NDOT Research Report

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Phase I: Evaluation of Low Flexural Strength for Northern Nevada Concrete Paving Mixtures

July 2017

Nevada Department of Transportation
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16. Abstract Production paving grade concrete in Northern Nevada having acceptable strength is universally acknowledged to be difficult; however understanding why this is true remains elusive. Current practice is to meet flexural strength requirements by using mixtures with high cementitious materials contents and low water/cement ratios (*w/cm*), which can negatively impact cost, workability, shrinkage, and durability. There are multiple factors that can contribute to poor concrete strength. Most focus on the characteristics of the interface that develops between the hydrated cement paste and the aggregate which can be compromised by localized high *w/cm*, the presence of coatings, and/or clustering of air bubbles at the aggregate surface. An experimental plan has been developed for Phase II of this study to evaluate the factors contributing to poor flexural strength of paving grade concrete produced in Northern Nevada with the purpose of developing cost effective solutions to improve the situation.		
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FINAL REPORT

Phase I: Evaluation of Low Flexural Strength for Northern Nevada Concrete Paving Mixtures

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ABSTRACT

Production of paving grade concrete in Northern Nevada having adequate flexural strength is universally acknowledged to be difficult; however understanding why this is true remains elusive. Current practice is to meet flexural strength requirements by using mixtures with high cementitious materials contents and low water/cement ratios (*w/cm*), which can negatively impact cost, workability, shrinkage, and durability.

There are multiple factors that can contribute to poor concrete strength. Most focus on the characteristics of the interface that develops between the hydrated cement paste and the aggregate which can be compromised by localized high *w/cm*, the presence of coatings, and/or clustering of air bubbles at the aggregate surface. An experimental plan was developed for Phase II of this study to evaluate the factors contributing to poor flexural strength of paving grade concrete produced in Northern Nevada with the purpose of developing cost effective solutions to improve the situation.

EXECUTIVE SUMMARY

Summary of Research Need

Production of paving grade concrete in Northern Nevada having adequate strength is universally acknowledged to be difficult; however understanding why this is true remains elusive. Current practice is to meet flexural strength requirements by using mixtures with high cementitious materials contents and low water/cement ratios (*w/cm*). While going to these extremes can in most cases result in a mixture that satisfies NDOT specified strength requirements, these extremes bring other problems with regards to cost, workability, shrinkage, and durability.

In contrast, mixtures produced in accordance with the same NDOT specifications routinely meet flexural strength requirements in Southern Nevada, demonstrating good strength at cement contents at the lower limits of NDOT's minimum requirement. Aggregates in Northern and Southern Nevada have vastly different minerology, and there is very little overlap between cement producers, ready-mix producers, and contractors. This makes it difficult to identify a single obvious explanation for the large difference in concrete strength. Providing such an explanation is the necessary first step for enabling the development of strategies for improving concrete flexural strength in Northern Nevada.

Review Findings

There are multiple factors that can contribute to poor concrete strength. Most focus on the characteristics of the interface that exist between the hydrated cement paste and the aggregate along what is known as the interfacial transition zone (ITZ). The ITZ can be compromised by localized high *w/cm*, the presence of coatings, and/or clustering of air bubbles at the aggregate surface. Techniques are available to assess this zone including optical and scanning electron microscopy, nanoindentation, and other means. A review of NDOT and neighboring DOT specifications revealed no major differences with regards to paving grade concrete, suggesting that the difficulties in Northern Nevada are not a result of specifications, but instead in the local materials.

Representative mixture designs based on trial batching from Southern and Northern Nevada were obtained and evaluated, demonstrating that Southern Nevada mixtures typically achieve higher flexural strength at lower cementitious materials contents. Differences in aggregates, cements, supplementary cementitious materials, and admixtures were also documented.

Phase II Laboratory Experimental Plan

An experimental plan was developed for Phase II of this study. This plan involves obtaining all components for three NDOT-approved concrete mixtures – one from Southern Nevada and two from Northern Nevada (one with a high absorptivity volcanic aggregate and one with a low absorptivity granodiorite aggregate). After re-creating and verifying the previous observed behavior of the approved mixtures in the laboratory, the components will be systematically exchanged between mixtures to explore how changes in components affect flexural strength. Optical, and potentially scanning electron, microscopy is discussed as a tool to explore the mechanism behind these effects. Due to uncertainties in available funding levels and the complexity of the problem, an alternate to this experimental design is presented, in which the investigation will proceed in an iterative manner using a "troubleshooting" flowchart that is intended to allow for expeditious targeting of specific mixture components as the cause of low flexural strengths. This plan can be expanded to a full experimental matrix if funding allows.

CHAPTER 1: INTRODUCTION

1.1 Project Background

It is well-documented that achieving NDOT's specified 28-day flexural strength of 650 psi (in third-point loading according to ASTM C78) for concrete paving mixtures (Class PCCP) in Northern Nevada is challenging. The commonly stated reason for the difficulty encountered is that aggregates in Northern Nevada are of "poor quality." Yet the challenge persists even when a quarried aggregate of with good physical characteristics is used (e.g., the Spanish Springs aggregate has a specific gravity approaching 2.70 and an absorption of less than 1 percent). Further, if the issue was solely "weak" coarse aggregate, the mode of failure in flexural beams would predominately be through the coarse aggregate particles as they would be the weakest link in the composite system. Yet in many instances, the failure is observed along a fracture surface that passes around the coarse aggregates, with the aggregate particles observed to have "pulled out" of the hydrated cementitious paste (HCP) as shown in Figure 1-1. In these cases, the fracture surface is largely through the interfacial transition zone (ITZ) that exists between the HCP and the coarse aggregate surface. This is indicative of either an exceptionally weak ITZ and/or poor bonding of the HCP to the aggregate surface.

Figure 1-1. Fracture surface in concrete beams tested in flexure. Note multiple aggregate "pull-outs", some circled in red (Van Dam 2015).

The ITZ is known to be a potential "weak link" in concrete, contributing to increased permeability and lower strength (Mehta and Montiero 2006). Further, cases exist where poor HCP adhesion to the aggregate surface has resulted in poor strength characteristics (Cross et al. 2000, Muñoz et al. 2007). There are a number of factors that can contribute to weakening of the ITZ and/or the ability of the HCP to bond to the aggregate surface including:

- Aggregate coatings interfering physically and/or chemically with the bond with the HCP.
- Poor moisture conditioning of the aggregates prior to batching resulting in severely deficient or excessive moisture at the aggregate interface.

- Poor packing characteristics of cementitious materials particles resulting in the "wall effect" producing localized high water-to-cementitious materials ratio (*w/cm*) and a weak ITZ.
- Poorly reactive or chemically incompatible mixture constituents (e.g., cementitious materials, admixtures) that results in the formation of a weak ITZ or poor bonding of the HCP to the aggregate.
- Aggregate surface chemistry and/or coatings that have an affinity for water, resulting in high *w/cm* at the interface and a weak ITZ or poor bonding of the HCP to the aggregate.
- Aggregates surface chemistry has an affinity for entrained air bubbles contributing to air void clustering at aggregate-HPC boundary weakening the bond.

Multiple strategies have been attempted to address the problem, including reducing the mixture *w/cm* and increasing the total cementitious content, but to-date, no strategy has proven universally successful. Determining the cause of this problem and developing cost effective strategies to increase flexural strengths is warranted as it will reduce NDOT's cost for pavung grade concrete while enhancing the long-term performance of the concrete pavement.

NDOT's experience with low flexural strength concrete in Northern Nevada is seemingly unique. A review of national literature revealed periodic cases of low strength, but there is no record of such low strength reported on such a broad time scale as experienced by NDOT. The nature of the failure strongly indicates a weakness at the HCP-aggregate boundary, suggesting an inability of the HCP to bond to the aggregate and/or a weakness in the ITZ itself. Only through a a detailed laboratory study that includes systematic characterization of the HCP-aggregate boundary can the cause of the problem in Northern Nevada be identified and a solution developed.

1.2 Overall Program and Phase I Objectives

The overall program objective of this research is to determine the cause(s) of the poor flexural strength of Class PCCP mixtures in Northern Nevada and to develop cost effective strategies, enforceable through specifications, to improve strength. The objective will be achieved in three research phases, conducted over multiple years. The research reported herein specifically focuses on Phase I, which has the objectives to synthesize the local, regional, and national body of knowledge on factors contributing to low concrete strength and to develop a research plan for Phase II of the program. The remaining two future phases of the research effort are as follows:

- Phase II laboratory investigation to identify the cause of the problem and develop strategies to address it suitable for implementation in Nevada.
- Phase III is to develop a prototype specification for implementation by NDOT.

At the conclusion of this program, NDOT will have a prototype Class PCCP specification to achieve desired flexural strength in Northern Nevada that is not dependent upon the use of high cementitious content, low *w/cm* mixtures.

1.3 Phase I Research Methodology

The research methodology presented below is exclusively for Phase I, which concludes with this Final Report that includes a recommended research plan for Phase II.

1.3.1 Task I-1: State-of-the-Practice Synthesis

Task I-1 was conducted in the following four subtasks:

- **Task I-1a –** Reviewed current NDOT specifications, special provisions, and practices including a review of concrete paving projects conducted in Northern Nevada over the last fifteen years. Specific information regarding Class PCCP mixture proportions, aggregates, cementitious materials, and construction practices and test results was reviewed.
- **Task I-1b –** Synthesized detailed information on regional practices regarding concrete paving mixtures, especially from states that may use the same or similar constituent materials. Detailed information was sought from California, Idaho, Oregon, Utah, and Wyoming. Focus was on hot, arid regions as this is more typical of conditions prevalent in Northern Nevada.
- **Task I-1c –** Reviewed national literature regarding concrete paving mixture design, specifications, and strength.
- **Task I-1d –** Reviewed mechanisms responsible for low concrete flexural strength and methodologies available for the characterization of the HCP-aggregate boundary. These include advanced characterization such as optical and scanning electron microscopy and nanoindentation.

The Task I-1 deliverable is a comprehensive synthesis presented herein of NDOT, regional, and national information regarding concrete paving mixture design practices and factors contributing to low flexural strength. In addition, a synthesis of applicable advanced techniques for the characterization of the HCP-aggregate boundary has also been prepared.

1.3.2 Task I-2: Develop Draft Phase II Research Plan

Based on Task I-1, a draft Phase II Research Plan was developed. Phase II is envisioned as an 18 month long laboratory study designed to identify causes of low flexural strength and develop implementable strategies to eliminate it. It will be conducted in the following subtasks:

- **Task I-2a –** Develop a plan to sample existing pavements that are known to have had difficulty in meeting NDOT's flexural strength requirements, and characterize the concrete with particular emphasis on the nature of the HCP-aggregate boundary.
- **Task I-2b –** Develop a plan to conduct laboratory investigation to evaluate a number of concrete constituent and proportioning variables to identify those contributing to low strength characteristics of Northern Nevada Class PCCP. The investigation will include material selection, initial mixture designs and testing, and detailed testing of selected

mixtures including advanced characterization of the HCP-aggregate boundary. The concrete mixtures will be fabricated and tested locally and specimens fabricated for advanced characterization sent out to university facilities.

The Task I-2 deliverable is a draft Phase II Research Plan.

1.3.3 Task I-3: Phase I Final Report and Phase II Research Plan

Task I-3 meets NDOT's requirements to have a Task dedicated to producing a Final Report. This final report synthesizes the information collected in Task I-1 and provides a Phase II Research Plan. The Task I-3 deliverable is this Phase I Final Report, incorporating NDOT comments.

1.4 Anticipated Benefits

The difficulties encountered in achieving the specified 650 psi 28-day flexural strength in Class PCCP in Northern Nevada is well-documented and continues to plague concrete pavement projects throughout the region including the recent concrete pavement work conducted on I-580 in Reno in 2016. The impacts of this problem are three-fold:

- According to the 1993 AASHTO Guide to Pavement Design and the new AASHTOWare Pavement ME Design software, the number of traffic loads (i.e., trucks) that can be carried by a concrete pavement will be reduced as flexural strength decreases. For example, using typical NDOT inputs in the 1993 AASHTO design equation, a change in flexural strength from 750 psi to 650 psi will reduce predicted life by over 60 percent.
- The current strategy to achieve the specified flexural strength is to increase the amount of cementitious material used beyond what is common regionally and nationally. This adds approximately 5 percent to the cost of the Class PCCP.
- More importantly, increasing cementitious content increases concrete drying shrinkage, resulting in increased cracking and slab warping. It also increases the potential for durability problems such as alkali-silica reactivity. This leads to increased pavement roughness, which negatively impacts the comfort of the riding public, reduces vehicle fuel efficiency, and shortens pavement life.

There is thus a significant need to understand the cause(s) of the problem and to develop implementable strategies and specifications that cost effectively increase flexural strength while reducing the required amount of cementitious materials. The payoff for achieving this research program's objectives will have immediate cost savings, reducing the cost of Class PCCP mixtures by approximately 5 percent once fully implemented. At time of implementation, there may be a slight increase in material costs, but these would likely be offset by the reduction in the required cementitious content.

The long-term cost benefits for NDOT are more substantial. As discussed above, the long-term performance benefits incurred through increased flexural strength and a reduction in cementitious content have the potential to significantly increase pavement life, reduce the expense of future repairs and user delay costs, improve ride quality, and improve safety by shortening the frequency and duration of construction work zones.

The cost of implementation beyond the cost of the full study (all three phases) is minimal. Once changes are made to existing specifications, there will likely be small costs incurred in the way constituent materials are acquired and/or processed.

CHAPTER 2: REVIEW OF LOW CONCRETE STRENGTH MECHANISMS AND CHARACTERIZATION METHODS

2.1 Introduction

This chapter reviews a number of studies focused on mechanisms that can result in low concrete flexural strength. These mechanisms can be placed in one of two categories: i) weak interfacial transition zone (ITZ), and ii) poor bonding between the aggregate surface and the hydrated cementitious paste (HCP).

The ITZ is more conceptual than a tangible, identifiable component of concrete. Figure 2-1 provides a schematic representation of the ITZ, showing it as a relatively porous zone with a higher proportion of calcium hydroxide (CH – note that here and throughout the common practice of using ceramic notation to identify cement and HCP phases will be used) and ettringite than is present in the bulk paste which is dominated by calcium silica hydrate (CSH).

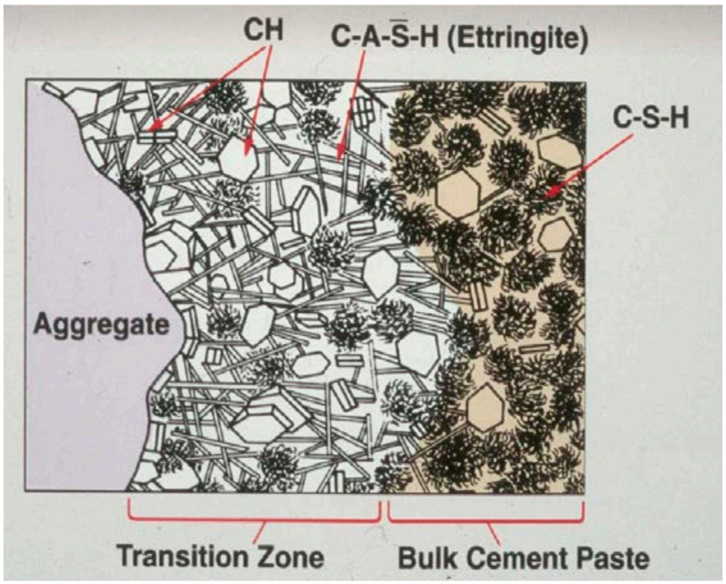

Figure 2-1. Schematic representation of the interfacial transition zone between an aggregate particle and the bulk cement paste. (after Mehta and Monteiro 2006).

Essentially, the term ITZ is used to describe the transition from the bulk HCP to the aggregate surface. The ITZ results from the nature of how cement particles pack against the surface of the aggregate. Given the size of an aggregate relative to a cement particle, the aggregate is effectively a flat "wall." This wall prohibits the dense packing of the cement particles against it, increasing the volume of water filled space at the aggregate-cement particle interface. This leads in turn to a zone of locally high *w/cm* at the interface, which in turn favors the formation of CH as a reaction product as CH precipitates in water-filled space whereas CSH forms on the surface of cement grains. Further, the increased water-filled space ultimately results in increased localize porosity, increasing the permeability of the HCP in the ITZ compared to that in the bulk HCP. In turn, the available porosity in the ITZ provides space for deposition of other secondary phases such as ettringite. The net result is a zone of higher porosity and the presence of HCP phases that tend to be larger-grained and weaker than found in the fine-grained and less permeable bulk HCP.

The ITZ will vary in thickness, but because of the connection to cement particle packing, the thickness of the ITZ is on the order of the diameter of a few cement particle (i.e., ~ 30 to 50 microns). In addition to being more permeable, the combination of porosity and increased CH formation results in the ITZ having a lower strength as compared to the bulk HCP. Often, the ITZ is referred to as the “weak link” in the concrete microstructure and in many cases fractures initiate in the ITZ as concrete is loaded, or propagate through the ITZ from another point of origination. As will be discussed later, the formation of excessive air bubbles in the ITZ has also been shown to adversely affect permeability and strength.

The bond between the HCP and the aggregate is primarily physical. Surface roughness and texture have clearly been demonstrated as key factors in paste-aggregate bond (Roy 1993). In some cases there is also a chemical-related bond as well. For example, calcium carbonate aggregates typically bond to the paste more tenaciously, and the resulting concrete has a higher compressive strength compared to siliceous aggregates. The specific cause of this chemical bond is not fully understood (Roy 1993). Under either scenario, the nature of the surface affects bond and when contaminate phases are present, such as fine dust or clay, the active physical and chemical mechanisms are disrupted. Therefore, aggregate cleanliness is important to concrete performance.

The remainder of this chapter discusses various factors that affect the porosity of the aggregate-paste interface and aggregate-paste bond, as reported in the literature.

2.2 Factors Affecting the Characteristics of the Aggregate-Paste Interface

Roy et al. (1993) conducted a study for the Strategic Highway Research Program (SHRP) that investigated factors affecting the characteristics of the aggregate-paste interface. The key factors investigated in their study was i) the original particle density, which they equated with *w/cm*, ii) the travel distance of particles to the surface of the aggregate, iii) the “sticking probability”, which they equated to the tendency of particles to flocculate, and iv) the amplitude of particle motion that was assumed to be related to the energy of mixing (Roy et al. 1993). This study used a combination of modeling with limited laboratory testing. Their results confirmed that a significant variation in HCP porosity existed at the aggregate interface, where porosity was a maximum compared to the bulk HCP. This transition was observed to occur over a distance approximately equal to the diameter of two to three cement particles. The initial packing of cement particles at the interface at time of mixing determined the nature of the interface as the concrete matured. Related, this study reported that cement particle flocculation strongly impacted the degree of packing, with flocculated systems having higher porosity and poor packing characteristics.

The effect of the amplitude of motion (i.e., energy imparted during mixing) on the quality of packing was found to be dependent on multiple factors. For one, particle packing in cement systems that were flocculated benefited from lower mixing energies whereas systems that were not flocculated (e.g., a superplasticizer was used to disperse the cement grains) benefited from vigorous mixing (Roy et al. 1993). An interesting finding was that the thickness of the water film around the aggregate, which relates to particle travel distance and the original *w/cm* of the mixture, was found to have no effect on particle packing and thus no effect on the porosity of the ITZ (Roy et al. 1993).

In summary, Roy et al. (1993) demonstrated that the ITZ existed (a zone of higher porosity at the aggregate-paste interface compared to bulk HCP) in most all systems they evaluated (ordinary portland cement, Class F fly ash, Class C fly ash, and condensed silica fume), although it was reduced or nearly eliminated through the use of Class C fly ash or silica fume. In addition to changing the cementitious system, mitigating particle flocculation through the use of a superplasticizer also reduced interface porosity. And finally, mixing energy may be a very important factor in reducing porosity at the aggregate-paste interface, although this variable was evaluated only through modeling.

Others have also concluded that mixing energy can be an important factor influencing the quality of the ITZ. Tamimi (1994) reported on the impacts of a two-stage mixing procedure that resulted in reduced bleeding, increased micro-hardness of the HCP, and a high rate of strength development. In the two-stage procedure, also called SEC for “sand enveloped with cement”, the first stage pre-mixes the aggregate with 25 percent of the water, and a latter addition of the cementitious materials. In the second stage, the remaining constituents are added, with the water added last. Tamini (1994) reported up to a 25 percent increase in compressive strength at early age, and 5 percent increase at later age. One feature of this study was the characterization of the crystallographic orientation of CH occurring at the aggregate-HCP interface, which showed the tendency for CH to grow with a preferred orientation due to two-stage mixing. In a follow-up paper, Tamimi (1996) used a scanning electron microscope (SEM) to study the aggregate-HCP interface in concrete produced using the same two-stage mixing procedure and monitored the morphology change with time.

Other researchers have investigated altering the properties of the ITZ through the addition of supplementary cementitious materials (SCMs). Silica fume is widely considered to positively modify the ITZ structure given its very small particle size, which improves particle packing and through pozzolanic activity, converts CH to CSH in the ITZ (Mehta and Monteiro 2006). Ping and Beaudoin (1992) investigated the use of silica fume to modify the ITZ by coating aggregate particles with silica fume prior to mixing in concrete. This densified the ITZ, which was verified by SEM analysis. They also found the coating method was not dependent on aggregate type or *w/cm* (Ping and Beaudoin 1992). Scrivener et al. (2004) investigated the ITZ in typical concrete mixtures, concluding that the use of extremely fine SCMs, such as silica fume or metakaolin, could fill in the gaps minimizing the "wall" effect. The filling of these gaps strengthened the ITZ by reducing porosity, resulting in higher 28-day mechanical properties. Kong et al. (2010) combined the use of pozzolans and multi-stage mixing to improve ITZ properties for concrete made with recycled concrete aggregate. They used a self-termed "triple mixing" process, which is shown graphically in Figure 2-2. Pozzolans were added as admixtures. W_1 and W_2 were determined by $W_1 = 1.2(W_t - W_f)$ and $W_2 = (W_t - W_1)$, where W_t is total water content and W_f is free water content. The SCMs used were a combination of slag cement and fly ash. The researchers reported improvement in strength and a reduction in chloride ion penetration with slag cement, and to a lesser degree with fly ash.

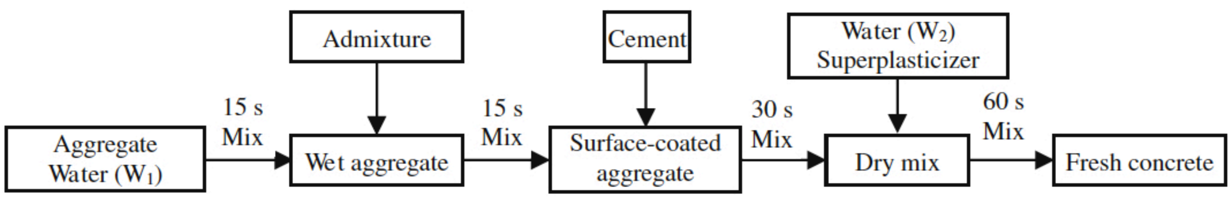

Figure 2-2. Schematic presentation of the triple mixing procedure used by Kong et al. (2010).

Buenfeld and Okundi (1998) investigated the release of air from unsaturated aggregate as concrete set. Eight different concrete mixtures were prepared and tested in a laboratory setting to evaluate the rate of compressive strength gain. Two batches for each concrete mix were produced: one used aggregates that were dry of saturated surface dry (SSD) condition and the second used aggregates that were saturated, having been immersed in water for seven days. The results showed that the concrete made with saturated aggregates had compressive strengths approximately 85 percent higher at 1 day, 26 percent higher at 7 days, and 23 percent higher at 28-days. SEM images of the concrete made with dry aggregates showed concentrations of air voids at the surface of the larger aggregate particles, whereas the concrete with the soaked aggregates were free of these air void clusters. The conclusions of the study were that aggregates dry of SSD absorbed water from the fresh paste, and the air within the aggregate was displaced into the concrete prior to setting. This displaced air remained at the aggregate-HCP interface and may have been responsible for the lower compressive strengths. One observation made by Buenfeld and Okundi (1998) was that the 24-hour water absorption of the combined aggregate gradation was 2.81 percent, whereas the 7-day absorption was 3.36 percent, representing a 20 percent increase in absorption value. Many of the concrete aggregate sources in Northern Nevada are highly absorptive and it is likely that these aggregates required more than 24 hours soaking to become saturated.

A report by Cross et al. (2000) describes an investigation conducted by the South Dakota Department of Transportation (SD DOT) to determine the cause of low compressive strengths that occurred during the 1997 construction season. It was determined that the predominant cause of low strength was poor aggregate-HCP bond associated with air void clustering and poorly formed HCP in the ITZ. This is shown in Figure 2-3. This problem was attributed to an interaction between non-Vinsol® resin air entraining admixtures (AEAs) and low-alkali cement in combination with high summertime temperatures during construction.

