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HIGH-PERFORMANCE CONCRETE USING NEVADA AGGREGATES

NEVADA DEPARTMENT OF TRANSPORTATION

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16. Abstract

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Laboratory testing based on tests suggested by FHWA's High-Performance Concrete Committee was used to evaluate the performance of each trial batch. The test program consists of chloride ion penetration, scaling resistance, shrinkage, compressive strength, modulus of elasticity, alkali-silica reactivity, and freeze/thaw durability.

The results of trial batch testing show that different raw materials investigated have a significant impact on long-term performance and durability. In order to optimize concrete for the different effects of the raw materials and their proportions, NDOT should adopt a performance based specification system for concrete. As a part of this study, a proposed performance based specification system has been developed for NDOT.

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DEPARTMENT OF TRANSPORTATION

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High-Performance Concrete Using Nevada Aggregates

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Abstract

The performance of concrete bridge decks depends on basic materials as well as external factors such as exposure conditions. The main objective of this research project is to develop concrete performance specifications for Nevada and determine measures which might be used to make concrete which passes those requirements. In order to do this, a series of concrete mix designs were developed and evaluated for their suitability to be used in Northern Nevada bridge decks and other structures exposed to harsh environments. In this research program, a total of 37 mix designs were completed using various combinations of three coarse aggregate sources, two Type I/II cement sources, two Class F fly ash sources, and four rates of cement replacement with fly ash. Testing of trial batches made from these materials yields data which shows how different materials and combinations of materials affect the final performance of the concrete.

Laboratory testing based on tests suggested by FHWA's High-Performance Concrete Committee was used to evaluate the performance of each trial batch. The test program consists of chloride ion penetration, scaling resistance, shrinkage, compressive strength, modulus of elasticity, alkali-silica reactivity, and freeze/thaw durability.

The results of trial batch testing show that different raw materials investigated have a significant impact on long-term performance and durability. In order to optimize concrete for the different effects of the raw materials and their proportions, NDOT should adopt a performance based specification system for concrete. As a part of this study, a proposed performance based specification system has been developed for NDOT.

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CHAPTER 1
Introduction

1.1 General

Concrete performance is influenced by the type and quality of the raw materials used, the mix design used to combine those materials, and the methods used to mix, consolidate, finish, and cure the concrete. The demands on concrete performance include not only structural demands, but also demands due to the specific environment and exposure conditions under which the concrete will be placed. There may also be additional demands on the concrete due to the use of sub-standard materials. In some regions, quality raw materials may not be available. In these cases, the concrete designer must take appropriate measures in the mix design to compensate for the necessary use of non-ideal materials and to ensure that the concrete will still be able to perform as required.

1.2 Need for Performance Specifications

Currently, the concrete performance specifications required by the State of Nevada Department of Transportation (NDOT) are compressive strength, slump, air content, and unit weight (NDOT 501.02.04). While these are important properties, this method of specification does not allow for the wide variety of environmental demands, exposure conditions, and materials that are encountered in the field. Of these properties, the specification for compressive strength is the only one that requires a test on the actual performance of the cured concrete. The slump, air content, and unit weight are all important properties of the fresh concrete and they each have an effect on the concrete's final performance. However, these properties are not absolute indicators of the long-term performance and durability of the concrete.

In order to ensure good performance and durability of concrete in the field, other requirements which relate to the long-term outcome of concrete elements must be specified by the design engineer. These long-term behavior specifications could relate to mechanical properties, such as strength and elastic modulus, or to durability issues, such as abrasion resistance, chloride ion permeability, freeze-thaw resistance, and alkali-silica reactivity. In order for these requirements to be met, materials engineers would have to engineer concrete mix designs to pass the performance criteria necessary for each job. Implementing the use of additional specifications may incur higher initial costs due to additional tests for acceptance of concrete batches and the possible use of more specialized materials. However, long-term benefits from more durable, longer lasting concrete with fewer maintenance requirements should outweigh the increase in initial cost.

1.3 Objectives

The main objective of this research program is to develop concrete performance specifications for Nevada and determine measures which might be used to produce concrete which passes those requirements. In order to achieve this goal, the performance of several concrete trial batches must be evaluated. For this project, it is necessary to develop a series of trial mix designs using a range of materials readily available in Northern Nevada and to identify the environmental exposure conditions and mechanical properties that are important factors for bridge decks in Nevada. Finally, the trial batch concrete must be subjected to the appropriate test methods in order to evaluate performance and durability.

Although high strength is often associated with high-performance concrete, compressive strength is not the only factor which influences concrete durability. Due to this fact, obtaining high strengths was not an objective of this project.

The results of this research will enable NDOT design engineers to specify performance requirements on structural concrete. The results of trial batch testing should aid materials engineers in optimizing mix designs for various exposure conditions and performance criteria that may be required.

1.4 High-Performance Concrete

The use of high-performance concrete (HPC) is a relatively new strategy for specifying long-term concrete behavior. The Strategic Highway Research Program (SHRP), the American Concrete Institute (ACI), and the Federal Highway Administration (FHWA) each have a different definition for HPC. FHWA's report, "High-Performance Concrete Defined for Highway Structures" (37), gives the FHWA definition and briefly summarizes the ACI and SHRP definitions. For this project, the FHWA definition has been used to evaluate concrete performance. However, the SHRP and ACI definitions will be reviewed briefly.

1.4.1 ACI Definition of HPC

ACI defines HPC as concrete which meets special performance and uniformity requirements which cannot always be obtained using conventional ingredients, normal mixing procedures, and typical curing practices (3). These requirements may relate to longer life in severe environments, volume stability, toughness, high early strength, ease of placement and consolidation without affecting strength, and long-term mechanical properties.

1.4.2 SHRP Definition of HPC

Unlike the ACI definition, SHRP's definition for HPC incorporates mix design proportions (w/c) and performance criteria. A SHRP report defines HPC as concrete having a maximum water-cementitious materials ratio (w/c) of 0.35; a minimum relative dynamic modulus of elasticity (durability factor) of 80%, as determined by the ASTM C666 freeze-thaw test; and a minimum strength meeting one of the criteria given in the Table 1.1 (51).

1.4.3 FHWA's Proposed HPC Definition

FHWA's high-performance concrete definition is given in "High-Performance Concrete Defined for Highway Structures" (37). This HPC definition is based only on long-term performance criteria and does not specify fresh concrete properties. It considers eight performance parameters, four of which are related to deterioration resistance. The following table shows the FHWA high-performance concrete parameters and the test method specified for each one.

Table 1.1 - SHRP High-Performance Concrete Strength Criteria

HPC Strength Rating	Strength, Age
Very Early Strength (VES)	3000 psi, 4 hours
High Early Strength (HES)	5000 psi, 24 hours
Very High Strength (VHS)	10000 psi, 28 days

Table 1.2 - FHWA High-Performance Concrete Parameters and Tests[1]

Performance Parameter	Standard Test Method
Freeze-Thaw Durability	AASHTO T 161 ASTM C 666 Procedure A
Scaling Resistance	ASTM C672
Abrasion Resistance	ASTM C944
Chloride penetration	AASHTO T 277 ASTM C 1202
Strength	AASHTO T 2 ASTM C 39
Modulus of Elasticity	ASTM C 469
Shrinkage	ASTM C 157
Creep	ASTM C 512

[1]Table based on Reference (37).

FHWA assigns a performance grade to different levels of performance in each of the eight HPC parameters. This is meant to allow design engineers to specify exactly how concrete for a given job should perform in each of the eight performance parameters. Table 1.3 shows FHWA's performance grades for structural concrete. FHWA also provides some suggested guidelines as to which performance grades should be specified for different exposure conditions. Table 1.4 summarizes these guidelines

To better understand how HPC performance grades might be used, consider the following example that may be encountered in Northern Nevada: The design of a concrete bridge deck requires a compressive strength of 6000 psi. This bridge deck will experience approximately 30 freeze-thaw cycles each year. Because this bridge deck will be exposed to a freezing environment, it is estimated that deicer salt will be applied at a rate of 5 tons/lane-mile-year. The average daily traffic is 100,000 and chains or tire studs are allowed for driver safety during icy conditions. Also, it is desired that this concrete experience minimum shrinkage in order to minimize long-term crack formation.

Table 1.3 - FHWA Performance Grades for HPC[1]

Performance Parameter	High Performance Concrete Grade 1	2	3	4
Freeze-Thaw Durability (x = relative dynamic modulus of elasticity)	$60\% \le x < 80\%$	$x \ge 80\%$		
Scaling Resistance (x = visual rating of surface after 50 cycles)	x=4, 5	x=2, 3	x=0, 1	
Abrasion Resistance (x = avg. depth of wear in mm)	$2.0 > x \ge 1.0$	$1.0 > x \ge 0.5$	$0.5 > x$	
Chloride Penetration (x=coulombs)	$3000 \ge x > 2000$	$2000 \ge x > 800$	$800 \ge x$	
Strength (x = compressive strength)	$6 \le x < 8$ ksi	$8 \le x < 10$ ksi	$10 \le x < 14$ ksi	$x \ge 14$ ksi
Elasticity (x = modulus of elasticity)	$4 \le x < 6(10^6)$ psi	$6 \le x < 7.5(10^6)$ psi	$x \ge 7.5(10^6)$ psi	
Shrinkage (x = microstrain)	$800 > x \ge 600$	$600 > x \ge 400$	$400 > x$	
Creep (x=microstrain/pressure unit)	$0.52 \ge x > 0.41$/psi	$0.41 \ge x > 0.31$/psi	$0.31 \ge x > 0.21$/psi	$0.21/\text{psi} \ge x$

[1]Table based on Reference (37).

Table 1.4 – FHWA Guidelines for the Application of HPC Grades[1]

	Recommended HPC Grade for Given Exposure Condition				
Exposure Condition	N/A[2]	Grade 1	Grade 2	Grade 3	Grade 4
Freeze/Thaw Durability Exposure (x=F-T[3] cycles per year)	$x<3$	$3 \le x < 50$	$50 \le x$		
Scaling Resistance AppliedSalt (x=tons/lane-mile-year)	$x<5.0$	$5.0 \le x$			
Abrasion Resistance (x=avg. daily traffic, studded tires allowed)	no studs/chains	$X \le 50(10^3)$	$50{,}000 < x \le 100{,}000$	$100{,}000 \le x$	
Chloride Penetration Applied Salt (x=tons/lane-mile-year)	$x<1$	$1.0 \le x < 3.0$	$3.0 \le x < 6.0$	$6.0 \le x$	

[1]Table based on Reference (37).
[2]N/A stands for "not applicable" and indicates a situation in which specification of an HPC performance grade is not necessary.
[3]F-T stands for "freeze-thaw."

For this example, the design engineer would specify the following HPC performance grades for this concrete: Table 1.3 shows that for a strength of 6000 psi, a compressive strength grade of 1 applies. The bridge deck will experience 30 freeze-thaw cycles per year. Table 1.4 shows that for this condition, a freeze-thaw grade of 2 should be specified. Five tons/lane-mile-year of deicer salt will be applied. Table 1.4 recommends that, for this amount of deicer, a scaling resistance grade of 1 and a chloride penetration grade of 2 should be specified. For abrasion resistance, Table 1.4 shows that when the average daily traffic is 100,000 and studded tires are allowed, an abrasion resistance grade of 3 should be required. Finally, in order to minimize shrinkage, the highest shrinkage performance grade should be required. The highest shrinkage grade, as given in Table 1.3, is a grade of 3.

1.4.4 High-Performance Concrete in this Project

For this research project, the FHWA definition of high-performance concrete has been used as a guideline in developing durable concrete and for determining which mix design factors affect the performance parameters. The FHWA definition was chosen for this project because, of the three HPC definitions previously outlined (ACI, SHRP, and FHWA), it seems to be best suited for use in specifying concrete for the wide variety of loading and exposure conditions that can exist.

The ACI definition is good in the respect that it allows for the specification of any possible performance criteria that may be required. However, the ACI definition gives no guideline as to how such a system of specification could be implemented. The SHRP definition is too limited in its performance criteria requirements and makes little allowance for the range of loading and exposure conditions that exist in the field.

