reached or exceeded. The main surface preparation technique used for Oregon DOT bridge deck overlays is hydrodemolition which was difficult to replicate in laboratory conditions which were done mechanically. It was particularly difficult to achieve a uniform surface removal depth. The results from bond strength evaluation showed high variability. After 14 days, mixtures containing a proprietary pore filling additive did not achieve the ODOT required bond strength of 175 psi. Further, the remaining mixtures contained values falling both above and below the 175 psi minimum. In the case of PVA modifiers and increase latex to binder ratio can decrease bond capacity [Mehri, B. et al., 2022]. It is recommended that bond strength be evaluated in the field (as is current practice) without any change to existing ODOT specifications.

4.2.3 Task 4: Volume change and cracking risk

The results for AASHTO T363 dual ring evaluation on the CSA2 mixture are shown in Figure 4-16. The air content of these mortar mixtures was not determined.

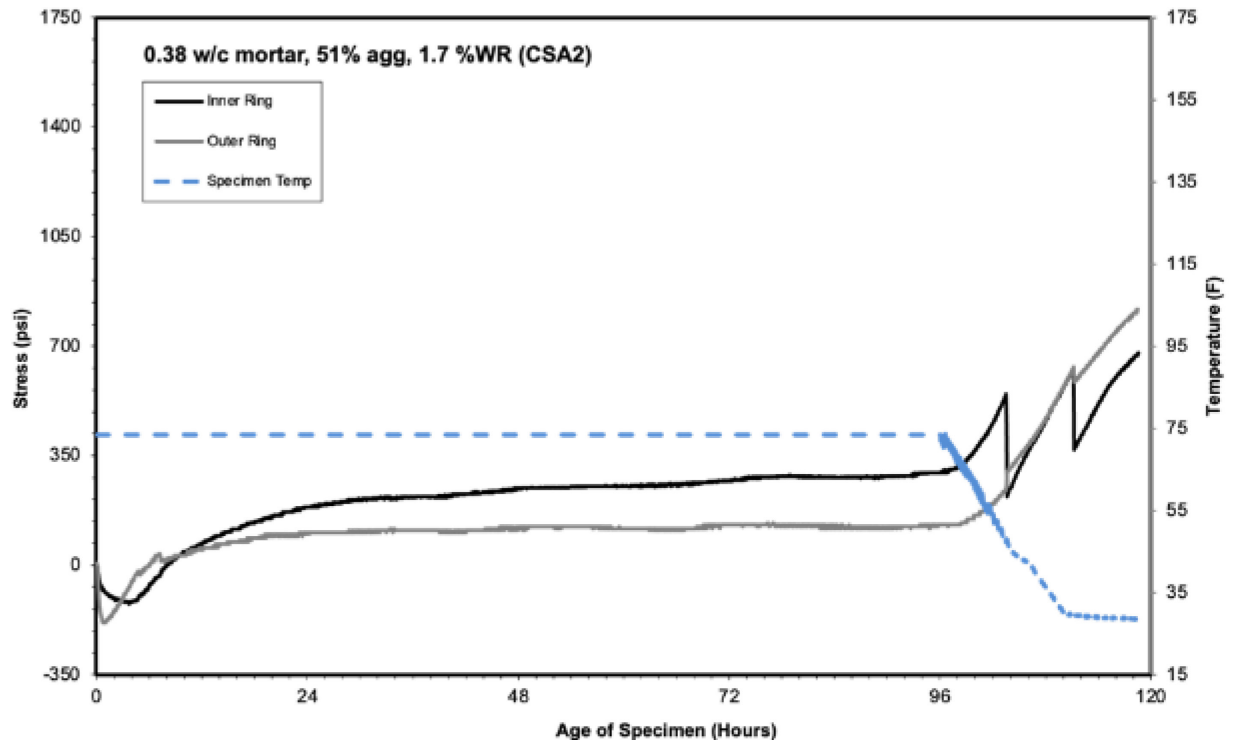


Figure 4-16. AASHTO T363 dual ring results for CSA 2

The CSA2 mixture showed early-age expansion as evidenced by the decrease in stress from 0 MPa after casting. This was followed by a gradual shrinkage with a maximum shrinkage stress before cooling of 294 psi. No cracking was observed prior to artificial cooling. This mixture first exhibited cracking at a temperature of 48°F.

Figure 4-17 and Figure 4-18 show the AASHTO T363 dual ring results for the two proprietary CSA mixtures, PrCSA2 and PrCSA1, respectively.

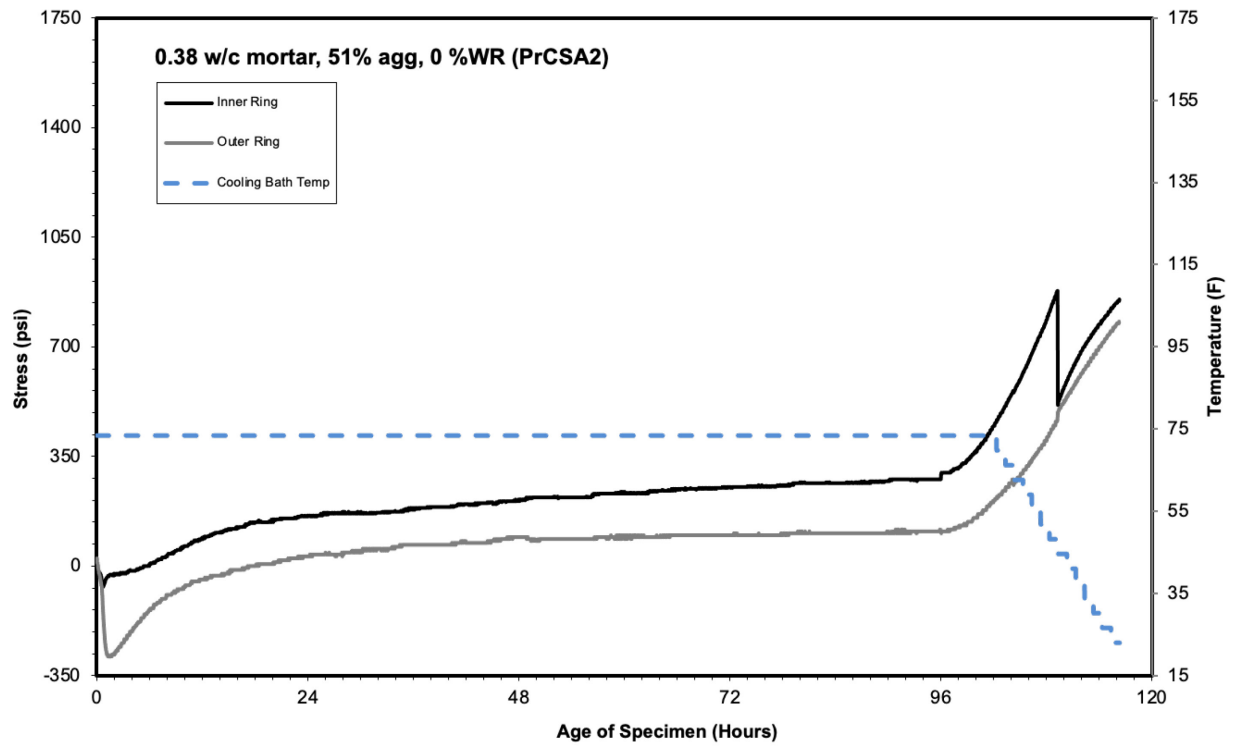


Figure 4-17. AASHTO T363 dual ring results for PrCSA2

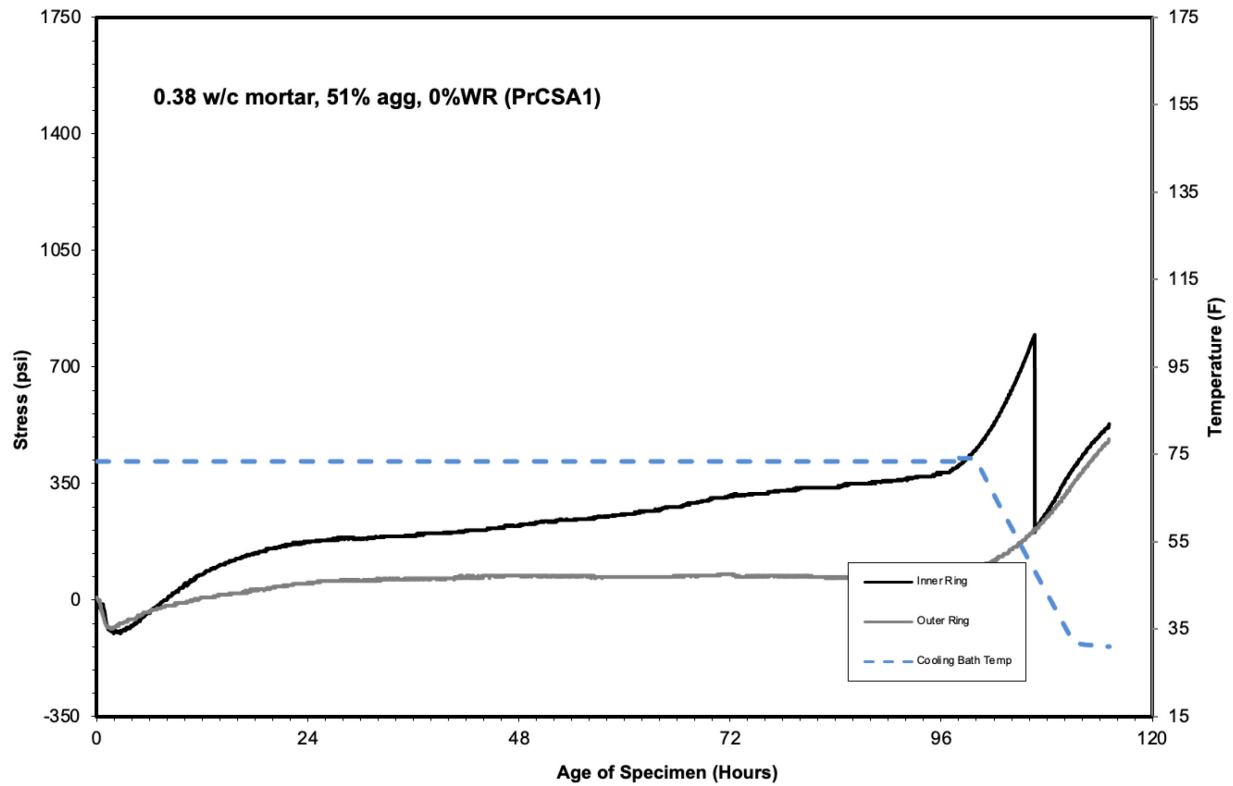


Figure 4-18. AASHTO T363 dual ring results for PrCSA1

Both proprietary CSA mixtures showed early age expansion during hydration followed by gradual increase in shrinkage. Prior to cooling neither mixture exhibited cracking. PrCSA2 cracked at a slightly lower (45°F) temperature than PrCSA1 (49°F).