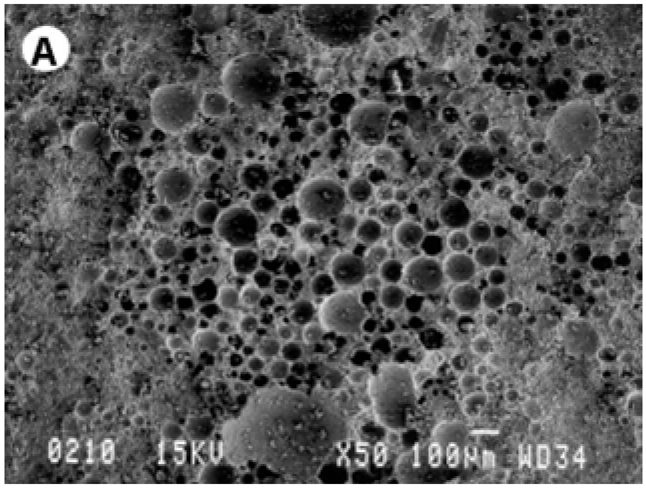

Figure 2-3. SEM micrograph showing air void clustering at aggregate-HCP interface (Cross et al. 2000).

Kozikowski et al. (2005) looked at several factors to determine those that had the most influence on air void clustering around coarse aggregate particles. Cylindrical specimens were cast and evaluated for 28-day compressive strength and petrographic examination was conducted to determine the extent of air void clustering around coarse aggregate particles. The results showed that concrete mixtures made with a Vinsol® resin-based AEA did not experience air void clustering regardless of whether they were retempered (i.e., late addition of water) or not. On the other hand, retempered mixtures made with non- Vinsol® resin-based AEA regularly showed signs of air void clustering and reduced strengths. Furthermore, as the mixing time was increased, the severity of air void clustering also increased, suggesting that mixing times should be carefully monitored. Lastly, the aggregate shape and minerology was found to have an impact on the amount of strength loss.

An article published in Concrete Producer Magazine (2012) summarized the continuing problem of air void clustering causing low concrete strengths. It cites problems observed by Department of Transportations (DOTs) in New Jersey, Delaware, South Dakota, Minnesota, Virginia, New York, Michigan, and Ohio, linking them to the switch from Vinsol® resin-based to “synthetic” air AEAs. Multiple factors are cited as contributing to the problem including increases in air content, the percentage of coarse aggregate, volume of crushed aggregate, and retempering being most strongly implicated. Strength losses were reported to be as high as 25 percent in some cases.

Ram et al. (2013) conducted a study for the Wisconsin DOT to evaluate loss of entrained air in the slipform paving process. In the course of this study, air void clustering was commonly observed. Figure 2-4 shows two examples of this phenomenon in polished concrete viewed through a stereo optical microscope. Although strength was not assessed in this study, it can be assumed that clustering would impact the bond at the aggregate-HCP interface.

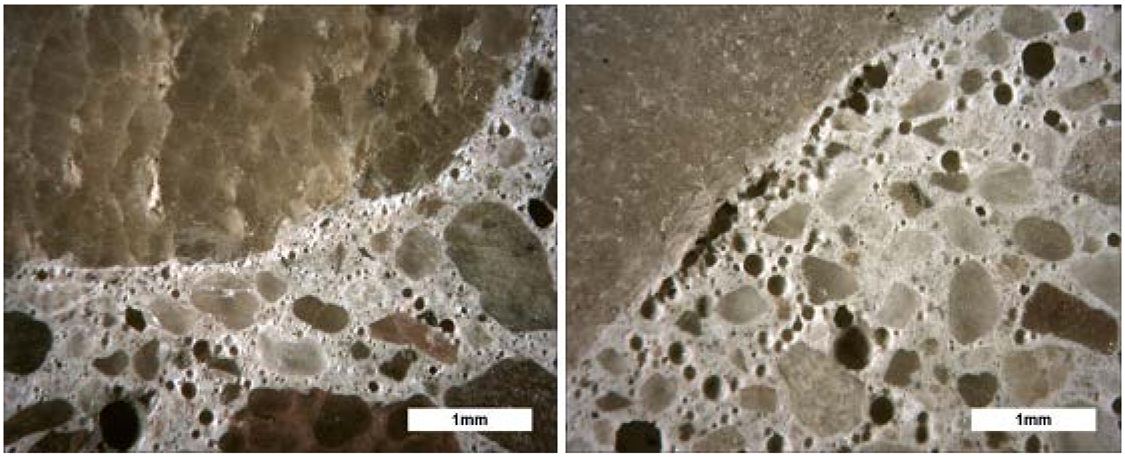

Figure 2-4. Stereo optical micrograph showing air void clustering along coarse aggregate-HCP interface (Ram et al. 2013).

Vosahilk (2014) investigated air void clustering around coarse aggregates as a potential cause of low concrete strength. Air void clustering was reproduced in the laboratory, but a relationship between clustering and compressive strength was not observed. Instead, loss of compressive strength was linked to the total air content in the concrete and the inhomogeneity of the HCP microstructure. Retempering of the concrete was strongly linked to the loss of strength due to the increase in the *w/cm*, air content, and zones of HCP inhomogeneity. Data in this study suggested that retempering had little effect on air void clustering around coarse aggregate. It was also observed that zones of inhomogeneity were far more common when the coarse aggregate was unwashed, with it being concluded that the "aggregate is generally susceptible to issues related to improper mixing" and especially to problems when retempering occurred. It was also concluded that the severity of air void clustering around coarse aggregate was not influenced by admixture chemical composition or aggregate type.

A study conducted by Leite and Monteiro (2015) evaluated the ITZ of concrete made using recycled concrete aggregate (RCA) in a laboratory setting. Two different RCA sources were used and each RCA was evaluated in both dry and SSD moisture conditions at the time of mixing. The pore sizes in the ITZ of the different concrete mixes were evaluated using a combination of synchrotron microtomography and SEM analysis. The results showed that the pore size distribution for the mixtures with the dry RCA consisted primarily of pore sizes above 50 micrometers. In comparison, the mixtures batched with RCA at SSD exhibited much smaller pore sizes around 30 micrometers. These results support earlier observations that batching absorptive aggregates in a condition dry of SSD can released air into the ITZ as water is absorbed into pores, resulting in a high-porosity ITZ.

2.3 Factors Affecting Bonding Between Aggregate and the HCP

Beyond the porosity of the ITZ, another factor that causes poor bonding between the aggregate surface and the HCP relates to microfine coatings on aggregate surfaces. Microfine particles are defined as those that pass the No. 200 sieve but may remain adhered to aggregate surface even after cleaning. Research on surface coatings dates back to the 1930's when Goldbeck (1933) presented properties of several different types of aggregate coatings including: clay, dust, organic and so forth (Gullerud and Cramer 2002).

Aggregate coatings can be classified as clay, dust, or carbonate coatings (Schmitt 1990). Clay coatings tightly adhere to the aggregate surface by means of electrostatic forces. The strength of the bonds vary from weak to very strong. Thus, some clay coatings can be removed from the surface during normal concrete handling and mixing whereas other clay particles remain strongly adhered to the surface throughout the construction process. The latter can negatively affect the bonding between aggregate and the HCP. Dust coatings, on the other hand, can normally be removed by mechanical means, typically increasing the fines in the mixtures. Carbonate coatings, consisting of calcium carbonate material derived from calcite (limestone or dolomite) deposits, are typically not strongly adhered to the surface (Gullerud and Cramer 2002).

Gullerud and Cramer (2002) discussed the effect of aggregate surface coatings on workability and strength problems experienced in Wisconsin. Coatings were characterized to determine composition and their impact on concrete strength and durability was assessed. In general, most of the coatings were found to be innocuous, having no identifiable impact on concrete strength or durability. But the nature of the coating mineralogy was found to very important, with clay coatings having the potential to be detrimental to concrete. The main effect was not on strength, but on increased water demand, which negatively impacted workability, increased drying shrinkage, and potentially caused air void clustering at the aggregate-paste boundary. Vigorously washing the aggregates was effective at removing the coatings, but it was not felt to be cost effective as the overall impacts of the surface coatings was considered minor.

Munoz et al. (2007) further investigated the impact of clay coatings on concrete in Wisconsin. Three igneous coarse aggregates were studied, two of which were naturally coated with clay whereas the third was a clean aggregate that was coated in the laboratory. SEM examination was used to characterize the nature of the ITZ for each of the aggregate-clay coating combinations. It was found, depending on the nature and availability of the clay that some clay detached during concrete mixing. Clays that undergo crystalline swelling (i.e., calcium montmorillonite) or no swelling (kaolin) detached most easily and thus increased the rate of the cement hydration. Clay with macroscopic swelling (i.e., sodium montmorillonite) was the most difficult to detach from the aggregate surface, and actually slowed cement hydration. The water demand increased significantly for sodium and calcium montmorillonite coatings, inhibiting adhesion between the coarse aggregate and HCP. The result was a reduction in the compressive and tensile strength of the concrete (shown in Figure 2-5) increased shrinkage, and reduced freeze-thaw performance.

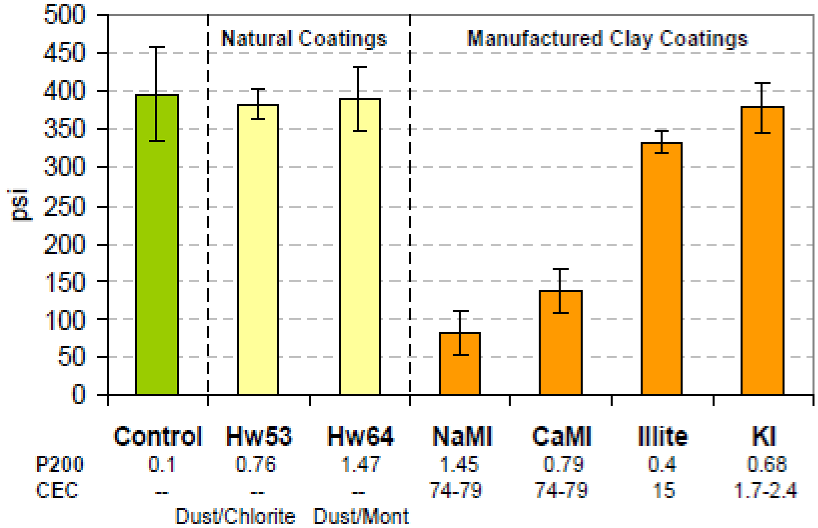

Figure 2-5. Tensile strength for concrete made with aggregates with clay coatings. Note that the yellow bars represent natural clay coating whereas the orange bars represent aggregates coated in the laboratory (Muñoz et al. 2007).

Figure 2-6 presents the effect of additional mix water on tensile strength. Additional mix water increased the tensile strength of the concrete with NaMI and CaMI aggregate coatings but reduced the strength of the concrete with kaolin aggregate coatings.

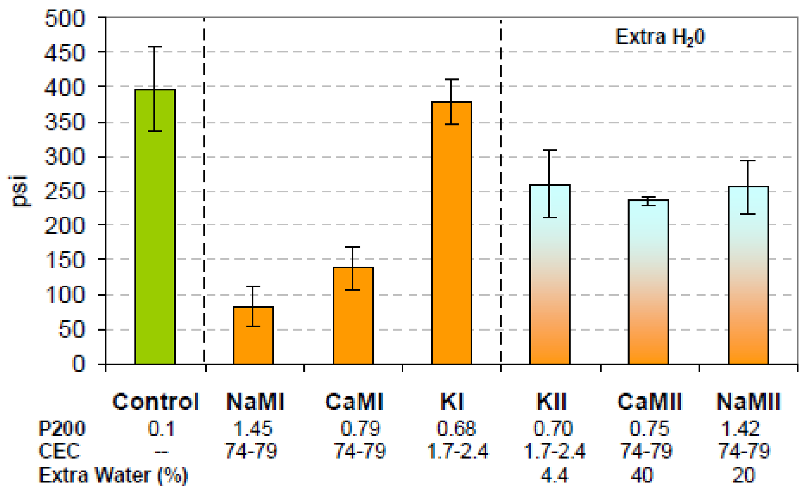

Figure 2-6. Effect of additional mix water on tensile strength of concrete made with aggregates with clay coatings (Muñoz et al. 2007).

Forster (2006) describes coatings as adhering materials that may be strongly or weakly bound to the aggregate's surface, including silt and clay (fine material passing the 75 μm sieve) that occur in sand and gravel deposits as well as the dust of fracture that occurs through crushing, processing, and transporting aggregates. He states that strongly adhering material, particularly clay, may create problems, including "poor coarse aggregate-to-paste bond." Forster (2006) distinguishes between natural coatings, which occur in sand and gravel deposits, and artificially generated coatings due to aggregate processing. Petrographic evaluation of aggregates in accordance with ASTM C295 can determine the nature of the coating and concrete petrography in accordance with ASTM C856 can characterize the fracture surface between aggregate and HCP, which is helpful in identifying detrimental impacts. Forster (2006) states that the number one effect of coatings on concrete is on strength, where they may be detrimental if the bond of the HCP to the coating is greater than the bond strength of the coating to the aggregate.

2.4 Methods to Characterize the Aggregate-HCP Interface

Several methods can be used to characterize the aggregate-HCP interface. These methods range in cost, complexity, equipment requirements, technical knowledge, and desired outcome. The methods presented herein are those identified from literature as being most applicable to address the objectives of the next phase of this research program.

Optical Microscopy (Petrography) – The application of petrography to the study of concrete is described in ASTM C856 – *Standard Practice for Petrographic Examination of Hardened Concrete*. The standards practice provides a description of methods to prepare and study concrete, primarily through the use of optical microscopy including both stereo optical microscopy (SOM) and petrographic optical microscopy (POM)   SOM can be used to view fractured surfaces as well as polished slabs using reflected light. The SOM has good depth of field and is an excellent tool for initial examination of concrete. When used under ASTM C457 – *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*, the SOM can be used to determine the characteristics of the entrained air-void system as well as mixture volumetrics.

The POM is used to identify rocks and minerals, which are prepared thin enough (i.e., ~20 microns) to allow light to pass through. These types of specimens are referred to as thin sections and standard procedures have been developed for preparing these specimens. As the light passes through the specimen, it interacts with the material depending on whether the material is crystalline (ordered) or amorphous (disordered). For crystalline materials, the light interaction further differs depending upon the mineral crystal structure. The resulting effects can be interpreted by a trained petrographer and used for characterizing the material. For example, materials such as CH, ettringite, and calcium carbonate are readily identified by their strong birefringence (i.e., light transmission as a function of angle; the specimen is rotated in polarized light) and pleochroism (i.e., change in color of the transmitted light).

Concrete petrography refers to the application of petrography to the analysis of "man-made" rocks (i.e., concrete). One of the first reports of application of petrographic techniques to concrete was published by N.C. Johnson in 1915 (Erlin 1993) and related observations of the concrete microstructure to performance. Since then numerous publications have been written on the subject, including textbooks (St John 1998; Campbell 1985).

Optical petrographic microscopy has the advantages of being relatively inexpensive and also relatively reliable, assuming the microscopist has experience examining concrete structures (Campbell-Allen and Roper 1991). An optical microscope can be used to examine amorphous materials found in concrete as well as crystalline materials such as CH prevalent in the ITZ of concrete. Optical microscopy is the best choice for studying concrete microstructure and it plays a key role in everyday laboratory analysis of concrete.

Of special importance in the area of petrography is one technique that has been developed based on epifluorescent microscopy. First proposed by Christensen (Thaulow et al. 1982), the use of UV-fluorescent dye has been developed for the determination of capillary porosity in hardened cement paste. Thaulow et al. (1982) used this method to estimate the compressive strength of concrete samples using fluorescence microscopy. Since its development, the method has become accepted by many as a measure of *w/cm* in hardened concrete (Mayfield 1990; Jakobsen et al. 2000). Additionally, the UV-fluorescent dye clearly highlights cracks, microcracks, and air voids and delineates the structure of the ITZ. Cracks or voids filled with reaction products also stand out prominently from the surrounding cement paste.

Scanning Electron Microscopy (SEM) – The use of the SEM for the examination of concrete, cement paste, and mortar characterization is described in ASTM C1723 **–** *Standard Guide for Examination of Hardened Concrete Using Scanning Electron Microscopy*. The cement paste microstructure and the quality of the aggregate-HCP interface is of main interest when studying the low concrete strength problems in Northern Nevada and these features can be readily observed using the SEM. For HCP and the ITZ, the key parameters affecting strength are total porosity, pore size distribution, the degree of hydration, and the physical/chemical characteristics of phases within the paste.

One significant capability of the SEM is the ability to perform chemical analysis of micro-constituents in hardened cement paste. Using the energy dispersive x-ray spectrometer (EDS) commonly included in a SEM system, phase identification is readily accomplished. Additionally, x-ray mapping can be used to identify micro-constituents and clearly show their spatial distribution within the material. Quantification of CH, ettringite, and other phases found in the ITZ or bulk HCP is commonly performed. The SEM uses the same specimens examined by the POM and often the same area in a specimen is examined using both techniques.

Nanoindentation: The use of nanoindentation to measure the elastic properties of HCP is a relatively recent technique that provides significant opportunities to characterize the material at the micro- to nano- level (i.e., micrometer to nanometer). Nanoindentation tests are used to determine the elastic modulus (E) of a material's microstructure and can be used to study changes in stiffness that might occur across the ITZ. The local mechanical properties can be determined from the indentation load and displacement measurement. Information on the micro-mechanical properties is obtained from a matrix of a minimum of 120 indents covering a representative area of at least $40 \times 230\ \mu m^2$ on the surface. The indentation depth should be within the range that allows access to the in situ properties of CSH, CH, and un-hydrated cement phase (Aquino et al., 2013). In the next phase of this study, nanoindentation might provide a valuable tool to first characterize the nature of the ITZ in concrete made in accordance with current practice and then to assess the effectiveness of various strategies to improve the ITZ.

Small-Angle Scattering (SANS and SAXS): Small-angle scattering is a powerful tool for characterizing complex micro-structures. An intense beam of either neutrons (SANS) or x-rays (SAXS) is passed through the specimen, and a small component is scattered out of the incident beam direction by interactions with microstructural features within the bulk material. For HCP and similar porous materials, the resulting scattering profile, which is the intensity of scattered neutrons or x-rays as a function of scattering angle, is effectively a form of Fourier transform of the solid/pore microstructure. It can be used to determine, for example, size distributions and volume fractions of microstructural features, fractal components within the microstructure, and the total surface area. Both SANS and SAXS use the principle of diffraction (Wang, 1995).

Mercury Intrusion Porosimetry: MIP is a widely used technique for directly measuring porosity-related characteristics of a porous material (porosity and pore size distribution). In this method, the dried porous sample (in this project, the HCP) is placed into a vacuum chamber, the chamber is evacuated, mercury is added inside the chamber, and then an external pressure is applied on the mercury. As the pressure gradually increases, mercury is slowly forced into the pores of the cement paste, with the largest pores filling first. The applied pressure, P (MPa) is inversely proportional to the size of the pores as defined by the Washburn equation (Aquino et al., 2013):

$$d = \frac{-4\gamma * \cos(\varphi)}{P}$$

where:

P = applied pressure (MPa),
d = apparent pore diameter (m),
γ = mercury surface tension (N/m), and
φ = contact angle between mercury and pore wall

2.5 Test Methods to Evaluate Aggregate Coatings

Several test method have been used to detect the presence of deleterious or microfine coatings on coarse aggregate. The tests summarized by Gullerud and Cramer (2002) are as follows:

ASTM C117 – Standard Test Method for Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregate by Washing: This test method determines the amount of material that passes the No. 200 sieve by washing the aggregate. The P_{200} value is a widely used measure but does not necessarily describe the extent of coating as it only considers dispersed fines and not those attached to aggregate surfaces. Also, the parameter cannot tell the difference between harmful microfines and types of stone dust that may not affect concrete performance.

California Test 227 – Method of Test for Evaluating Cleanness of Coarse Aggregate: This test measures the relative amount of clay-sized particles attached to aggregates, and is similar to the sand equivalent test. The cleanliness value indicates the quantity, particle size and activity of the adhered material. NDOT uses a slight variation of the California Test 227 (Test Method Nev. T228B).

AASHTO T 330 - Standard Method of Test for The Qualitative Detection of Harmful Clays of the Smectite Group in Aggregates Using Methylene Blue: The relative amount of clay in an aggregate sample is detected by absorption of methylene blue. A methylene blue value (MBV) is calculated as a measure of the amount of clay present. Dust and carbonate coatings cannot absorb the dye so the test is useful in detecting clay.

Modified Methylene Blue Value Index (MMBV) – Uses results from ASTM C117 and AASHTO T 330: This is a calculated value to account for the fact that the MBV only accounts for the clay present in the material passing the No. 200 sieve. This value uses the P200 and MBV to determine the overall clay content in the aggregate.

X-ray Diffraction (XRD): The mineral phases in a powder sample are identified through a pattern of waves diffracted through a crystalline structure. The diffraction pattern is matched to patterns for known minerals as a means to identify the presence of minerals. However, the test is not practical outside of applications for research and forensic evaluations. The test is very useful as it can detect if aggregate coatings fall into the three categories: dust, clay, or carbonate.

Gullerud and Cramer (2002) examined the correlation between P_{200}, California Cleanness Value, MBV, and MMBV to the performance of hardened concrete. P_{200} and MBV had the lowest correlation with strength properties. The authors concluded that the California Cleanness Value and MMBV were stronger predictors of strength properties but other factors such as strength of adherence may also influence performance and are not captured in these tests. Further examination of specific aggregate showed that the cleanness value and the MMBV may not accurately describe strength performance.

Similarly, Munoz (2012) examined the correlation between the aforementioned tests (with the exception of x-ray diffraction) and properties of fresh and hardened concrete. The P_{200} value in ASTM C117 was the poorest predictor of all properties examined (i.e., slump, AEA demand, fresh air content, compressive strength, flexural strength, shrinkage, stiffness, and weight durability). With respect to compressive and flexural strength, the authors found that the MMBV, a combination of P_{200} and the methylene blue value, had the highest correlation to compressive and flexural strengths.

2.6 Chapter Summary

Research examining the occurrence of low concrete strength, particularly flexural strength, has focused on the interface between the coarse aggregate and the HCP. In particular, problems have been associated with the characteristics (porosity and/or hydration products) of the aggregate-HCP interface or with bonding of the aggregate to the HCP due to aggregate surface coatings. The aggregate-HCP interface, or the ITZ, is known to often be more porous and contain coarser hydration products with a higher proportion of crystalline CH and ettringite than the bulk HCP. Further, issues with high air contents occurring at the interface due to the batching of absorptive aggregates dry of SSD and air void clustering at the interface due to retempering concrete made with non-Vinsol® resin-based AEA have been noted. Various methods have been investigated to improve the characteristics of the ITZ, most notably the use of certain, fine SCMs (particularly silica fume or metakoalin), superplasticizers to disperse flocculating cement particles, and increased mixing energy.

Coatings, particularly clay coatings that are strongly bonded to the aggregate surface, can prevent the HCP from directly adhering to the aggregate surface as well as potentially having negative impacts on cement hydration. No easy methods are available to clean dirty aggregates beyond traditional washing.

There are multiple test methods that can be employed to evaluate the interface between aggregates and HCP, the most common of which feature optical petrography and SEM/EDS examinations of prepared concrete specimens. Nanoindentation also shows promise as a technique to characterize the ITZ and bulk paste.

The most promising methods for characterization of aggregate coatings are variations on CA Test 227 in which methyl blue is used to further characterize clay. XRD is also identified as a useful research tool for the characterization of aggregate coatings.

CHAPTER 3: REGIONAL AND NATIONAL PRACTICE

3.1 Introduction

This chapter presents a synthesis of specifications for concrete paving mixtures and practices used for projects in Nevada and the surrounding states. In addition, national practice pertaining to mixture design, specifications, and flexural strength is presented for comparison with regional practices.

Both NDOT and regional DOT specifications are summarized. Additionally, typical mixture proportions and performance characteristics are presented to compare concrete paving mixtures used by NDOT to those used by surrounding states including California, Idaho, Oregon, Utah, and Wyoming. The version of the specification used for these discussions are as follows:

- Nevada : NDOT 2014 Standard Specifications for Road and Bridge Construction
- California: 2015 Caltrans Standard Specifications
- Idaho: ITD Standard Specifications for Highway Construction - 2012
- Oregon: ORDOT Standard Specifications for Construction - 2015
- Utah: UDOT 2012 Standards and Specifications
- Wyoming: WYDOT Standard Specifications for Road and Bridge Construction – 2010 Edition.

The purpose of evaluating specifications from other states is to identify potential differences in specification in hopes of identifying items that can be incorporated by NDOT to address the low flexural strength challenges in Northern Nevada. It is noted that state specifications provide a general sense of how materials may differ from state to state, but do not necessarily represent the typically materials and mixtures that are available and actually used in each state. For instance, ORDOT specifications may require higher minimum total cementitious materials content than NDOT specifications, but mixtures in Northern Nevada typically have more cement than paving mixtures typically used in Oregon.