1.5 State DOTs' Experience with HPC

Many states have begun to use high-performance concrete specifications on selected trial projects. Texas, Ohio, New Hampshire, and New York are examples of states which have begun to use HPC as a specification method for selected concrete bridge projects. These states have all successfully completed bridge projects using HPC. Some of their results are briefly outlined here.

1.5.1 HPC in Texas

The Texas Department of Transportation (TxDOT) has completed two bridge projects using HPC. These projects resulted in the construction of the Louetta Road overpass in Houston and the San Angelo U.S. 67 bridge. In an article for "HPC Bridge Views", Wes Heald, executive director of TxDOT, cites several factors which made the use of HPC practical (38). Quality control and quality assurance (QC/QA) were very important aspects of these two projects because they help to ensure that the right materials are used for the job and provide a check to make sure that the final product meets specifications. Partnering and teamwork are also cited as an important factor in these projects. State DOTs must work in conjunction with universities, contractors, fabricators and researchers to ensure that HPC is implemented properly and to make sure that all of the parties involved are able to complete their tasks.

One of the main HPC parameters used in the Texas bridge projects was high strength. For the Louetta Road overpass a compressive strength of 13,100 psi was required for some of the beams. This corresponds to an FHWA HPC performance grade of 3 for strength. Using 54 inch deep precast, prestressed U-beams at spacings up to 15.8 ft, the Louetta overpass spans up to

135.5 ft. Certain beams in the San Angelo bridge required strengths as high as 14,700 psi (50). This would require a performance grade of 4 for strength based on FHWA guidelines.

In his article, Wes Heald states some of the advantages in using HPC. First, when high strength concrete is used, HPC bridges can be designed with fewer spans and longer beams. This saves time and money because fewer substructure elements are required and the bridge may be fabricated more quickly. Also, HPC can be more durable than conventional concrete. Durable and impermeable bridge decks should not experience damage from scaling, freeze-thaw action, shrinkage cracking, or reinforcement deterioration. This is expected to increase the life-span of the bridge and reduce the maintenance and rehabilitation costs that will be required over the bridge's life-span (38).

1.5.2 HPC in New Hampshire

The New Hampshire Department of Transportation's (NHDOT) first HPC structure is a bridge in Bristol, NH (56). This bridge carries Route 104 over the Newfound River.

To minimize maintenance and prolong the bridge life-span, it was desired that the bridge deck be highly impermeable, freeze-thaw resistant, and free of cracks. The specifications shown in Table 1.5 were used to achieve these properties.

Table 1.5 - New Hampshire HPC Bridge Deck Specifications[1]

Cement	Type II
Cement Replacement with Silica Fume	7.5%
w/cm max.[2]	0.38
Air Content	6 to 9%
28-day Cylinder Strength*	7200 psi
Chloride Ion Permeability	1000 coulombs
Corrosion Inhibitor	4 gal/yd^3
Curing Procedure	4-day wet cure with cotton mats

[1]Values in table are based on Reference (56).
[2]w/cm stands for water/cementitious materials ratio.
*Actual specified strength was 6000 psi. The 7200 psi value was used as a basis for concrete mix proportions.

Before the actual deck pour, a 5 yd^3 trial pour was performed to simulate the actual placement, finishing, and curing methods that would be used. This enabled the contractor to adjust the mix design and admixture dosage for acceptable workability. It was also an opportunity to make sure that the proper equipment was being used.

For the actual bridge deck, standard construction techniques were used. A self-propelled finishing machine was used strike to off the surface. A finishing pan and burlap drag were attached to the screed machine to finish and texture the surface. Within 15 minutes after the surface was finished the concrete was covered with cotton mats and wetted. Care was taken not to over-finish the surface. Concrete placement was not allowed if the evaporation rate was over 0.1 $lb/ft^2/hr$ or if the temperature was over 85°F. This was done to limit water evaporation and surface drying which can cause cracking.

Table 1.6 gives the test results for the deck concrete used in New Hampshire's project. It should be noted that, although the air content results are lower than the specified values, the results of freeze-thaw testing are good.

Table 1.6 - NHDOT Concrete Deck Test Results[1]

Test	Result
Slump	3 to 5 inches
Unit Weight	144 to 147 lb/ft^3
Air Content	4.0 to 5.8%
w/cm ratio	0.39
28-Day Compressive Strength	8160 to 9610 psi
Modulus of Elasticity	4.2 to $4.3x10^6$ psi
Chloride Ion Permeability	610 to 900 coulombs
Freeze-Thaw Durability	96 to 99%
Scaling	0 to 1

[1]Values in table are based on Reference (55).
[2]w/cm stands for water/cementitious materials ratio.

In his article, "Crack Free HPC Bridge Deck - New Hampshire's Experience", Christopher M. Waszczuk of NHDOT cites trial batches and the trial pour as important elements which led to the success of New Hampshire's HPC project. Other important factors were a proper finishing technique in which "over-finishing and bullfloating the surface were strongly discouraged" and proper curing procedures (56). NHDOT's HPC project was very successful and led to the construction of a bridge deck with no shrinkage or transverse cracking.

1.5.3 HPC in Ohio

Ohio's HPC bridge is located on U.S. 22 near Cambridge, OH (43). The required length of this bridge was 116 feet and it was originally designed as a three span, adjacent box girder bridge with 21-inch deep simply supported boxes.

The use of HPC in this project yielded several benefits. First, HPC allowed this bridge design to be changed from three spans to a single span with 42 inch deep box girders. This yields economic benefits because the need to construct piers is eliminated. Secondly, the use of HPC makes the concrete less permeable and, therefore helps protect steel reinforcement from corrosion due to salt water penetration. For the HPC bridge in Ohio, HPC yields both immediate benefits, in the form of construction savings, and long-term benefits, through lower maintenance costs due to better corrosion resistance (43).

1.5.4 HPC in New York State

In 1994 the New York State Department of Transportation (NYSDOT) established a bridge deck task force with the goal of making longer lasting bridge decks (52). The objective of this task force was to produce a concrete that is more durable, less permeable, resistant to cracking, and easily placed and finished. These objectives were achieved through the use of a lower cement content, the use of pozzolans, a lower water/cementious materials ratio, and limited use of admixtures (specifically, no high range water reducers were allowed). Trial mix designs developed by this task force were tested for compressive strength, permeability, resistance to cracking, scaling, workability, and finishability.

The results of this research yielded a new concrete specification, Class HP concrete, which is essentially a modification to NYSDOT's Class H concrete. Class HP concrete has a maximum water/cementitious materials ratio of 0.40. The proportions of materials making up the cementitious portion of this concrete are given in Table 1.7. Table 1.8 compares the test results of trial batches of NYSDOT's class H and class HP concrete.

Table 1.7 - NYSDOT Class HP Concrete Cementitious Materials[1]

Material	% of Total Cementious Content	Pounds/(Yd^3 of Concrete)
Cement	74%	500 lb.
Fly Ash	20%	135 lb.
Microsilica	6%	40 lb.

1. Table based on Reference (52).

Table 1.8 - NYSDOT Trial Batch Test Results [1]

Test Parameter	Class H Concrete	Class HP Concrete
Scaling Rating	1	2
Permeability	4344 Coulombs	1270 Coulombs
Cracking	191 inches/yd^2	6 inches/yd^3

1. Table based on Reference (52).

As table 1.8 shows, class HP concrete has much better permeability and cracking characteristics than class H concrete. The scaling resistance of class HP concrete is slightly worse than class H, but the amount of scaling damage was deemed as acceptable. The early rate of strength gain for class HP concrete was slower than class H but the 14 and 28 day strengths were comparable.

The first field application of class HP concrete was for a bridge deck on Route 78. A finishing machine was used to finish the deck and no bullfloating was necessary. Turf drag texturing was employed and wet burlap curing was applied within 10 minutes of texturing. The concrete used in this deck had a compressive strength of 7500 psi at 28 days and a permeability of 600 coulombs. Twenty months after construction there were no cracks in the concrete (52).

Since NYDOT's first application of class HP concrete, it has been used in over 60 placements on 40 bridges. In 1996 class HP concrete became required for all bridge decks in

New York and it began use in substructures as well. Some beneficial properties that class HP concrete has shown over time are very good handling and workability characteristics and no cracking related to materials. One problem that has been experienced is that some scaling has occurred in areas which were over-worked due to hand finishing. Regular use of class HP concrete has resulted in cost stabilization of bid prices (52).

1.6 Summary

There are many environmental and exposure factors that can damage concrete structures. There are also several mechanical demands on concrete structures. In order for concrete to withstand the environment and perform as required, it must be designed for the specific conditions that it will face. The use of high-performance concrete is a method which can be used to specify and design engineered concrete for various applications. Several states have already successfully completed high-performance concrete bridge projects and more projects are underway. Some factors which have led to the success of these projects are cooperation between states, researchers, and contractors, the use of quality raw materials combined in the right proportions, and proper consolidation, finishing, and curing procedures.

The purpose of this research project is to develop concrete trial mix designs and to test trial batches which will provide data to determine how high-performance concrete could be produced and utilized to make durable bridge decks in Nevada. Chapter 2 of this report gives a description of the various mechanical factors and exposure conditions that are important for concrete structures as well as the test program used for this project. Chapter 2 also contains descriptions of the test methods used to determine how concrete performs in each of the eight performance parameters set forth in FHWA's high-performance concrete definition. Chapter 3 describes the raw materials, mix designs, mix procedure, and fresh concrete tests used for this project. Chapter 4 gives the results of the tests performed as well as some discussion of those results. Chapter 5 presents a suggested concrete performance grading system for Nevada. Chapter 6 provides recommendations as to how high-performance concrete should be used in Nevada bridge decks and how it might be integrated into use for other applications.

CHAPTER 2
Background and Test Program

2.1 General

Concrete bridge decks must be able to withstand the structural loads they are subjected to, as well as the environmental conditions to which they are exposed. In Northern Nevada, these environmental factors include wet and dry moisture conditions, repeated freezing and thawing, exposure to deicing chemicals, and aggregates which have the potential to participate in alkali-silica reactions.

2.2 Test Program

The choice of tests to be included in this research program was based largely on the tests suggested in FHWA's "High-Performance Concrete Defined for Highway Structures" (37). Table 2.1 summarizes the high-performance concrete tests that were used in this project. The test program used for this research deviates from FHWA's HPC test program. FHWA specifies tests for creep and abrasion resistance and also suggests using ASTM C 227 to test alkali-silica reactivity. Because the creep test requires a time period longer than the scope of the project and the abrasion test requires specialized equipment which was not available, these tests were not included in this research. ASTM C 1260 was used to determine the alkali-silica reactivity of the aggregates. Even though they were not included as part of the test program for this research, creep, abrasion, and the ASTM C 227 test will be briefly described in this chapter.

Table 2.1 - Research Parameters and Tests

Performance Parameter	Standard Test Method
Freeze-Thaw Durability	ASTM C 666 Procedure A
Scaling Resistance	ASTM C 672
Chloride penetration	ASTM C 1202
Strength	ASTM C 39
Modulus of Elasticity	ASTM C 469
Shrinkage	ASTM C 157
Alkali-Silica Reactivity	ASTM C 1260

2.3 Drying Shrinkage

Some regions in Northern Nevada remain very dry throughout most of the year. In a dry environment, concrete can experience drying shrinkage. Drying shrinkage is essentially a volume change that takes place over time due to moisture loss. Loss of moisture is the main cause for drying shrinkage, however, the fundamental processes underlying drying shrinkage are not yet fully understood. Drying shrinkage is a major concern because it can cause cracking and warping of concrete elements due to structural restraints on the concrete (44). Drying shrinkage is effected by a wide range of variables which include the shape of the element, the mix proportions, the chemical and physical properties of the raw materials, and the environment to which the element is exposed.

2.4 Length Change of Hardened Hydraulic Cement Mortar and Concrete (ASTM C 157)

The purpose of this test is to determine the length change or shrinkage of concrete due to factors other than externally applied forces or temperature changes. Concrete specimens measuring 3 inches by 3 inches by 10 inches are cast with a metal gage stud at the center of each end. After one day, each specimen is removed from its mold and a length comparator is used to take an initial length reading of the distance between the gage studs. Figure 2.1 shows a length comparator with a concrete shrinkage specimen.