The AASHTO T363 dual ring results for the CSA/OPC blend are shown in Figure 4-19.

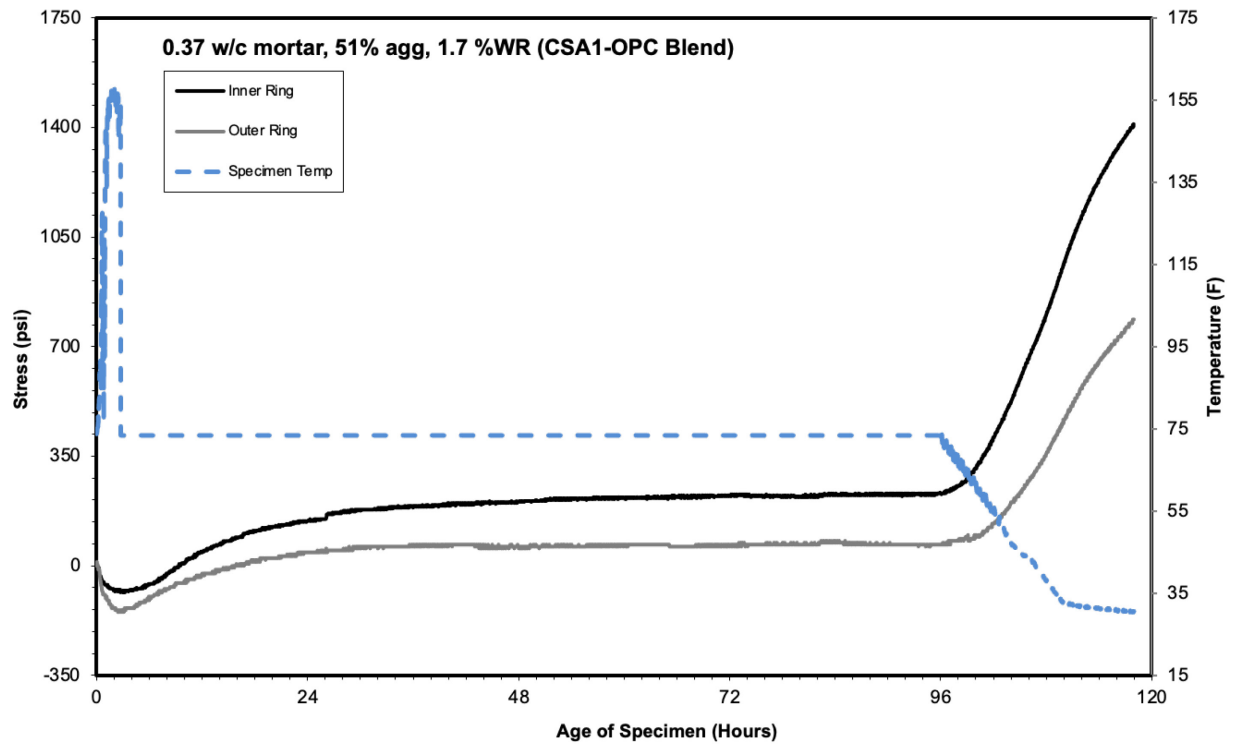


Figure 4-19. AASHTO T363 dual ring results for CSA-OPC Blend

The CSA-OPC blend also showed early age expansion due to heat of hydration. No cracking was observed prior to cooling. The maximum shrinkage stress observed prior to cooling was 228 psi. During cooling to 32°F no cracking was observed for this mixture indicating a high resistance to cracking.

Figure 4-20 shows the results of the dual ring testing for the T3A mixture.

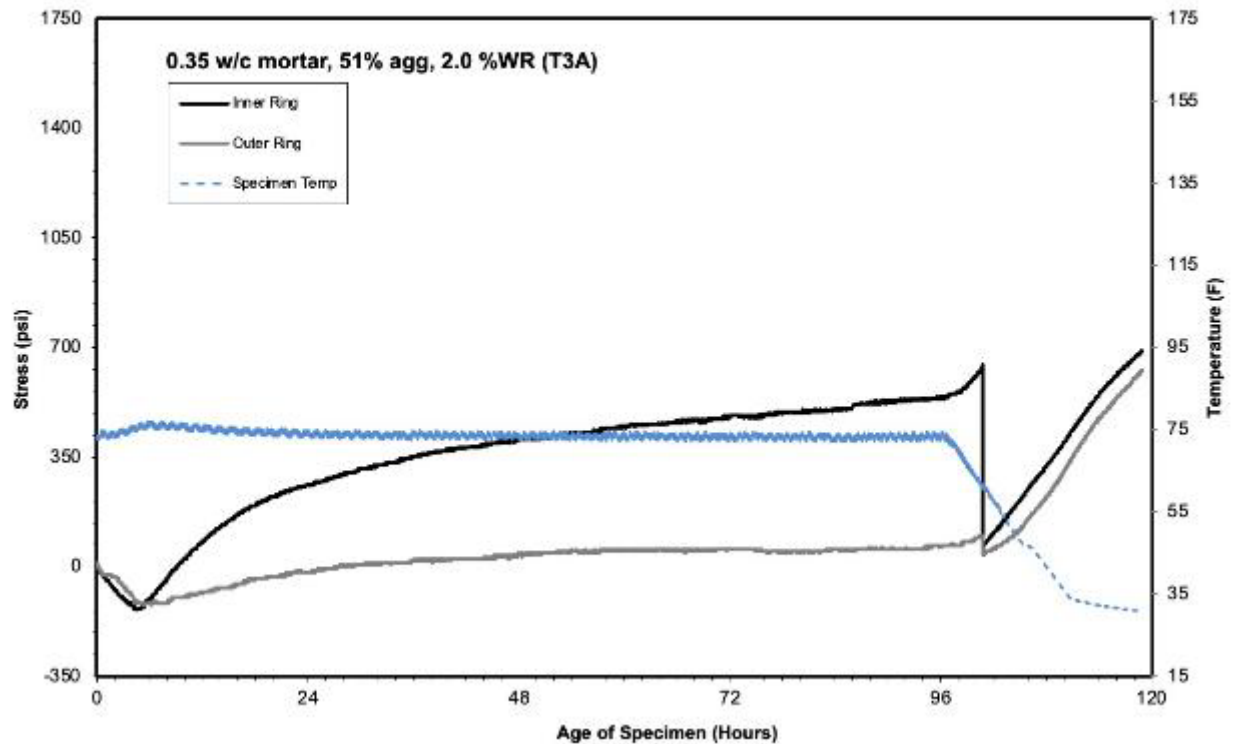


Figure 4-20. AASHTO T363 dual ring results for T3A

The T3A mixture showed a more rapid increase in shrinkage stress between 0-24 hours after casting compared to the other mixtures in this study. It also exhibited the highest cracking temperature (61°F) upon cooling. This mixture also showed a very high strength gain and could indicate a mixture more susceptible to cracking than other mixtures in this study.

The results of dual ring testing for the HPC mixture are shown in Figure 4-21.