Regional practices with regard to handling, mixing, and curing concrete are also presented. Finally, a summary of constituent material properties used in Nevada and the surrounding states is presented to determine how the concrete paving mixtures and constituent materials differ between Northern Nevada and Southern Nevada, as well as with the materials used in the surrounding states.

3.2 Aggregate Grading for Concrete Paving Mixtures

Mixture proportioning entails the selection of material proportions to produce economical concrete that meets the required specification parameters and design goals identified in the mixture design. Laboratory mixture proportioning is often used to develop an acceptable mixture design, and this should be considered a reference point for field mix trials carried out to ensure the mixture is workable for the paving conditions. Adjustments, permitted by specification, may be required prior to full-scale paving (Tayabji, Fick, and Taylor 2012). The following criteria list the qualities of a properly proportioned concrete mixture (ACI 1991; Kosmatka and Wilson 2016):

- Good workability of the fresh concrete.
- Acceptable strength and durability of the hardened concrete.
- Economic and cost effective mixture.
- Sustainable material selection and usage.

Several mixture proportioning methods are in use, the most widely being the absolute volume method detailed in ACI 211.1 (ACI 1991; Taylor et al. 2006). Other approaches to proportioning mixtures include proportioning from field data and proportioning from trial mixtures (see Kosmatka and Wilson 2016). One available mixture proportioning tool is the COMPASS Software, which provides guidelines to optimize job specific concrete mixtures (Transtec 2004). Another is an approach and software tool developed by Taylor et al. (2015) in which the aggregate system and paste quality are established and relative volumes of each selected. Kosmatka and Wilson (2016) recommend caution when using software tools as local material properties may not perform as predicted by computer models. The best way to determine constructability and performance is through field trials of mixtures established through laboratory testing.

Detailed information on conducting mixture proportioning can be found in ACI 211.1 (ACI 1991), Taylor et al. (2006), and Kosmatka and Wilson (2016), including step by step processes that can be used to determine initial proportions. However, a few key mixture proportioning topics deserve more detailed discussion herein. These include aggregates, cementitious materials, water content, *w/cm*, workability, chemical admixtures, and durability.

3.2.1 Aggregates

Aggregates used in concrete are most often derived from natural sources (either mined from gravel pits or quarried), although recycled (e.g., recycled concrete aggregates) and industrial byproduct materials (e.g., air-cooled blast furnace slag) are also used. The aggregates must be clean, hard, strong, and durable, and must be free of materials that will adversely affect the hydration of the cement and bonding of the hydrated cement to the aggregate particle (Kosmatka and Wilson 2016). Fine and coarse aggregates used in State DOT paving concrete are typically specified in accordance with AASHTO M 6, *Standard Specification for Fine Aggregate for Hydraulic Cement Concrete* and AASHTO M 80, *Standard Specification for Coarse Aggregate for Hydraulic Cement Concrete*, respectively. Alternatively, some State DOTs use ASTM C33, *Standard Specifications for Concrete Aggregates.* These specifications provide basic requirements for the aggregates to be used in concrete including permissible amounts of deleterious materials, physical properties with respect to soundness (AASHTO T 104) and abrasion (AASHTO T 96), and grading. Aggregate durability, with respect to freeze-thaw resistance and alkali-aggregate reactivity (AAR), are addressed through source approval. Source approval for aggregate resistance to freeze-thaw deterioration commonly requires testing using a variant of AASHTO T 161 (ASTM C666), with passing criteria customized by individual State DOTs based on their experience. Screening aggregates for AAR has been standardized in AASHTO R 80, which is the basis for ASTM C1778-16: *Standard Guide for Reducing the Risk of Deleterious Alkali-Aggregate Reaction in Concrete*.

The effect of aggregate grading on concrete performance cannot be overstated (Taylor 2015). Ley, Cook, and Fick (2012) demonstrated that for a constant paste volume and *w/cm*, aggregate grading, nominal maximum size, and aggregate types all impact concrete strength, workability, and ability to consolidate the concrete under vibration. When discussing aggregate grading, it is common to refer to an aggregate blend as being well-graded or gap-graded. Well-graded aggregate are distributed somewhat uniformly across sieve sizes whereas gap-graded aggregate typically exclude or have a very low percentage of aggregate retained on the intermediate sieve sizes. Well-graded aggregate are thought to be more densely packed in the mixture, leaving less void space to be filled by cement paste. Recently, considerable efforts have been expended investigating improved aggregate packing to develop “optimized aggregate grading” that improves concrete workability and permits a reduction in paste content (Cook et al. 2013a; Taylor et al. 2015). The key to success is to carefully develop the aggregate proportions to ensure that the mixture workability is not compromised. Taylor (2015) provides a good summary of aggregates blending for concrete paving mixture optimization.

Methods that have been investigated to improve aggregate grading include the coarseness factor chart, individual percent retained chart (8-18 curves), the 0.45 power curve, and most recently the “tarantula” curve. These as described below:

- The coarseness factor chart, developed by Shilstone (1990), uses a coarseness factor (CF) and workability factor (WF) to examine the distribution of coarse, intermediate, and fine aggregates in the combined grading. These factors are plotted on the modified coarseness factor chart (see Figure 3-1), which features pre-defined zones associated with certain levels of workability (Shilstone 1990; Cook et al. 2013b; Taylor 2015). Zone II is the most desirable for slipform paving of concrete having nominal maximum aggregate sizes between ¾ to 2 inch (19 to 50 mm).

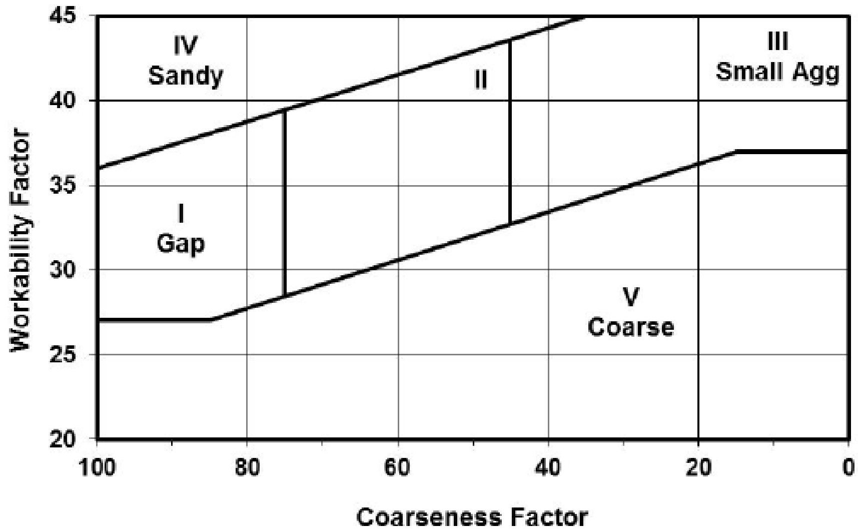

Figure 3-1. Shilstone coarseness factor chart (Taylor 2015).

- The individual sieve percent retained chart (also referred to as the “8-18” chart), also developed by Shilstone (1990), plots percent retained for the combined gradation for each individual sieve size. This chart provides a deeper understanding of the distribution of the combined aggregate gradation for each sieve size. Previous experience has suggested the use of lower and upper limits of 8 and 18 percent, respectively, but there is limited research to justify these limits (Cook, Ghaeezadah, and Ley 2013a).
- The 0.45 power curve plots the cumulative percent passing of the combined aggregate grading versus sieve size (commonly in millimeters), raised to the 0.45 power. A maximum density line is drawn from the origin to the nominal maximum size, and maximum and minimum percent passing limit lines are extended from the origin to one sieve size larger and smaller than the nominal maximum aggregate size, respectively (Taylor et al. 2006). Ley, Cook, and Fick (2012) and Cook, Ghaeezadah, and Ley (2013a) suggest this approach alone is not suitable for proportioning concrete mixtures for slipform paving.
- The tarantula curve (see Figure 3-2) has been proposed by Cook et al. (2013b) as a modification to the individual percent retained approach. Recommended limits for fine and coarse aggregate were developed based on research results and field performance, in which over 500 concrete mixtures were studied (Kosmatka and Wilson 2016). This approach places limits on specific aggregate sizes for the combined gradation that were found to improve workability and resistance to segregation.

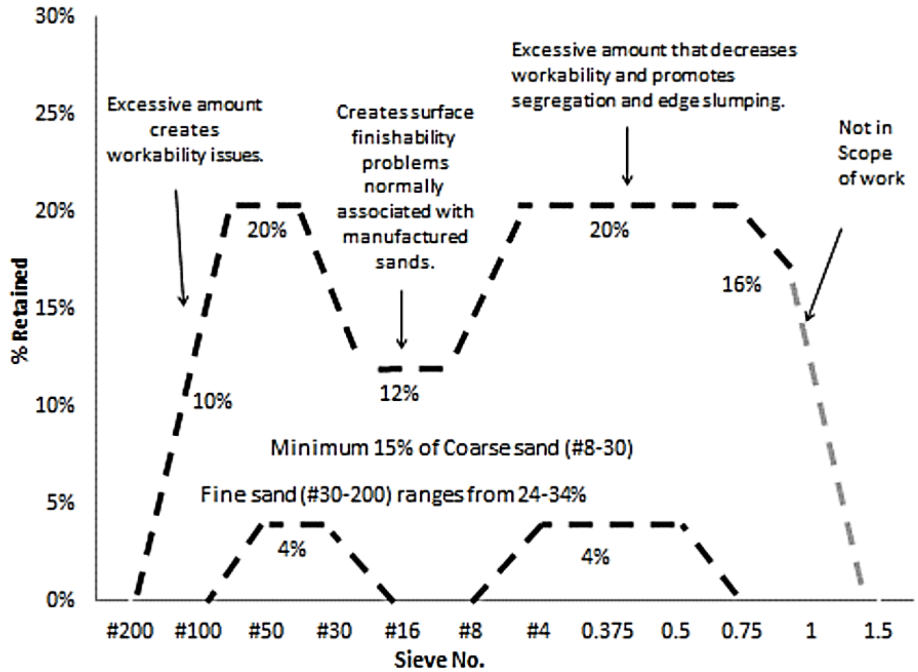

Figure 3-2. Tarantula curve with proposed limits for fine and coarse sand (Cook, Ghaeezadah, and Ley 2013a).

3.3 Constituent Material Specifications

3.3.1 Aggregate Properties

Concrete paving aggregate specifications for NDOT and the surrounding states present the coarse and fine aggregates separately, and many State DOT specifications include a sentence that makes a general statement regarding aggregate quality for concrete applications as follows:

Caltrans- *"Aggregates must be free from deleterious coatings, clay balls, roots, bark, sticks, rags, and other extraneous material."*

ORDOT - *"Coarse aggregate shall consist of rock, or other approved inert material of similar characteristics having hard, strong, durable pieces free from adherent coatings." And the fine aggregate material "shall also be reasonably free from all other harmful substances, such as shale, alkali, mica, coated grains, and soft and flaky particles."*

WYDOT - *"Wash coarse aggregate to remove adherent soil coatings and reduce the amount of material passing a No. 200 sieve at least 50 percent."*

One observation to note is the above specifications make specific reference to aggregate coating. For instance, WYDOT requires that the coarse aggregates be washed to remove adherent soil coatings. Caltrans states that the coarse aggregate should be "free from deleterious coatings" and

ORDOT states that aggregate must be "free from adherent coatings." There is recognition by these states that aggregate coatings are potentially harmful. NDOT, on the other hand, does not make a statement of this type for aggregates to be used in concrete, but does state for aggregate to be used in HMA:

"The mineral aggregate shall be clean, hard, durable, free from frozen lumps, deleterious matter, and harmful adherent coatings."

Table 3-1 and 3-2 summarize the regional State DOT coarse and fine aggregate specifications for use in PCCP, respectively. In comparing the coarse aggregate requirements in different states, it is easy to see similarities and differences between the NDOT specification and the regional specifications. Of the12 aggregate properties considered, NDOT has specification limits for six. Caltrans and UDOT specify only two and five, respectively. ORDOT, WYDOT, and ITD have more specified limits on properties than NDOT, with seven, eight, and nine, respectively. For the specified properties that NDOT has in common with the other five states, the values are very similar. The requirements for sand equivalent, percent passing the No. 200 sieve, sodium soundness, and potential for ASR all match the limits other states specify. Further, NDOT has a lower allowable amount of clay lumps than the other states that specify this property. However, the NDOT specification for aggregate durability in the LA Abrasion test is significantly less stringent than the surrounding states, allowing 50 percent mass loss compared to an average of 32 percent for Caltrans, ITD, ORDOT, and WYDOT.

Table 3-2 summarizes the fine aggregate specifications for use in PCCP. NDOT and ITD have the highest number of specified properties with five. For the sand equivalent, clay lumps, sodium soundness, and percent passing the No. 200 sieve, NDOT has similar required values as the other states. Furthermore, NDOT is the only state that has a specification to evaluate effect of organic impurities in fine aggregate on mortar strength.

3.3.2 Portland Cement

Historically, minimum cementitious materials content has been specified by state highway agencies with intent to ensure sufficient strength, good workability, and adequate durability. More recently, the minimum total cementitious materials content requirements have been reduced for many states as optimized aggregate gradings have been adopted. These optimized gradations accommodate the use of reduced cementitious contents while maintaining workability and durability (Taylor et al. 2006; Cook et al. 2013). It was common in the past to recommended a minimum cementitious content of "six sacks" (564 lbs/yd^3) for improved durability. This guidance has been withdrawn, with new recommendations suggesting that high cementitious materials contents (greater than 600 lbs/yd^3) should be avoided as such mixtures cost more and are prone to excessive drying shrinkage and increased risk of durability issues (Kosmatka and Wilson 2016).

Table 3-1. Summary of regional specifications for coarse aggregate.

Property	NDOT	Caltrans	ITD	ORDOT	UDOT	WYDOT
Cleanness/Sand Equivalent	71 min (Nev. T228)	71 min (Cal. T227)	-	-	-	-
Clay lumps (AASHTO T112)	0.3% max	-	0.5% max	-	-	0.5% max
Clay lumps and Friable Particles (AASHTO T 112)	-	-	2% max	-	2% max	2% max
Coal and Lignite (AASHTO T 113)	-	-	1% max	-	0.5% max	0.1%
Percent Passing No. 200 sieve	1% max	-	1% max	1% (1.5% if free of clay and shale)	1% (1.5% if free of clay and shale)	2% max
Sodium Soundness (AASHTO T 104)	12% max after 5 cycles	-	12% max after 5 cycles	12% max	12% max	12% max (MTM 403.0)
Fractured Faces (AASHTO T 335)	-	-	40% min	50% min [A]	50% max	50% min [B]
Alkali-Silica Reactivity (AASHTO T 303)	0.1% max (combined gradation)	-	0.1% max	-	-	0.1% max (combined gradation)
LA Abrasion (AASHSTO T 96)	50% max	25% max [C] (Caltrans T 211)	35% max	30% max	-	40% max
Flat and Elongated	-	-	15% max	10% max for plus No 4 material	-	-
Wood Particles (TM225)	-	-	-	0.05% max	-	-
Air Degradation (TM 208)	-	-	-	3% all, (30% minus No. 20 material)	-	-

[A] *At least 2 fractured faces, for particles retained on 3/8", ½", ¾", 1", and 1/5" sieves.*
[B] *At least 1 fractured face, for all material retained on the No. 4 sieve.*
[C] *For the high desert and high mountain climate regions.*

Table 3-2. Summary of regional specifications for fine aggregate.

Property	NDOT	Caltrans	ITD	ORDOT	UDOT	WYDOT
Cleanness/Sand Equivalent (AASHTO T 176)	71% min Nev. T228	71 min Cal. T227	70 min	68 min	-	-
Clay lumps (AASHTO T112)	1% max	-	1% max	-	-	1% max
Percent Passing No. 200 sieve	5% max	-	2% max (3% max where SE<80)	4% (6% where it is free of clay)	-	4% max
Sodium Soundness (AASHTO T104)	10% max after 5 cycles	-	10% max after 5 cycles	10% max	-	-
Coal and Lignite (AASHTO T 113)	-	-	1% max	-	-	1% max
Mortar Strength (ASTM C87)	95% min	-	-	-	-	-

Regional specifications related to cement used in PCCP are summarized in Table 3-3. There are quite a few similarities between states with regards to cement type, as all six states specify the same types of cement, with Type II (ASTM C150/AASHTO M 85) and Type IP (ASTM C595/AASHTO M 240) being allowed by all of the states (Type II is used exclusively by UDOT). However, NDOT is the only state that has additional requirements of the Type IP cement in that the cement portions shall meet the Type II cement requirements and that the pozzolan constituent is at least 20 percent. Another similarity is that all six states require the use of low alkali cement, with a maximum equivalent alkali content of 0.6 percent by mass as taken by the sum of Na_2O + 0.658 * K_2O content.

Table 3-3. Summary of cement types permitted by regional specifications.

Spec.	NDOT[A]	Caltrans[A]	IDDOT[A]	ORDOT[A, F]	UDOT[A]	WYDOT[A]
ASTM C150	(II or V)	(II[E], III, or V)	-	-	II[C]	II[D]
ASTM C595	IP[B]	-	-	-	-	I(PM)
AASHTO M 85	-	-	(I, II, III)	(I or II)	-	-
AASHTO M 240	-	IS, IP	IP, I(PM), IS(20)	IP, I(SM)	-	-

[A] *Maximum of 0.6% equivalent alkalis by mass (Na_2O + 0.658 K_2O)*
[B] *Minimum of 20% pozzolan constituent, cement portion shall meet type II requirements*
[C] *Unless otherwise specified (AASHTO M 85 types I, III, V or ASTM C595 types IP, IP(MS)*
[D] *Unless otherwise specified (ASTM C150 type V)*
[E] *C_3S content less than 65%*
[F] *The maximum fineness (specific surface) as determined by the air permeability test shall be 430 m^2/kg for any field-sampled check test.*

3.3.3 Supplementary Cementitious Materials

In general, replacement of portland cement with SCMs reduces costs while positively impacting the fresh and hardened concrete properties, including increased workability, improved long-term strength, reduced permeability, increased resistance to ASR and sulfate attack (Taylor et al. 2006; Kosmatka and Wilson 2016). The exception is natural pozzolans, which in some cases can significantly increase water demand and thus negatively affect workability.

The most common SCM is fly ash, being classified as either Class F or Class C under ASTM C618/AASHTO M 295. Class F fly ash is typically lower in lime (CaO) than Class C fly ash and is more effective at mitigating ASR and sulfate attack (at lower replacement levels, Class C fly ash can make ASR worse in what is referred to as a "pessimum limit"), but typically retards strength gain at early ages. In comparison, Class C fly has less impact on early strength gain but is often not as effective in mitigating ASR as Class F fly ash, and thus has to be used at higher addition rates. And Class C fly ash cannot be used to mitigate sulfate attack (ACI 2016). Recently, there has been renewed interest in natural pozzolans (ASTM C618/AASHTO M 295 Class N), such as volcanic ash, calcined clay, calcined shale, and metakaolin, as shortages of fly ash continue to occur. In Northern Nevada, natural pozzolans are commonly used in paving concrete.

Slag cement (ASTM C989/AASHTO M 302) is another common SCM, but its usage is limited to certain areas of the U.S. with access to blast furnace slag (parent material) or shipping terminals. When used in proper proportions with portland cement, slag cement can significantly reduce the permeability of concrete and effectively mitigate ASR and sulfate attack.

Typical SCM replacement/addition rates for paving concrete are in the range of (Taylor et al. 2006):

- Class C fly ash – 15 to 40 percent.
- Class F fly ash – 15 to 25 percent.
- Natural pozzolans – 15 to 25 percent
- Slag cement – 35 to 50 percent

Table 3-4 summarizes the specifications for SCMs used in concrete paving mixes in Nevada and regional states. Regardless of whether a state chooses to follow ASTM or AASHTO, the six states all specify the same types of SCMs. Class F fly ash is the allowed in all six states, with ITD and WYDOT using Class F exclusively. NDOT and ORDOT are the only two states that allow the use of Class C fly ash. Five of the states also allow for the Class N natural pozzolans, with WYDOT being the only state that does not allow it. Lastly, Caltrans and ORDOT allow blast furnace slag to be used in the concrete mixes.

Table 3-4. Summary of regional specifications for supplementary cementitious materials.

Spec.	NDOT[A]	Caltrans[B]	ITD	ORDOT	UDOT	WYDOT
ASTM C618	C, F, N	-	-	C, F, N	-	F
AASHTO M 295	-	F, N	F[C]	C, F, N	F[D], N	-
AASHTO M 302	Slag Cement	Slag Cement Grade 100, 120	-	Slag Cement	-	-

[A] *Loss of ignition less than 5%.*
[B] *Na_2O + 0.658 K_2O less than 1.5% per ASTM C311 or 5% per AASHTO T 105.*
[C] *Loss of ignition less than 1.5%, Na_2O less than 1.5%, CaO less than 11%.*
[D] *Loss on ignition less than 3%, CaO less than 15%.*
[E] *Allowed as alternative to pozzolan at an addition rate of 35%.*

3.3.4 Admixtures

Admixtures are used to modify fresh and hardened properties of concrete mixtures, typically air content, water demand, and set time. It is recognized that although admixtures can be used to enhance concrete properties, the use of admixtures is not a replacement for proper mixture proportioning. In addition, unintended interactions should be identified and adjusted for through producing trial batches (prior to production) that simulated anticipated ambient conditions during placement. Further, trial batches should be used if constituent materials change during production. Typical categories of admixtures include air entraining, water reducing, retarding, and accelerating (Taylor et al. 2006).

Table 3-5 summarizes the specifications for the admixtures used in PCCP for the six states. Five of the six states refer to ASTM C494 for specifications related to water-reducing admixtures (WRA), accelerators (ACC), and for set retarders (SRT). UDOT is the only state that used AASHTO specifications exclusively. For air-entraining admixtures (AEA), NDOT and Caltrans use ASTM standards while ITD and UDOT use AASHTO. ORDOT and WYDOT specify admixtures that meet either ASTM or AASHTO specifications.

Table 3-5. Summary of regional admixture specifications.

Spec.	NDOT	Caltrans	ITD	ORDOT	UDOT	WYDOT
ASTM C494	WRA, ACC, SRT	WRA, ACC, SRT	WRA, SRT	WRA, ACC, SRT	-	WRA, SRT
ASTM C260	AEA	AEA	-	AEA	-	AEA
AASHTO M154	-	-	AEA	AEA	AEA	AEA
AASHTO M194	-	-	-	WRA, ACC, SRT	WRA, ACC, SRT	WRA, SRT

3.4 Concrete Paving Mixture Specifications

Table 3-6 summarizes the specifications for the concrete paving mixtures for each state. As is seen, there are a lot of similarities between the different specifications, but also notable differences.

3.4.1 Total Cementitious Material Content

Five of the six state DOTs (ORDOT being the exception) specify the amount of total cementitious materials per cubic yard of concrete. WYDOT and UDOT require a minimum cementitious materials content of 564 lb/yd^3, Caltrans requires a minimum of 590 lb/yd^3 if concrete is being placed in a freeze-thaw climate zone (505 lbs/yd^3 in non-freeze-thaw environment), and NDOT 611 lb/yd^3. Utah increases the required minimum total cementitious materials content to 611 lb/yd^3 if nominal maximum aggregate size is reduce to 1 inch or smaller. Three state DOTs also specify a maximum allowable total cementitious material contents: 675 lb/yd^3 by Caltrans and 705 lb/yd^3 for NDOT and WYDOT. ITD requires 660 lb/yd^3, although this can be reduced by 20 percent (to 528 lbs/yd^3) if an optimized gradation is used.

Table 3-6. Summary of regional concrete paving mixture specifications.

Spec. Property	NDOT	Caltrans	ITD	ORDOT	UDOT	WYDOT
Cementitious Materials (lbs/yd^3)	611 to 705	505 to 675 (min 590 if freeze-thaw climate)	660 (if fly ash 550 cement, 138 fly ash)	-	564 (2" and 1.5" NMAS) 611 (1" and 0.5" NMAS)	564 to 705 (470 min cement if using fly ash)
Supplemental Cementitious Material Content	20% min pozzolan or 35% min slag cement (Max 50% of slag and pozzolan replacement)	-	25% max fly ash	-	20% min 30% max (unless approved or specified)	20 to 25% Class F allowed (required for level 1 control)
Maximum w/cm	0.45	-	0.44 (0.42 if using fly ash)	0.44	0.44	0.45
28-Day Compressive Strength (psi)	4,000[1]	3,600	4,500	4,000	4,000	
28-Day Flexural Strength (psi)	650[2] (550[3])	570 (650 at 42 days)	-	600	650	650
Slump (in)	1" to 5" (< 8" if using Type F HRWR)	-	2" max	4" max without WRA, 5" max with WRA, 5.5" ± 2.5" with HRWR	3.5" with no WRA 5" with WRA 9" With HRWR	0.5" to 2" for slip-form paver, 4" (6" with WRA) for placed in forms
Air Content (%)	4% to 7% (The air range indicated is not required in Clark County)	6% ± 1.5% for high desert and high mountain regions 4% ± 1.5% for low and south mountain regions	4% to 7%	5% ± 1.5% for severe exposure (everywhere but coast)	4% to7% (2"NMAS) 4.5% to 7.5% (1.5" NMAS) 5% to 7.5% (1" and 0.5")	4.5% to7.5%
Other Requirements	-	-	-	-	-	44% max fine aggregate (weight)

[1] Basis for acceptance.