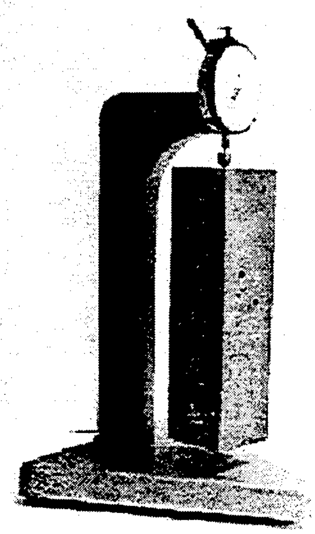

Figure 2.1 - Length Comparator with Shrinkage Specimen

The specimen is placed in moist storage for 28 days, at the end of which time it is removed from moist storage and another length comparator reading is taken. The specimen is then placed in a drying room for the remainder of the test. The temperature of the air in the drying room is maintained at 73.4°F with a relative humidity of 50%. Successive length comparator readings are taken at 7 days, 14 days, 28 days, 8 weeks, 16 weeks, and 32 weeks after being placed in the drying room.

The length comparator readings at each test age are compared to the initial length comparator reading to calculate the shrinkage of the concrete which is measured in microstrains. The values in Table 2.2 are used to evaluate test results.

Table 2.2 - Shrinkage Grades

Shrinkage (microstrains)	Description	FHWA HPC Performance Grade*
<400	Good	3
400-600	Moderate	2
600-800	Poor	1
>800	Very Poor	

*Values based on Reference (37).

2.5 Freeze-Thaw Environment

Northern Nevada's climate is such that, during certain periods in late fall, winter, and early spring, concrete structures experience daily cycles of freezing and thawing. This is of concern because freeze-thaw action can cause micro-cracking of concrete and, over time, very severe damage and concrete failure can occur.

In order for freeze-thaw damage to occur, the concrete must be in a moist condition. When pore water in concrete freezes, it expands and causes hydraulic pressures which induce stresses within the concrete (49). After repeated cycles of freezing and thawing, micro-cracks can begin to form and water can enter the small voids formed by the micro-cracks. When the water in the micro-cracks freezes, the micro-cracks are forced to expand even more and severe concrete damage may result. Factors influencing a concrete's resistance to freeze-thaw action include air content, soundness of aggregates, and concrete maturity (37). Air entrainment is the most commonly used method for ensuring good freeze-thaw resistance. Air voids in the concrete matrix allow space into which the residual pore water can diffuse so that no stresses are induced in the concrete (44).

2.6 Resistance of Concrete to Rapid Freezing and Thawing (ASTM C 666)

This test is used to determine the resistance of concrete to rapidly repeated cycles of freezing and thawing. Concrete specimens measuring 3 inches by 3 inches by 10 inches are cast and placed in moist storage for 14 days. Each specimen is then removed from moist storage and its fundamental transverse frequency is measured using a sonometer. Next, the specimen is placed in water in a freeze-thaw apparatus and exposed to rapid cycles of freezing and thawing in which the temperature cycles between 0 and 40°F. After 300 cycles the fundamental transverse frequency is measured again and the specimens are removed from the test.

The initial and final values of the fundamental transverse frequency are used to calculate the relative dynamic modulus of elasticity of the concrete after 300 cycles. The relative dynamic modulus of elasticity, in percent, is a measure of how much of the original dynamic modulus of elasticity remains after exposure to repeated freeze-thaw cycles. Higher values for the relative dynamic modulus of elasticity indicate better resistance to freezing and thawing. Table 2.3 gives FHWA's performance grades for freeze-thaw resistance.

Table 2.3 - FHWA Freeze-Thaw Resistance Performance Grades[1]

Relative Dynamic Modulus of Elasticity	FHWA Performance Grade
60% to 80%	1
80% or greater	2

1. Table based on Reference (37).

2.7 Scaling

Due to the fact that Northern Nevada experiences freezing temperatures throughout parts of the year, deicing salts are applied to roadways for driver safety. When the roadway consists of concrete pavement or a concrete bridge deck, deicing chemicals can cause surface damage such as scaling, spalling, flaking, and pitting.

The exact reason why deicing chemicals cause concrete to be more susceptible to scaling, spalling, flaking, and pitting is not fully understood. One reason for increased surface damage in the presence of deicing chemicals may be that deicer application causes additional freeze-thaw cycles to take place (37). When deicing chemicals are applied to a concrete roadway, the ice on the surface of the roadway melts and the water absorbed in the surface of the concrete thaws. The melt water then saturates the surface concrete and the deicer solution becomes more diluted. If the surface of the concrete freezes again, then the application of deicing chemical has caused a freeze-thaw cycle that would not have taken place without its application. This cycle can repeat and cause deterioration to concrete without adequate freeze-thaw resistance (37).

Another possible cause of surface damage due to deicing chemicals is that melting ice is an endothermic process. As the ice melts it absorbs thermal energy from the concrete which causes the temperature of the concrete to drop. This may cause damage to the concrete due to differential thermal strains (37).

Factors which may affect a concrete's susceptibility to surface damage from the application of deicer salts include air content, water-cement ratio, the concentration of deicer salt, the thermal properties of the aggregates, and the method used to finish the concrete surface.

2.8 Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals (ASTM C 672)

This test is used to determine the resistance of a horizontal concrete surface to scaling when exposed to deicing chemicals and repeated freeze-thaw cycles. For this test, specimens measuring approximately 8.5 inches square by 3 inches deep are cast and placed in moist storage for 14 days followed by 14 days in air storage. The specimen is then exposed to a calcium chloride solution which is 1/4 inch deep on its top surface. The specimen is exposed to repeated cycles consisting of 16-18 hours in a freezing environment maintained at 0°F followed by 6-8 hours in laboratory air at 73°F. After every five cycles the surface of the concrete is flushed with water and a visual rating of the surface is made after 5, 10, 15, 25, and 50 cycles.

The scaling resistance of a concrete is evaluated using qualitative visual examination. The scaling resistance of each specimen is rated using a number scale which is based on the condition of the surface. Table 2.4 is used to rate scaling resistance.

Table 2.4 - Scaling Resistance Rating Scale

Condition of Surface	**Rating[1]**	**FHWA Grade[2]**
no scaling	0	3
very slight scaling (1/8 in. depth, max., no coarse aggregate visible)	1	3
slight to moderate scaling	2	2
moderate scaling (some coarse aggregate visible)	3	2
moderate to severe scaling	4	1
severe scaling (coarse aggregate visible over entire surface)	5	1

[1]Rating scale is based on descriptions given in ASTM C 672.
[2]Grades based on Reference (37).

2.9 Permeability and Steel Reinforcement Corrosion

When deicing chemicals melt ice, the melt water becomes a solution containing chloride ions. The melt water solution can then travel through pores in the concrete matrix and come into contact with steel reinforcement. When this happens, the chloride ions react with the steel reinforcement and cause it to corrode. Steel corrosion is an expansive reaction which will induce tensile stresses within the concrete and, when these stresses exceed the capacity of the concrete, it begins to spall (37).

One of the main measures used to reduce concrete's permeability, and, therefore, its resistance to steel corrosion, is to include pozzolanic materials as part of the cementitious content of the mix. The pozzolanic reaction that takes place in concrete containing pozzolans produces hydration products which are very efficient at filling pore space in the concrete matrix (42). This reduces the permeability of the concrete thereby reducing the amount of chloride ions that are able to penetrate into the concrete.

Another method used to protect steel reinforcement from corrosion is to use epoxy coated rebar. This method is very effective at protecting the steel but it may not be as economically efficient as certain methods of reducing the permeability of the concrete itself (42).

2.10 Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration (ASTM C 1202)

This test determines a concrete's ability to resist the penetration of chloride ions. This test was performed at 56 days and 120 days after casting for each trial batch. For this test, cylindrical specimens having dimensions of 4 inches in diameter and 8 inches in height are cast and moist cured until testing. The test requires only a 2 inch thick slice of the specimen, so, prior to testing, 1 inch is cut off the top of the eight inch cylinder and a 2 inch thick test specimen is cut from the top of the remaining cylinder. The top inch of the original cylinder is not included in the 2 inch test specimen so that any surface effects due to finishing or bleeding will not effect the test.

The test specimen is then saturated with water in a vacuum saturation apparatus and placed in an applied voltage test cell. In the test cell, one end of the specimen is exposed to a 3% concentration NaCl solution and the other end is exposed to a 0.3 N concentration NaOH solution. Figure 2.2 shows a cross-section of the test cell.

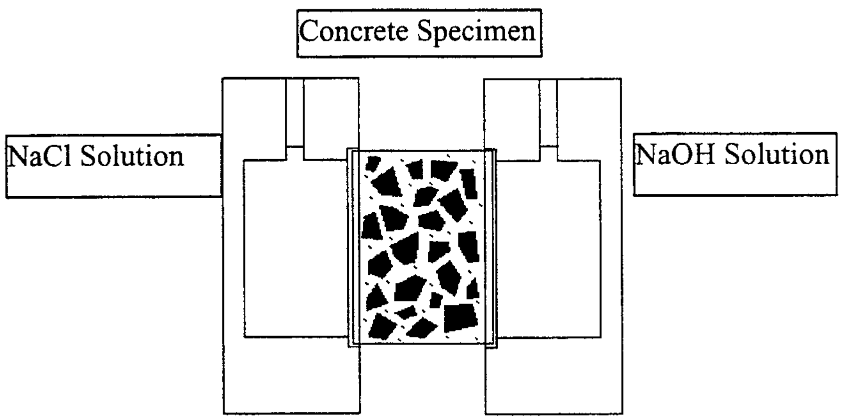

Figure 2.2 - ASTM C 1202 Test Cell Cross-Section

A potential difference of 60 V dc is maintained across the ends of the specimen and the current passing through the specimen is measured at half hour intervals. The total charge passed through the specimen is calculated by integrating the area under the current versus time curve. Figure 2.3 shows a typical current vs. time curve. The area underneath the curve represents the total charge passed in ampere-seconds or coulombs. The total charge passed, in coulombs, is related to the concrete's ability to resist chloride ion penetration. As more chloride ions migrate into the concrete, more current can pass through and the total charge passed increases. A high value for total charge passed indicates that the concrete is highly permeable. A low value for total charge passed indicates that the concrete has low permeability. Table 2.5, based on ASTM C1202, is used to evaluate test results.

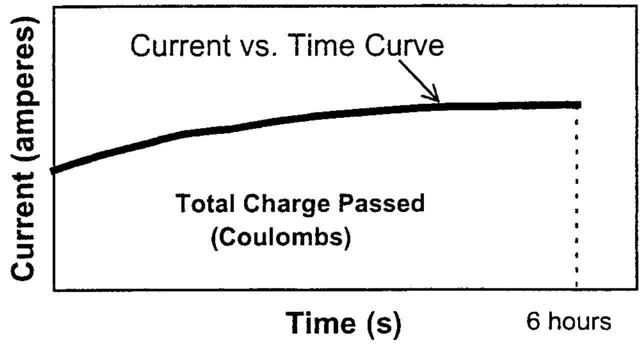

Figure 2.3 - Current vs. Time Curve

Table 2.5 - Chloride Ion Penetrability Based on Charge Passed*

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

*Values in Table are based on ASTM C 1202

Table 2.6 gives FHWA's high performance concrete performance grades for permeability.

Table 2.6 - FHWA Performance Grades for Permeability*

Charge Passed (Coulombs)	HPC Performance Grade
2000 to 3000 Coulombs	1
800 to 2000 Coulombs	2
0 to 800 Coulombs	3

*Table based on Reference (37).

2.11 Alkali-Silica Reactivity

Depending on the weather, concrete in Northern Nevada can experience a moist condition for significant time periods throughout the year. A moist environment contributes to alkali-silica reactivity. Alkali-silica reactivity can take place between reactive silica in the aggregates and alkalis contained in the cement. This reaction forms a hydrous alkali silicate gel and weakens the aggregate significantly. The gel can then absorb water and expand causing stresses that are able to crack the weakened aggregate and the cement paste around it (44). This process can eventually cause extensive cracking, spalling, popouts, and failure of concrete structures.