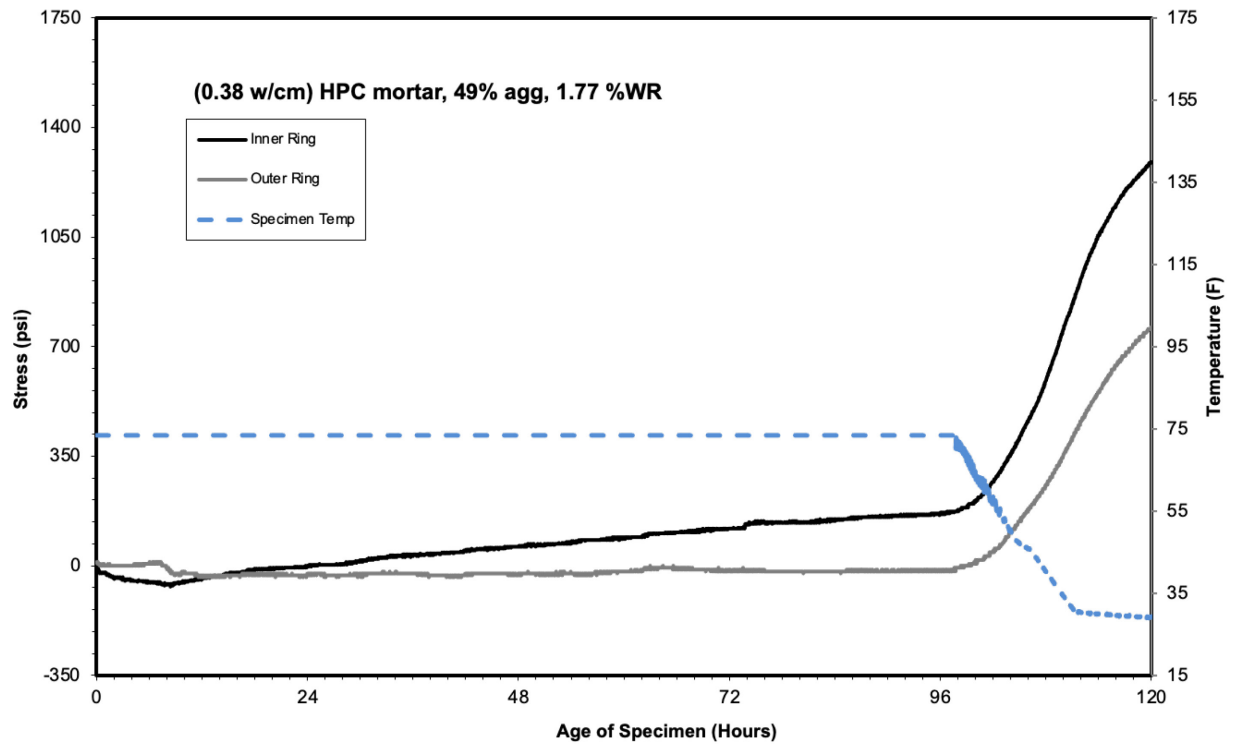


Figure 4-21. AASHTO T363 dual ring results for HPC

For the HPC mixture very little early age expansion was observed and shrinkage after the first 24 hours was gradual and low. Upon artificial cooling this mixture did not show any cracking to 32°F. It should be noted that the maturity of this mixture would not be expected to be the same as the other mixtures which gain strength rapidly (e.g. in the first 3-24 hours). However, this does follow the recommended practice in AASHTO T363.

Table 4-9 provides a summary of the AASHTO T363 dual ring data.

Table 4-9. Summary of AASHTO T363 dual ring data including maximum expansive stress, stress at first cracking, and cracking temperature

Mixture ID	Maximum Expansive Stress (psi) <i>Outer ring</i>	Maximum stress before cooling (psi) <i>Outer ring</i>	Maximum Shrinkage Stress Before Cooling (psi) <i>Inner ring</i>	Stress at First Cracking (psi)	Cracking Temperature (F) and ΔT
CSA2	-184	129	294	544	48 (25)
PrCSA2	-289	104	277	876	45 (28)
PrCSA 1	-86	83	377	790	49 (24)
CSA/OPC	-148	67	228	1403*	N/A (43)
T3A	-123	74	648	737	61 (12)
HPC	12	12	174	1289*	N/A (44)

Note negative stress indicates compression (e.g. expansion), positive stress indicates tension (e.g. shrinkage)

**No thermal cracking observed, maximum stress at end of test*

The ΔT represents the change in temperature from 73 °F (the initiation of cooling) until cracking (or no observation of cracking). No mixtures exhibited cracking prior to artificial cooling. The CSA/OPC and HPC mixtures were the most resistant to cracking with no cracking observed even during the artificial cooling period to 32°F. The CSA, PrCSA2, and PRCSA1 mixtures all exhibited a relatively similar behavior and a cracking temperature between 7-9.3°C. The stress at first cracking was similar for PrCSA1 and PrCSA2 (790 psi and 876 psi, respectively) and lower for CSA2 (544 psi). The mixture with the highest cracking temperature (least crack resistant mixture) was T3A.

4.2.4 Task 5: Durability

4.2.4.1 Freeze-Thaw

The results for samples exposed to freeze-thaw cycling, expressed in terms of the relative dynamic modulus, are shown in Figure 4-22. The air contents are provided in Table 4-6 with full mixture proportions. An abbreviated Table 4-10 for quick reference is shown below

Table 4-10 Finalized concrete air contents and air entrainer dosages used in ASTM C666 testing.

Constituents	CSAP	PrCSA1	PrCSA2	CSA-OPC	T3A	HPC
Air entrainer, oz/cwt	34	14	0	3	40	17
Air content, %	5	5	10	10	9	7

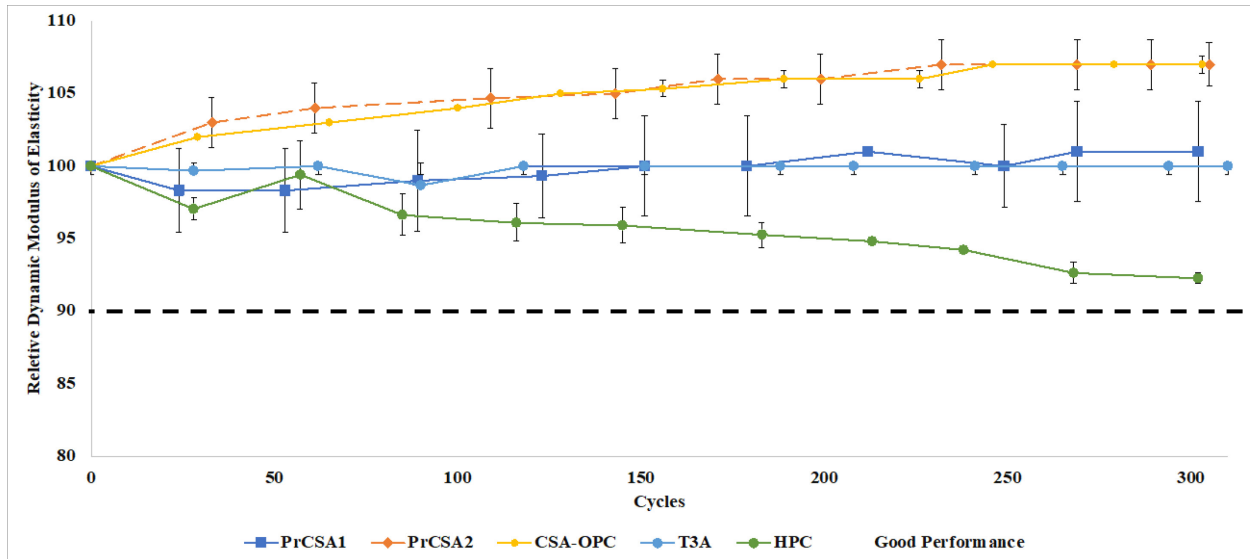


Figure 4-22 Relative dynamic modulus of elasticity RDME for concrete prisms undergoing ASTM C666 freeze-thaw testing. An RDME of 90% or more indicates good performance, 60% is failing. Error bars are one standard deviation

Most of the CSA concretes maintained a RDME of 100 or more through the entire exposure length. With increased air content and adequate 28-day strength, these designs were able to withstand the freeze-thaw cycling. The HPC samples were used as the control group. From the results the concretes made with CSA cement performed as well or better than the HPC design. Even in CSAs with air contents below the typical air content of 6-7% that is usually desired in aggressive freeze-thaw conditions, higher performance was observed compared to the HPC control. This could be due to the coarser microstructure of CSAs (de Bruyn, 2017)

The CSAP mixture did not perform as well as other mixtures as shown in Figure 4-23 likely due to consolidation issues at the time of casting.

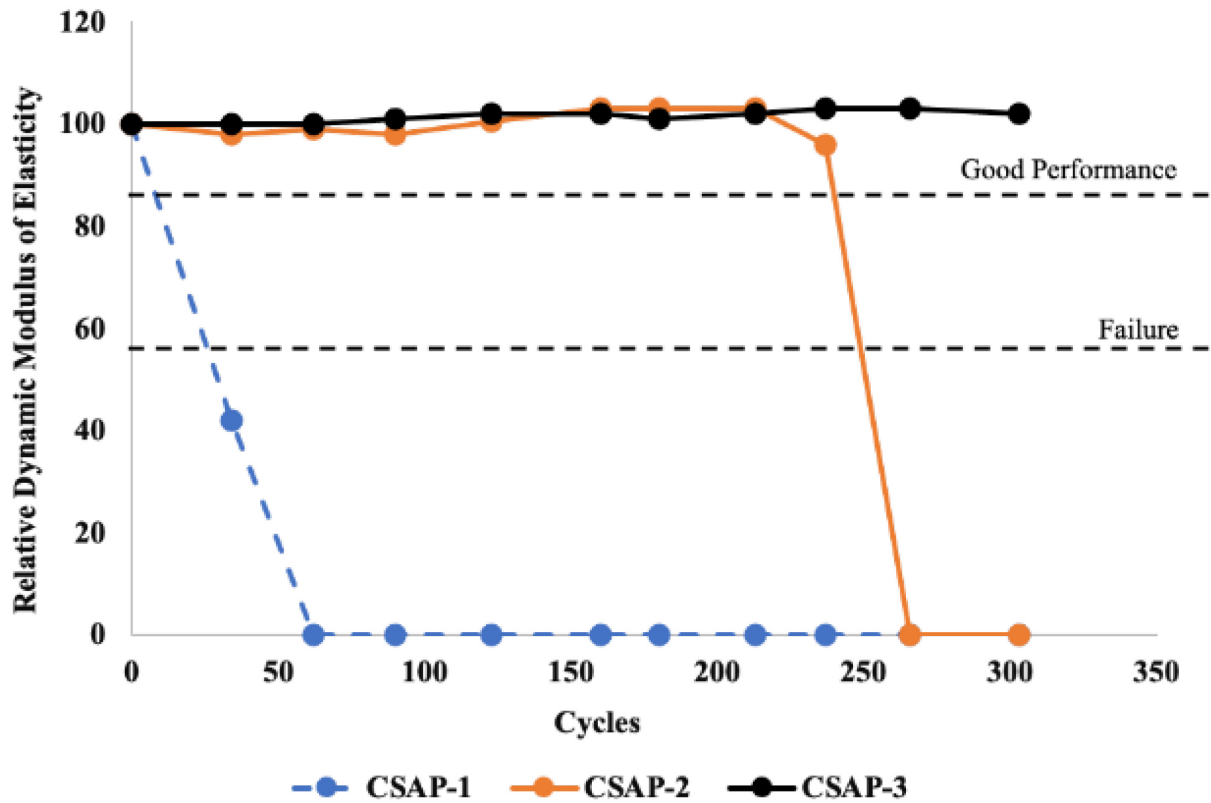


Figure 4-23 All samples tested in freeze-thaw for CSAP

Two of the samples had poor compaction due to lack of working time for the number of samples needed. Sample 1 dropped past the failure criteria after the first set of cycles. Sample 2 had a sudden failure shortly after with less than 50 cycles remaining. The specimen had lost 4.5% of its mass, all from the edge, and the frequency could not be recorded due to deterioration. Sample 3 lasted the entire duration and showed no signs of cracking, pop outs, or scaling. The early failures can likely be attributed to the poor compaction in casting and not the material itself.