[2] Basis for mixture design approval.

[3] Basis for opening to traffic.

For the amount of pozzolan, NDOT and UDOT each require a 20 percent replacement of the cementitious material with pozzolan. Alternatively, NDOT allows 35 percent slag to be used in lieu of pozzolan, or a ternary blend of pozzolan and slag not to exceed 50 percent replacement. WYDOT allows, and at times may require, a 20 to 25 percent replacement with fly ash for concrete paving mixes. ITD has specifies a maximum pozzolan content of 25 percent, stating that the minimum portland cement content is 550 lb/yd^3 if using fly ash. Caltrans and ORDOT do not have specification limits on SCMs.

Nationally, many DOT specifications have minimum cementitious materials contents requirements, with the intent of ensuring sufficient strength, good workability, and adequate durability. In recent years, the required minimum cementitious materials content requirements have been reduced by many DOTs as optimized aggregate grading has been implemented, as these gradations accommodate the use of reduced cementitious contents while maintaining workability and durability (Taylor et al. 2006; Cook et al. 2013). Previous guidance that recommended a minimum cementitious content of 564 lbs/yd^3 for durability have been dropped, with new recommendations suggesting that high cementitious materials contents (in excess of 600 lbs/yd^3) should be avoided as such mixtures have increased cost and higher risk of durability issues due to excessive drying shrinkage (Kosmatka and Wilson 2016). Colorado DOT for instance has a minimum cementitious materials content requirement for paving mixtures (Class P) set at 660 lbs/yd^3, but reduces this to 520 lbs/yd^3 if acceptance is based on flexural strength.

3.4.2 Water to Cementitious Materials Ratio (w/cm)

The maximum water-to-cementitious materials ratio (*w/cm*) is specified by five of the states, with Caltrans being the lone state that does not specify this value. The maximum value ranges from 0.42 (in Idaho when using fly ash) to a maximum of 0.45 by NDOT and WYDOT.

The selection of the proper *w/cm* is a key parameter in any concrete mixture design as it strongly influences the strength and durability of the concrete mixture. The *w/cm* selected for proportioning should be the lowest value needed to achieve the desired strength and meet expected exposure conditions (Kosmatka and Wilson 2016). ACI (2016) recommends that the maximum *w/cm* for plain concrete subjected to freezing and thawing in a saturated condition or subjected to severe sulfate exposure be 0.45, whereas very severe exposure to sulfates in the soil requires a maximum *w/cm* of 0.40. A further reduction in *w/cm* may improve strength and durability, but using a *w/cm* much below 0.40 may have undesirable negative impacts including an increased risk of autogenous shrinkage, increased difficultly in entraining air into the mixture, and increased issues with workability (Tayabji, Fick, and Taylor 2012). Thus a minimum *w/cm* of 0.40 is recommended for paving concrete, although some states (most notably Minnesota) have had success with *w/cm* as low as 0.37 (Taylor et al. 2006).

3.4.3 Workability

The slump requirement of the concrete paving mixtures is specified for five of the six states, which maintain different requirements for the slump depending on method of placement and what admixtures are present within the paving mixture. ITD and WYDOT have a 2-inch maximum slump requirement for concrete paving mixtures, while ORDOT and UDOT have similar maximum slump limits of 4 inches and 3.5 inches, respectively, if no water-reducing admixture (WRA) is used. If a WRA is used, ORDOT and UDOT allow slump values up to 5 inches. NDOT requires a slump between 1 and 5 inches, except for mixtures with HRWR, where the slump can be a maximum of 8 inches. Caltrans does not have a slump requirement for concrete paving mixtures, but rather tests for penetration of the concrete under California Test 533, with a maximum penetration of 1 inch. Note that a 2-inch maximum slump is normally specified for slipform paving, whereas 3 to 4 inch slump is common for sideform paving.

3.4.4 Strength

Acceptance is most often based on 28-day strength, but this can also vary, especially for early-age mixtures (time is shortened) or if high amounts of SCMs are used (time is lengthened). Traditionally, concrete flexural strength (routinely called modulus of rupture) has been specified for acceptance as it is linked directly to design. Yet in recent years, compressive strength has become more common for quality control, opening time, and even acceptance (Taylor et al., 2006). Four states have specific reference to compressive strength (Caltrans and WYDOT do not) and five states reference flexural strength (ITD does not) for various. For compressive strength, NDOT, ORDOT, and UDOT all specify a minimum of 4,000 psi at 28 days whereas ITD specifies 4,500 psi. For flexural strength, NDOT, UDOT, and WYDOT all specify 28-day strengths of 650 psi (for NDOT, this is the limit for approval of the mixture design), whereas Caltrans specifies a 28-day flexural strength of 570 psi and a 42-day strength of 650 psi. Caltrans's specification allows the concrete an extra two weeks to reach 650 psi flexural strength compared to NDOT, UDOT, and WYDOT allowing for the slower reactivity of SCMs to contribute.

3.4.5 Air Content

The specifications for air content are quite similar amongst the six states. NDOT (except for Clark County) and ITD require the air content to be between 4 and 7 percent, with WYDOT and Caltrans (in freeze-thaw zones) requiring a value between 4.5 and 7.5 percent. ORDOT requires a value from 4.5 to 6.5 percent for "severe exposure" areas, which are defined as areas where the elevation is 1,000 feet and above (the area between the coast and beginning of the Cascades mountain range is the only part of Oregon at or below 1,000 feet). UDOT requires a minimum air content of 4, 4.5, or 5 percent and a maximum air content of 7 or 7.5 percent depending on the nominal maximum aggregate size (NMAS) of the combined aggregate gradation, with lower air content as maximum aggregate size increases. This is consistent with ACI (2016), which links required air content to severity of exposure and maximum size of the aggregate.

3.4.6 Aggregate Grading

For paving grade concrete, NDOT requires that a three aggregate system be used, consisting of a blend of two coarse aggregates and a fine aggregate. Grading limits are established in Section 409.02.01 of the 2014 NDOT Standard Specifications. The cited method for combining these aggregates to achieve the grading limits is ACI 302.1R-04, Section 5.4.3 *Combined Aggregate Grading* (ACI 2004). It is noted that this ACI guide document is specific to concrete floor and slab construction. Further, the guide was updated in 2015 and the section cited in the NDOT Standard Specifications is not current, but has been replaced by Section 8.9.2 *Aggregate Blending*, which describes the use of the coarseness factor-workability factor chart previously described in Section 3.2.1 of this report. Caltrans, ITD, and UDOT allow the use of a combined gradation, but it is not required. ORDOT and WYDOT use a coarse and fine aggregate.

3.5 Mixing, Placing, and Curing

3.5.1 Aggregate Handling

Combined, the coarse and fine aggregate fractions makes up approximately 70 percent of the total volume of concrete. Because of the high amount of aggregate in concrete mixtures, the properties of the aggregate have a large influence of the concrete mixture's properties. In addition to the specified properties of the aggregates presented in Section 3.3.1, there are other properties of the aggregates that can affect the mechanical and durability properties of concrete mixtures. As discussed, aggregate coatings can result in lower strengths as they can interfere with aggregate particles bonding with the cement paste. Additionally, absorptive aggregates that are batched dry of SSD may release air into the concrete as they soak up water early in the hydration process, causing a weak zone in the concrete around the aggregate particles. Both aggregate coatings and moisture content at batching are influenced by aggregate handling.

NDOT specifications require that the aggregate be brought to fully saturated condition (at or above SSD) prior to batching. This is to be accomplished through the addition of water to aggregate stockpiles, with testing to be conducted on the aggregates to determine amount of free water so that mix water can be adjusted during batching. The NDOT specifications also restrict the use of aggregates that show visible separation of water and aggregates during transportation from stockpile to batch plant.

The Caltrans specifications state that at the time of batching, the aggregates must be dried and drained to a stable moisture content such that no visible separation of water from the aggregate occurs during transportation from the proportioning plant to the point of mixing. Additionally, the free moisture content of the fine aggregate must not exceed 8 percent of its SSD-weight at the time of batching. Lastly, the aggregates shall be stored in such a way that segregation of the aggregate particles does not occur.

ORDOT specifications state that the aggregates be placed in stockpiles and removed from stockpiles in a manner that holds segregation to a minimum. Additionally, the specifications restrict the use of aggregates that become segregated, mixed with earth or foreign material, or contain lumps of hardened material. Additionally, stockpiles with frozen aggregates or aggregates containing frozen lumps shall be thawed before mixing.

UDOT has limited specification language related to handling of aggregates. There is one specification that states that all washed aggregates shall be allowed to drain to uniform moisture content before use (12 hours minimum). Additionally, UDOT requires stockpiles be constructed to minimize segregation of the aggregates.

ITD specifications state that the supplier shall transport aggregates from stockpiles or other sources to the batching plant so that uniform grading and stable moisture content is maintained. Additionally, any aggregate placed in stockpiles or bins shall be allowed to drain for at least 12 hours before batching any aggregates produced or handled by hydraulic methods or any washed aggregates.

WYDOT does not have any specifications that discuss the handling of aggregates prior to batching.

Table 3-7 summarizes State specifications with respect to the handling of aggregate prior to batching. A review of these specifications shows that only two states explicitly require that the moisture content of the aggregates be in SSD condition or above prior to batching. Four of the states discuss having excessive free moisture in the aggregate. Of those, two states require a 12 hour time for the aggregates to drain and two states require no visible separation of water from the aggregates. While the specifications address the moisture content of the aggregates, none of the six states specify that the aggregates should be washed if the surface appears dirty or coated with clay or other deleterious matter.

Table 3-7. Summary of regional specifications for handling of aggregate prior to batching.

Specified Property	NDOT	Caltrans	ITD	ORDOT	UDOT	WYDOT
Minimum Moisture Content	Yes	Yes	No	No	No	No
Maximum Moisture Content	Yes	Yes	Yes	No	Yes	No
Segregation of Particles	No	Yes	Yes	Yes	Yes	No
Washing to Remove Coatings	No	No	No	No	No	No

3.5.2 Batching, Mixing, Retempering, and On-Site Adjustments of Concrete

This section discusses specifications related to the mixing, transport, and adjustments of the concrete paving mixtures. Specifications related to the mixing of the concrete should reduce batch-to-batch variability and will increase the quality of the paving concrete mixes. Also important is whether or not any adjustments are able to be made to the concrete at the job site. These adjustments, most notably the addition of water, can cause problems to arise during or after construction that are not easy to detect or may go completely unnoticed until much later. The specifications related to the mixing and field-adjustments to paving mixes for the six states are discussed below.

3.5.2.1 Batching and Mixing Specifications

NDOT specifications for mixing concrete state that the concrete shall be mixed between 60 and 90 seconds. The mixing is specified to begin within 30 minutes of intermingling the damp aggregates with cement. The mixing water shall enter the drum in advance of the cement and aggregates, and all of the water shall be uniformly added within the first 15 seconds of the mixing period. The throat of the drum must be kept free of accumulations that may restrict the flow of materials into the drum. The mixing time will begin after all materials have been completely charged into the mixing drum, and will end when the mix discharge begins. The temperature of the materials as charged into the mixer should be such that the temperature of the mixed concrete at placement in the final position is between 50 °F and 90 °F. However, the temperature of the aggregates and mixing water shall not exceed 150 °F at time of batching. For truck mixing, each batch must be mixed for between 70 and 100 revolutions of the drum.

The Caltrans specifications for mixing concrete state that the start of mixing must occur within 30 minutes after the damp aggregates and cement are intermingled. The mixing sequence requires that the water must be added within the first one fourth of the mixing time, the cement and aggregates must be charged simultaneously with the cement, and the air-entrainment admixture must be the first admixture incorporated into the mixture. However, Caltrans allows the concrete producers to add the admixtures in a different order if a trial batch demonstration shows improved performance. The mixing time must be from 50 seconds to 5 minutes. The temperature of the aggregates and water may not exceed 150 °F and be adjusted so the temperature of the mixed concrete prior to placement is from 50 °F to 90 °F. Central-mixed concrete shall be mixed for at least 90 seconds.

ITD specifies that the minimum mixing time for mixers of 10 yd^3 or less capacity is 50 seconds when a stationary mixer is used for the complete concrete mixing. The supplier is to obtain the Engineer’s approval of the mixing time for mixers of 10 yd^3 or more capacity. The mixing time is from the time cement and aggregates are in the drum to discharge of the concrete from the mixing drum. Charging the batch into the mixer will be such that some water will enter in advance of cement and aggregates, and all water to be added at the mixer is to be in the drum by the end of the first one fourth of the specified mixing time. Concrete mixed in a truck mixer shall be mixed from between 70 to 100 revolutions. The volume of concrete being mixed shall not exceed 70 percent of the drum gross volume. The temperature of the materials prior to mixing shall be such that the temperature of the concrete at placement is between 50 °F and 85 °F. The mixing water may be between 70 °F and 150 °F for cold-weather concrete.

In ORDOT, the supplier is required to mix all concrete in accordance with ASTM C94, *Standard Specification for Ready-Mixed Concrete*. Some mixing water is required to be charged into the drum before the solid components are added. The concrete is to be mixed for at least 60 seconds in stationary mixers or 70 to 100 revolutions at mixing speed for truck mixers. The temperature of the concrete at time of placement must be between 50 °F and 90 °F.

In UDOT, the concrete must be mixed for at least 80 seconds after all materials are in the drum. Mixing must continue for at least 30 seconds after the last addition of water or cement is made after initial batching. The temperature of the concrete during placement shall be between 50 °F and 90 °F, unless otherwise specified.

WYDOT specifies that the mixing time shall be in accordance with the plant manufacturer's recommendations, but shall not be less than 50 seconds. The mixing time begins when all of the dry materials are in the drum and ends with the start of the discharge. All of the mix water shall be added to the batch within the first 15 seconds of the mixing time, but a portion of the water shall be in the mixer ahead of the cement and aggregate. The concrete temperature shall be between 50 °F and 90 °F during placement.

3.5.2.2 Allowable Adjustments

NDOT specifications allow on-site adjustments, stating that one attempt may be made when the concrete is delivered to the job site by adding approved admixtures to bring the slump and air content of the concrete into specification. These adjustments must be made before 10 percent of the originally batch concrete has been discharged from the delivery vehicle. If any admixtures or additional water are added to the concrete, the drum must be revolved at mixing speed for 45 revolutions or until the concrete is homogeneous. Water may not be added to the concrete once a water-reducing admixture has been incorporated into the mix. NDOT does not allow concrete to be retempered and any concrete where initial set has begun must be wasted. Lastly, central-mixed concrete transported in non-agitating equipment must be completely discharged within 45 minutes while concrete delivered using a truck mixer or agitator must be completely discharged within 90 minutes after addition of the cement to the aggregates.

Caltrans allows one addition of water to the concrete if authorized by the Engineer, but no more than what is allowed by the authorized mix design. This is in compliance with the requirements of ASTM C94, *Standard Specification for Ready-Mixed Concrete*. After adding water, the drum must revolve at least 30 times at mixing speed before discharging the concrete. Caltrans does not allow for concrete to be retempered. Lastly, concrete transported in non-agitating equipment must be completely discharged within 60 minutes while concrete delivered using a truck mixer or agitator must be completely discharged within 90 minutes after addition of the cement to the aggregates.

The only specification ITD has for on-site adjustments is that the Engineer may allow a one-time addition of water during the discharge of the concrete. If approved, a minimum of 30 additional revolutions of the truck mixer drum at mixing speed is required before discharging any of the concrete. Lastly, the concrete shall be discharged from the delivery truck within 90 minutes after the cement and aggregates are mixed together.

ORDOT does not allow the concrete to be retempered by adding water or by other means. Any concrete that has been retempered, or has begun to take an initial set before placement, will be rejected. Concrete must be discharged within 60 minutes (90 minutes if delivered in a ready-mix truck) and all concrete not discharged will be rejected.

UDOT specifies that air-entraining agents may be added in one addition per load using pre-measured admixtures. Also, the amount added must be recorded on the batch ticket and the mixing drum must revolve at least 30 revolutions at the mixing speed before discharging. The concrete must be discharged from non-agitating haul equipment within 45 minutes and agitating haul equipment within 75 minutes. Concrete must also be discharged before initial set begins.

The WYDOT specifications allow for two mix adjustments, defined as the addition of water or an on-site admixture. Adjustments are not allowed for partial loads. Adjusting the mix on-site does not increase the allowable placing time requirements. However, air-entrainment admixtures are not allowed to be used for on-site adjustments. If on-site adjustments are made, additional mixing revolutions in accordance with the admixture manufacturer's recommendations are required. Mix for at least 30 additional revolutions at mixing speed if water is added at the site. On-site adjustments to the mix must be performed while the concrete is plastic and within 45 minutes of the start of initial mixing. Water or other materials are not allowed to be added to concrete that has started to set.

3.5.2.3 Summary Curing

NDOT specification for curing PCCP is described in Section 409.03.13 of the 2014 *Standard Specifications for Road and Bridge Construction*. The specifications allow a curing compound method and waterproof membrane method. The curing method prohibits application of the curing compound until all patching and surface finishing is completed (grinding excluded). Curing compound must be applied immediately after the moisture sheen disappears from the surface but prior to the onset of plastic shrinkage cracking. If fine cracks appear in the newly placed concrete, the concrete should be fogged until the finishing operations are completed and the curing is applied until it is no longer needed. The waterproof membrane method, presented in Subsection 501.03.08 (b), calls for keeping the concrete damp with water from an atomizing nozzle until set and then the entire concrete surface should be covered with membrane that is kept in place for not less than 7 days.

Table 3-8 summarizes the NDOT and regional state specifications regarding the mixing and on-site adjustments for concrete paving mixes. The minimum mixing time at central-mix plants is very similar with three states requiring 50 seconds, two requiring 60 seconds, and one requiring

Table 3-8. Summary of regional specifications for mixing and on-site adjustments.

Spec.	NDOT	Caltrans	ITD	ORDOT	UDOT	WYDOT
Central-Plant Mixing Time (sec)	60-90	50-300	Min 50	Min 60	Min 80	Min 50
Truck-Mixed Minimum Revolutions	70-100	Note 1	70-100	70-100	Note 2	Note 2
Maximum Temperature of Water and Aggregates at Time of Mixing	150°F	150°F	150°F	Note 2	Note 2	Note 2
Concrete Temperature at Placement (°F)	50-90	50-90	50-85	50-85	50-90	50-90
On-site Addition of Admixtures	Allow	Note 2	Note 2	Note 2	Allow	Allow
On-site Addition of Water	Allow	Allow	Allow	Note 2	Note 2	Allow
Retempering of Concrete	No	No	Note 2	No	Note 2	No
Waste Concrete in Initial Set	Yes	Yes	Note 2	Yes	Note 2	Note 2
Maximum Time Concrete must be Discharged from Truck after Mixing (minutes) Note 3	45/90	60/90	90	60/90	45/75	35/60

Note 1: The minimum required revolutions must be at least that recommended by the mixer manufacturer and must be enough to produce uniform concrete.
Note 2: There are no specifications that allow nor disallow this activity.
Note 3: First time is if non-agitating haul equipment is used, second time is if agitating haul equipment is used to transport concrete from batch plant to job site.

80 seconds. For the truck-mixed concrete, three states required 70 to100 revolutions, while a fourth state defers to the mixer manufacturer’s recommendations. All six states require the concrete be a minimum 50°F and a maximum of 85 °F or 90°F at time of placement. Three states allow the addition of admixtures in the field while the other three states do not have specifications addressing the addition on-site admixtures. Four states allow for the addition of water on-site and two states do not have a related specification. Four states do not allow retempering of concrete while three states require concrete that is in the initial set stage to be wasted. Lastly, the maximum amount of time that concrete can be hauled from the plant to the job site does vary between the states, but the average is about 50 minutes for non-agitated equipment and 90 for agitated equipment. Overall, the specifications NDOT have are very similar to the five surrounding states.

Subsection 40-1.03I of the 2015 Caltrans Standard Specifications state that curing pavement shall be accomplished using the waterproof membrane method or curing compound method. Curing compound shall be No.1 or No.2 and be applied using mechanical sprayers. Concrete shall be maintained at 40°F or great for the initial 72 hours.

Curing is described in Section 409.03 L of the 2012 ITD Standard Specifications for Highway Construction. Curing shall use a System 2 white pigmented membrane forming curing compound as described in Section 709.01. Two applications shall be applied (approved machine method) in opposite directions before initial set has taken place. Any damaged to the film that occurs within 72 hours shall be immediately sprayed with additional curing compound.

Approved curing methods are provided in Section 00756.53 of the 2015 ORDOT Standard Specifications for Construction include (1) liquid membrane-forming curing compounds and (2) other coverings to include polyethylene film, waterproof paper, or cotton/jute mats. Curing shall commence immediately after final finishing and while the concrete surface is still moist. The curing method shall remain in place for at least 72 hours.

The UDOT Supplemental Specification 02752M, *Portland Cement Concrete Pavement* in the 2012 Standard Specification book describes curing practices for PCCP. The specifications allow liquid membrane-forming compounds. Curing compound shall be applied to the surface and exposed edges immediately following the completion of finishing. Two applications shall be applied in opposite directions using an atomizing sprayer with wind protection hood.

3.6 Chapter Summary

This chapter provided a synthesis of specifications for concrete paving mixtures and practices used for projects in Nevada and the surrounding regional states including California, Idaho, Oregon, Utah, and Wyoming. In addition, aspects of national practice pertaining to mixture design, specifications, and flexural strength is interjected for comparison with regional practice. The following summarizes the observations noted in this review:

- The range of total cementitious materials specified by NDOT (611 to 705 lbs/yd^3) is at the high end of that specified by other regional state DOTs, and higher than currently recommended by national practice.
- NDOT's SCM requirements are within the range required by other regional DOTs and by national practice. The common use of natural pozzolans (ASTM C618 Class N) is a regional practice and is not common east of the Rocky Mountains.
- The maximum *w/cm* of 0.45 specified by NDOT is consistent with regional and national practice for plain concrete placed in a freeze-thaw environment and exposed to deicing chemicals.
- NDOT aggregate specifications are similar to those in other regional states. A few differences include many of the other states make a specific statement that concrete aggregates must be "clean" and "free of coatings." NDOT makes this statement for aggregates to be used in asphalt mixtures, but not for aggregates used in concrete mixtures. NDOT requires cleanness values for both coarse and fine aggregate that are similar to the Caltrans requirements. NDOT's allowable LA Abrasion results are regionally the highest at 50 percent.
- NDOT's concrete compressive strength requirement of 4,500 psi at 28 days is consistent with regional practice, although most states do not accept on compressive strength, instead basing acceptance on flexural strength. Most states use a flexural strength of 650 psi at 28 days, with Caltrans being the exception requiring 650 psi at 42 days as long as 570 psi is achieved in 28 days. This is to facilitate acceptance of mixtures containing

higher levels of SCMs as it is known that such mixtures have slower strength gain characteristics even though long-term strength is improved.

- Air content requirements for all states are similar and follow national practice for concrete exposed to freezing and thawing in the presence of deicers.
- NDOT requires a three aggregate blend (No. 4 stone, No. 67 stone, and fine aggregate) for concrete paving in which the Shilstone Coarseness Factor-Workability Factor chart is supposed to be used (note that the cited ACI reference for combining the aggregates has changed over the years). Some other states allow for combined aggregate grading.
- NDOT specifications require that the aggregate be brought to fully saturated condition (at or above SSD) prior to batching and restricts the use of aggregates that show visible separation of water and aggregates during transportation from stockpile to batch plant. None of the regional states require that concrete aggregates be washed prior to batching.
- NDOT specifications for mixing, transporting, placing, finishing, and curing are similar to those of other regional states and national practice.

CHAPTER 4: HISTORICAL REVIEW OF NORTHERN NEVADA CONCRETE PAVING JOBS AND CONSTITUENT MATERIAL PROPERTIES

This chapter presents a historical review of Northern Nevada concrete paving jobs (Class PCCP) constructed over the past 10 years and provides information regarding the properties of the constituent materials and flexural test results of these projects.

4.1 Historical Review of Concrete Paving Jobs

Table 4-1 provides an overview of recent concrete paving jobs constructed in Northern Nevada. Data was drawn from NDOT projects, as well as those completed by local agencies and at the Reno-Tahoe International Airport. The table includes the concrete mixture code for the material used, the NDOT Contract No. (as appropriate), and the specified 28-day design flexural strength.