The factors controlling alkali-silica expansion are the amount and nature of reactive silica, the particle size of reactive material, the amount of available alkali, and the amount of available moisture (44). Some measures that may be used to control alkali-silica reactions are to avoid the use of potentially reactive aggregates and to incorporate the use of low-alkali cements and pozzolanic admixtures. Avoidance of reactive aggregates is not always practical or economical but, with the use of service records and petrographic analyses, it may be feasible. Using pozzolanic admixtures has two advantages in controlling alkali-silica reactivity. First, using pozzolans reduces the pH of the pore solution. Second, pozzolans can reduce the permeability of concrete which may reduce the amount of water that is able to penetrate the concrete and participate in the expansion of the alkali-silica gel (44).

FHWA suggests using ASTM C 227 to evaluate HPC aggregates for alkali-silica reactivity. According to FHWA recommendations, this test would take 6 months. ASTM C 227 can be used to evaluate how different combinations of cement and fly ash effect the alkali-silica reaction that may take place with different aggregates. For this project, ASTM C 1260 was used to evaluate the aggregates for alkali silica reactivity. ASTM C 1260 only takes 16 days to perform. One of the limitations of ASTM C1260, relative to ASTM C 227, is that, according to ASTM C 1260, it can only be used to evaluate potential reactivity of aggregates and should not be used to determine the effect of different combinations of cement and fly ash. ASTM C 1260 was chosen for use in this project because of the shorter test duration. However, the ASTM C 227 test method will be described briefly.

2.12 Potential Alkali Reactivity of Cement-Aggregate Combinations (ASTM C 227)

For this test, four 1 inch by 1 inch by 10 inch mortar specimens are made with metal gage studs in each end. The molds are placed in moist storage for 24 hours at the end of which time the specimens are removed from their molds and an initial length comparator reading is taken. The specimens are then put in a container where they stand over, but not in contact with, water. The container is placed in a cabinet or room maintained at 100 °F for 12 days and then another length comparator reading is taken and the container is placed back in the cabinet or room maintained at 100 °F. FHWA recommends taking successive length comparator readings at 3 months and 6 months. According to FHWA, for good alkali silica reactivity performance, the expansion at 3 months should be less than 0.05% and the expansion at 6 months should be less than 0.10% (37).

2.13 Potential Alkali Reactivity of Aggregates (ASTM C 1260)

The purpose of this test is to evaluate an aggregate's potential to contribute to alkali-silica reactivity. For this test, three mortar bar specimens are made using the aggregate gradation requirements and proportions set forth in ASTM C 1260. The specimens are then moist cured for 24 hours and removed from their molds and an initial length reading is taken using a length comparator. The specimens are then placed in sealed containers containing water so that the specimens are fully immersed. The containers are placed in an oven maintained at 176 °F for 24 hours. At this time, the containers are removed from the oven and another length comparator reading is taken. Next, the specimens are placed in a container containing 1 N NaOH and placed back in the oven. At the end of 14 days, the containers are removed from the oven and final length comparator readings are taken.

The length comparator readings are used to calculate the expansion of the mortar bars from the time when they were placed in the NaOH to the time when they were removed. Higher expansions indicate a greater potential for alkali-silica reactivity.

2.14 Abrasion Resistance

Abrasion resistance is the ability of a concrete surface to resist wear due to abrasive action. Conditions which can cause abrasion include the use of tire chains and studded snow tires. FHWA includes abrasion resistance as a high performance concrete parameter but no abrasion resistance tests were included in this research.

2.15 Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method (ASTM C 944)

In "High-Performance Concrete Defined for Highway Structures," FHWA recommends using the ASTM C 944 test method to evaluate abrasion resistance. The main piece of equipment required by ASTM C 944 is an abrasion device. An abrasion device is essentially a drill press which rotates an abrading cutter and maintains a constant load forcing the abrading cutter against the concrete specimen. The abrading cutter is rotated at a rate of 200 rotations per minute and each test area on the surface of the concrete is exposed to 6 minutes of abrasion applied over 3 periods, each lasting 2 minutes. FHWA specifies that the abrading cutter be pressed against the specimen at a force equal to about 44 pounds (37). After the abrasion process is complete, the depth of wear is measured. Smaller values for depth of wear indicate better abrasion resistance. Table 2.7 gives FHWA's performance grades for abrasion resistance.

Table 2.7 - FHWA Performance Grades For Abrasion Resistance

Depth Of Wear (x = avg. depth of wear (mm))	Performance Grade
2.0 mm $\geq$ x > 1.0 mm	1
1.0 mm $\geq$ x > 0.5 mm	2
0.5 > x	3

Table based on Reference (37).

2.16 Mechanical Properties of Concrete

From a structural standpoint, concrete's most important properties are compressive strength, modulus of elasticity, and creep. Compressive strength is important to the structural engineer because it specifies what level of stress the concrete can withstand. The modulus of elasticity is important because it defines the relationship between stress and strain. This allows the design engineer to determine how much elastic deformation a structural element will experience under different loads. Creep is the long-term deformation that a material will experience under a constant load. No creep tests were performed for this project, however the test method will be briefly reviewed.

2.17 Factors Effecting Compressive Strength

Concrete is a composite material and its strength is effected by the properties of each one of its constituent raw materials as well as the proportions in which these materials are combined and the methods used to mix, place, consolidate, and cure the concrete. Assuming consistent mixing, placing, and curing methods, the main factors effecting concrete strength at a given age are the water/cement ratio, the aggregate, the cement, and the admixtures used.

2.17.1 Water/Cement Ratio and Porosity

Porosity is one of the most important factors that effect the strength of brittle materials such as concrete (48). Porosity is a measure of void space. Void space does not contribute to strength so a high porosity represents low strength. The water/cement ratio and air content control concrete's porosity. The water/cement ratio governs the amount of capillary voids that exist (42). When the cement and water in concrete are initially mixed together, the reaction between the two does not reach completion immediately. Rather, some of the mixing water reacts with cement to form hydration products and the remaining water resides in capillary pores which form within the matrix of hydration products and cement. As time passes and the cement continues to react with the water, the new hydration products form within the capillary pores. The initial water-cement ratio governs the porosity of the paste and the extent to which those pores become filled with hydration products. A high water/cement ratio causes greater porosity than a low water/cement ratio. This is because, with a high water/cement ratio, more pore space is required for mixing water and less space is taken up by hydration products due to a smaller amount of cement.

2.17.2 Aggregate

For normal weight aggregates, the main properties which influence strength are maximum size, shape, and texture. Each one of these parameters influences strength mainly through how they effect the bond between the aggregate and paste. For a given volume of coarse aggregate, a larger maximum size reduces the specific surface area of the aggregate. This reduces the bond strength and can have a detrimental effect on strength. A rough textured aggregate tends to have a better mechanical bond with the paste than a smooth aggregate (44). Because of this, crushed aggregate leads to higher concrete strengths than smooth aggregate. Aggregate shape can have a positive influence on concrete strength in the fact that it may increase aggregate surface area and therefore increase the bond strength. However, very irregular aggregate shapes may cause stress concentrations that lead to easier bond failure.

2.17.3 Cement

The main cement properties influencing concrete strength are chemical makeup and fineness of cement (44). Variations in both of these parameters influence the rate at which strength develops as well as the ultimate strength that is attained. Cement strength is mainly influenced by tricalcium silicate (C_3S) and dicalcium silicate (C_2S.) C_3S influences early strength and C_2S influences later strength. Concrete made with cement high in C_3S will have high rates of early strength gain but may experience lower strengths later on. Concrete made with cement high in C_2S will have higher strengths at later ages but lower strengths at early ages. Cements that hydrate more slowly tend to have higher strengths at later ages (44).

Cement fineness has a significant effect on the rate of strength gain. Very finely ground cements have a higher surface area per unit weight so they react more quickly with water. This causes a higher rate of early strength gain but may lead to lower strengths at later ages.

It should be noted that, even though ASTM has guidelines which govern the properties of different cement types, there can be considerable variation in cement characteristics between different plants and even between cement made at the same plant at different times. These variations come from the use of different raw materials and variation in production process. Certain estimates indicate that variations in cement quality lead to a coefficient of variation in concrete strengths of about 5% (44).

2.17.4 Admixtures

Three of the most commonly used admixtures are air entrainer, water reducers, and pozzolans. The way that these admixtures influence strength is through their effect on other properties of the concrete. Air entraining admixtures increase the amount of air voids in the concrete thereby increasing the porosity and decreasing the strength capacity. Water reducers reduce the amount of water necessary to achieve a given workability. This allows for a lower water-cement ratio which results in a higher strength. Pozzolans alter the chemical makeup of the cementitious portion of the concrete. "The effect of adding a pozzolan is to effectively raise the C_2S content of the cement" (44). The use of pozzolans often causes a low rate of early strength gain but may also result in higher strengths later on (44).

2.18 Compressive Strength of Cylindrical Concrete Specimens (ASTM C 39)

The purpose of this test is to determine the compressive strength of concrete specimens. This test was performed at 7 days, 14 days, 28 days, and 56 days after casting for each trial batch. For this test, cylindrical concrete specimens measuring 6 inches in diameter by 12 inches tall are cast and moist cured until testing. Neoprene pads are then placed on the ends of the specimen and a testing machine is used to apply a compressive axial load at a prescribed rate on the specimen until failure occurs. The maximum load attained during the test is recorded. The compressive strength of the concrete is determined by dividing the maximum load attained by the cross-sectional area of the specimen. Table 2.8 gives FHWA's recommendations for strength performance grades.

Table 2.8 - FHWA Compressive Strength Performance Grades*

Compressive Strength (x = strength)	FHWA Performance Grade
6000 psi $\leq$ x < 8000 psi	1
8000 psi $\leq$ x < 10000 psi	2
10000 psi $\leq$ x < 14000 psi	3
x $\geq$ 14000 psi	4

*Table based on Reference (37).

2.19 Factors Effecting Modulus of Elasticity

In practice, the modulus of elasticity is rarely determined by direct test but, instead, it is estimated using empirical relationships based on strength and density. Since there is a relationship between these two parameters and the modulus of elasticity, the same factors that affect strength and density also influence the modulus of elasticity. The main factors that influence the modulus of elasticity are the porosity of the cement paste and the amount and nature of the aggregate.

2.19.1 Porosity

As discussed earlier, the two properties that have the most influence on the porosity of the cement paste are water-cement ratio and air content. Higher water-cement ratios increase the porosity of the cement paste which reduces the concrete's ability to resist deformation due to applied loads and, therefore, reduces the modulus of elasticity. Air entraining admixtures increase the amount of air voids in the paste, thereby increasing the porosity and reducing the modulus of elasticity.

2.19.2 Aggregate

The main way that normal weight aggregates influence the modulus of elasticity is through the quality of the bond they form with the surrounding cement paste and through how well that bond is maintained when load is applied to the concrete. Surface texture is important in this respect. Compared to rough aggregates, aggregates with a smooth texture form a poorer bond with the cement paste (44). This makes it easier for micro-cracks to form at the aggregate-paste interface. One cause of micro-cracking in this interface zone is the shrinkage of the paste during the drying period. Another cause of micro-cracking is that, during loading, tensile and shear stresses form at the interface zone due differences in the elastic modulus of the aggregate and the paste (44). Formation of micro-cracks causes decreases in the modulus of elasticity.

Lightweight aggregates tend to have a lower modulus of elasticity than normal weight aggregates. This has a direct impact on the modulus of elasticity of concrete made with lightweight aggregates. For lightweight concrete, the modulus of elasticity is 40 to 80% that of normal weight concrete with the same compressive strength (44).

2.20 Static Modulus of Elasticity of Concrete in Compression (ASTM C 469)

The purpose of this test is to determine the chord modulus of elasticity (Young's modulus) of concrete in compression. This test was performed at a concrete age of 28 days after casting for each trial batch. For this test, cylindrical concrete specimens measuring 6 inches in diameter by 12 inches tall are cast and moist cured until testing. Before testing, the ends of the specimen are capped and a compressometer is placed on the specimen. Figure 2.4 shows a compressometer on a concrete cylinder.