4.2.4.2 Electrical Properties and formation factor

The bulk resistivity of each design was recorded following procedure found in literature. The results at 24 hours and 28 days can be seen in Table 4-11.

Table 4-11 Sealed and saturated resistivities of concrete cylinder at 24 hours and 28 days

Mix ID	Average Bulk Resistivities (kΩ.cm) 24 hr Sealed	Average Bulk Resistivities (kΩ.cm) 24 hr Saturated	Average Bulk Resistivities (kΩ.cm) 28 days Sealed	Average Bulk Resistivities (kΩ.cm) 28 days Saturated
CSA-P	67.10	-	125.06	140.50
PrCSA1	38.50	-	60.79	93.70
PrCSA2	44.30	-	75.81	27.90
CSA-OPC	40.40	-	53.95	14.03
T3A	2.45	-	6.38	8.09
HPC	3.06	-	16.20	20.21

The concretes made with CSAs have rather high resistivities as compared to T3A and HPC mixtures. The high resistivities of the CSAs may be due to the formation of large amounts of ettringite, which has a low density and high consumption of water, resulting in a lack of total pore water which increases the resistivity of the mixture (Gjorv, 1977). For these mixtures, it is possible the modifications might be needed in the testing setup or sample conditioning for the calculation of formation factors. The resistivity values for the HPC and T3A are similar to those that are found in the literature for high performance mixtures (Qiao, C., 2018). Because resistivity is an indicator of the resistance of concrete to chloride penetration, these materials should be very resistant to chloride ingress.

The samples vacuum saturated with calcium hydroxide solution showed a decrease in the concrete resistivity for all CSA samples, and an increase for the T3A and HPC. These two mixtures saw an increase in resistivity because the saturated lime solution is more resistive than the pore solution of the concrete.

Figure 4-24 shows the resistivity and pH over time for the paste pore solutions at 24 hours to 28 days.

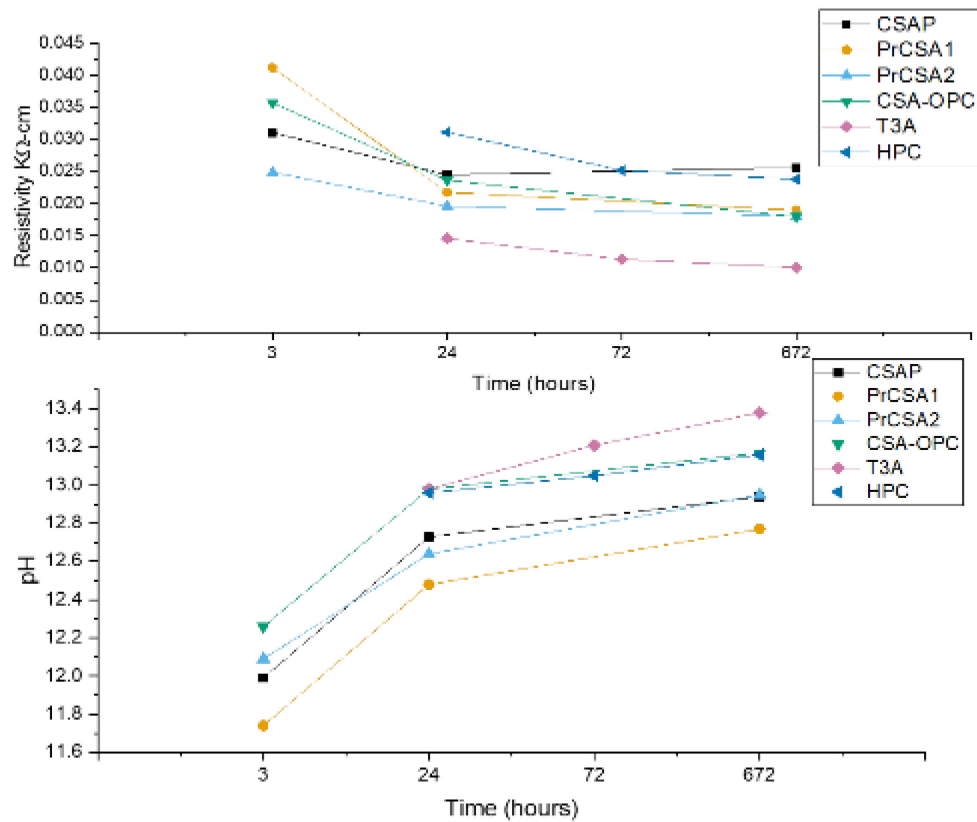


Figure 4-24 Resistivity (top) and pH (bottom) of pore solutions from paste samples for different length of sealed curing

There is a decrease in pore solution resistivity for all cements at later ages. From 3-hours to 24-hours is a more significant change in resistivity than from 24-hour to 28-day. After 28 days of curing, all CSA pore solutions had a pH above 12.5, which should have allowed for the passivation of reinforcing steel. The addition of 20 percent OPC to the CSA increased the pH from 12.8 to 13.1, matching the concentration of the HPC. The highest pH was seen in the T3A sample, which also had the lowest resistivity. The ion concentrations for the pore solution of the same samples above are shown Table 4-12.

Table 4-12 Ion concentrations of paste pore solution (w/c=0.5) at 3 hours, 24 hours, and 28 days for CSAs and 24 hours, 72 hours, and 28 days for HPC and T3A

Mix ID	Na⁺ (ppm) 3hr	Na⁺ (ppm) 24hr	Na⁺ (ppm) 72hr	Na⁺ (ppm) 672hr	K⁺ (ppm) 3hr	K⁺ (ppm) 24hr	K⁺ (ppm) 72hr	K⁺ (ppm) 672hr
CSAP	1380.7	1370.9	-	1433.8	7882.6	8093	-	7403.8
PrCSA1	6101.9	5933.4	-	6284.2	6721.7	6381.4	-	6619.9
PrCSA2	5897.5	6172.3	-	6528.2	6776.5	6773.5	-	6938
CSA- OPC	1121	1112.7	-	1429.1	6198.3	6198.7	-	6428.5
T3A	-	4159.8	5030.7	6085.7	-	7908.6	9565.8	12334.7
HPC	-	1058.8	1449.5	2329.3	-	2133.3	2438.9	2034.9
Mix ID	Ca (ppm) 3hr	Ca (ppm) 24hr	Ca (ppm) 72hr	Ca (ppm) 672hr	Cl (ppm) 3hr	Cl (ppm) 24hr	Cl (ppm) 72hr	Cl (ppm) 672hr
CSAP	3.3	0	-	0	1350.3	1411.5	-	1525.2
PrCSA1	6.2	5.3	-	6.3	1318.8	1416	-	1463.6
PrCSA2	12	1.4	-	0	1446.4	1533.4	-	1563.3
CSA- OPC	0	0	-	0	849.3	807	-	240.6
T3A	-	285.9	142.9	52.3	-	66.6	54.1	63.9
HPC	-	257.1	212.9	41.7	-	56.7	56	60.8
Mix ID	S (ppm) 3hr	S (ppm) 24hr	S (ppm) 72hr	S (ppm) 672hr	Al (ppm) 3hr	Al (ppm) 24hr	Al (ppm) 72hr	Al (ppm) 672hr
CSAP	2949	2200.9	-	1478.1	180.2	82.4	-	0
PrCSA1	5429.8	4466.9	-	2290.3	768.9	136.4	-	171.6
PrCSA2	5936.6	4653.3	-	1972.1	48.9	118.9	-	187.2
CSA- OPC	1819.8	944	-	156.3	386.3	9.2	-	0
T3A	-	103.4	40.9	82.3	-	0	0	0
HPC	-	39.3	0	4	-	0	0	0

The CSA pore solution has a relatively high ion concentration of Na and Cl compared to the HPC and T3A paste samples. The concentration of S ions in the CSAs is significantly greater than the OPC systems. This could lead to embedded steel struggling to form a dense passivation layer (L. Wang, 2019). The T3A shows an increased release of K ions from 24 hours to 28 days, possibly related to the chemical accelerator.

Using the sealed resistivities of concrete and pore solution, the sealed formation factor over time is calculated. The saturated resistivities along with the resistivity of the saturated lime water are used to calculate the saturated formation factor. The results are shown in Table 4-13.

Table 4-13 Apparent (sealed) and true (saturated) formation factor

Mix ID	Formation Factor 24 hr Sealed	Formation Factor 24 hr Saturated	Formation Factor 672 hr Sealed	Formation Factor 672 hr Saturated
CSAP	2737.7	-	4877.3	1433.7
PrCSA1	1767.2	-	3203.6	956.1
PrCSA2	2259.3	-	4199.9	284.7
CSA-OPC	1708.9	-	2999.6	143.2
T3A	168.1	-	634.2	82.6
HPC	98.2	-	680.4	206.2

The values for apparent formation factor (sealed) of concretes made with CSA are extremely high as compared to typical concretes (Isgor, 2020), mainly because of their high bulk resistivity, as discussed above, as the pore solution resistivities of all mixtures were rather similar. The value of apparent formation factor is largely influenced by the degree of saturation, pore connectivity, and the pore solution chemistry. As mentioned previously, the density and water consumption of ettringite in the CSA systems is likely one of the factors causing these results.