Table 4-1. Summary of recent Northern Nevada concrete paving projects.

Year	Project	Concrete Mix Code	NDOT Contract No.	28-Day Design Flexural Strength (psi)
2010	I-580 Meadowood	75	3389	550
2011	Reno-Tahoe Int'l Airport – Terminal Apron, Stage 15	8180G		650
2011	I-580 Reno, NV	PC-111	3292	550
2011	I-580 Moana Ln to I-80	7580	3292	650
2011	I-80 Robb to Vista	7580	3401	650
2011	I-80 Design Build Project	G75N	3441DB	550
2012	Washoe County – Moana Lane Widening	86RBN		650
2012	Reno-Tahoe Int'l Airport – Taxiway C Extension	1545598		650
2013	I-80 Carlin Tunnels	3D PCCP	3525	550
2013	Reno-Tahoe Int'l Airport – Terminal Apron, Stage 16	86RBN		650
2013	Reno-Tahoe Int'l Airport – Taxiway Q	46MLN3580		650
2014	Reno-Tahoe Int'l Airport – Runway 16L-34R	84DDF		650
2014	Reno-Tahoe Int'l Airport – Terminal Apron, Stage 17	84DDF		650
2015	Reno-Tahoe Int'l Airport – Taxiway C	85DDF		650

Table 4-2 and 4-3 provide a summary of the mixture sources, material proportions, and design properties for the concrete materials codes tied to the projects shown in Table 4-1. There are a number of important observations that can be made from this data including the following:

- There are only two sources of cement represented in the paving mixtures (Nevada Cement from Fernley, Nevada and Lehigh Redding from Redding, California). Both cements are classified as ASTM C150 Type I-II. In one case, the Nevada Cement was blended by the producer with 25 percent by mass natural pozzolan to create an ASTM C595 Type IP (25). This is representative of the current supply in Northern Nevada.
- There are only two sources of SCMs represented in the data. The first is an ASTM C618 Class F fly ash from the Jim Bridger Plant in Point of Rocks, Wyoming. This fly ash is marketed by Headwaters Resources. The second is an ASTM C618 Class N natural pozzolan provided by Nevada Cement from Fernley, Nevada. The composition of the natural pozzolan has changed over the years, and potentially will continue to change.
- Sources of aggregate varied from quarried igneous rock to water deposited gravels of varying lithology. The basic properties of the aggregate used in concrete mixture design are summarized in Table 4-4. This data shows that the coarse aggregate (No. 4 and No. 67 gradations) SSD specific gravities vary from 2.27 to 2.69 and the absorptions vary from 0.7 to 5.3 percent (note that there is an inverse relationship between specific gravity and absorption). The fine aggregate has similar variability with SSD specific gravities ranging from 2.30 to 2.62 and absorption from 1.01 to 5.7 percent. Interestingly, the fine aggregate absorptions for the Martin Marietta source had considerable variability, ranging from 1.01 to 2.0 percent, being higher than that of the coarse aggregate source. It is understood that this source is a blend of manufactured and natural fine aggregate, which likely contributes to the absorption being both higher and more variable than the absorption for the coarse aggregate. In addition, the fineness modulus for the fine aggregate ranged from 2.90 to 3.04, all of which are near the high end of that allowed by specification (3.1). More detailed discussion of the coarse aggregate classification and lithology is presented in Section 4.2.
- Admixtures are used in all mixtures to entrain air and improve workability. The latter is accomplished through a combination of low-range, mid-range, and even high-range water-reducing admixtures. Some mixtures used multiple water-reducing admixtures, indicating a heavy reliance on admixtures to enhance mixture workability.
- The *w/cm* varied for these mixtures from 0.30 to 0.42. This is a very wide range, and is lower than normal for concrete paving mixtures in much of the country, which typically vary from 0.40 to 0.45 for concrete placed in a freeze-thaw environment.
- The total cementitious materials contents for the mixtures ranged from 705 to 923 lbs/yd^3, with an average of 783 lbs/yd^3 This is high for paving grade concrete.

Table 4-2. Concrete mixture properties for NDOT projects.

Info / Property	Concrete Mixture Code				
	75	**PC-111**	**7580**	**G75N**	**3D PCCP**
Cement Supplier (Type)	Nevada Cement (II)	Nevada Cement (II)	Nevada Cement (II)	Nevada Cement (II)	Nevada Cement (IP(25))
SCM Supplier (Type)	Bridger (Class F)	Bridger (Class F)	Bridger (Class F)	Nevada Cement (Class N)	-
No. 4 Stone Supplier	Sierra Stone	Onsite	Martin Marietta	Martin Marietta	WS-2
No. 67 Stone Supplier	Sierra Stone	Onsite	Martin Marietta	Martin Marietta	WS-2
Sand 1 Supplier	Sierra Stone	Martin Marietta	Martin Marietta	Martin Marietta	WS-2
Sand 2 Supplier	-	-	-	-	-
Admixture Supp.	Grace	BASF	BASF / Grace	BASF / Grace	Euclid
Material Proportions (per cubic yard)					
Cement Wt. (lbs.)	529	688	738	564	705
SCM Wt. (lbs.)	176	192	185	141	0
Total Cementitious (lbs)	705	880	923	705	705
Water (lbs.)	268	364.8[2]	348.3	264	265
w/cm	0.38	0.42	0.38	0.37	0.38
No. 4 Stone (lbs.)	600	1017.2	1097.9	837	786
No. 67 Stone (lbs.)	925	1392.9	1439.6	1100	957
Sand 1 (lbs.)	953	1354.7	1258.8	959	1064
Sand 2 (lbs.)	-	-	-	-	-
Admixtures Supplier (Wt. oz.)	Darex II (9.9)	MicroAir (8.2)	MicroAir (14.7)	MicroAir (11.3)	AEA92 (7.8)
	Daracem 55 (42)	MasterPave Plus (27.4)	Daracem 55 (55.3)	Daracem 55 (56)	Eucon WR91 (28)
	-	-	-	-	Eucon 37 (28)
	-	-	-	-	-
Design Properties					
Slump (in)	1.75	2.5	2.0	1.5	1.5
Slump w/ admix (in)	-	-	-	-	3.5
Total Air (%)	5.0	5.0	5.0	5.5	5.4

[1] Martin Marietta Spanish Springs Pit

[2] Reclaimed water

A summary of the 28-day flexural strengths are presented in Table 4-5. In total, 6.6 percent of the test results obtained during construction fell below the 650 psi specification limit.

In combination, this data is a direct indicator of the difficulty that exists in obtaining even modest flexural strengths with the materials available in Northern Nevada. The cementitious materials contents are very high (averaging 783 lbs/yd^3) and the *w/cm* is low (average of 0.354) compared to national standards for paving concrete. These high cementitious materials contents and low *w/cm* require the use one or more water-reducers, often of mid-range or high-range. In addition, the higher paste content adds costs, contributes to increased drying shrinkage, and increases alkali loading increasing the risk of alkali-silica reactivity (ASTM C1778).

Table 4-3. Concrete mixture properties for Non-NDOT projects.

Info / Property	Concrete Mixture Code					
	8180G	**86RBN**	**1545598**	**46MLN3580**	**84DDF**	**85DDF**
Material Sources						
Cement Supplier (Type)	Nevada Cement (II)	Nevada Cement (II)	Nevada Cement (II)	Lehigh Cement (II)	Nevada Cement (II)	Nevada Cement (II)
SCM Supplier (Type)	Bridger (Class F)	Nevada Cement (Class N)	Bridger (Class F)	Nevada Cement (Class N)	Bridger (Class F)	Bridger (Class F)
No. 4 Stone Supplier	Martin Marietta	Rilite Pit	Sierra Stone Quarry	Martin Marietta	Dayton	Dayton
No. 67 Stone Supplier	Martin Marietta	Rilite Pit	Sierra Stone Quarry	Martin Marietta	Dayton	Dayton
Sand 1 Supplier	Martin Marietta	Paiute Pit	Sierra Stone Quarry	Martin Marietta	Dayton (Manufactured)	Dayton (Manufactured)
Sand 2 Supplier	-	Rilite Pit	-	-	Dayton (Natural)	Dayton (Natural)
Admixture Supp.	BASF	Euclid	Grace	BASF	Euclid	Euclid
Material Proportions (per cubic yard)						
Cement Wt. (lbs.)	602	639	639	602	639	599
SCM Wt. (lbs.)	150	160	160	150	160	200
Total Cementitious (lbs.)	752	799	799	752	799	799
Water (lbs.)	267	270	258	257	242	242
w/cm	0.36	0.34	0.32	0.34	0.30	0.30
No. 4 Stone (lbs.)	760	680	500	449	650	650
No. 67 Stone (lbs.)	1100	926	1050	1489	1180	1180
Sand 1 (lbs.)	983	440	876	918	387	383
Sand 2 (lbs.)	-	399	-	-	579	571
Admixtures Supplier (Wt. oz.)	MicroAir (10.5)	AEA 92 (4.8)	Darex II (12.8)	MicroAir (11)	AEA 92 (6.4)	AEA 92 (7.8)
	Polyheed 997 (45)	X15 (72)	WRDA-64 (32)	322-N (23)	X15 (71.9)	X15 (71.9)
	-	37 (24)	Daracem 55 (48)	RHOB1000 (23)	Eucon 37 (24)	Eucon 37 (24)
	-	-	Daracem 19 (32)	-	-	-
Design Properties						
Slump (in)	2	2	2	-	2	2
Slump w/ admix (in)	-	3.75	3.5	-	4	4
Total Air (%)	5.5	5.5	5.5	5.5	5.5	5.5

Table 4-4. Summary of aggregate properties for NDOT projects.

Concrete Mix	Source	Size	SSD Specific Gravity	Absorption (%)	Fineness Modulus
75	Sierra Stone	#4	2.27	4.7	
	Sierra Stone	#67	2.27	5.3	
	Sierra Stone	Fine	2.30	5.7	2.98
PC-11	Site	#4	2.624	1.76	
	Site	#67	2.622	2.37	
	Martin Marietta	Fine	2.621	1.01	3.04
7580	Martin Marietta	#4	2.69	0.7	
		#67	2.69	1.0	
		Fine	2.60	2.0	3.02
G75N	Martin Marietta	#4	2.69	0.7	
		#67	2.69	1.0	
		Fine	2.62	1.8	3.03
3D PCCP	WS-2	#4	2.59	0.9	
		#67	2.58	1.2	
		Fine	2.57	1.6	2.96

Table 4-5. Summary of 28-day concrete flexural strengths.

Project	**Mix Design Code**	**28-Day Flexural Strength (650 psi design, unless noted)**						
		Ave. (psi)	**StDev (psi)**	**Max (psi)**	**Min (psi)**	**No. Tests**	**No. Below Spec.**	**% Below Spec.**
I-580* Meadowood	75**	663	18.35	690	640	6	-	-
RTIA Apron Stage 15	8180G	703	81.34	845	530	42	9	21.4
I-580 Reno*	PC-111**	624	14.5	609	638	3	-	-
I-580 Reno	7580	718	46.79	860	610	79	2	2.5
I-580 Moana Ln to I-80	G75N**	754	66.14	920	650	44	0	0
Moana Lane Widening	86RBN	699	24.27	760	630	156	1	0.6
RTIA Twy C Extension	1545598	698	29.11	765	625	25	1	4.0
I-80 Carlin Tunnels*	3D PCCP	663	13.96	680	650	5	-	-
RTIA Apron Stage 16	86RBN	700	31.52	770	640	68	2	2.9
RTIA Twy Q	46MLN3580	655	38.30	745	550	31	11	35.5
RTIA Apron Stage 17	84DDF	715	28.60	785	645	40	1	2.5
RTIA Rwy 16L-34R	84DDF	717	25.52	800	660	71	0	0.0
RTIA Twy C	85DDF	676	37.40	740	610	46	10	21.7

** Break data from mix design; **550 psi mix (flexural)*

In contrast, concrete mixtures in Southern Nevada have not experienced the same difficulties. An example is presented in Table 4-6, which presents two trial batches prepared for initial submittal for Project NEON. The mixture designated as NDOT 409 represents a concrete mixture that meets NDOT's current concrete specifications, with a three aggregate blend (No. 4, No. 67, and sand), a total cementitious materials content of 615 lbs/yd^3 (exceeding NDOT's minimum cementitious content requirement of 611 lbs/yd^3), and a *w/cm* of 0.42. As an alternative, an optimized concrete mixture was prepared with a four aggregate blend and a reduced total cementitious materials content of 500 lbs/yd^3. Both mixtures exceeded 700 psi flexural strength within 3 days and exceeded 1000 psi flexural strengths by 14 days[1].

For comparison, the strength gain characteristics of these two NEON trial batch mixtures are charted in Figure 4-1 against trial batch results for four NDOT projects constructed in Northern Nevada. Indicated on the chart for each mixture is the total cementitious materials content (CMC, which varied from 705 to 923 lbs/yd^3) and *w/cm* (which ranged from 0.38 to 0.42). The four mixtures shown represented four different coarse aggregate sources (Sierra Stone, a source mined on-site in Reno, Martin Marietta, and WS-2), three sands sources (Sierra Stone, Martin Marietta, and WS-2), one cement type (Nevada Cement), and two pozzolan sources (Bridger Class F fly ash and Nevada Cement Class N natural pozzolan). The flexural strengths at 28 days for all four trial batch mixtures were basically the same, ranging from 624 to 670 psi. These results clearly suggest that the NDOT specification can be used to produce concrete having excellent strength gain characteristics with materials from Southern Nevada. But obtaining flexural strengths comfortably in excess of the required 650 psi at 28 days is difficult with materials readily available in Northern Nevada, even when using high amounts of cementitious materials and *w/cm* below 0.40.

The next section of this report will discuss two of these materials, the cement and aggregates, in more detail.

4.2 Constituent Material Properties

4.2.1 Cements

Table 4-7 summarizes the cement mill certificates reviewed as part of this study. In total, 164 mill certificates were examined from five cement suppliers, two who supply Northern Nevada and three who supply Southern Nevada. Other than the data provided by Nevada Cement, the number of mill certificates available is insufficient to draw trends, but each certificate provides a snapshot of the cement properties representing a month's production. All cements met the relevant ASTM requirements.

Tables 4-8 and 4-9 summarizes some key chemical and physical cement parameters, respectively.

[1] Note that these mixtures were not air entrained, and had 3 to 4 percent less air than mixtures prepared in Northern Nevada. Concrete strength is reduced approximately 5 percent for each 1 percent addition of air.

Table 4-6. Concrete mixture properties for Project NEON trial batch mixtures.

Info / Property	**Concrete Mixture**			
	NDOT 409	**Opt #1**		
Material Sources				
Cement Supplier (Type)	Ash Grove (II/V)	Ash Grove (II/V)		
SCM Supplier (Type)	Navajo (Class F)	Navajo (Class F)	SSD SG	
No. 4 Stone Supplier	Sloan	Sloan	2.757	
No. 67 Stone Supplier	Sloan	Sloan	2.731	
No. 8 Stone Supplier	Sloan	Sloan	2.748	FM
Sand Supplier	Sloan	Sloan	2.776	2.72
Admixture Supplier	Grace	Grace		
Materials Proportions (per cubic yard)				
Cement Wt. (lbs.)	460	400		
SCM Wt. (lbs.)	155	100		
Total Cementitious (lbs.)	615	500		
Water (lbs.)	258	210		
w/cm	0.42	0.42		
No. 4 Stone (lbs.)	1003	536		
No. 67 Stone (lbs.)	1160	1238		
No. 8 Stone (lbs.)	0	533		
Sand (lbs.)	1179	1265		
Admixtures Supplier (Wt. oz.)	WRDA 64 (18.40)	WRDA 64 (16.00)		
Concrete Properties				
Slump (in)	0	2		
Total Air (%)	2.2	1.4		
Design Unit Wt. (lbs./ft^3)	156.1	158.6		
Average Flexural Strengths (psi)				
3 days	730	760		
7 days	822	1002		
14 days	1035	1010		
28 days	1238	1098		

Table 4-7. Summary of data sources for cement suppliers.

Region	**Cement Supplier**	**Cement Type**	**No. of Certs.**	**Dates**
Northern NV	Lehigh Cement	I-II	1	8/16
	Nevada Cement	I-II	59	4/10 to 12/16
		I-II LA	35	1/13 to 12/15
		II-V	4	11/12 to 10/15
		IP	59	6/1 to 11/16
Southern NV	Ash Grove	V	3	6/12 to 4/14
	CalPortland	II-V	1	10/11
	Mitsubishi	II-V	2	10/10 to 10/12

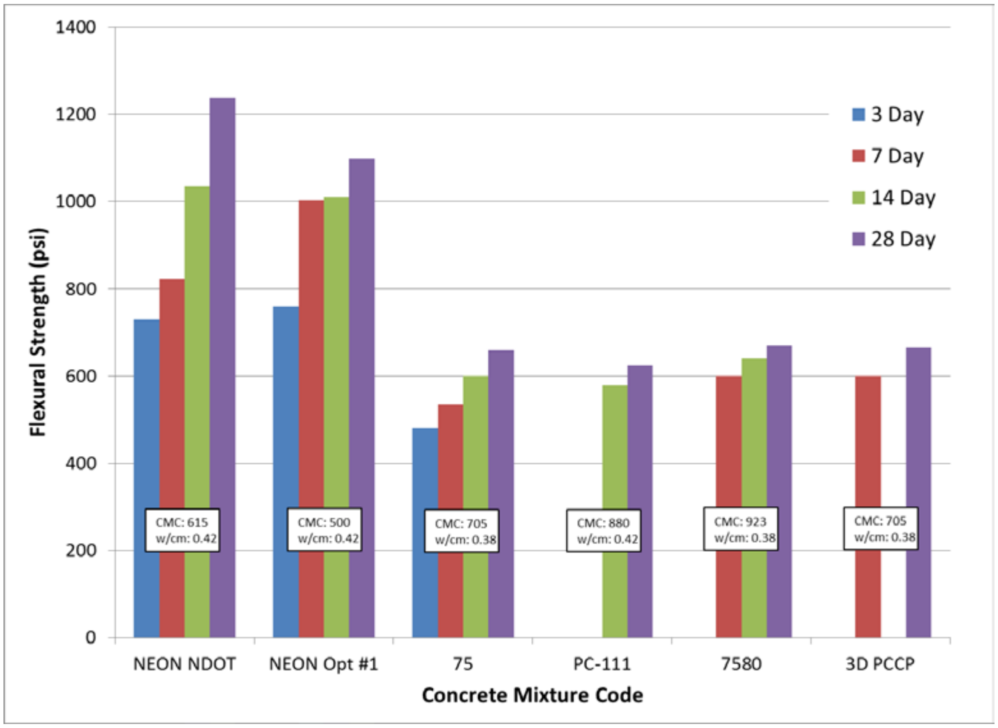

Figure 4-1. Comparison of flexural strength with time for Project NEON trial batch compared to four Northern Nevada NDOT mixtures. Note "CMC" stands for total cementitious materials content in lbs/yd^3.

Table 4-8. Average chemical properties for various cements.

Cement Supplier	Cement Type	Average Values (%)			
		C_3S[1]	C_2S[1]	C_3A[1]	$NaO_{2\,eq}$[2]
Lehigh Cement[3]	I-II	59	13	8	0.50
Nevada Cement	I-II	58.90	13.83	6.73	0.531
	I-II LA	61.49	11.91	6.80	0.482
	II-V	62.5	11.8	4.2	0.468
	IP	Not reported for a Type IP			
Ash Grove	V	56.7	15.3	4.0	0.497
CalPortland[3]	II-V	55	17	3	0.52
Mitsubishi	II-V	54.0	-	3.5	0.445

[1] C_3S: tricalcium silicate; C_2S: dicalcium silicate; C_3A: tricalcium aluminate – estimated in accordance with ASTM C150 using Bogue calculation.

[2] $NaO_{2\,eq}$ is the total alkali content which combines the sodium oxide and potassium oxide into a single equivalent as described in ASTM C150.

[3] Only one mill certificate available for review.

Table 4-9. Average physical properties for various cements.

Cement Supplier	**Cement Type**	**Average Values**				
		Blaine (cm^2/g)	**Setting Time (minutes)**	**Strength (psi)**		
				3 Day	**7 Day**	**28 Day**
Lehigh Cement[1]	I-II	4660	180	3963	4856	6509
Nevada Cement	I-II	3337.7	167.7	3192.2	3989.8	5705[1]
	I-II LA	3757.1	111.5	3612.4	4358.6	-[2]
	II-V	4066.8	127.0	3790.5	4545.8	5647.2
	IP	4717.3	169.3	3007.2	3809.8	5115.2
Ash Grove	V	4276.7	96.7	3622.3	4670.3	-[2]
CalPortland[1]	II-V	4040	150	3658	4710	6100
Mitsubishi	II-V	3815.0	157.0	3267.0	4417.0	6009.5

[1] Only one mill certificate available for review.

[2] No 28 day results reported.

In evaluating the bulk data, it is observed that for a given cement type, the average bulk chemistry (based on Bogue calculated C_3S, C_2S, and C_3A) is similar, but with one difference. This is in the higher estimated C_3S (tricalcium silicate) content with a corresponding decrease in the C_2S (dicalcium silicate) for the Nevada Cement Type II-V compared to that calculated for the other Type II-V cements.

On average, the alkali content of the cements are similar. Alkali content has two major impacts. The first is that higher alkalinity is generally associated with a higher rate of reactivity (although this is not the case with the Nevada Cement as will be discussed later). Higher alkali contents also contribute to ASR by increasing the "alkali loading" for a given cement content as described in ASTM C1778. On average, all of the cements are relatively low in alkalis with sodium equivalents below 0.55 percent (the maximum allowable for low alkali cement is 0.60 percent, although some state set more restrictive limits). But the average results do not tell the full story as is seen in Figure 4-2. This plot shows the alkali contents from all mill certificates, revealing that all remained below 0.55 percent, with the exception of the Nevada Cement Type I-II, which was at or above 0.55 percent about one-quarter of the time. Further, the alkalinity of the Nevada Cement Type I-II varied month to month. With this type of variability, and with the high amount of cement commonly used in Northern Nevada, the alkali loading for the concrete could change significantly over a single construction season.

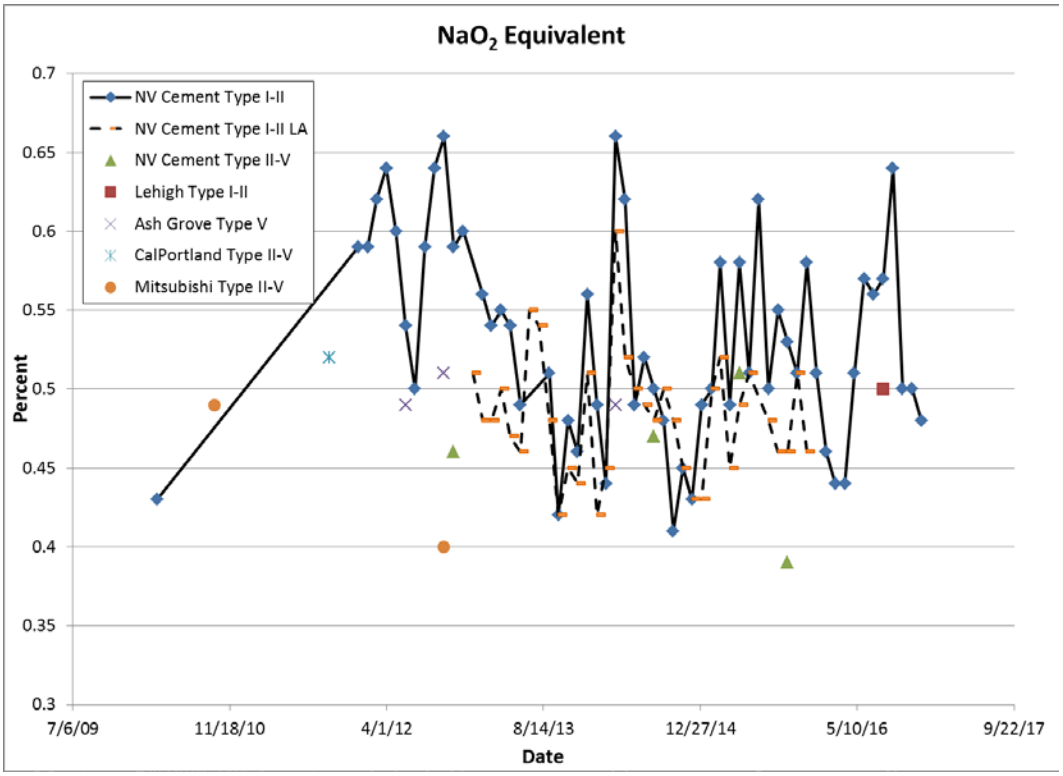

Figure 4-2. Plot of alkali content (in sodium equivalents) for all mill certificates.