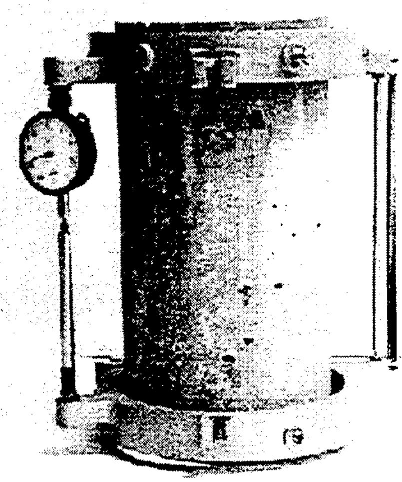

Figure 2.4 - Compressometer

The compressometer is a strain-measuring device used to measure axial deformations (strains) that take place when an axial force is placed on the specimen. The specimen is then placed in a testing machine and an axial load is applied at a prescribed rate until the applied load is equal to 40% of the ultimate load (obtained from ASTM C 39). The applied load is recorded at a strain equal to 50 millionths and the strain is recorded at 40% of the ultimate load. The chord modulus of elasticity is calculated as the slope of the chord connecting the point of 50 millionths strain to the point of 40% of the ultimate load on the concrete's stress vs. strain curve. Figure 2.5 shows a typical stress vs. strain curve for concrete as well as the chord elastic modulus. Each specimen is loaded three times. The first loading is done to make sure that the gage on the compressometer is seated properly. No readings are recorded during the first loading. Readings are taken during the two successive loadings and their results are averaged together. Table 2.9 gives FHWA's performance grades for modulus of elasticity.

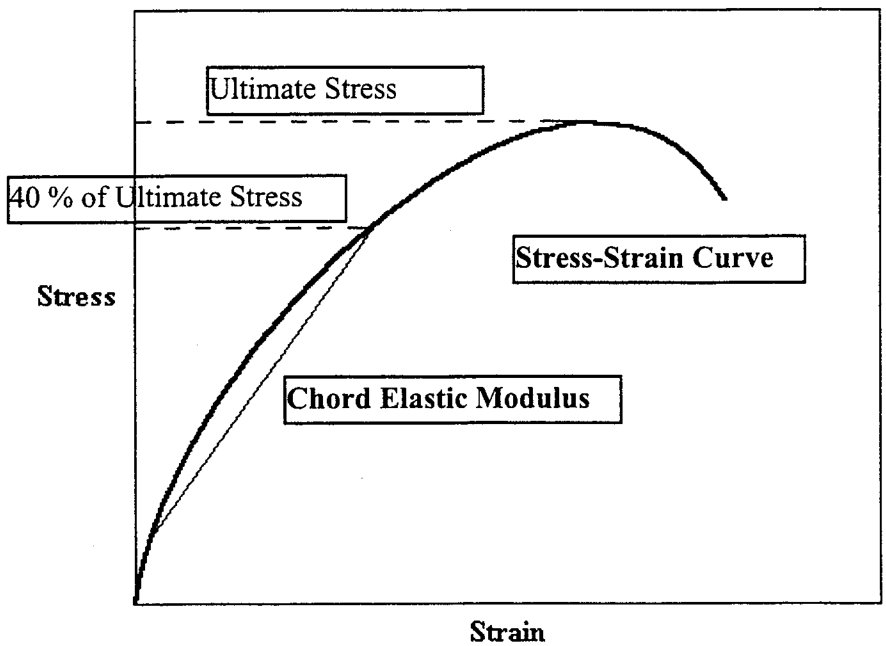

Figure 2.5 - Chord Elastic Modulus (Adapted from Reference (44)).

Table 2.9 - FHWA Performance Grades for Modulus of Elasticity*

Elasticity (x = modulus of elasticity)	FHWA Performance Grade
$4(10^6)$ psi $\leq x < 6(10^6)$ psi	1
$6(10^6)$ psi $\leq x < 7.5(10^6)$ psi	2
$x \geq 7.5(10^6)$ psi	3

*Table based on Reference (37).

2.21 Creep

Creep is the long term deformation of a material under constant load. In concrete, creep is an important factor because it can cause cracking and loss of prestress. Some factors which influence creep are the water/cement ratio, curing conditions, temperature, the moisture state of the cured concrete, cement composition, the use of admixtures, aggregate content, paste content, and the geometry of the concrete member (44). No creep tests were performed for this research project.

2.22 Standard Test Method for Creep of Concrete in Compression (ASTM C 512)

For this test, 6" diameter by 12" long concrete specimens are made and moist cured until testing. FHWA recommends moist curing the specimens for 28 days and then performing the test for an additional 180 days (37). When testing begins, the specimens are placed in a loading frame which applies a load equal to 40% of the concrete's ultimate load on the specimen. This load is maintained until the end of the test which should be after 180 days, according to FHWA, or after 1 year, according to ASTM C 512. The strain in the specimen is measured at the beginning and end of the test as well as at intervals in between. The amount of strain measured is an indication of the amount of creep that occurred in the specimen. Table 2.10 gives FHWA's performance grades for creep in concrete.

Table 2.10 - FHWA Performance Grades for Creep*

Creep (x = microstrains/psi)	Performance Grade
0.52/psi ≥ x > 0.41/psi	1
0.41/psi ≥ x > 0.31/psi	2
0.31/psi ≥ x > 0.21/psi	3
0.21/psi ≥ x	4

*Table based on Reference (37).

2.23 Summary

Results of the test program discussed in this chapter are given in chapter 4. The tests performed in this research project are meant to evaluate concrete in various areas of performance that are likely to be required in the field. However, while important information is gained from laboratory testing, it is important to realize that field conditions are highly variable and different from the controlled environment of the laboratory. In evaluating and comparing data from the laboratory and the field, it is important to recognize these differences.

CHAPTER 3
Materials, Mix Design, Mix Procedure, and Fresh Concrete Tests

3.1 Introduction

The final, long-term performance of concrete is largely influenced by its raw constituent materials and how those materials are combined. This chapter describes the properties of the raw materials used in this project as well as the mix design and mixing process used for trial batches. The tests used to determine the fresh concrete properties of trial batches are also described in this chapter.

3.2 Materials

For trial batch mix design, raw materials were chosen which represent materials commonly used and available in Nevada. The final set of trial mix designs includes various combinations of three sources of coarse aggregate, two sources of cement, and two sources of fly ash used at 0, 15, 20, and 25 % rates of replacement of cement. All of the trial mix designs incorporate the same source of fine aggregate as well as the same air entraining admixture and high range water reducer.

3.2.1 Portland Cement

The two Portland cement sources used in the trial mix designs are Nevada Cement and Calaveras Cement. Both cements are Type I/II low alkali cements meeting the requirements of ASTM C 150 (15) and the requirements of NDOT 701 (47). Cement test report results showing the specifications of both cement sources have been included in the appendix.

3.2.2 Fly Ash

Fly ash is a by-product of coal burning power plants. Fly ash is used as a mineral admixture because of its pozzolanic properties. Fly ash is classified into two ASTM classes, Class C and Class F. In Class C fly ash, the sum of SiO_2, Al_2O_3, and Fe_2O_3 is between 50% and 70%. Class C fly ash has a high calcium oxide content. In Class F fly ash, the sum of SiO_2, Al_2O_3, and Fe_2O_3 is greater than 70%. Class F fly ash has low calcium oxide content. Fly ash with high calcium content tends to gain more early strength than low calcium fly ash (46).

The two sources of fly ash used for this project are the Jim Bridger power plant in Wyoming and the Intermountain Power Service Corporation in Delta, Utah. Both fly ashes are supplied by ISG Resources, Inc. Both conform to the specifications for Class F fly ash given in ASTM C 618 and the specifications for pozzolanic admixtures set forth in NDOT 702.03.05. Chemical and physical analysis results for the two fly ash sources are included in the appendix. The trial mix designs use cement replacement with fly ash at rates of 0, 15, 20, and 25%. The cement is replaced by fly ash on a 1 to 1 ratio based on mass.

3.2.3 Fine Aggregate

All of the trial mix designs use fine aggregate from Paiute Pit Aggregates' Wadsworth Pit. This aggregate passes the limits for fine aggregate set forth in ASTM C 33 and in NDOT 706.03.03. The gradation and properties of this aggregate are shown in Table 3.1 and Figure 3.1.

Table 3.1 - Fine Aggregate Properties

Properties

Source	Paiute Pit
Bulk Specific Gravity (Dry)	2.39
Absorption(%)	4.1
Dry-Rodded Unit Weight	97.61
Fineness Modulus	2.88

Gradation

		NDOT Limits		ASTM C 33 Limits	
Sieve Size	% Passing	Lower	Upper	Lower	Upper
#4	99.42	95	100	95	100
#8	83.64			80	100
#16	67.16	45	80	50	85
#30	44.3			25	60
#50	15.12	10	35	10	30
#100	2.41	2	12	2	10

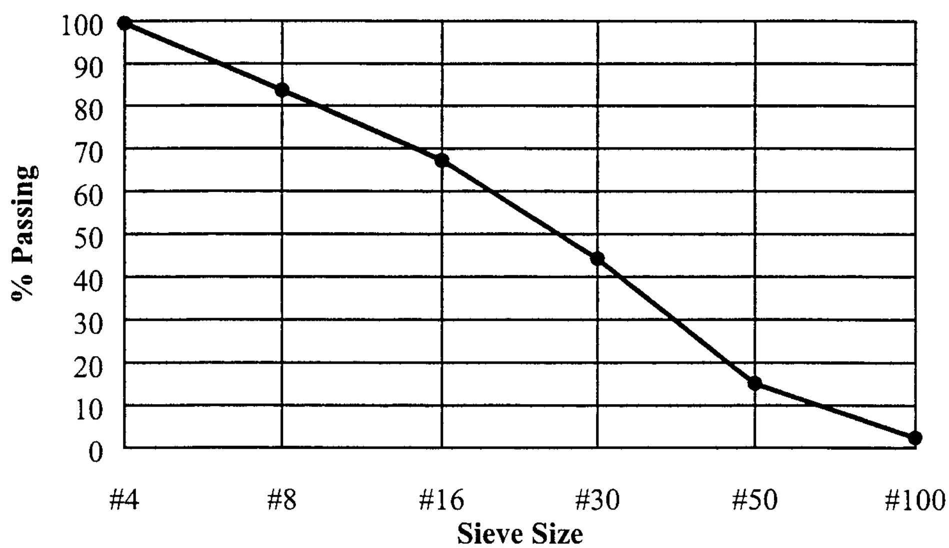

Figure 3.1 - Fine Aggregate Gradation

3.2.4 Coarse Aggregate

The three coarse aggregate sources are Paiute Pit Aggregates' Wadsworth Pit, All-Lite Pit in Lockwood, and Vega Construction's Elburz Pit. All three aggregate gradations have a ¾" maximum size. The Paiute and Vega aggregates are very similar in shape and texture. Both appear to be smooth rounded river gravel with an irregular shape which has been crushed to meet grading requirements. Paiute coarse aggregate is mainly composed tertiary volcanic rock form the Sierra Nevada mountain range and the Truckee river canyon (36). Vega aggregate is mainly composed of quartzite, chert, and siltstone (36). The All-Lite aggregate is a lighter crushed stone with an irregular angular shape and rough texture. All-Lite coarse aggregate is composed of Washington Hill rhyolite (36). These aggregate's gradations are given in Figures 3.2, 3.3, and 3.4. The properties of these aggregates are given in Table 3.2.