The degree of saturation for the samples is in Table 4-14. Dried samples were placed in a 230°F oven until 2 consecutive readings where the same mass was recorded.

Table 4-14 Degree of saturation for 3 cylinders of each mixture

Mixture ID	Sealed	Dried	Saturated	Average DoS	SD DoS
PrCSA1	8.62	8.12	8.44	156.34	5.87
PrCSA2	7.90	7.41	8.07	74.71	0.54
CSAP	8.53	8.03	8.38	144.08	6.91
CSA-PC	8.68	8.18	8.73	90.63	5.66
T3A	8.65	8.24	8.66	96.52	1.23
HPC	8.73	-	8.82	98.91	0.22

A particularly interesting result is the PrCSA1 and CSAP, both of which contained a latex modifier; these samples were not able to be fully re-saturated under vacuum following drying as indicated by an average DoS above 100%.

4.2.4.3 Critical chloride threshold

The properties of the mortars used in testing are outlined below in Table 4-15 as the average of three samples.

Table 4-15. Mortar compressive strength and bulk resistivity
OC_{crit} Mortar Summary

Mixture	Compressive Strength ksi (MPa)	Bulk Resistivity (k Ω .cm)
CSAP	7.8 (54.0MPa)	132.0
PrCSA1	9.4 (57.9)	104.5
PrCSA2	5.7 (39.5)	7.2
CSA-OPC	9.4 (57.9)	4.9
T3A	16.7 (115.5)	6.7
HPC	5.1 (35.1)	6.8

Like the concrete mixtures, the CSAs containing latex modifier exhibited a high resistivity value. The PrCSA2 mixture has a resistivity that is like the T3A and HPC mixture. The compressive strength of the cubes also followed the same trends as the concrete mixtures with T3A having been significantly higher than the other materials.

Results for half-cell potential measurements of samples with the same anodic and cathodic environments are shown in Figure 4-25. All three materials show distinct decrease in the OCP measurement below -350 mV which are interpreted as the activation due to corrosion. The HPC samples required the highest time to activation at an average of around 30 days. One sample met the activation criteria at only 3 days, indicating it may be an outlier because of poor compaction or a defect that was not observed and fixed prior to testing. and in chloride testing had a total chloride content close to 0. The T3A sample took the second longest to activate all samples with an average of 17 days. Time to activation was like the T3A samples for the PrCSA1 mixture. The acid and water-soluble chlorides were determined for each sample following activation. Figure 4-26 show these values for the mixtures in Figure 4-25.

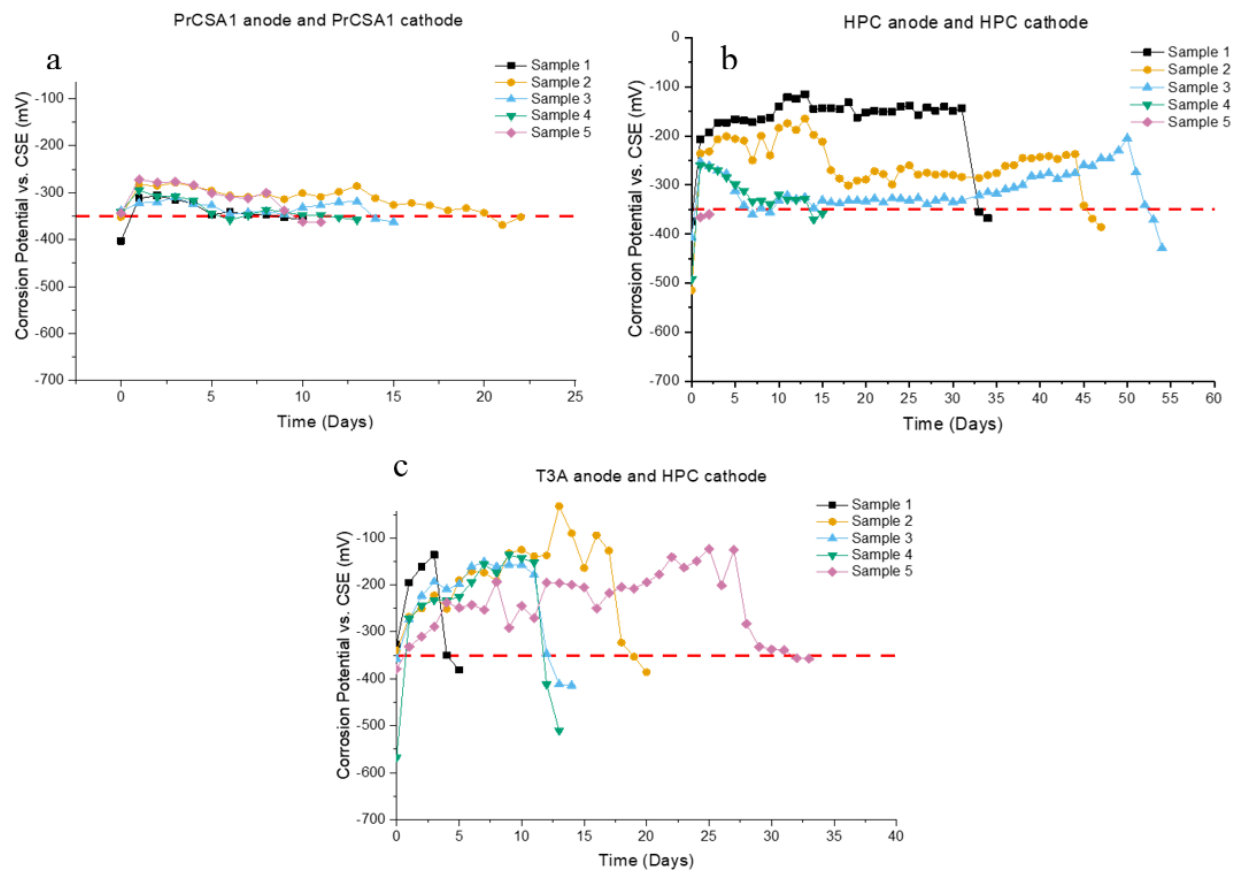


Figure 4-25 Half-cell potential of same anodic and cathodic environments for a) PrCSA1, b) HPC, and c) T3A

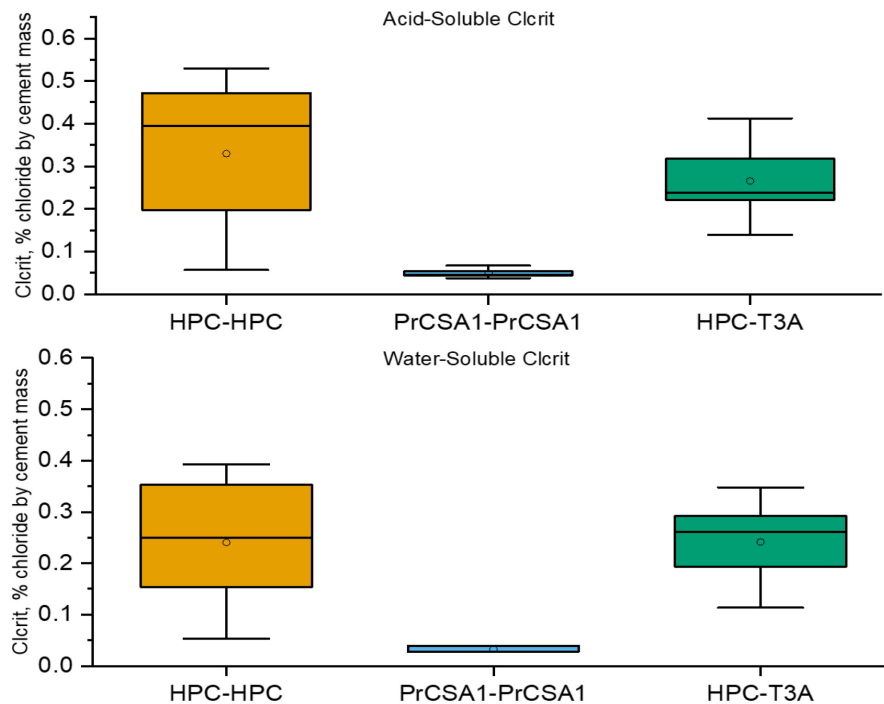


Figure 4-26 Critical chloride threshold as percent mass of cement for same anode and cathode environment with total (top) and free (bottom) chlorides

One sample for water-soluble chlorides in the PrCSA1 reported as a negative value with respect to cement mass. This is not physically possible and should just be considered as a zero. The HPC and T3A samples observed similar Cl_{crit} values for water-soluble chlorides. With total chlorides the HPC mixture has a higher value, likely due to additional binding from the SCMs. An average of 77% of the total chlorides were free chlorides in for the HPC, so nearly a quarter of the total chlorides were being bound transporting through the mortar. The T3A samples had similar results with about 85% of the chlorides.

The PrCSA1 samples had a significantly lower Cl_{crit} at 5-7 times smaller than either of the OPC samples. This likely means that the time to activation was entirely based on the CSAs ability to prevent the transport of fluids with the addition of the latex modifier, but as soon as chlorides were able to reach the steel, corrosion began. This could also be related to the passivation layer of the steel formed in the lower pH of the CSA.

The results for half-cell potential measurements for samples with different environments are in Figure 4-27.