With regards to physical properties, the Blaine fineness is a measure of particle surface area and thus is an indirect measure of particle size distribution. The finer the particles, the higher the Blaine fineness. All things equal, increasing Blaine fineness increases cement reactivity as more surface area is exposed to water. But this increase in surface area increases water demand to maintain the same level of workability. The Blaine fineness data in Table 4-9 reveal that the Nevada Cement Type I-II has, on average, the coarsest grind of any cement evaluated, followed by the Nevada Cement Type I-II LA. The Nevada Cement Type IP is the most finely ground cement. Figure 4-3 clearly demonstrates this trend.

The average setting times vary from just under 100 minutes (Ash Grove) to 180 minutes for the one sample of Lehigh. But as seem in Figure 4-10, the sample to sample variability is high with the setting time for Nevada Cement Type I-II ranging from 108 to 238 minutes. The set time for the Nevada Cement Type I-II LA is much lower (111.5 minutes) and more uniform (91 to 139 minutes). It is unknown how Nevada Cement produced the “low alkali” Type I-II LA cement, yet it is clear that its behavior with regards to setting time is significantly different from the Type I-II cement.

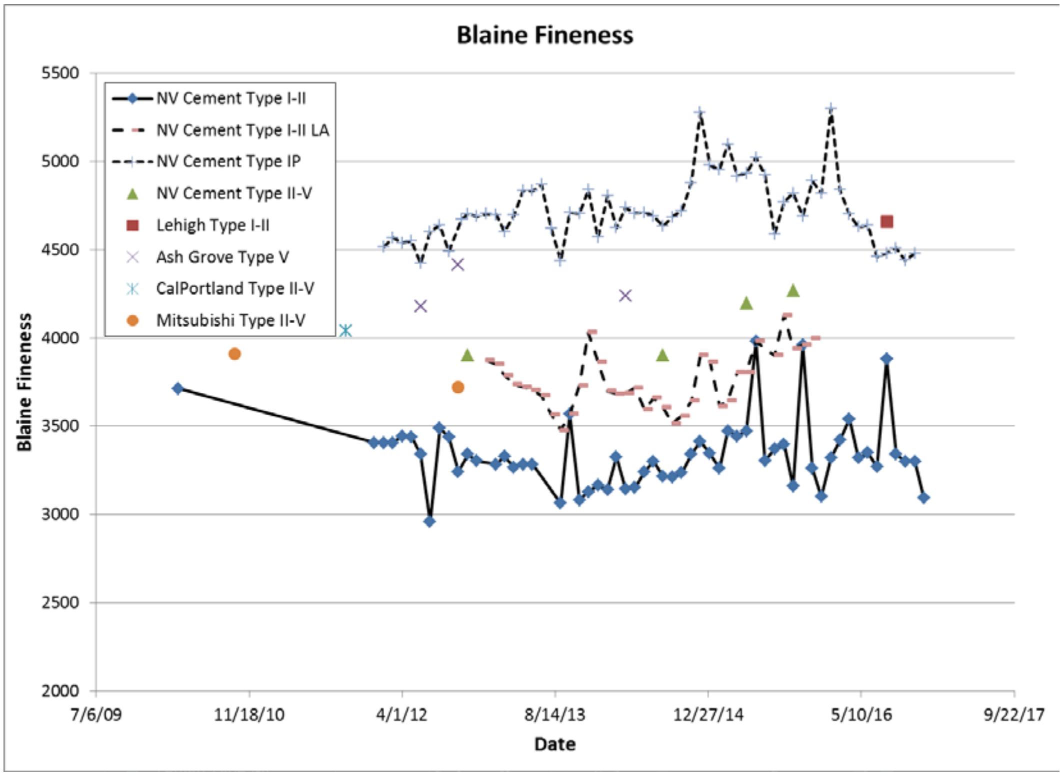

Figure 4-3. Plot of Blaine fineness (cm^2/g) for all mill certificates.

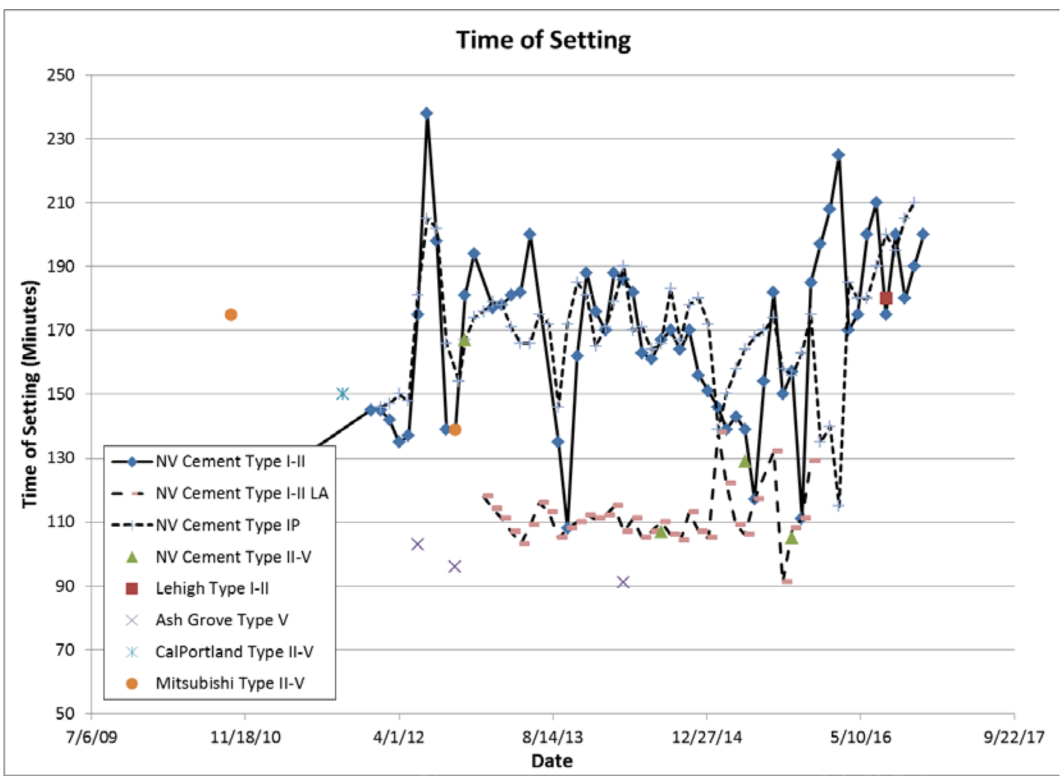

Figure 4-4. Plot of setting time (minutes) for all mill certificates.

The strength data was difficult to assess for cements other than Nevada Cement as the data was not complete. Nevada Cement and CalPortland did not report 1-day strength data, and 28-day strength data was also incomplete, with Nevada Cement only reporting a single 28-day strength test for in Type I-II cement. Examining the 3-day strength data (see Figure 4-5), it is seen that, on average, the Nevada Cement Type IP had the lowest strength of any cement, followed by the Nevada Cement Type I-II. Further, the trend (as shown in Figure 4-5) is that the 3 day strength has been decreasing of the Nevada Cement Type I-II over the last four years.

The same trends are observed in the 7-day strength data, with the four Nevada Cements having the four lowest strengths as seen in Figure 4-6. It can be seen in Figure 4-7 that at 28 days the strengths for the Nevada Cement products continue to increase, although the Type IP is still consistently the lowest strength of all cements evaluated. This is not unexpected as the pozzolanic reaction is known to be slow and strength gain would be anticipated well beyond 28 days.

In closing, an evaluation of cement mill certificates shows that all cements met the relevant ASTM requirements and no systemic issue were identified with the cements used in Northern Nevada compared to Southern Nevada. This analysis is limited as the data evaluate was primarily from a single source (Nevada Cement). Of interest is that the Nevada Cement products have the lowest strengths at 3 and 7 days, but appear to be recovering by 28 days. This suggests that the cement reactivity may be lower, particularly for the Nevada Cement Type IP, than other cements but continued strength gain is likely beyond the assessment period. Of interest is that the strength trends for the Nevada Cement Type I-II and Type I-II LA appear to be decreasing from 2012 through 2016, although the variability in the data makes it impossible to draw definitive conclusions.

4.2.2 Aggregate

A geologic investigation was performed to determine the rock classification and lithologies that make up the No. 67 stone gradation from the following four Nevada aggregate quarries as shown in Figure 4-8:

- Location 1: 3D Concrete WS-2 (Battle Mountain, NV).
- Location 2: 3D Concrete Dayton Pit (Dayton, NV).
- Location 3: Martin Marietta-Spanish Springs Pit (Spanish Springs, NV).
- Location 4: Sierra Stone Pit (Lockwood, NV).

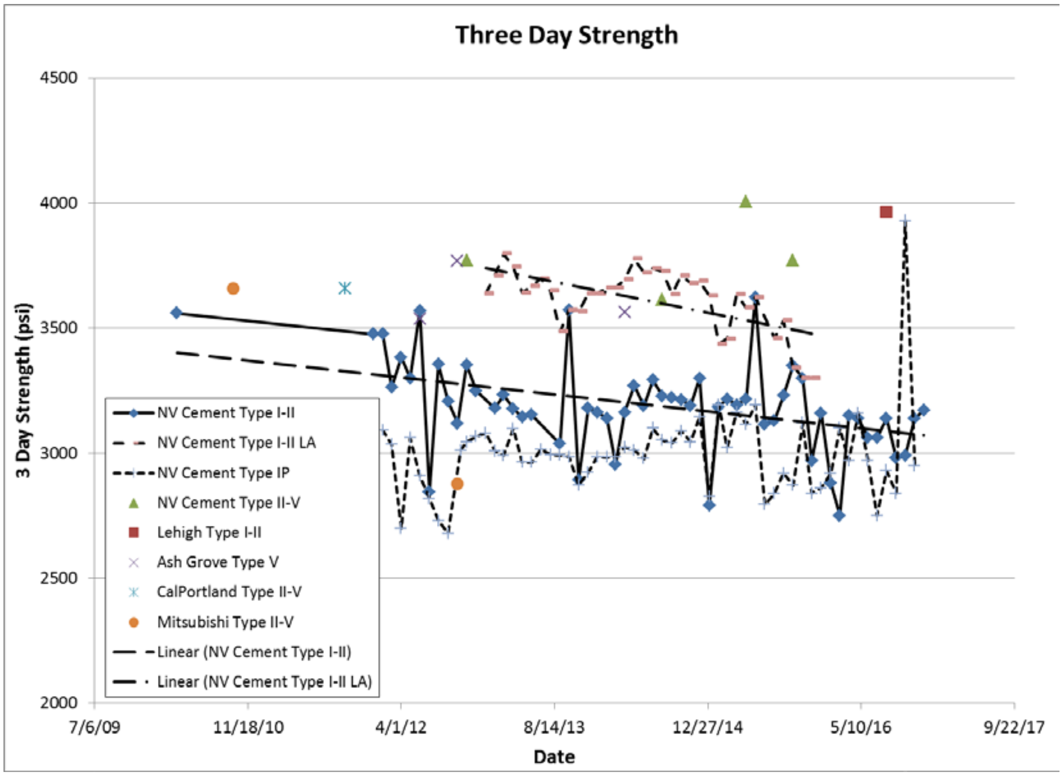

Figure 4-5. Plot of 3-day strengths for all mill certificates.

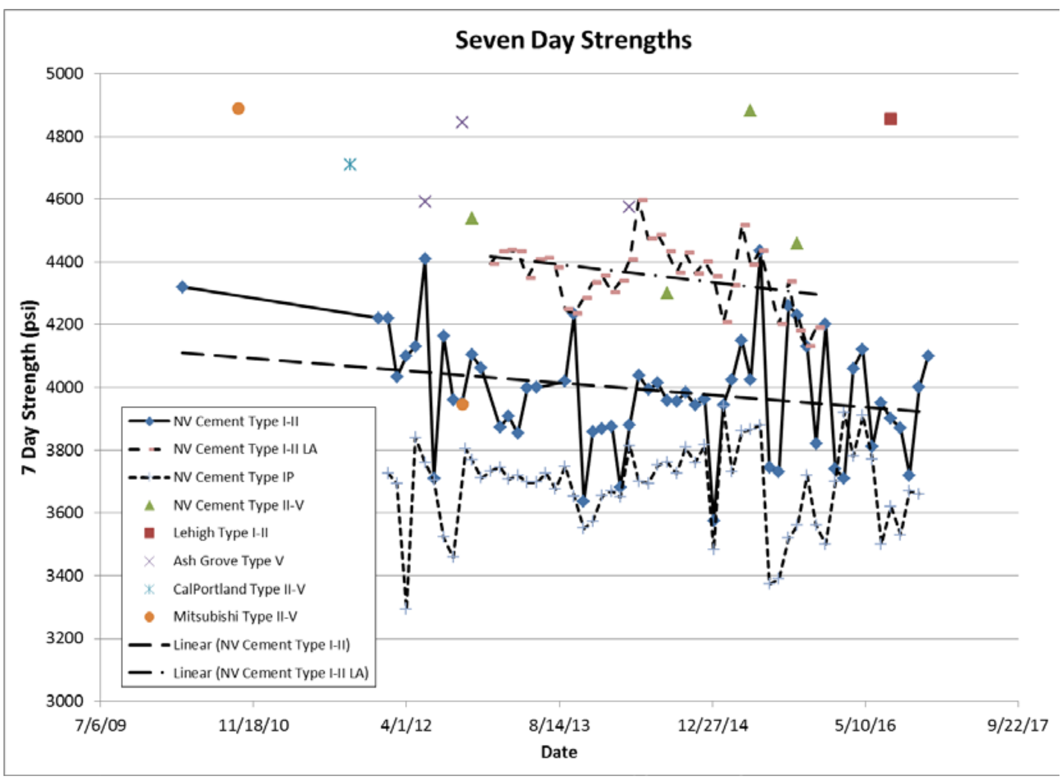

Figure 4-6. Plot of 7-day strengths for all mill certificates.

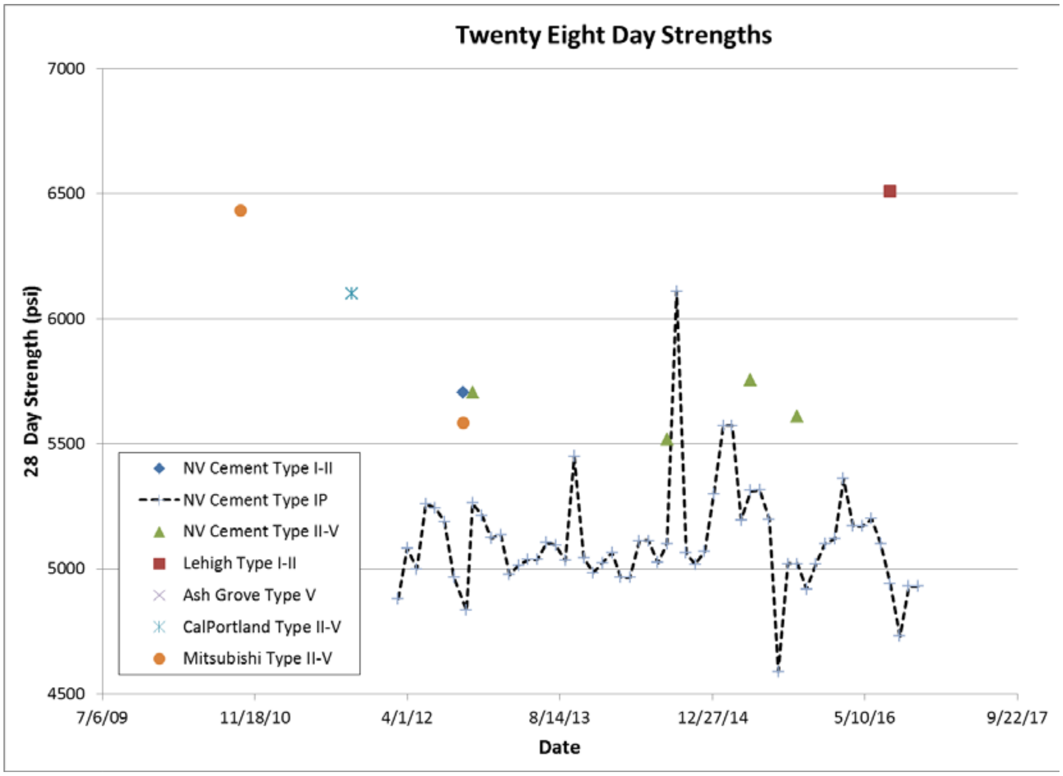

Figure 4-7. Plot of 28-day strengths for all mill certificates.

Figure 4-8. Nevada quarry locations.

The results of the geologic investigation are summarized in Tables 4-10 through 4-13. At Location 1 and Location 2, the aggregate materials were identified as sourced from water deposited gravels and consequently contain mixed lithology-rock types. Location 3 and Location 4 were pit quarry runs from one relatively homogenous geologic bedrock source and were therefore classified as single rock types.

The various constituents of the river deposited aggregates for Location 1 and Location 2 were classified and sorted based on rock type and lithology identification. After separation, the percent of each lithology that made up the sample was determined and the specific gravity and absorption calculated for each lithology type. These data were then compared to specific gravity and absorption test results from bulk aggregate studies at these locations. The summation of the weighted average results for the separated aggregates closely match the bulk sample specific gravity and absorption data determined by the research team from aggregate qualification testing. This exercise was performed to demonstrate how the various rock types from mixed aggregate sources can influence the overall bulk aggregate properties. This further leads to the fact that in water-lain channel deposits, there can be great variability at the point of extraction and the percentage of various rock lithologies that make up the sample for processing.

As anticipated, the specific gravity and absorption data showed that the non-porous, high silica sedimentary aggregates had the lowest absorption, while the volcanic and igneous sourced aggregates introduced higher variability. The specific gravity and absorption data presented (Table 4-12 and Table 4-13) for the single rock types at Location 3 and Location 4 were from aggregate qualification test data compiled previously by the research team. A review of these test results over several years show absorption and specific gravity values have remained relatively consistent.

The mineral content of the various rock types is also summarized by reporting the whole rock chemistry for the identified lithology. The data were compiled to present a generic average for the different rock classifications. Silica contents from the aggregate sources ranged from 65 to over 95 percent SiO_2. The higher average silica contents for the water-deposited aggregates (average of 76.7 percent) reflects their hardness and is a function of their durability when subjected to normal erosion and weathering processes as the harder, high silica aggregates survive water transport and abrasion. In contrast, the pit quarried Sierra Stone rhyolite, being highly siliceous but porous, would not be a common aggregate found in water-deposited gravels due to rapid deterioration during water transport. This again relates back to the physical and chemical properties of the parent rock lithology.

Table 4-10. Rock classification and lithology for 3D Concrete WS-2 (Battle Mountain, NV).

Constituent Material Properties			Date: 9-8-16		
Aggregate Sample Source: 3D Concrete WS-2 #67 stone			Battle Mountain, NV		
Aggregate General Description: Mixed lithology, rounded river gravels-generally smooth surface fragments					
Aggregate Constituents:			**Aggregate Properties:**		
Rock Classification	**Lithology**	**Rock Description**	**Average Chemical Composition***	**Bulk SSD S.G. (1)**	**Percent Absorption**
Sedimentary 45% of sample	Chert	Cryptocrystalline silica (marine floor chemical precipitate)	> 95% silica	2.55	1.3
Sedimentary 42% of sample	Orthoquartzite	Siliceous quartz sandstone	> 90% silica	2.57	1.1
Igneous 13% of sample	Granite	Equigranular medium crystalline with variable amounts of the minerals hornblende, pyroxene, and biotite-muscovite micas	71.5% SiO^2; 14.0 % Al^2O^3; 1.5% Fe^2O^3; 1.4% FeO; 0.6% MgO; 1.6% CaO; 3.4% Na^2O; 4.3% K^2O	2.46	5.0
			Weighted Average	**2.55**	**1.7**
			CME #67 stone aggregate qualification data of single bulk sample	*2.56*	*1.7*

*Average whole rock chemistry for lithology as defined by: Russell B. Travis, Quarterly of the Colorado School of Mines, Volume 50, Number 1.
(1) ASTM C127 Bulk specific gravity and absorption as determined by CME.

Table 4-11. Rock classification and lithology for 3D Concrete Dayton Pit (Dayton, NV).

Constituent Material Properties			Date: 9-8-16		
Aggregate Sample Source: 3D Concrete Dayton Pit #67 stone			Dayton, NV		
Aggregate General Description: Mixed lithology of smooth rounded to rough angular gravels					
Aggregate Constituents:			**Aggregate Properties:**		
Rock Classification	**Lithology**	**Rock Description**	**Average Chemical Composition***	**Bulk SSD S.G. (1)**	**Percent Absorption**
Igneous/Volcanic 38% of sample	Quartz Latite	Microcrystalline groundmass with phenocrysts of feldspar, hornblende, pyroxene, and biotite	66.8% SiO^2; 15.8 % Al^2O^3; 2.3% Fe^2O^3; 1.3% FeO; 1.0% MgO; 2.8% CaO; 3.7% Na^2O; 4.2% K^2O	2.54	3.6
Igneous/Volcanic 32% of sample	Dacite	Microcrystalline to fine crystalline groundmass of quartz and feldspar. Occasional minerals of hornblende and biotite mica	65.3% SiO^2; 16.1% Al^2O^3; 2.1% Fe^2O^3; 2.3% FeO; 1.7% MgO; 3.9% CaO; 3.8% Na^2O; 2.7% K^2O	2.65	1.6
Igneous 30% of sample	Granite	Equigranular medium crystalline with variable amounts of the minerals hornblende, pyroxene, and biotite-muscovite micas	71.5% SiO^2; 14.0 % Al^2O^3; 1.5% Fe^2O^3; 1.4% FeO; 0.6% MgO; 1.6% CaO; 3.4% Na^2O; 4.3% K^2O	2.60	2.0
			Weighted Average	**2.59**	**2.5**
			CME #67 stone aggregate qualification data of single bulk sample	*2.61*	*2.2*

**Average whole rock chemistry for lithology as defined by: Russell B. Travis, Quarterly of the Colorado School of Mines, Volume 50, Number 1.*
(1) ASTM C127 Bulk specific gravity and absorption as determined by CME.

Table 4-12. Rock classification and lithology for Martin Marietta-Spanish Springs Pit (Spanish Springs, NV).

Constituent Material Properties			Date: 9-8-16		
Aggregate Sample Source: Martin Marietta (Spanish Springs Pit) #67 stone			Spanish Springs, NV		
Aggregate General Description: Monolithologic crushed aggregate, angular, rough surface					
Aggregate Constituents:			**Aggregate Properties:**		
Rock Classification	**Lithology**	**Rock Description**	**Average Chemical Composition***	**Bulk SSD S.G. (1)**	**Percent Absorption**
Igneous 100% of sample	Granodiorite	Equigranular medium to coarse crystalline matrix with phenocrysts of hornblende, pyroxene, and biotite mica	65.3% SiO^2; 16.1 % Al^2O^3; 2.1% Fe^2O^3; 2.3% FeO; 1.7% MgO; 3.9% CaO; 3.8% Na^2O; 2.7% K^2O	2.68	0.9

**Average whole rock chemistry for lithology as defined by: Russell B. Travis, Quarterly of the Colorado School of Mines, Volume 50, Number 1*
(1) ASTM C127 Bulk specific gravity and absorption as determined by CME.

Table 4-13. Rock classification and lithology for Sierra Stone Pit (Lockwood, NV).

Constituent Material Properties			Date: 9-8-16		
Aggregate Sample Source: Sierra Stone #67 stone			Lockwood, NV		
Aggregate General Description: Monolithologic crushed aggregate, angular, rough surface					
Aggregate Constituents:			**Aggregate Properties:**		
Rock Classification	**Lithology**	**Rock Description**	**Average Chemical Composition***	**Bulk SSD S.G. (1)**	**Percent Absorption**
Igneous/ Volcanic 100% of sample	Flow Banded Rhyolite	Microcrystalline groundmass with bands and masses of fine cellular pumice and volcanic glass. Occasional phenocrysts of hornblende and biotite mica	71.5% SiO^2; 14.0 % Al^2O^3; 1.5% Fe^2O^3; 1.4% FeO; 0.6% MgO; 1.6% CaO; 3.4% Na^2O; 4.3% K^2O	2.23	5.4

**Average whole rock chemistry for lithology as defined by: Russell B. Travis, Quarterly of the Colorado School of Mines, Volume 50, Number 1*
(1) ASTM C127 Bulk specific gravity and absorption as determined by CME.

Finally, the general aggregate surface condition was visually reviewed and described. The water-deposited aggregates tend to have a smooth surface condition due to water transport and abrasion while the quarry aggregates are a crushed product with angular, rough surfaces. It would seem on face value that the angular, rougher surface of a quarried crushed product would provide a better bonding surface that abraded smooth surface of water-deposited gravel. Yet this has not been borne out in observations of failed flexural strength bean specimens made with the Martin Marietta Spanish Springs Pit quarried aggregate, which based on its physical characteristics (relatively high density and low absorption) should be the high quality aggregate.