Table 3.2 - Coarse Aggregate Properties

Aggregate Source	Vega	Paiute	All-Lite
Bulk Specific Gravity (Dry)	2.558	2.491	2.292
Absorption(%)	1.32	3.87	5.64
Dry-Rodded Unit Weight (pcf)	96	95.1	82.01
Gradation Sieve Size	% Passing	% Passing	% Passing
1.5"	100	100	100
1"	100	100	100
0.75"	86.68	90.84	86.89
0.5"	45.34	42.34	39.49
3/8"	19.33	16.18	20.32
#4	2.48	1.26	3.33

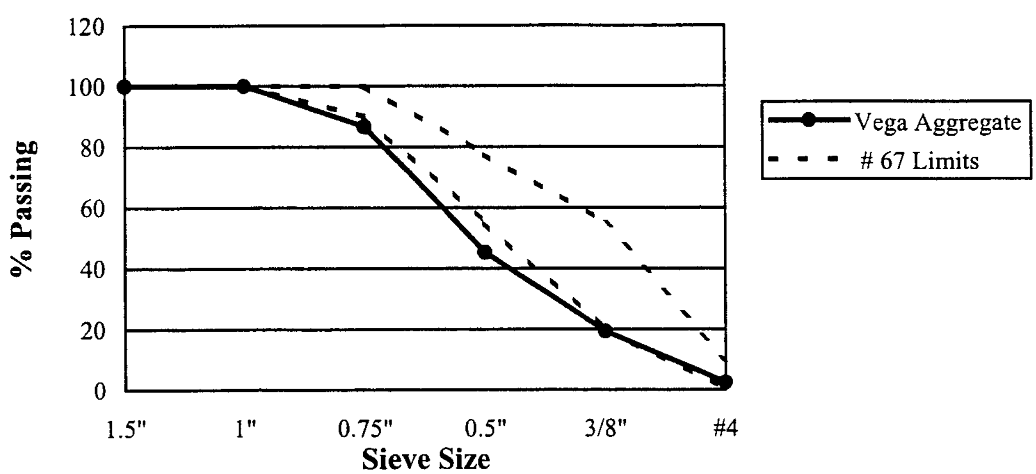

Figure 3.2 - Vega Coarse Aggregate Gradation

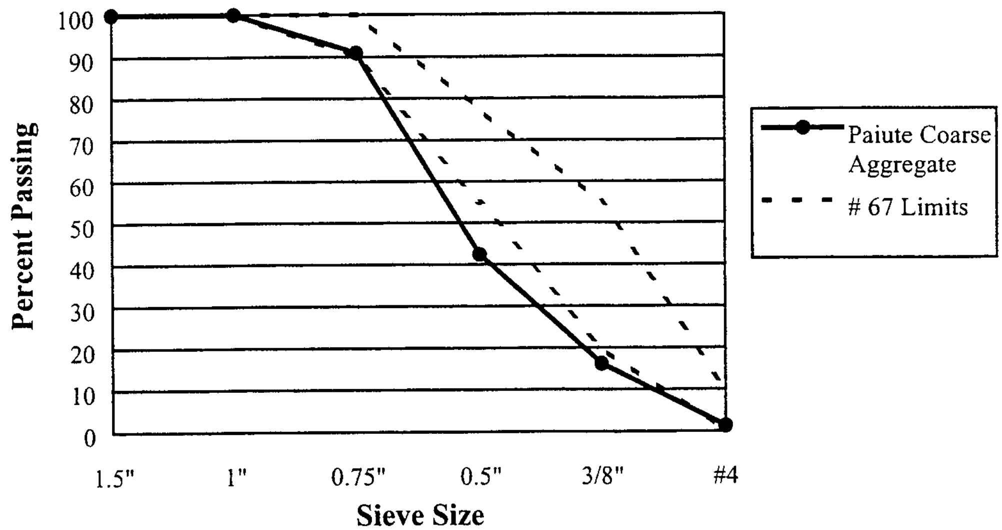

Figure 3.3 Paiute Coarse Aggregate Gradation

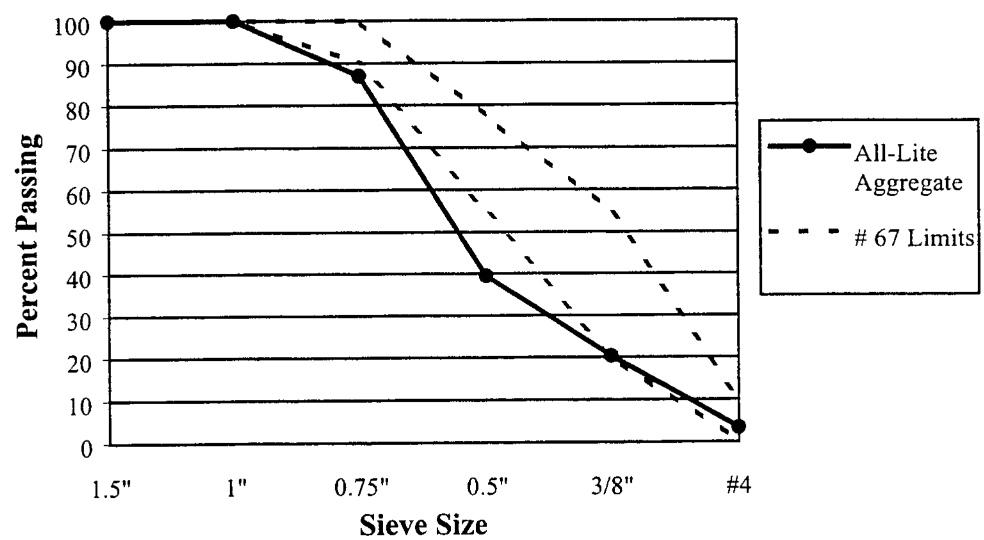

Figure 3.4 - All-Lite Coarse Aggregate Gradation

3.2.5 Chemical Admixtures

All mix designs incorporate the use of W.R. Grace Daravair 1000 air entrainment admixture and W.R. Grace Daracem 19 high range water reducer. Both of these admixtures are in liquid form. Daravair 1000 conforms to the specifications of ASTM C 260 and the specifications for air entraining admixtures in NDOT 702.03.02. Daracem 19 conforms to the specifications of ASTM C 494 and the specifications in NDOT 702.03.03. The rates of admixture addition vary between mix design based on the specific properties of each batch. Table A3.3 in the appendix shows the exact rates of admixture addition used in each trial batch.

3.2.6 Water

The water used in this project is from the City of Reno. No tests were performed to determine the suitability of this water, but it is assumed to be adequate because it meets safe drinking water standards.

3.3 Trial Mix Design Overview

For this research, a total of 37 different trial concrete mixes were designed. The range of trial mixes includes various combinations of the three coarse aggregates, the two cements, and the two fly ashes.

3.3.1 Mixture Type Designation

Each mix design is identified by a mixture designation based on cement source, fly ash source, percent of cement replacement with fly ash, and coarse aggregate source. The following Table identifies the symbols used for mixture identification.

Table 3.3 - Mix Identification Symbols

Material Component	Identification Symbol
Cement	
Nevada	N
Calaveras	C
Fly Ash	
Bridger, Wyoming	B
Delta, Utah	D
Percent Cement Replacement with Fly Ash	
0%	0
15%	15
20%	20
25%	25
Coarse Aggregate Source	
Vega Pit	V
Paiute Pit	P
All-Lite Pit	A

As an example, consider a mixture containing Nevada cement, Bridger fly ash used to replace cement at a rate of 20%, and Paiute aggregate. The symbol for Nevada cement is "N," the symbol for Bridger fly ash at 20 % replacement is "B20," and the symbol for Paiute coarse aggregate is "P." The mixture designation for this combination of materials is "N-B20-P."

As a second example, take a mixture with Calaveras cement, no fly ash, and All-Lite coarse aggregate. The symbol for Calaveras cement is "C," the symbol for no fly ash is "0," and the symbol for All-Lite coarse aggregate is "A." The mixture designation for this mix design is "C-0-A."

3.3.2 Research Mix Designs

The mix designs used in this research did not include all of the possible combinations of the materials chosen for use in this project. For mixes with Vega or Paiute aggregates, Nevada cement is used with all 4 rates of addition of the 2 fly ash sources. For the Vega and Paiute coarse aggregate mixes, Calaveras cement is only used in selected combinations of fly ash. This was done in order to limit the number of required trial batches. Even though this limits the amount of data, it still provides enough data to compare the behavior of different combinations of fly ash and cement for each coarse aggregate sources. For mixes with All-Lite aggregate, all possible combinations of cement and fly ash are used. This was done so that, for one coarse aggregate source, it is possible to compare every combination of fly ash and cement. Table A3.1 in the appendix lists all of the mix designs completed for this project. Table A3.2 in the appendix gives a summary of the quantities of materials used for each trial batch. Table A3.3 in the appendix gives the rates of admixture addition used for each trial batch as well as the fresh concrete properties of each trial batch.

3.4 Mix Design Procedure

Table I of NDOT section 501 specifies the use of class EA Modified concrete for roadway deck slabs, slab spans of bridges, approach slabs, and barrier rails. With the exception of aggregate gradation, the basic proportions and properties required for class EA Modified concrete were used in the mix designs for this project. The selection of mix proportions used for mix design followed the procedures and guidelines set forth in ACI 211.1, "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete" (4).

3.4.1 Slump

The first step is to choose the desired slump. Slump is a measure of the concrete's workability and is an important factor relating to ease of placement and consolidation. Slump is effected by water/cement ratio, aggregate shape, size, and texture, cement content, admixtures, and air content. ACI 211.1.6.3.1 states that mixes with the "stiffest consistency that can be placed efficiently should be used."

Table 6.3.1 in ACI 211.1 recommends using a slump between 1" and 3" for pavements and slabs. Table I of NDOT 501 requires a slump range of 1" to 2 ½" for Modified EA concrete before the addition of high range water reducer. For the mix designs used in this project, a target slump of 1" was used for mix proportioning and an allowable slump range of 1" to 2 ½" was used as acceptance criteria for trial batches. Both the values for the target slump and the criteria for acceptable slump range are for the concrete before the addition of high range water reducer.

3.4.2 Maximum Size of Aggregate

The maximum size of aggregate effects the strength of concrete and how well the concrete may be consolidated. For a given water/cement ratio, smaller maximum sizes of aggregates yield higher strength concrete. ACI 211.1.6.3.2 requires that the maximum size of the aggregate should not "exceed one-fifth of the narrowest dimension between sides of forms, one-third the depth of slabs, nor three-fourths of the minimum clear spacing between individual reinforcing bars." This is to ensure that the concrete can pass easily between reinforcing bars and into the forms. This also helps avoid segregation and the formation of voids and pockets within the concrete.

Table I in NDOT 501 specifies the use of coarse aggregate graded to size number 57 specifications for gradation. According to NDOT 706.03.01, size number 57 specifies a maximum aggregate size of 1". Based on conversations with people in NDOT's materials division, it was decided that, for this project, size number 67 coarse aggregate should be used. Size number 67 coarse aggregate has a maximum size of ¾". Table 3.4 gives NDOT specifications for size number 57 and size number 67 coarse aggregate gradation. Because it has a smaller maximum size than the size number 57 aggregate, size number 67 aggregate should yield stronger concrete which is more workable at a given water/cement ratio.

Table 3.4 - NDOT Coarse Aggregate Gradation Specifications

Sieve Size	NDOT Size # 67 Limits		NDOT Size # 57 Limits	
	Lower	Upper	Lower	Upper
1.5"			100	100
1"	100	100	95	100
0.75"	90	100		
0.5"			25	60
3/8"	20	55		
#4	0	10	0	10

It should be noted that the aggregate samples obtained from the three coarse aggregate sources did not meet size number 67 specifications. While they all passed the requirement for maximum size, they all failed to pass at least one of the requirements for a smaller sieve size. Specifically, the Vega coarse aggregate fails to pass the requirements for the ¾ inch and 3/8 inch sieves. For both of these sieves, the percent of coarse aggregate passing the sieve is slightly below the lower limit for size number 67. The Paiute coarse aggregate fails to pass specifications for the 3/8 inch sieve. The percent passing is below the specified range for size number 67. The All-Lite coarse aggregate fails to pass the specifications for the ¾ inch sieve. The percent passing is below the specified range. All of the coarse aggregate samples pass the specifications for size number 57 coarse aggregate.

Because all three of the coarse aggregates have a relatively similar gradation and they are close to the specified gradation, they have been used in this research even though they do not pass specifications for size number 67 gradation. The gradations for all three coarse aggregates are given in Table 3.2 and Figures 3.2, 3.3, and 3.4.

3.4.3 Air Content

The amount of air entrained in concrete effects the workability of the concrete and how well the concrete will resist damage due to freeze-thaw action. NDOT 501 specifies an air content between 5 % and 7 % for Modified EA class concrete. This is consistent with Table 6.3.3 of ACI 211.1 which recommends an air content of 6 % for concrete with severe freeze-thaw exposure. For this project, a target air content of 6 % is used for mix proportioning and an air content range of 5 % to 7 % is used for acceptance of trial batches.