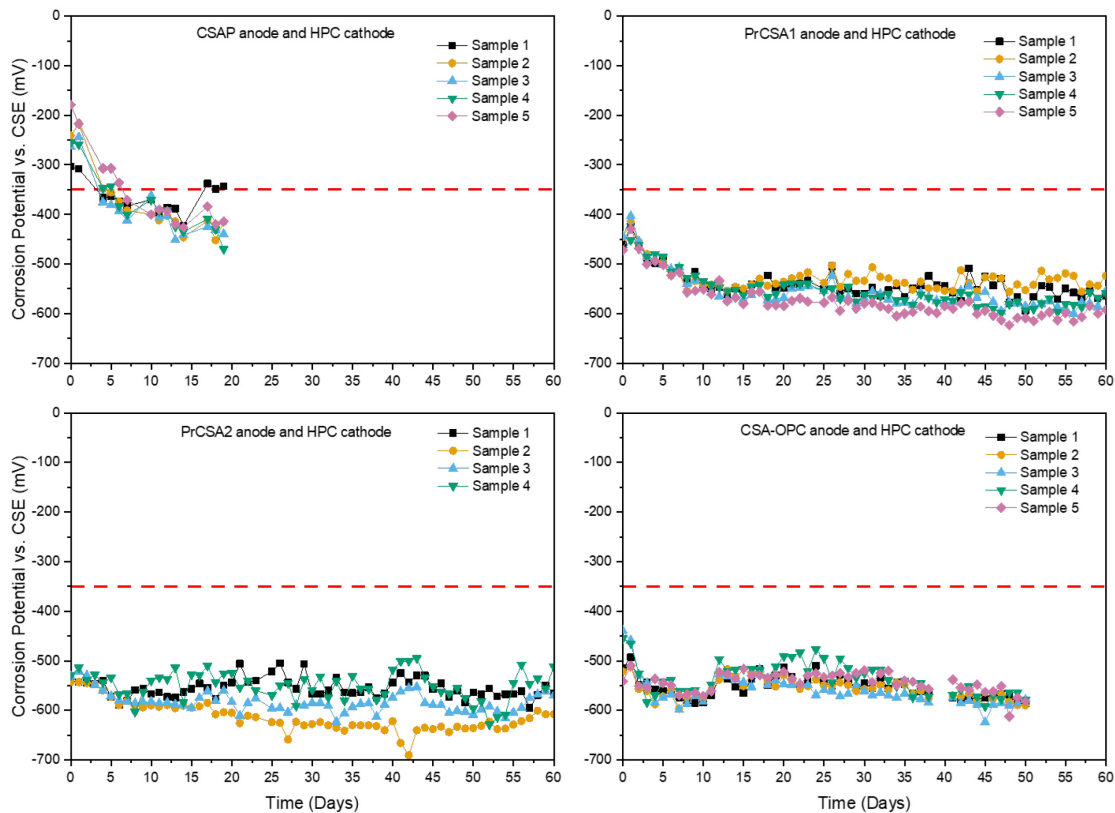


Figure Error! No text of specified style in document.-1 Corrosion potential
Figure 4-27 Corrosion potential measurements of samples with different anodic and cathodic environments

Unlike the OC_{crit} samples with similar anodic and cathodic environments, the 4 cements Figure 4-27 could not be evaluated for critical chloride threshold. When the samples were connected to the cathode, the OCP never became more positive than -350 mV versus the Cu-CuSO₄ electrode, meaning that from connecting there was a 90% chance of corrosion occurring. Samples were left connected and in the exposure solution, with daily measurements being made to try and notice a drop in the OCP that could be related to corrosion activation from chlorides. This was not able to be accomplished, and the samples began showing signs of staining seen in Figure 4-28. The CSAP samples were not exposed to chloride solution. They were submerged in saturated calcium hydroxide solution and connected to a cathode with the HPC mixture.



Figure 4-28. Corrosion precipitate and staining on PrCSA2 sample

With the onset of corrosion, and no way to state the cause, determining the corrosion rate of these samples became necessary.

4.2.4.4 *Corrosion rate*

A typical linear polarization resistance (LPR) scan from the VersaStudios software for a rebar in mortar sample is in Figure 4-29. The top graph is plotting the potential versus the log of the current, and the bottom is the measured potential in volt versus time in seconds. The first 300s of the scan is determining the open circuit potential of the specimen. The LPR scan is beginning at 301s.

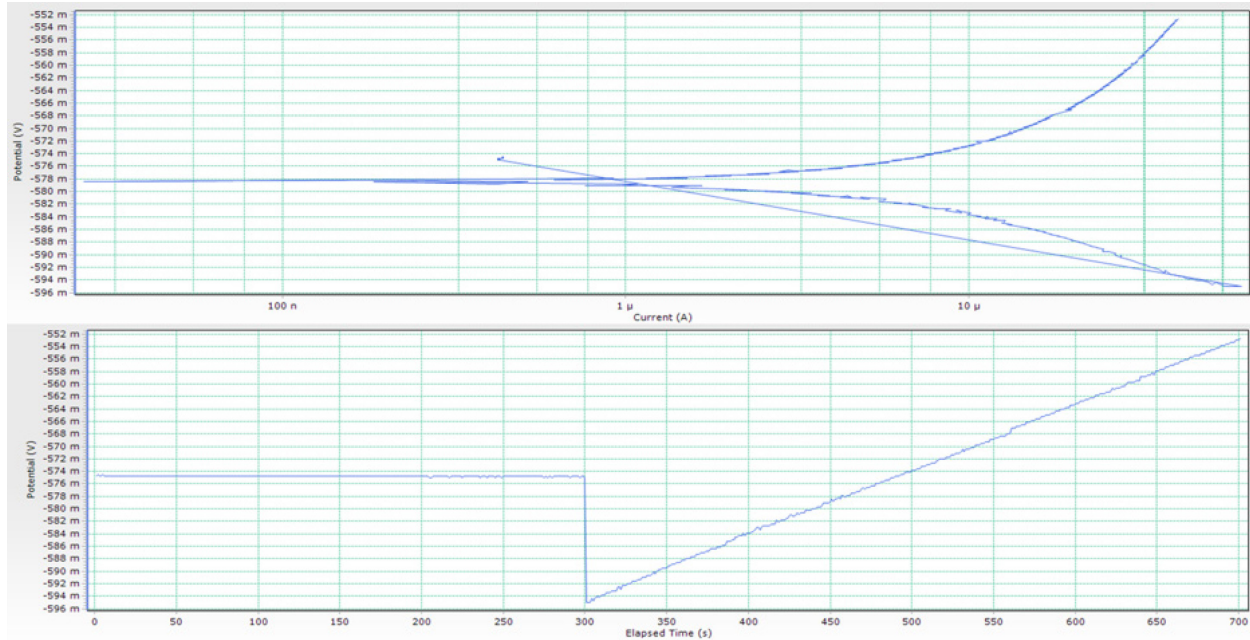


Figure 4-29. The potential vs log(I) (top) and potential vs time (bottom) for LPR measurements

The data for a scan is extracted and used to plot the potential in mV versus the corrosion current density, i . To calculate i , the surface area of the 140-mm long bar with a 16-mm diameter was found to be 73.6-cm². Once plotted, the slope of the line at the OCP is R_p . An example of the $\frac{\Delta V}{\Delta i}$ plots for one of each mortar are in Figure 4-30.

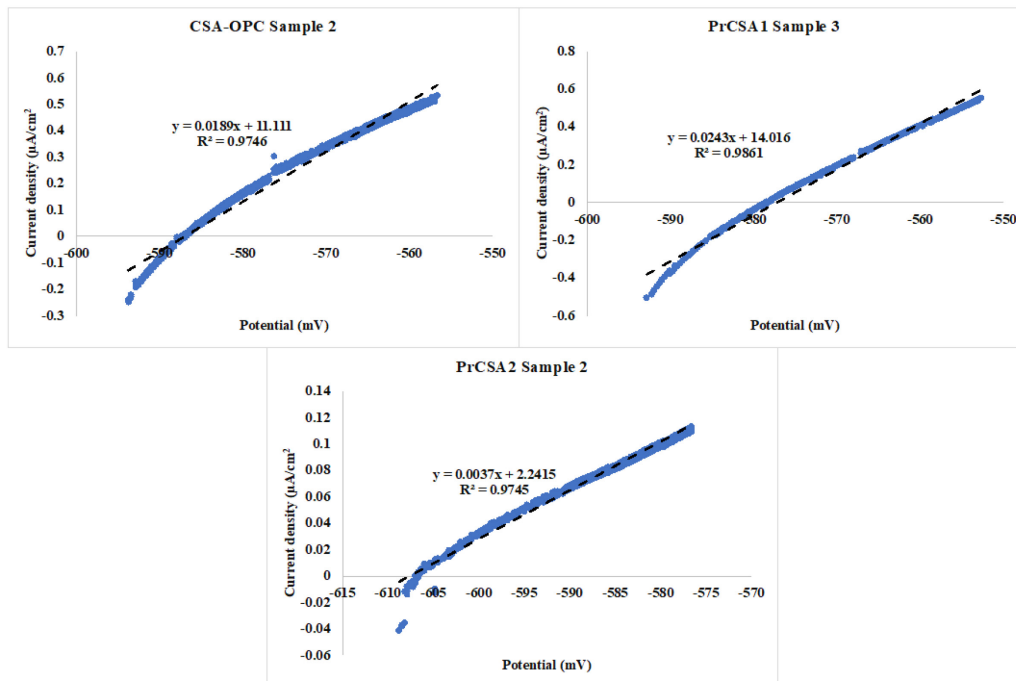


Figure 4-30. Potential versus current density for one of each mortar investigated

All samples from each of the mortar mixes were measured for corrosion rate using the method just described. Table 4-16 show the complete results.

Table 4-16. Results for corrosion rate of steel in mortar

Cement	Sample	R_p ($\Omega \cdot \text{cm}^2$)	B (V)	i_{corr} ($\mu\text{A}/\text{cm}^2$)	CR (mm/yr)
PrCSA1	1	0.020	0.026	1.275	0.015
	2	0.018	0.026	1.436	0.017
	3	0.024	0.026	1.070	0.012
	4	0.022	0.026	1.176	0.014
	5	0.023	0.026	1.156	0.013
	Avg	0.021	0.026	1.223	0.014
	SD	0.002	0.000	0.140	0.002
PrCSA2	1	0.004	0.026	7.027	0.082
	2	NA	NA	NA	NA
	3	0.039	0.026	0.670	0.008
	4	0.020	0.026	1.333	0.015
	5	0.019	0.026	1.376	0.016
	Avg	0.020	0.026	2.602	0.030
	SD	0.014	0.000	2.968	0.034
CSA-OPC	1	0.014	0.026	1.844	0.021
	2	0.019	0.026	1.376	0.016
	3	0.019	0.026	1.354	0.016
	4	0.013	0.026	2.000	0.023
	5	0.003	0.026	10.400	0.121
	Avg	0.014	0.026	3.395	0.039
	SD	0.007	0.000	3.926	0.046

As mentioned, the B value has been well studied for embedded concrete and is 26 mV for active corrosion of reinforcement. The severity for the service life of the reinforcement has also been defined by researchers and shown in Table 4-17 (Andrade, 2005).