As a conclusion to this geologic review, aggregates sourced from homogenous pit run quarries tend to offer the highest predictability for use in concrete mix designs, with the understanding that they may be from a deposit of porous rocks that may be susceptible to abrasion during transport and processing, as well as in use when subjected to studded tires. It is also understood that physical and chemical variances to the rock quality can occur within the sites.

4.3 Chapter 4 Summary

This chapter presented a historical review of Northern Nevada concrete paving jobs (Class PCCP) constructed over the past 10 years, providing information regarding the properties of the constituent materials and flexural test results of these projects. In total, 14 projects were evaluated (six of which were NDOT projects) that used 11 different concrete mixture designs. In addition, cement mill certificates were evaluated over a four year period and detailed analysis was conducted of four aggregate sources. The following observations were made:

- Two suppliers of cement (Nevada Cement and Lehigh Redding) were represented in the concrete mixture. The Lehigh Redding and Nevada Cement portland cements were classified as ASTM C150 Type I-II. In one project, an ASTM C595 Type IP(25) cement was used, being sourced from Nevada Cement. These two cement suppliers are representative of the current supply in Northern Nevada.
- There are only two sources of SCMs represented in the concrete mixture data: an ASTM C618 Class F fly ash from the Jim Bridger Plant in Wyoming and an ASTM C618 Class N natural pozzolan provided by Nevada Cement. These two sources are representative of the current supply in Northern Nevada.
- Sources of aggregate varied from quarried igneous rock to water deposited gravels of varying lithology. The data shows that the coarse aggregate (No. 4 and No. 67 gradations) SSD specific gravities vary from 2.27 to 2.69 and the absorptions vary from 0.7 to 5.3 percent. The fine aggregate has similar variability with SSD specific gravities ranging from 2.30 to 2.62 and absorption from 1.01 to 5.7 percent. In addition, the fineness modulus for the fine aggregate ranged from 2.90 to 3.04, all of which are near the high end of that allowed by specification (3.1). These sources are representative of the current supply in Northern Nevada.
- Admixtures are used in all mixtures to entrain air and improve workability. Some mixtures used multiple water-reducing admixtures, indicating a heavy reliance on admixtures to enhance mixture workability.
- The *w/cm* for the 14 projects varied from 0.30 to 0.42. At the low end, the *w/cm* are much lower than the NDOT maximum specified at 0.45 or what is normally observed for concrete paving mixtures nationwide, which typically vary from 0.40 to 0.45.
- The total cementitious materials contents for the mixtures ranged from 705 to 923 lbs/yd^3, with an average of 783 lbs/yd^3. The NDOT specification limit is 705 lbs/yd^3, and thus all these mixtures either just met or exceeded this limit. These total cementitious materials contents are much higher than what is typical for paving grade concrete used through much of the country.

- An evaluation of the 28-day flexural strengths obtained during construction for these projects revealed that 6.6 percent of the test results for mixtures designed for 650 psi flexural strength at 28 days fell below the specification limit. This is in spite of the high total cementitious materials contents and low *w/cm*.
- A comparison between trial batches prepared using materials from Southern Nevada and trial batches prepared for four of the NDOT projects in Northern Nevada clearly suggested that the NDOT specification can be used to produce concrete having excellent flexural strength gain characteristics with materials from Southern Nevada. But obtaining flexural strengths comfortably in excess of the required 650 psi at 28 days is difficult with materials readily available in Northern Nevada, even when using extreme quantities of cementitious materials and *w/cm* below 0.40.
- In total, 164 cement mill certificates were examined from five cement suppliers, two who supply Northern Nevada and three who supply Southern Nevada. Other than the data provided by Nevada Cement, the number of mill certificates available is insufficient to determine trends. All cements met the relevant ASTM requirements.
- On average, all of the cements would be considered "low alkali" as the average alkali content in sodium equivalents was below 0.65 percent. But the alkali content of the Nevada Cement Type I-II was variable with over one-quarter of the samples have a total alkali content of 0.55 percent or above. This variation occurred month to month, and with the high amount of cement commonly used in Northern Nevada, the alkali loading for the concrete could change significantly over a construction season.
- The Nevada Cement Type I-II has, on average, the coarsest particle distribution of any cement evaluated. This is followed by the Nevada Cement Type I-II LA. The Nevada Cement Type IP has the finest particle distribution of any cement.
- The average setting times for the cements varied from just under 100 minutes (Ash Grove) to 180 minutes for the one sample of Lehigh. But sample to sample variability is high with setting time for Nevada Cement Type I-II ranging from 108 to 238 minutes. The set time for the Nevada Cement Type I-II LA is much lower (111.5 minutes) and more uniform (91 to 139 minutes).
- On average, the Nevada Cement Type IP had the lowest strength at any age up to 28 days of any cement, followed by the Nevada Cement Type I-II. Further, trends indicate that the strength has been decreasing for both Nevada Cement Type IP and Type I-II over the last four years. It is noted that the pozzolanic reaction is known to be slow and strength gain would be anticipated well beyond 28 days for the Type IP cement.

- Four aggregate sources were examined in more detail. Two were identified as being sourced from water deposited gravels and consequently contain mixed lithology-rock types. The other two pit quarry runs from one relatively homogenous geologic bedrock source and were therefore classified as single rock types. These four sources (listed below) are broadly representative of the coarse aggregate available in Northern Nevada:
  - 3D Concrete WS-2: specific gravity 2.55, absorption 1.7 percent.
  - 3D Concrete Dayton Pit: specific gravity 2.59, absorption 2.5 percent.
  - Martin Marietta-Spanish Springs Pit: specific gravity 2.68, absorption 0.9 percent.
  - Sierra Stone Pit: specific gravity 2.23, absorption 5.6 percent.
- Specific gravity and absorption data revealed that the non-porous, high silica sedimentary component of the gravel deposits had the lowest absorption, whereas the volcanic and igneous component introduced higher variability that contributed to the overall variability of the water-lain gravel sources. Aggregates sourced from homogenous pit run quarries tend to offer the highest predictability for use in concrete mix designs, yet for some deposits, the low specific gravity and high porosity may make them susceptible to abrasion during transport and processing, as well as in use when subjected to studded tires. It is also understood that physical and chemical variances to the rock quality can occur within the quarry sites.

CHAPTER 5: PROPOSED PHASE II RESEARCH PLAN

5.1 Introduction

This chapter presents a proposed experimental plan to be implemented in Phase II of this research project. The experimental plan was developed using the findings presented in the preceding chapters to identify experimental variables and develop a testing protocol to determine the cause(s) of the low flexural strength concrete that is ubiquitous in Northern Nevada with the purposed of addressing the problem.

5.2 Constituent Materials

Based on the work presented herein, the following constituent materials are proposed for consideration in the Phase II research.

5.2.1 Coarse Aggregate Types

It is proposed that three coarse aggregates be included in the study as presented in Table 5-1. These broadly represent the coarse aggregates available in Northern Nevada and one source from Southern Nevada that has demonstrated good strength characteristics in concrete. Note that the two Northern Nevada sources have already undergone detailed characterization as described in Section 4.2.2 of this report. A similar characterization will be conducted on the Sloan aggregate source from Southern Nevada. In addition, a detailed chemical and mineralogical analysis will be conducted on the fines coating the aggregate.

Table 5-1. Proposed coarse aggregate sources for Phase II.

Coarse Aggregate ID	Source	SSD Specific Gravity	Absorption (percent)
CA-MM	Martin Marietta	2.68	0.9
CA-SS	Sierra Stone	2.23	5.6
CA-SLV	Sloan	~2.75	unknown

5.2.2 Fine Aggregate Types

It is proposed that three fine aggregates be included in the study as presented in Table 5-2. Two of these sources broadly represent the fine aggregates available in Northern Nevada and one is a Southern Nevada source that has demonstrated good performance. Detailed characterization will be conducted on all three including a chemical and mineralogical analysis of the fines.

Table 5-2. Proposed fine aggregate sources for Phase II.

Fine Aggregate ID	Source	SSD Specific Gravity	Absorption (percent)	Fineness Modulus
FA-MM	Martin Marietta	2.60 to 2.62	1.01 to 2.00	3.01 to 3.04
FA-SS	Sierra Stone	2.30	5.7	2.98
FA-SLV	Sloan	2.776	unknown	2.72

5.2.3 Cement Types

It is proposed that two cement types be included in the study as presented in Table 5-3. One of these sources represents the cement most commonly used in Northern Nevada and one is a Southern Nevada source that has demonstrated good strength gain characteristics. Detailed characterization will be conducted on all three sources completing the same testing as reported on the ASTM C150 mill certificates. In addition, XRD will be done to characterize the mineralogy and SEM conducted to establish angularity and other microscopic parameters.

Table 5-3. Proposed cement sources for Phase II.

Cement ID	Source	Classification
PC-NC	Nevada Cement	Type I-II
PC-AG	Ash Grove	Type II-5

5.2.4 Supplementary Cementitious Material Types

It is proposed that two SCM types be included in the study as presented in Table 5-4. One of these sources represents an SCM often used in Northern Nevada and one is a Southern Nevada source that has demonstrated good performance. Detailed characterization will be conducted on the two sources completing the same testing as reported on the mill certificates for ASTM C618. In addition, XRD will be done to characterize the mineralogy and SEM conducted to establish angularity and other microscopic parameters.

Table 5-4. Proposed SCM sources for Phase II.

SCM ID	Source	Classification
SCM-NP	Nevada Cement	Class N
SCM-NF	Navajo	Class F

5.2.5 Water

It is proposed that most mixtures be prepared with municipal water as is customary. But limited work will be completed using reclaimed water as this is not uncommon in Northern Nevada. Both water sources will be chemically analyzed. Further, mortar specimens will be made with each will each cement source to determine what, if any, impact the water source has on strength development. If an impact is noted, additional work will be completed to fully characterize the impact of reclaimed water on the concrete.

5.2.6 Admixtures

It is assumed that two admixtures will be used: an air-entraining admixture and a low-range to mid-range water-reducing admixture (not polycarboxylate-based). Both admixtures will be from a single producer to minimize the risk of incompatibility.

5.3 Mixture Proportions

Mixture proportions will be established to assess the impact on mixture strength and other fresh and hardened concrete properties. Mixture proportioning variables to be investigated include aggregate grading, total cementitious materials content, and *w/cm*. Note that aggregate volume will have to vary as paste volume changes due to changes in total cementitious materials content and *w/cm*.

5.3.1 Total Cementitious Materials Content

The total cementitious materials content will have two levels: 611 lbs/yd^3 (TCM-L), and 752 lbs/yd^3 (TCM-H). These three levels are just within and in excess of NDOT's current specification limits.

5.3.2 Water-to-Cementitious Materials Ratio

Two levels of *w/cm* will be investigated: 0.38 (WCM-L) and 0.45 (WCM-H). These represent a typical low *w/cm* used in Northern Nevada and the upper limit of NDOT's specification.

5.3.3 Other Mixture Proportions

The aggregate grading will meet the NDOT PCCP requirements using No.4 stone, No. 67 stone, and fine aggregate. Per the NDOT 2014 specification, the aggregate grading will use the guidance presented in ACI 302.1R-04, Section 5.4.3 *Combined Aggregate Grading* (will look at the updated version in ACI 302.1R-15 Section 8.9.2), blending the aggregates so that the combined grading lies within Zone II of the coarseness factor-workability factor chart.

Admixtures will be used in dosages needed to achieve slumps of 1.5 ± 0.5 inches and total air content of 5.5 ± 1.0 percent.

SCM contents will be based on SCM type. For ASTM C618 Class F and N materials, a 20 percent replacement by mass of cement will be used. If ASTM C989 slag cement is also investigated, a 35 percent replacement by mass of cement will be used.

5.4 Batching

Batching will be done consistently for all mixtures with two exceptions. The first is whether the aggregates are batched as delivered in an unwashed state (WA-UW) or batched with all aggregates being washed (WA-W).

The second variable will be the moisture condition of the coarse aggregate. The first condition is all the aggregates are batched at SSD (MC-SSD). The second is that the coarse aggregates are batched dry of SSD, being allowed to equilibrate to laboratory ambient conditions (MC-DRY). In this latter case, the moisture content of the aggregates will be determined prior to batching and additional mix water added to meet the target *w/cm*.

5.5 Mixture Testing

Both the fresh and hardened properties of the concrete will be measured as described below.

5.5.1 Fresh Concrete Testing

The fresh concrete properties of interest include those typically used to monitor concrete quality during construction, and include:

- Workability – Workability will be assessed by two methods. The first is the slump test (ASTM C143) which is required by NDOT specifications. The second will be the “Box Test” described in Appendix X3 in AASHTO PP 84-17, which provides a measure of a concrete’s ability to be consolidated under vibration yet maintain a vertical edge once vibration ceases.
- Air Content – The total air content in fresh concrete will be determined by the pressure meter (ASTM C231). Optionally, air content can be determined by the sequential pressure method (AASHTO TP 118) which will provide not only the total air content in the fresh concrete but also a SAM Number which has been correlated to the ability of the air-void system to protect the paste against freeze-thaw damage.
- Unit Weight – The unit weight will be measured in accordance with ASTM C138.
- Semi-Adiabatic Calorimetry – As an optional test, it is recommended that semi-adiabatic calorimetry be conducted on all fresh concrete to determine the heat signature generated from each mixture as a result of the exothermic hydration reactions. The type of semi-adiabatic calorimeter recommended would be similar to the Calmetrix series (https://www.calmetrix.com/f-cal-calorimeters). This testing would provide valuable information on the rate of hydration for the various mixtures.

5.5.2 Hardened Concrete

The hardened concrete properties of direct and indirect interest include:

- Compressive Strength – Compressive strength will be determined in accordance with ASTM C39 at 3, 7, 28 and 56 days.
- Flexural Strength – Third point flexural strength will be determined in accordance with ASTM C78 at 3, 7, 28 and 56 days.
- Surface Resistivity – Surface resistivity will be determined in accordance with AASHTO TP 119 on all cylindrical specimens tested for compressive strength and thus data will be collected at demolding, 3 days, 7 days, 28 days, and 56 days.

- Sorptivity – Sorptivitity will be assessed in accordance with ASTM C1585.
- Optical Microscopy – Optical microscopy will be conducted in accordance with ASTM C856 to observe the general condition and uniformity of the concrete microstructure. Further, optical microscopy will be used to evaluate fractured faces of flexural strength beams to investigate low strength breaks. Of particular interest will be the interfaces at aggregates to determine whether aggregate coatings and/or air void clustering is interfering with paste-aggregate bonding.
- Scanning Electron Microscopy/Energy Dispersive Spectrometry – SEM/EDS will be conducted in accordance with ASTM C1723 to supplement optical microscopy. The higher resolution of the SEM will allow detail assessment of the interfaces between the aggregate and paste, as well as being useful in generally categorizing the density of the hydrated cement paste. Further, the ability to conduct spot chemical analysis using EDS will be helpful in identify observed mineral phases that might be present at the paste-aggregate interface or elsewhere in the hydrated paste microstructure.
- Nanoindentation – As an optional test, nanoindentation may prove useful to assess variations within the strength of the hydrated cement microstructure to identify specific areas of weakness.
- Small-Angle Scattering (SANS and SAXS) – As an optional procedure, SANS and SAXS can be used to determine the size distributions and volume fractions of microstructural features within the microstructure and may prove useful in better characterizing the nature of the paste-aggregate interface.

5.6 Experimental Plan

Table 5-5 summarizes the variables that could be considered in Phase II of this study. If all variables were to be tested in all mixtures, this would amount to 576 mixture combinations, which is well beyond the realm of possibility. Instead it is proposed that the study be conducted in a series of smaller experiments, each of which will reduce the number of variables until a manageable number is obtained.

5.6.1 Initial Concrete Experiment

Table 5-6 presents the full matrix with the assumption that the aggregates are batched unwashed and at SSD (this eliminates the batching variables for the initial phase of the study). The green cells represent the trial batch mixture prepared for Project NEON, which is expected to exceed design strength even after being air entrained. The orange cells represent typical Northern Nevada mixtures that have had a very difficult time meeting the design flexural strength of 650 psi at 28 days. It is anticipated to never produce mixtures represented in cells with an “X” through them as these mixtures have either never performed or have no economic advantage.

Table 5-5. Summary of variables in Phase II study.

Variable	Level	Source	Value/Comment
Coarse Aggregate (CA)	MM	Martin Marietta	
	SS	Sierra Stone	
	SLV	Sloan	
Fine Aggregate (FA)	MM	Martin Marietta	
	SS	Sierra Stone	
	SLV	Sloan	
Portland Cement (PC)	NC	Nevada Cement	Type I-II
	AG	Ash Grove	Type II-5
Supplementary Cementitious Materials (SCM)	NP	Nevada Cement	Class N
	NF	Navajo or Bridger	Class F
Total Cementitious Materials Content (TCM)	L		611 lbs/yd^3
	H		752 lbs/yd^3
Water-to-Cementitious Ratio (WCM)	L		0.38
	H		0.45
Washed Aggregate (WA)	UW		Unwashed
	W		Washed
Moisture Content (MC)	SSD		Aggregates at SSD
	Dry		Aggregates dry of SSD

Table 5-6. Experimental matrix with recommendations for 16 initial mixtures.

		Nevada Cement								Ash Grove								Cement Type
		Class N				Class F				Class N				Class F				SCM Type
Coarse	Fine	611		752		611		752		611		752		611		752		TCM
Aggregate	Aggregate	0.38	0.45	0.38	0.45	0.38	0.45	0.38	0.45	0.38	0.45	0.38	0.45	0.38	0.45	0.38	0.45	WCM
MM	MM																	
	SS																	
	SLV																	
SS	MM																	
	SS																	
	SLV																	
SLV	MM																	
	SS																	
	SLV																	

For this experiment, it is proposed that the mixtures represented by the six green and orange cells be produced to verify that they will perform as expected. If so, the eight mixtures represented in the yellow cells will be produced and tested to determine:

- For mixtures made with Nevada Cement Type I-II, whether changing the sand from the MM to the SLV improves performance.
- For mixtures made with Nevada Cement Type I-II, whether changing both the coarse aggregate and sand to SLV improves performance.
- For mixtures made with Ash Grove Type II-V, whether changing from SLV aggregate to SS aggregate changes performance.
- For mixtures made with Ash Grove Type II-V, whether changing from SLV aggregate to MM aggregate changes performance.

The results of testing conducted on these14 mixtures will guide the second phase of work which will be designed to answer the questions raised by the initial work. For example, if it is found that changing the cement source from Nevada Cement to Ash Grove results in a significant increase in flexural strength, additional testing will be conducted to determine why, and whether mixtures can be with lower cement contents and higher *w/cm* using Northern Nevada aggregates.

If on the other hand it is found that changing aggregate sources from those available in Northern Nevada to those available in the south results in significant improvements, additional research will be conducted to determine what property of the Northern Nevada aggregates are responsible for the poorer performance. One possibility is coatings. Another is air void clustering. Each of these can be evaluated in a follow up study or studies.

Although it is impossible to know which direction the laboratory experiment will take, it is safe to assume that another 14 to 20 mixtures will need to be prepared and tested to identify the full extent of the problem. Appendix A provides decision trees outlining a potential path to execute an iterative approach for conducting this experiment. This approach has the advantage of systematically collecting data through a series of experiments that are informed by the work conducted previously. This is potentially a more cost effective approach but will require more time as well as more direct management by NDOT and more flexibility in funding the work as it progresses. Alternatively, Phase II work can be funded in its entirety as described in the experimental matrix provided in Table 5-6. This will allow the issuance of a single, fixed cost contract, which will simplify the management of the effort. Regardless of the approach taken, the results of this Phase II experimental study will provide the knowledge needed to develop cost effective solutions to improve the performance of concrete paving mixtures in Northern Nevada.

CHAPTER 6: SUMMARY

Production of high flexural strength paving concrete in Northern Nevada is universally acknowledged to be difficult; however understanding why this is true remains elusive. Current practice is to meet flexural strength requirements by using mixtures with high cementitious materials contents and low *w/cm*, which can negatively impact cost, workability, shrinkage, and durability.

Research examining the occurrence of low concrete strength, particularly flexural strength, has focused on the interface between the coarse aggregate and the HCP. In particular, problems have been associated with the characteristics (porosity and/or hydration products) of the aggregate-HCP interface or with bonding of the aggregate to the HCP due to aggregate surface coatings. Issues with air bubble clustering at the interface and clay coatings that are strongly bonded to the aggregate surface can prevent the HCP from directly adhering to the aggregate.

There are multiple test methods that can be employed to evaluate the interface between aggregates and HCP, the most common of which feature optical petrography and SEM/EDS examinations of prepared concrete specimens. Nanoindentation also shows promise as a technique to characterize the ITZ and bulk paste.

A synthesis of specifications for concrete paving mixtures and practices used for projects in Nevada and the surrounding regional states including California, Idaho, Oregon, Utah, and Wyoming revealed many similarities. In general, NDOT’s practices are consistent with those of other states, although the requirements for minimum cementitious materials content is higher than most.

A historical review of Northern Nevada concrete paving projects (Class PCCP) constructed over the past 10 years provided information regarding the properties of the constituent materials and flexural test results. In total, 14 projects were evaluated (six of which were NDOT projects) that used 11 different concrete mixture designs. It was found that there are limited suppliers of cement and fly ash/pozzolan in Northern Nevada that the total cementitious contents of concrete mixtures used for paving were relatively high, and that the *w/cm* was considered low compared to national practice. Yet difficulties were still encountered in meeting the minimum flexural strength requirements. These difficulties were not noted in mixtures prepared for Southern Nevada, even when considering that these mixtures were not air-entrained.

It is recommended that a Phase II experimental program be initiated to evaluate the effects of aggregate, cement, and pozzolan properties on concrete strength. An experimental matrix for the initial stages of the study has been proposed, as have decision trees that outline an iterative process for conducting the study. It is believed that the execution of this experimental plan will provide NDOT with cause of the poor flexural strength performance of paving grade concrete in Northern Nevada, leading to a path forward to cost effectively improve performance.