3.4.4 Estimation of Mixing Water

The mixing water is the total amount of water that will be available to mix with the cement. ACI 211.1.6.3.3 states that, for a given slump, the amount of mixing water required depends on "maximum size, particle shape, and grading of the aggregates; the concrete temperature; the amount of entrained air; and use of chemical admixtures." Table 6.3.3 of ACI 211.1 recommends the use of 280 pounds of water per cubic yard of concrete for a 1" to 2" slump with the use of ¾" maximum size aggregate. NDOT makes no specification for mixing water. For mix proportion purposes, a value of 280 pounds of water per cubic yard of concrete was used.

Mixing water is the total amount of water that will mix with the cement. This means that, depending on the moisture state of the aggregates, the actual amount of water that is added to the mix has to be adjusted. For example, if the aggregates are at a state in which their moisture content is greater than the saturated-surface-dry condition, then the excess moisture on the aggregate will contribute to the mixing water. In this case the actual amount of water added to the mix will be less than the estimated amount of mixing water. If, on the other hand, the aggregates are dryer than the saturated-surface-dry condition, they absorb water from the mixture and reduce the amount that is available to mix with the cement. For this condition, the actual amount of water added must be increased to compensate for the water absorbed by aggregate. For the trial batches produced in this project, tests for the moisture content of the aggregates were performed before final water adjustments were made to the mix designs.

3.4.5 Design Strength

For Modified EA class concrete, Table I of NDOT 501 states that the minimum compressive strength of concrete at an age of 28 days is to be "specified on plans." Based on discussions with people in the structural and materials divisions at NDOT, it was decided to use a design strength of 5000 psi at 28 days.

3.4.6 Selection of Water/Cement Ratio

The water/cement ratio (w/c) is the ratio of mixing water to cementitious materials based on mass. For this project, the cementitious materials consist of cement and fly ash. The water/cement ratio has a large effect on the porosity of concrete so it effects strength and permeability. Table I of NDOT 501 specifies a maximum water/cement ratio of 0.44 for Modified EA class concrete. Table 6.3.4(a) of ACI 211.1 recommends the use of a water/cement ratio of 0.40 for air entrained concrete with a desired 28 day compressive strength of 5000 psi. Table 6.3.4 (b) of ACI 211.1 recommends the use of a maximum water/cement ratio of 0.45 for thin sections in severe exposures. For this project, the maximum water/cement ratio is taken as 0.40.

3.4.7 Determination of Cement Content and Amount of Fly Ash

The amount of cementitious materials to be used is indirectly determined based on the selected amount of mixing water and the water/cement ratio. Because the amount of water and the ratio of water to cementitious materials are known, the amount of cementitious materials may be calculated.

The cementitious materials used in this project consist of fly ash and cement. Fly ash is used to replace a certain percentage of the cement. For this project, the calculation of cement replacement is carried out based on the weight equivalency method outlined in ACI 211.1.6.3.4. Essentially, this method is a direct replacement of a certain weight of cement with the same weight of fly ash. The method does not take the differences in specific gravity between cement and fly ash into account in the calculation of the amount fly ash. Because a certain mass of fly ash occupies more volume than the same amount of cement, the total volume of cementitious materials in a mix with fly ash is greater than the volume of cementitious materials in a mix without fly ash. This greater volume of cementitious materials is compensated by a reduction in the volume of fine aggregate.

It should be noted that, while NDOT does allow for cement replacement with pozzolans such as fly ash, it only allows for a maximum replacement of 17% by mass of the required amount of Portland cement (NDOT 501.02.03.) Several of the mix designs created for this project use cement replacement rates of 20% or 25% and therefore do not conform to NDOT specifications. Also, NDOT 501.02.03 specifies that "the replacement rate of cement with pozzolan shall be at a rate of 1.2 [lb.] of pozzolan for each [lb.] of Portland cement." For this project, the cement replacement with fly ash is at a rate of 1 lb. of fly ash for each 1 lb. of cement. This method of cement replacement does not conform to NDOT standards.

NDOT 501 specifies a cement content range of 600 to 750 lbs/yd^3 for Modified EA class concrete. All of the mix designs in this project conform to this specification with a total weight of cementitious materials equal to 700 lbs/yd^3.

3.4.8 Coarse Aggregate Content

The amount of coarse aggregate has a significant influence on the workability of the mix. Table 6.3.6 of ACI 211.1 gives suggested values for the volume of coarse aggregate per unit volume of concrete to yield "a degree of workability suitable for usual reinforced construction." The values in this table are volumes of oven-dry-rodded coarse aggregate and are dependent on the maximum size of coarse aggregate and the fineness modulus of the fine aggregate. The fine aggregate used has a fineness modulus of 2.88 so, according to ACI, a volume of coarse aggregate between 0.60 and 0.62 of the volume of concrete should be used (4). For this project, a slightly higher value of 0.65 is used for the volume of coarse aggregate as a fraction of the total volume of concrete. To determine the actual weight of oven-dry coarse aggregate, the value of 0.65 must be multiplied by the desired volume of concrete and the oven-dry-rodded unit weight as determined by ASTM C 29. The actual weight of coarse aggregate in its field moisture condition is determined by adjusting the weight of the oven dry aggregate according to the actual aggregate moisture content.

3.4.9 Fine Aggregate Content

The fine aggregate content is determined by the absolute volume method described in ACI 211.1.6.3.7.2. Based on the mix proportions already determined, it is possible to calculate the volumes displaced by the cementitious materials, water, air, and coarse aggregate for a unit volume of concrete. The required volume of fine aggregate is calculated as the volumetric difference between a unit volume of concrete and the sum of the volumes of the constituent material excluding fine aggregate. Similar to the coarse aggregate, the actual weight of fine aggregate must be adjusted according to field moisture conditions.

3.4.10 Combined Aggregate Properties

NDOT 706 gives grading limits for the combination of coarse and fine aggregates. These limits, and the combined gradations of aggregates for concrete mixes included in this research are given in Table 3.5 and Figure 3.5.

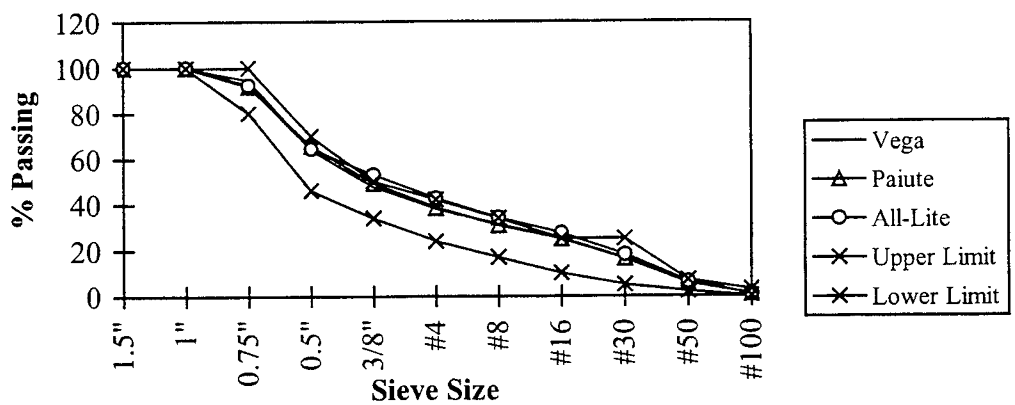

Figure 3.5 - Combined Aggregate Gradations

Table 3.5 - Approximate Combined Gradations

	Coarse Aggregate Source			NDOT Limits	
	Vega	Paiute	All-Lite	Lower Limit	Upper Limit
Sieve Size	%Passing	%Passing	%Passing	%Passing	%Passing
1.5"	100	100	100	100	100
1"	100	100	100	100	100
0.75"	94.26	91.61	92.26	80	100
0.5"	63.91	65.56	64.30	46	70
3/8"	47.53	49.18	52.99*	34	50
#4	37.97	38.35	42.73*	24	42
#8	31.28	30.95	34.29*	17	34
#16	25.12*	24.85	27.54*	10	25
#30	16.57	16.39	18.16	5	25
#50	5.65	5.59	6.20	2	7
#100	0.90	0.89	0.99	0	3

*Gradation specifications are violated.
Actual combined gradations vary slightly depending of fine aggregate content.

It should be noted that the combined gradations for all three coarse aggregates with the fine aggregate are very close to the upper limit and, for certain sieve sizes, the combinations with Vega or All-Lite coarse aggregate exceed this limit. Because these aggregate combinations are close to the upper limit and similar to one another, they have been included in this research even though they do not meet specifications.

3.4.11 Air Entrainment Admixture Quantities

For the air entraining admixture, Daravair 1000, the quantity to be used was based on trial and error as well as the air content results from previous trial batches. Throughout the course of this project, the required quantity of air entraining admixture varied between 1.5 and 5 ounces of admixture per 100 pounds of cementitious materials. The exact cause and factors effecting this variation are unknown. The exact amount of air entrainment admixture used for each trial batch are given in Table A3.3 in the appendix. Because of differences in mix procedure and batch size, concrete batches produced at a batch plant would probably require different quantities of air entrainment admixture than those used for this research project.

3.4.12 High Range Water Reducer

For purposes of the mix design, a high range water reducer addition rate of 8 ounces per 100 pounds of cementitious materials was used. However, the actual amount of high range water reducer added varies based on the slump of the concrete before its addition. Based on discussions with people in NDOT's materials division, a target slump of 5" was used for concrete after the addition of high range water reducer. As stated in section 3.2.4.1 of this report, a slump range of 1" to 2 ½" was used as acceptance criteria for concrete before the addition of high range water reducer. Because of this variation in allowable initial slump (the slump before addition of high range water reducer), the amount of high range water reducer required to raise the slump to 5" will also vary. The exact rate of high range water reducer addition used in each trial batch is given in Table A3.3 in the appendix. Similar to air entrainment admixture, the quantities of high range water reducer used in this project would probably have to be adjusted for concrete production at a batch plant.

3. 5 Materials Storage

The cement was stored in a dry environment and kept in paper cement sacks until use. The fly ash was stored in sealed plastic 5 gallon buckets until use. Both coarse and fine aggregates were stored in sealed plastic 55 gallon drums until use. The coarse aggregate seemed to maintain a relatively consistent moisture content within each drum but the moisture content of the fine aggregate in each drum varied greatly. The moisture in the fine aggregate drums settled to the bottom so that the moisture content at the top of the drum was much lower than the moisture content at the bottom. To account for the fact that the moisture states of the aggregates vary from the saturated-surface-dry condition, the mix designs must incorporate moisture content.

Before each mix design was completed, the moisture contents of the aggregates had to be determined. For the coarse aggregate moisture test, representative samples were taken directly from the storage drum. The moisture content test for coarse aggregate followed the procedure set forth in ASTM C 566 with the exception of the specified sample size. ASTM C 566 requires an aggregate sample weighing a minimum of 3 kg or about 6.6 lb. The coarse aggregate moisture content tests done for this project used aggregate samples weighing approximately 3.3 lb.

For the fine aggregate moisture test, samples were prepared by first sealing a quantity of the sand in a metal 55 gallon drum and then tipping the drum on its side and rolling it repeatedly to mix the sand and attain a uniform moisture content. A moisture content sample was then taken from the drum and tested for moisture content according to ASTM C 566.

3. 6 Mix Preparation

Before mixing a trial batch, the materials were weighed out into 5 gallon buckets and plastic cylinder molds and covered to prevent moisture loss. For each trial batch, enough materials were weighed out to make one 4 ft^3 batch and one ¼ ft^3 batch with the same proportions. The ¼ ft^3 batch was used to coat or "butter" the inside of the mixer in order to help prevent material from sticking to the sides of the mixer. The 4 ft^3 batch was used to make test specimens. Before the 4 ft^3 batch was made, the excess material from the ¼ ft^3 batch was taken out of the mixer and wasted.

3. 7 Mixing Equipment

The mixer used to make concrete for this project is an electric Whiteman model WC-92S. This concrete mixer has a 12.35 ft^3 capacity steel drum and a 1.5 HP electric motor. The actual mix capacity of this mixer is 9 ft^3.