Table 4-17 Interpreting values of corrosion current and corrosion rate to severity for the service life of the reinforcement [C. Andrade, et al., 2005]

i_{corr} ($\mu\text{A}/\text{cm}^2$)	CR (mm/yr)	Corrosion level
≤ 0.1	≤ 0.001	Negligible
0.1 – 0.5	0.001 – 0.005	Low
0.5 – 1	0.005 – 0.010	Moderate
> 1	> 0.010	High

Based on the interpretation information in Table 4-17, the results shown in Table 4-16 indicate all the bars are corroding at a high rate. This is likely due to the concentration difference between the anodic and cathodic environments and then further worsened by the chlorides reaching the steel. In this case, the anodic environment has a lower concentration due to the pH of the CSAs (12.6-12.9) while the cathodic environment is the HPC design (pH ~13.1). This difference may be large enough to drive the corrosion of the reinforcement (2-5x different). More research is needed in this area to definitively determine the potential for corrosion when an overlay material is used that has a different chemistry from the substrate material, in particular the pH of the pore solution of that overlay material. It would also be important to compare the results of these laboratory tests to field applications of HESC materials to determine the accuracy of laboratory test methods to the field experience. A summary of average corrosion rate, exposure solution, cathodic connection, and corrosion severity level shows in Table 4-18.

Table 4-18 Summary of corrosion rate testing

Mix ID	i_{corr} ($\mu\text{A}/\text{cm}^2$)	CR (mm/yr)	Severity	Solution	Cathode connection
PrCSA1	1.223	0.014	High	Sat. CaOH+2%NaCl	HPC
PrCSA2	2.602	0.030	High	Sat. CaOH+2%NaCl	HPC
CSAP	4.266	0.049	High	Sat. CaOH	HPC
CSA- OPC	3.395	0.039	High	Sat. CaOH+2%NaCl	HPC

4.2.5 Field evaluation

The Yamhill Crossing Overflow bridge deck (BR# 08492) on Oregon 18 Highway 39 was overlaid with a HESC over the weekend of July 26, 2020. The surface of the deck was prepared by hydrodemolition, and the depth of concrete removal varied with the top of rebar being visible in certain parts as shown in Figure 4-31.



Figure 4-31 Hydrodemolition preparation of Yamhill Crossing bridge deck (BR# 08492), July 26, 2020.

PI Jason Ideker and Co-PI David Trejo were present for the placement of HESC on the first lane of the overlay early in the morning on July 26, 2020. They observed the construction process (Figure 4-32) and met with ODOT personnel led by David Dobson, Nation's Mini-Mix, and the HESC supplier (CTS). The Rapid Set cementitious material supplied was used in conjunction with Strux BT50 fibers. Immediately after final screeding saturated curing blankets, followed by plastic sheeting were placed over the concrete to prevent evaporation and left in place for approximately 3 hours until the bridge deck was readied for opening. More details about the mixture design can be found in Appendix A and in ODOT construction records.



Figure 4-32. Final finishing and curing of Yamhill Crossing bridge deck (BR# 08492), July 26, 2020

On October 4 and 5th, 2022 (overnight), PI Jason Ideker, Postdoctoral Scholar Anuj Parashar, PhD student Lamiya Noor and MS student Jeremy Smith participated with ODOT in a condition survey of the Yamhill Crossing Overflow bridge deck (BR# 08492) on Oregon 18 Highway 39. ODOT was led by Statewide Structural Materials Engineer David Dobson. ODOT conducted a visual survey and chain drag on the overlay to assess its integrity. The chain drag did not reveal any concerning areas of delamination, and the overall integrity of the overlay seemed sound from this part of the investigation.

The research team at Oregon State University also did a visual assessment and a more detailed cracking survey of the eastbound travel lane. Appendix B contains seven figures showing the location, extent and width of any cracking observed in the overlay. While some cracking was observed overall this was considered minor. Most of the cracks were hairline (under 0.010 in in width, visible but not measurable using a standard crack comparator card). Some minor cracking between 0.010 to 0.06 in was observed in discrete locations. Most of any observed cracking occurred in the transverse location and was located over the bents of the bridge deck indicating these were in the negative moment region and caused by movement of the deck not due to shrinkage of the material.

Six, 4 in. diameter cores were taken from three different locations on the bridge deck roughly in pairs of two. These cores were removed intact and sound with a general observation of good bond between the overlay and the existing bridge deck. Two of these cores were used for coefficient of thermal extraction determination as shown in Figure 4-33.

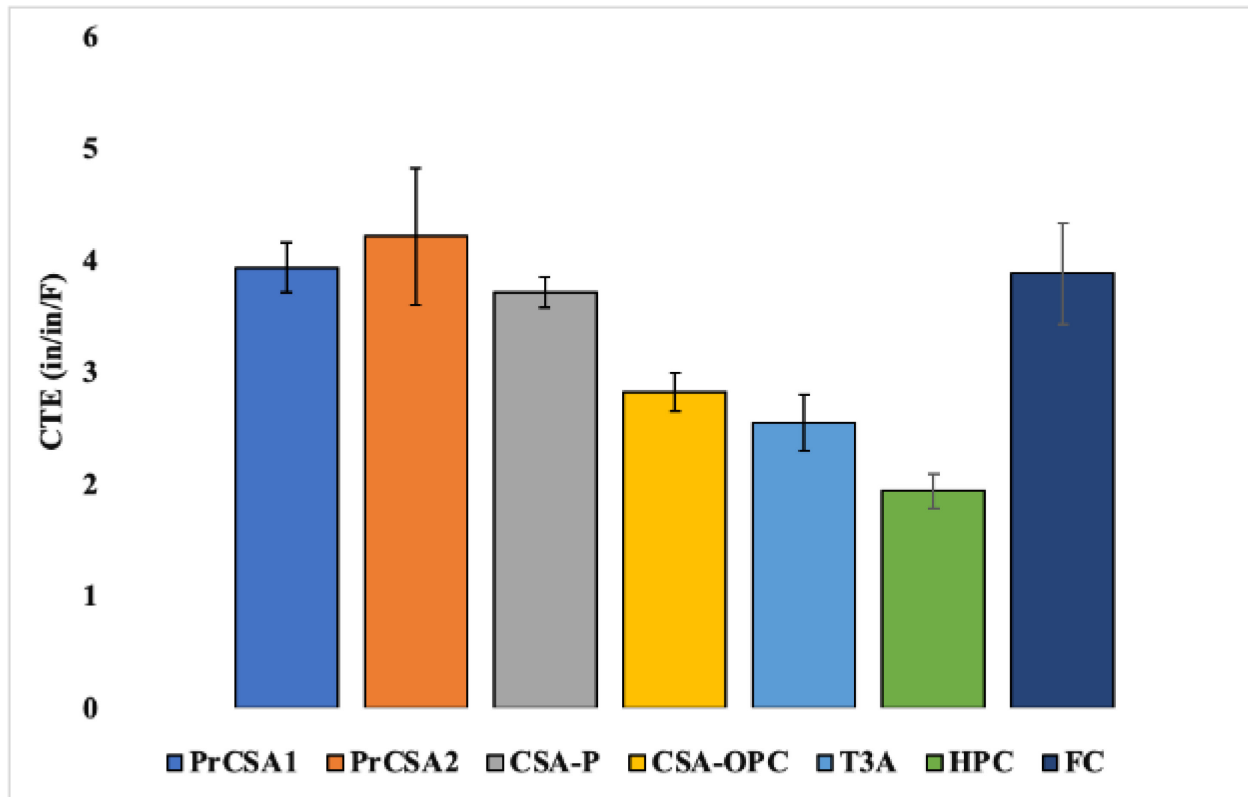


Figure 4-33 Coefficient of thermal expansion of field cores compared to the other HESC materials in this study following AASHTO T336. Note that the field cores contain both the HESC overlay and the underlying substrate concrete. Error bars are one standard deviation

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The objective of this research project is to identify the obstacles or impediments to successfully and reliably using HESC for structural bridge deck overlays on Oregon bridges and to develop the standards, specifications, processes, and practices necessary to using them on ODOT bridges

Rapid strength gain is a desired property for fast and efficient bridge deck overlay reconstruction. Three calcium sulfoaluminate (CSA) cement systems, an accelerated ASTM C 150 Type III ordinary portland cement (OPC), and a CSA-OPC blended system were evaluated in this research project for potential use as high early strength structural overlay materials for Oregon Department of Transportation (ODOT) concrete bridge decks. A high-performance concrete (HPC) mixture that is representative of ODOT bridge deck mixture designs from about the early 2000s to present was also analyzed in this project. In some evaluation approaches the HPC mixture was used as a substrate material to approximate field applications of HESC overlays. Where appropriate, the performance of the HPC was compared to that of the HESC candidate materials (e.g., durability performance). These materials were analyzed for fresh properties including workability, density, and entrained air content. They were also analyzed for mechanical properties namely rapid strength gain with a particular interest in the first 3-6 hours for HESC materials. Other hardened properties that were measured included coefficient of thermal expansion (CTE) and bond strength. The bond strength was evaluated by applying the HESC materials to existing small scale (3 ft², 1 m²) slabs that were made of a similar HPC mixture used in this project but cast as part of an earlier (2015) ODOT project and stored in the outdoor exposure site at Oregon State University. These slabs had the same cementitious materials and content, similar w/cm and aggregates from the same source – though not the same sample. The effectiveness of different curing techniques was evaluated on the HESC materials where the HPC served as a substrate material using neutron radiography. The materials were evaluated for early-age volume change and cracking resistance in a dual ring cracking resistance test. Durability properties investigated included electrical properties (e.g., bulk resistivity), pore solution analysis, freeze-thaw performance, chloride penetration resistance and corrosion rate.