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APPENDIX A: DECISION TREES FOR CONDUCTING PHASE II EXPERIMENT

Key to flowcharts

Coarse Aggregate	CA
Fine Aggregate	FA
Cement Producer	Cem
Supplementary Cementious Materal	SCM
Total cementitious content (lbs per yard	TC
Water cement ratio	wcm
Air Content	AC

SLV = Sloan (Las Vegas)
MM = Martin Marietta (Spanish Springs)
SS = Sierra Stone (Reno)

AG = Ash Grove
NC =Nevada Cement

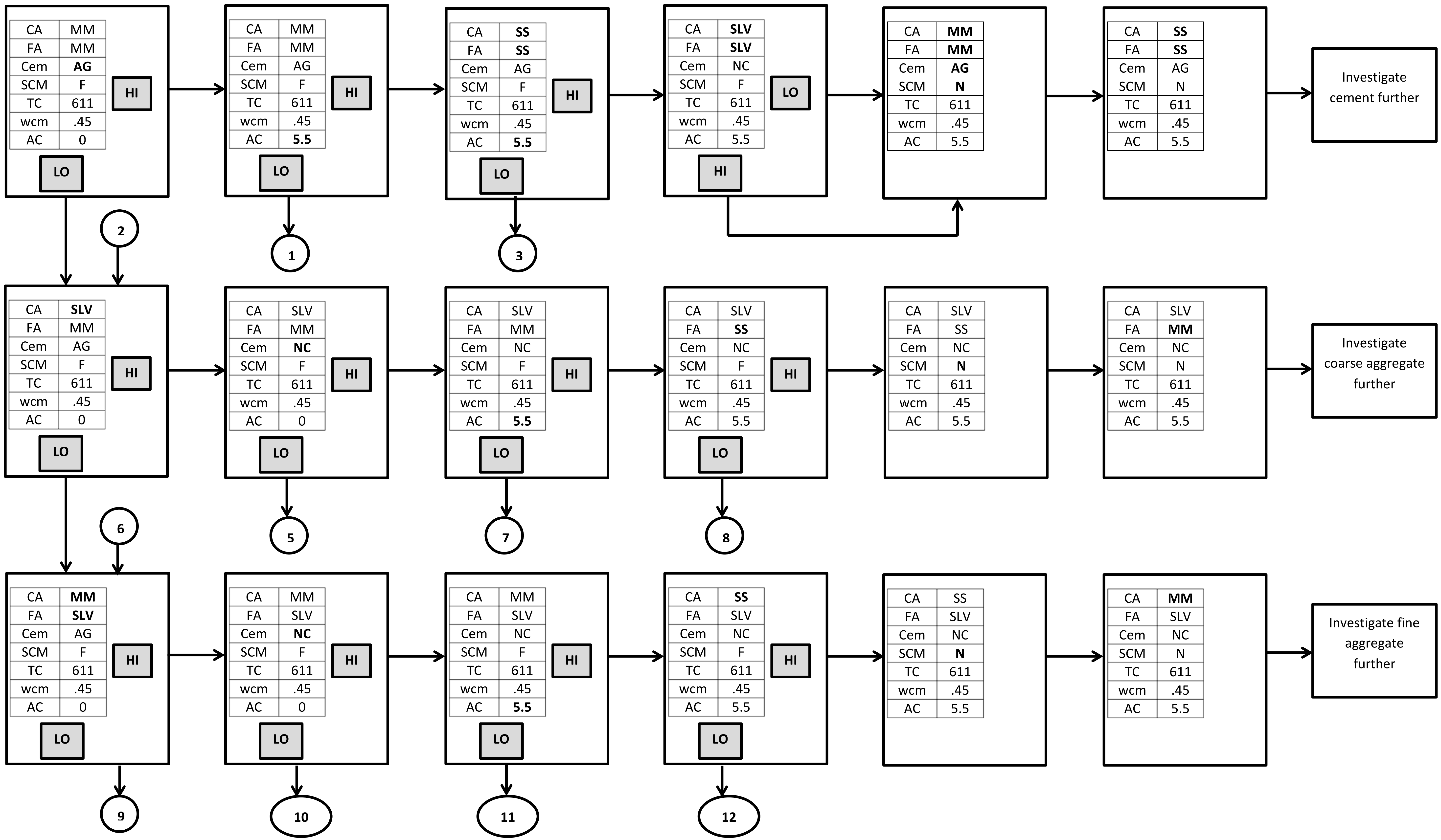
F = Bridger Type F Ash
N = Nevada Cement Natural Pozzolan

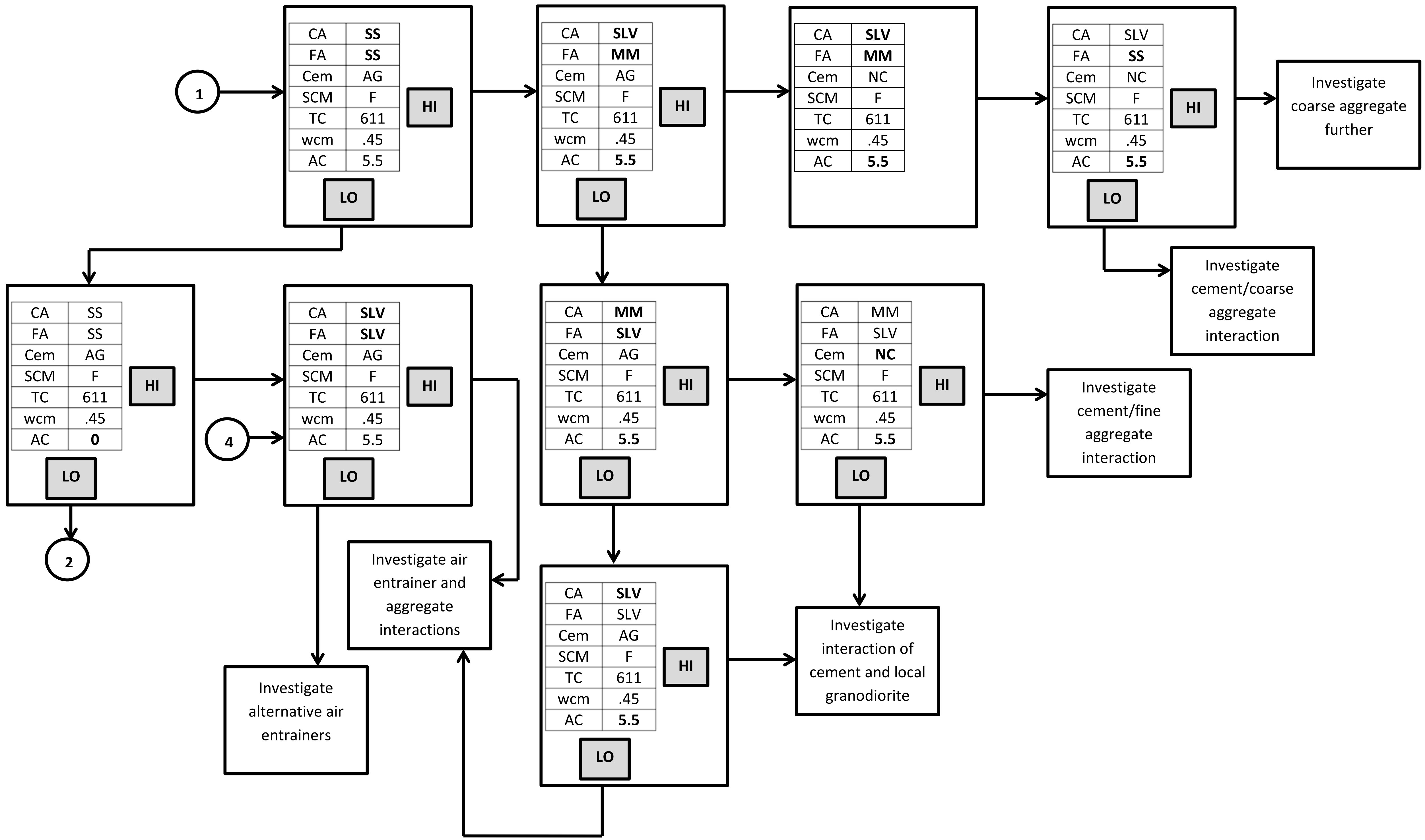
HI

> 700 PSI Flexural Strength

LO

< 700 PSI Flexural Strength


CA MM FA MM Cem AG SCM F TC 611 wcm .45 AC 0
HI
LO
CA MM FA MM Cem AG SCM F TC 611 wcm .45 AC 5.5
HI
LO
CA SS FA SS Cem AG SCM F TC 611 wcm .45 AC 5.5
HI
LO
CA SLV FA SLV Cem NC SCM F TC 611 wcm .45 AC 5.5
LO
HI
CA MM FA MM Cem AG SCM N TC 611 wcm .45 AC 5.5
CA SS FA SS Cem AG SCM N TC 611 wcm .45 AC 5.5
Investigate cement further
2
1
3
CA SLV FA MM Cem AG SCM F TC 611 wcm .45 AC 0
HI
LO
CA SLV FA MM Cem NC SCM F TC 611 wcm .45 AC 0
HI
LO
CA SLV FA MM Cem NC SCM F TC 611 wcm .45 AC 5.5
HI
LO
CA SLV FA SS Cem NC SCM F TC 611 wcm .45 AC 5.5
HI
LO
CA SLV FA SS Cem NC SCM N TC 611 wcm .45 AC 5.5
CA SLV FA MM Cem NC SCM N TC 611 wcm .45 AC 5.5
Investigate coarse aggregate further
6
5
7
8
CA MM FA SLV Cem AG SCM F TC 611 wcm .45 AC 0
HI
LO
CA MM FA SLV Cem NC SCM F TC 611 wcm .45 AC 0
HI
LO
CA MM FA SLV Cem NC SCM F TC 611 wcm .45 AC 5.5
HI
LO
CA SS FA SLV Cem NC SCM F TC 611 wcm .45 AC 5.5
HI
LO
CA SS FA SLV Cem NC SCM N TC 611 wcm .45 AC 5.5
CA MM FA SLV Cem NC SCM N TC 611 wcm .45 AC 5.5
Investigate fine aggregate further
9
10
11
12



1
CA SS
FA SS
Cem AG
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
CA SLV
FA MM
Cem AG
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
CA SLV
FA MM
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
CA SLV
FA SS
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate coarse aggregate further
Investigate cement/coarse aggregate interaction
CA SS
FA SS
Cem AG
SCM F
TC 611
wcm .45
AC 0
HI
LO
2
4
CA SLV
FA SLV
Cem AG
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
CA MM
FA SLV
Cem AG
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
CA MM
FA SLV
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate cement/fine aggregate interaction
Investigate air entrainer and aggregate interactions
CA SLV
FA SLV
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SCM F
TC 611
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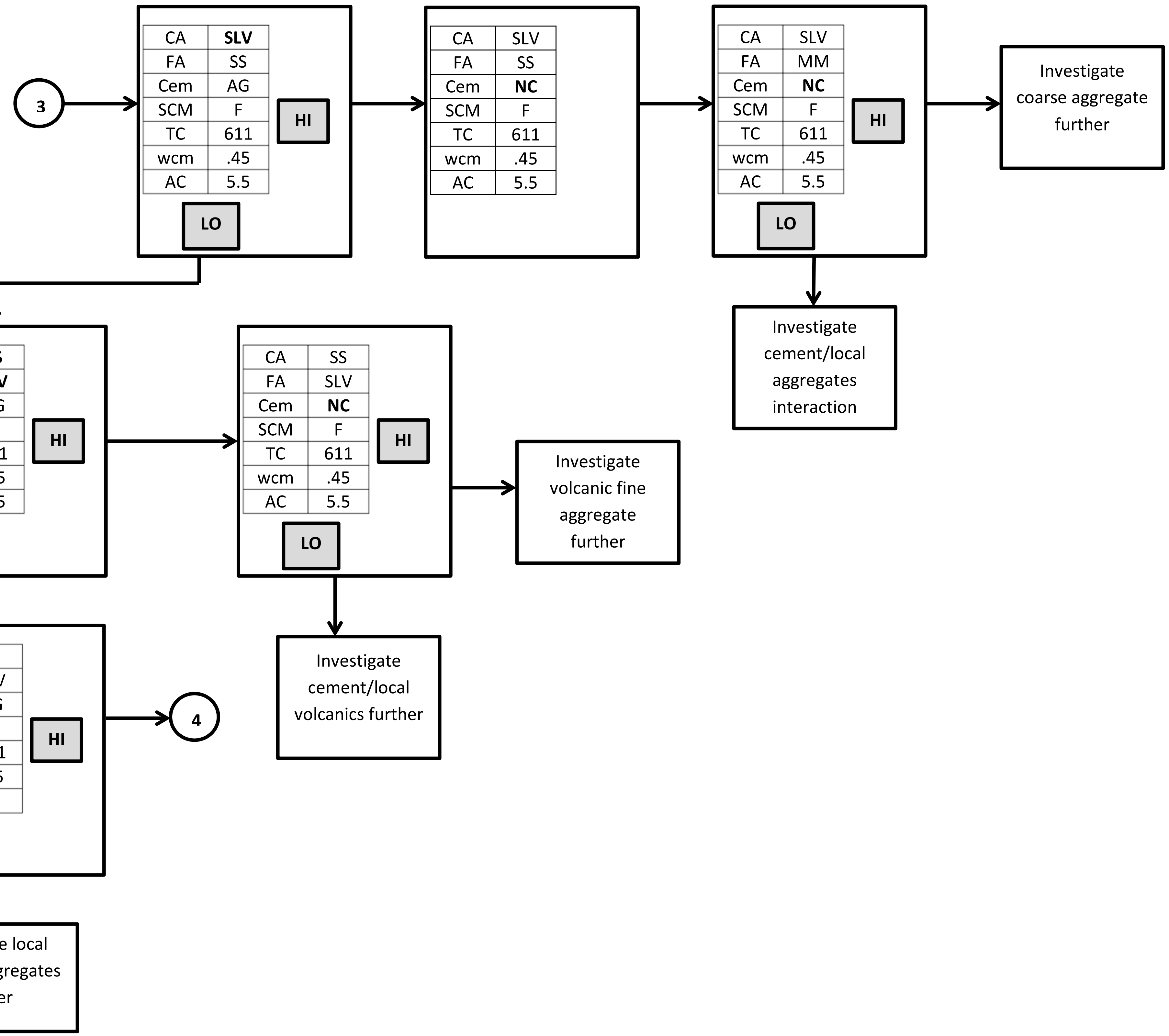
HI
LO
Investigate interaction of cement and local granodiorite
Investigate alternative air entrainers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HI
LO
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Cem NC
SCM F
TC 611
wcm .45
AC 5.5
CA SLV
FA MM
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate coarse aggregate further
Investigate cement/local aggregates interaction
CA SS
FA SLV
Cem AG
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
CA SS
FA SLV
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate volcanic fine aggregate further
Investigate cement/local volcanics further
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Cem AG
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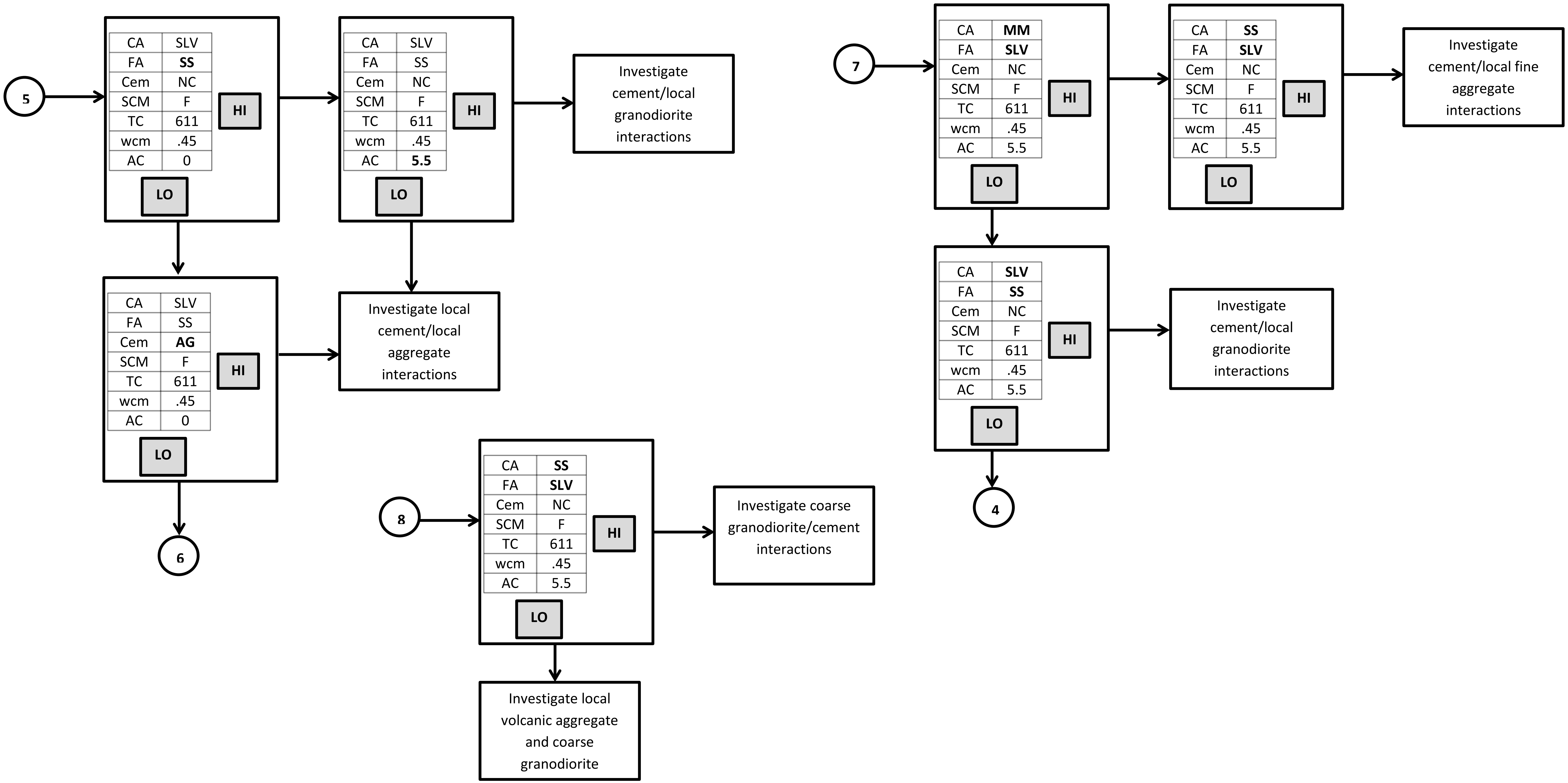
AC 0
HI
LO
4
Investigate local volcanic aggregates further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Cem NC
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AC 0
HI
LO
CA SLV
FA SS
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TC 611
wcm .45
AC 5.5
HI
LO
Investigate cement/local granodiorite interactions
CA SLV
FA SS
Cem AG
SCM F
TC 611
wcm .45
AC 0
HI
LO
6
Investigate local cement/local aggregate interactions
8
CA SS
FA SLV
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate coarse granodiorite/cement interactions
Investigate local volcanic aggregate and coarse granodiorite
7
CA MM
FA SLV
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
CA SS
FA SLV
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate cement/local fine aggregate interactions
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FA SS
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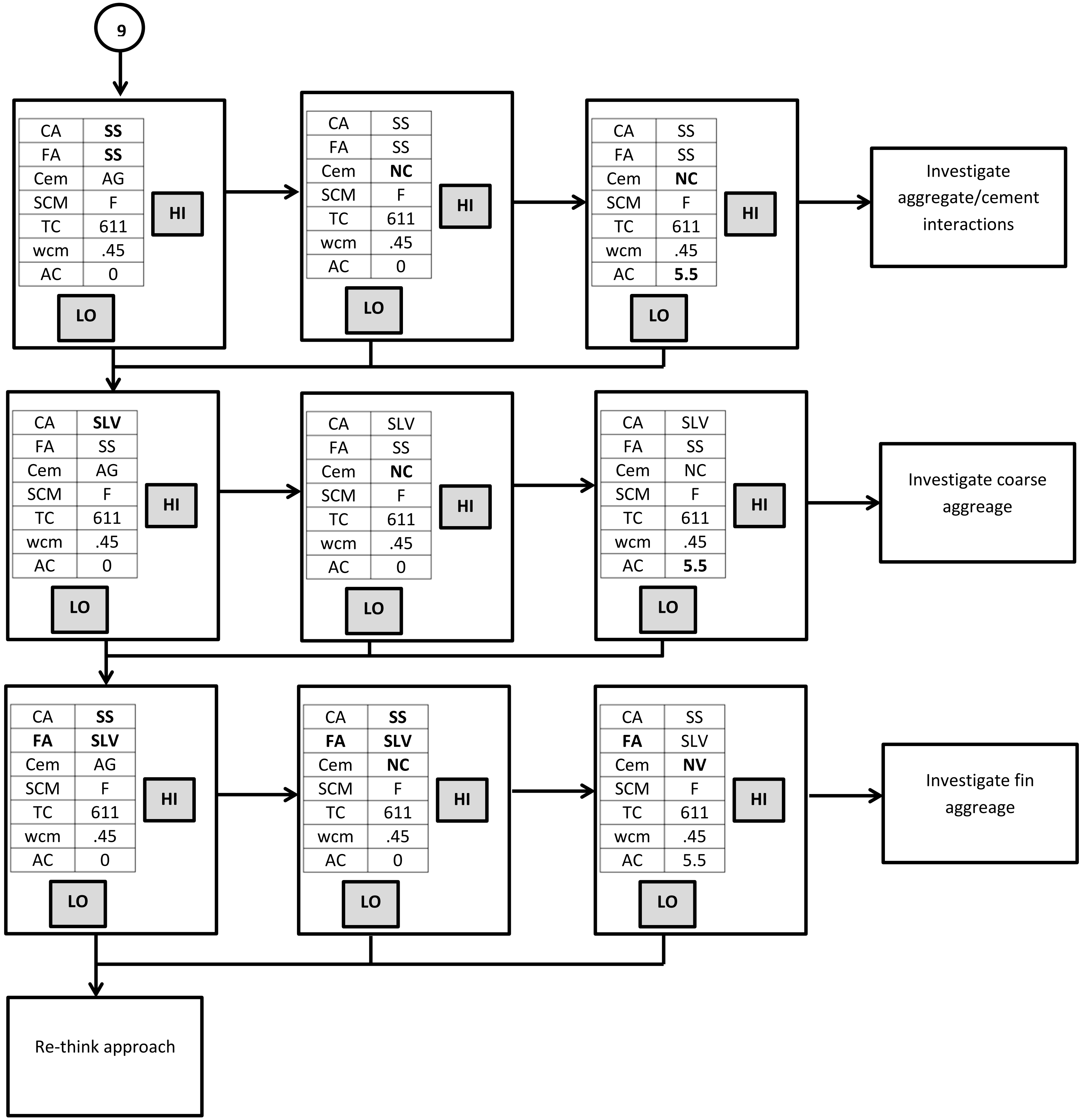
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate cement/local granodiorite interactions
4


9
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FA SS
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LO
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wcm .45
AC 0
HI
LO
CA SS
FA SS
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate aggregate/cement interactions
CA SLV
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Investigate coarse aggreage
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FA SLV
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SCM F
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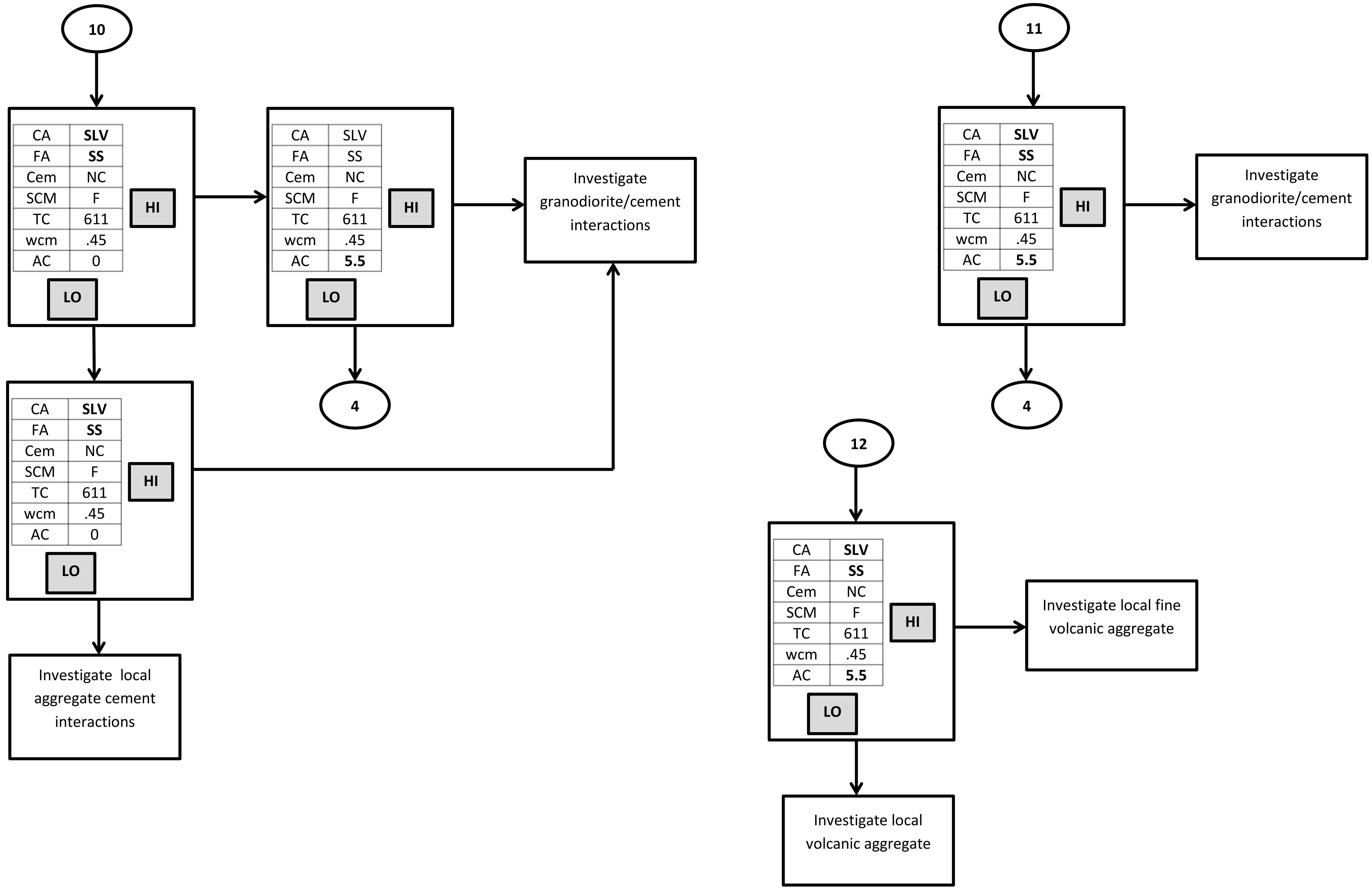
Investigate fin aggreage
Re-think approach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HI
LO
CA SLV
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SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate granodiorite/cement interactions
4
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FA SS
Cem NC
SCM F
TC 611
wcm .45
AC 0
HI
LO
Investigate local aggregate cement interactions
11
CA SLV
FA SS
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate granodiorite/cement interactions
4
12
CA SLV
FA SS
Cem NC
SCM F
TC 611
wcm .45
AC 5.5
HI
LO
Investigate local fine volcanic aggregate
Investigate local volcanic aggregate

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