From these investigations the following conclusions were drawn.

- Compressive strength: A 3-hour compressive strength exceeding 3,000 psi (20 MPa) was achieved with CSA concretes even with increased air contents.
- Workability: With proper admixture dosage an adequate working time of at least 15 minutes could be achieved with all cementitious materials in the project.
- Curing: Overall, the use of a curing compound, as indicated by the lowest curing affected zone, was the most effective for all but one mixture in this study.
 - The mixture with 80% CSA and 20% OPC cured with a 3-hour application of wet burlap and plastic sheeting resulted in the lowest curing affected zone (CAZ), indicating this was the most effective curing method.

- Using no curing at all still provided a smaller curing affected zone in any mixtures containing CSA compared to the ordinary portland cement mixtures referenced in the literature.
- For the accelerated ASTM C150 Type III ordinary portland cement (T3A), the most effective curing approach was a curing compound which can be used for early opening of HESC overlays on bridge decks.
- The porosity of mixtures containing CSA were generally lower compared to the T3A mixture, although none of the porosities would be considered high relative to OPC.
- Activation energy: In this project, the derived E_a values of CSA, PrCSA1, PrCSA2, CSA-OPC, and T3A mixture were 54039 J/mol, 43156 J/mol, 52650 J/mol, 86889 J/mol, and 28210 J/mol.
- Bond strength: the concrete surface to be overlaid should be prepared in a clean, SSD condition prior to paving.
- The main surface preparation technique used for Oregon DOT bridge deck overlays is hydrodemolition which was difficult to replicate in laboratory conditions. The results from bond strength evaluation showed high variability. After 14 days mixtures containing a proprietary bonding agent did not achieve the ODOT required bond strength of 175 psi. Further, the remaining mixtures contained values falling both above and below the 175 psi minimum. It is recommended that bond strength be evaluated in the field (as is current practice) without any change to existing ODOT specifications. The main surface preparation technique used for Oregon DOT bridge deck overlays is hydrodemolition which was difficult to replicate in laboratory conditions. After 14 days mixtures containing a proprietary bonding agent did not achieve the ODOT required bond strength of 175 psi. The other mixtures contained values falling both above and below the 175 psi minimum. It is recommended that bond strength be evaluated in the field (as is current practice) without any change to existing ODOT specifications.
- Dual ring cracking evaluation:
 - Early-age beneficial expansion was observed in all mixtures containing CSA. This is in line with the current state-of-understanding, and is a benefit in terms of reducing the potential for early age cracking.
 - In this testing no mixtures cracked prior to artificially cooling the specimens. The CSA/OPC and HPC mixtures were the most resistant to cracking with no cracking observed even during the artificial cooling period to 32°F. The CSA, PrCSA2, and PRCSA1 mixtures all exhibited a relatively similar behavior and a cracking temperature between 45-49°F. The stress at first cracking was similar for PrCSA1 and PrCSA2 (790 psi and 876 psi, respectively) and lower for CSA2 (544 psi). The mixture with the highest cracking temperature (least crack resistant mixture) was T3A.

- Freeze-thaw performance: CSA concretes perform as well, or better, than a common HPC mixture in freeze-thaw cycling even with lower air content.
 - The high strength developed by accelerated ASTM C150 Type III system made air entrainment a lower concern to achieve desired performance.
- Coefficient of thermal expansion (CTE): The CTE of CSA concrete, although higher than HPC, should provide limited thermal cracking concern and good performance as an overlay.
- Resistivity and formation factor: It is critical to measure the resistivity of CSA specimens in a saturated state and to determine the degree of saturation to obtain an accurate representation of transport properties.
 - The resistivity of CSA concrete is very high compared to OPCs if measured immediately after sealed curing and without regard to saturation state. This has been observed by other researchers and is a continuing source of debate in the scientific and engineering communities.
 - The results of these formation factor measurements do not accurately represent the transport properties of these systems. Nor are they directly transferrable to the chloride ingress and potential for corrosion of reinforcing steel in CSA systems. More research in this area is merited to fully elucidate the critical chloride threshold for CSA systems and to ensure proper test methods and sample conditioning are done.
 - There is a strong influence from the hydration of the material, the formation of ettringite which depletes the pore structure of moisture, connectivity of the pores, degree of saturation, and the pore solution chemistry of the material.
- Pore Solution: CSAs have high concentrations of sulfite ions and low concentrations of calcium. The resulting lower pH could be leading to a less well-formed passivation layer on reinforcing steel than what forms in OPC concrete where the pH is typically ~13.4 or higher. CSA has a pH of 12.6 to 12.7 at 28 days. Adding 20% OPC to CSA can increase the pH to above 13. The resistivity of CSA pore solution is not significantly different from OPC or high SCM mixtures.
- Formation Factor: The formation factor of CSA is largely influenced by the degree of saturation. When vacuum saturated in saturated limewater, there is a large decrease in the formation factor of CSA. Latex modifying admixtures prevent full re-saturation of the concrete after drying and thus results incorporating latex should be carefully interpreted.
 - Further modifications to formation factor evaluation may be needed when assessing concrete incorporating latex modifying admixtures.
- Critical Chloride Threshold: Accelerated ASTM C150 Type III OPC systems perform well against chloride ingress and have a critical chloride threshold like HPC. When

exposed to the same anodic and cathodic environments, CSAs containing latex modifiers perform like accelerated OPCs at resisting chlorides. However, the critical chloride threshold of the CSA material is much lower.

- Corrosion Rate: In this research, different cement chemistries were investigated in the OC_{crit} test to simulate a bridge deck overlay where the substrate was HPC, and the overlay was HESC. The corrosion rate of steel in the OC_{crit} test increased when the anodic and cathodic environments of the steel were different in pH. Due to the concentration difference of these half-cells, corrosion initiates almost immediately and is then worsened once the chloride ions reach the (un/lightly) passivated steel.

5.2 LESSONS LEARNED AND RELATION TO PROJECT OBJECTIVE

The objective of this research project was to identify the obstacles or impediments to successfully and reliably using HESC for structural bridge deck overlays on Oregon bridges and to develop the standards, specifications, processes, and practices necessary to using them on ODOT bridges.

5.2.1 Positive Outcomes

The most promising aspects identified from this research were that meeting the 3,000 psi (20 MPa) goal in 3 hours was achievable for all mixtures that contained CSA. The research team also identified that it was possible to accelerate a 100% portland cement concrete to achieve 3,000 psi (20 MPa) in 6 hours, which may still be beneficial to ODOT for certain HESC structural bridge deck overlays. Evaluation of early-age cracking risk showed that all mixtures evaluated should have a low risk of cracking in field applications. The freeze-thaw performance of all mixtures was generally excellent compared to the HPC reference material. Several samples of one proprietary CSA mixture that were not properly consolidated did not perform well, however a third sample in this mixture that was properly consolidated performed excellently. This demonstrated the need for proper compaction and consolidation in laboratory tests. Another promising outcome of the research project was that a curing compound was very effective for all mixtures except for one. The CSA-OPC blend was most effectively cured with a 3-hour application of wet burlap and plastic sheeting. Although the CTE values of HESC mixtures were higher than the reference HPC, these mixtures should still have good compatibility with the underlaying HPC material provided that the bond strength is met (see Section 5.2.2).

5.2.2 Impediments Identified

Replicating the field conditions (e.g. hydrodemolition) to prepare substrate slabs of HPC for these HESC materials in laboratory conditions was challenging and did not produce conclusive or confirmatory results. It is recommended that bond strength be evaluated in the field (as is current practice) without any change to existing ODOT specifications. The potential for corrosion of reinforcing steel in systems with CSA and/or latex modifiers is unclear. The results of the very high formation factor measurements in this project do not accurately represent the transport properties of these systems. Nor are they directly transferrable to the chloride ingress and potential for corrosion of reinforcing steel in CSA systems. More research in this area is

merited to fully elucidate the critical chloride threshold for CSA systems and to ensure proper test methods and sample conditioning are done.

5.3 RECOMMENDATIONS

Recommendations from the outcomes of this research project include the following:

- Further broaden the materials on the ODOT QPL for HESC. This can be done following the specifications developed, in part, from this project.
- Review and incorporate recommended specification updates provided in Appendix C of this report.

5.4 FUTURE RESEARCH AND CHALLENGES

Future recommended work based on the results of this project include:

- Wholistically assess the role of chlorides in CSA-based systems that focuses on longer (e.g. 56-90 day) curing times for belitic calcium sulfoaluminate cement-based systems is merited based on the outcomes of this research.
 - Investigate the role of dry versus moist curing in these ettringite accelerated systems where a belite (C_2S) will hydrate slowly to form CH (calcium hydroxide) and C-S-H (calcium silicate hydrate).
 - Critical chloride threshold using the OC_{crit} approach
 - Chloride permeability
- Perform a full-scale half-cell survey of the Yamhill Crossing overlay to establish a baseline for current half-cell readings with full map. It would provide a useful comparison for future half-cell surveys if initial numbers are of concern, or over time as the HESC overlay is exposed to deicing chemicals. This may also be advantageous on other HESC overlays and ODOT is recommended to consider this. ODOT may consider doing this at regular intervals, e.g. every 3-5 years to establish overall trends and provide insight as to the long-term corrosion performance of HESC overlays.
- Investigate the role of coefficient of thermal expansion and bond on HESC overlay performance using pavement design tools (e.g. finite element modeling and relevant design guide tools/software).

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