3. 8 Mix Procedure

The following procedure was used to mix all of trial batches for this project. After the drum was "buttered", all of the coarse aggregate, ½ of the sand, and 1/3 of the water was added to the mixer. The drum was then turned a few times to mix the aggregates. Next all of the cement and fly ash were placed in the mixer with the rest of the sand. One-half of the remaining water mixed with the air entrainment admixture was poured on top of the sand. The mixer was then turned on and the rest of the water was added. It should be noted that this water was added incrementally until the concrete appeared to have reached the desired consistency and stiffness. If the concrete attained the desired consistency before all of the mixing water was added then water addition stopped.

As soon as the concrete reached a homogenous state with the desired consistency, it was mixed for an additional 2 minutes. Then the mixer was stopped for 1 minute followed by 2 more minutes of mixing. At this point a slump test was performed. If the slump was less than the minimum requirement of 1 inch, then some of the remaining mixing water would be added to the concrete and it would be mixed for 1 additional minute. This was followed by an additional slump test.

As soon as an acceptable initial slump was achieved, the high range water reducer was poured on the concrete in the mixer and the mixer was turned on for 1 more minute. At this point, another slump test was performed. If the slump was too low then more high range water reducer was added followed by 1 minute of mixing and another slump test. As soon as the slump was at an acceptable level, the air content test was performed and all of the test specimens were made.

3. 9 Fresh Concrete Tests

The fresh concrete tests performed for this project are slump, air content, temperature, and unit weight. The values determined by these tests are summarized in Table A3.3 in the appendix.

3.9.1 Test Method for Slump of Hydraulic Cement Concrete (ASTM C 143)

The slump test was performed according to ASTM C 143. The slump test is used as an indicator of the consistency, stiffness, and workability of concrete. The equipment necessary for this test are a slump mold and a 5/8 in diameter steel tamping rod. The slump mold is made of metal and shaped like the frustum of a cone with a base diameter of 8" and a top diameter of 4". For the test, the mold is placed on a flat, rigid surface and filled with concrete in three layers.

Each layer is rodded with 25 strokes of the tamping rod and the top of the concrete is made level with the top of the mold. Then the mold is lifted off the concrete allowing the concrete to settle. The mold is placed next to the concrete and the slump is measured as the difference between the height of the mold and the height of the concrete.

For this project, a slump range of 1" to 2 ½" was used as acceptance criteria for concrete before the addition of high range water reducer. A slump range between 4" and 6" was used for acceptance of concrete after the addition of high range water reducer.

3.9.2 Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method (ASTM C 173)

The procedure to determine the air content of fresh concrete followed the procedure of ASTM C 173. This test is a volumetric method to determine air content.
The main piece of equipment used for this test is the airmeter. The airmeter consists of a bowl and a top section which fits on top of the bowl. The top section contains a transparent scale which is graduated to measure a range of 0% to 9% of the volume of the bowl.

For the test, the bowl is filled with three equal layers of concrete. Each layer is rodded with 25 strokes of a 5/8" steel tamping rod. For each layer the side of the bowl is tapped 10 to 15 times with a rubber mallet to close any voids in the concrete. After the bowl is filled, a strike off bar is used to finish the top of the concrete. Next, the top section is attached to the bowl and a funnel is used to fill water up to the 0% mark on the transparent scale. The top section is then sealed with a lid and the airmeter is inverted several times and rolled on the ground repeatedly to release all of the entrained air from the concrete. As air is separated from the concrete, the liquid level in the transparent scale drops accordingly. The air content of the concrete is read directly from the scale. For this project, an air content range of 5% to 7% was used as acceptance criteria for trial batches.

For this project, the airmeter was rolled extensively and repeatedly for each trial batch test to ensure that all of the entrained air became separated from the concrete. Also, because of foam buildup in the scale, a calibrated cup was used to add isopropyl alcohol to the airmeter. This helps clear up the foam, allowing a clear view of the liquid level in the scale. When alcohol is added, the air content is attained by adjusting the value read off the scale by the amount of alcohol added.

3.9.3 Test for the Temperature of Freshly Mixed Concrete

The procedure for this test followed the procedure of ASTM C 1064 with the exception of the specifications for the temperature measuring device. ASTM C 1064 requires the use of a temperature measuring device which has been calibrated using reference temperature measuring device. The temperature measuring device used for this test is a digital readout thermometer and it was not calibrated according to ASTM standards.

For this test, the thermometer is placed in a container containing concrete so that the temperature sensor will have a minimum of 3" of cover concrete in all directions. The temperature reading is taken as soon as the temperature has stabilized.

3.9.4 Test for Unit Weight of Fresh Concrete

The test method for unit weight used in this research did not follow an established standard method. The test specimens made for the compressive strength test were used to determine unit weight. These specimens are 6" diameter cylinders and were made according to method set forth in ASTM C 39. The plastic cylinder molds are filled in three equal layers. Each layer is rodded with 25 strokes of a 5/8" tamping rod and the mold is tapped 10 to 15 times with a rubber mallet for each layer. The top of the specimen is finished with a strike off bar. To determine unit weight, a cylinder mold containing fresh concrete was weighed. The weight of the empty plastic cylinder mold was subtracted from the combined weight of the concrete and mold to determine the weight of the concrete. The unit weight was then calculated as the weight of the concrete divided by the volume of a 6" diameter by 12" long cylinder.

3.10 Trial Batches

The total number of trial mix designs made for this project is 37. However, during trial batch production, several of the trial batches did not meet the acceptance criteria for fresh concrete. These trial batches, which failed either slump or air content criteria, had to be redone until they met acceptable levels of performance. A total of 51 trial batches were made. The trial batches which did not meet acceptance criteria were not wasted but, instead, were tested along with the acceptable batches.

CHAPTER 4
Results

4.1 Introduction

Test results for all of the trial batches are given in the appendix. For certain mix designs, several trial batches were made before acceptable fresh concrete performance was achieved. Unless stated otherwise, all of the results discussed in this chapter are from the acceptable trial batches. Table A4.1 in the appendix indicates which trial batches have been deemed as acceptable. This chapter will present the test results and provide some discussion as to how the mix design variables effect the long term performance of the concrete.

4.2 Compressive Strength (ASTM C 39) Results

Table A4.2 in the appendix gives the compressive strength results for all of the trial batches. Unless noted otherwise, each of the results given in Table A4.2 is the average test result from three specimens broken at the same age.

The 28-day strengths for the trial batches produced for this project range from about 3500 psi to 6500 psi. The "56-day" strengths range from about 4500 psi to greater than 7500 psi. Based on this very general statement alone, it can be seen that a strength gain of around 1000 psi after 28 days is not unusual.

4.2.1 Effect of Cement Source on Strength

Figures 4.1 through 4.6 show how strength is affected by cement source, fly ash source and percent replacement for each coarse aggregate source. Figures 4.1 and 4.2 show the 28-day and 56-day strengths for concrete made with Vega coarse aggregate. As the figures show, for concrete made with Vega coarse aggregate, Nevada cement mixes tend to have higher 28 and 56-day strengths than Calaveras cement mixes. There is a similar trend for concrete made with All-Lite coarse aggregate. Figures 4.3 and 4.4 show that, for All-Lite aggregate concrete, Nevada cement mixes tend to be stronger than Calaveras cement mixes for both 28 and 56-day ages. Paiute coarse aggregate mixes, however, tend to exhibit the opposite trend. Figures 4.5 and 4.6 show that Calaveras cement mixes tend to have higher 28 and 56-day strengths than Nevada cement mixes for mix designs containing Paiute coarse aggregate.

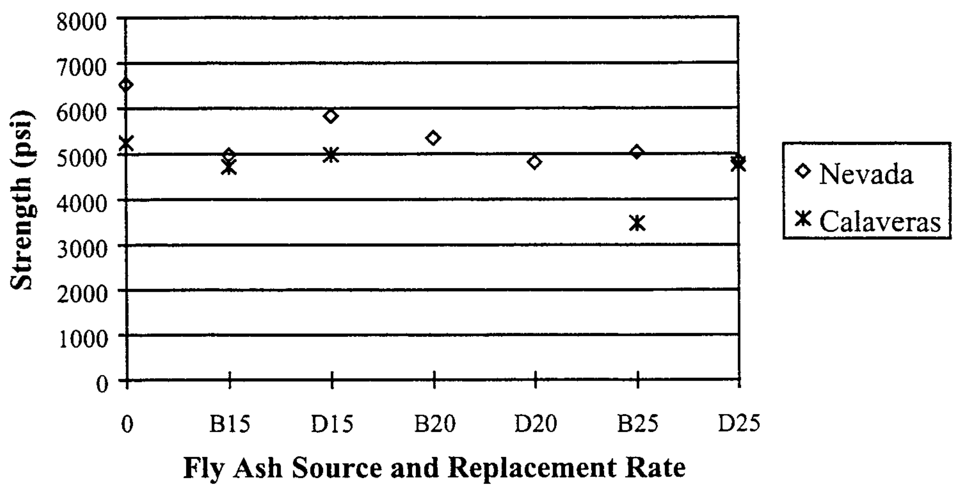

Figure 4.1 - 28-Day Strengths for Vega Aggregate Concrete

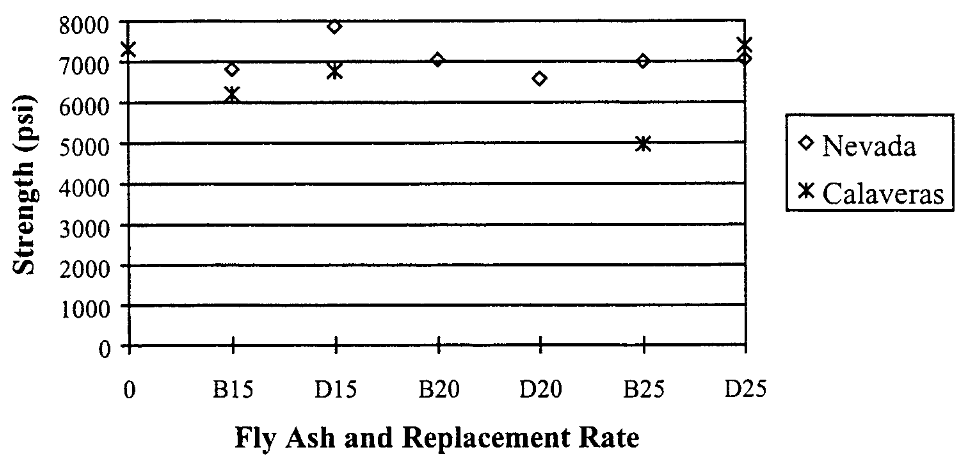

Figure 4.2 - 56-Day* Strengths for Vega Aggregate Concrete

***Actual test age is greater than 56 days.**

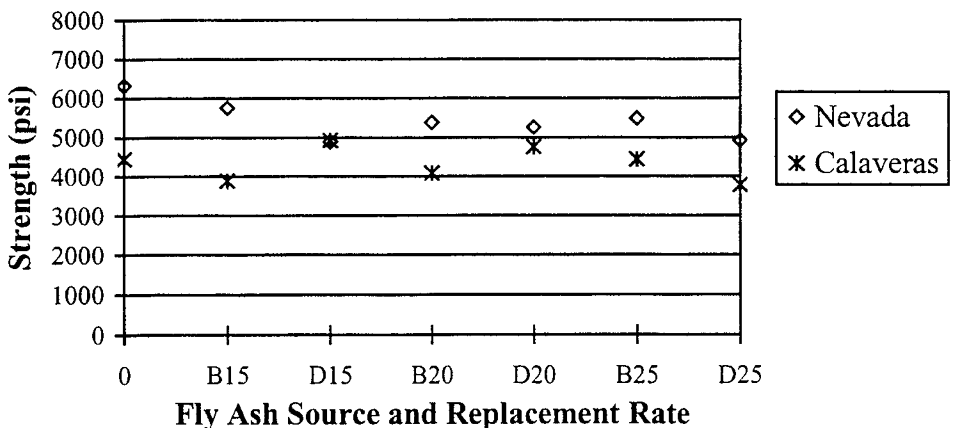

Figure 4.3 - 28-Day Strengths for All-Lite Aggregate Concrete

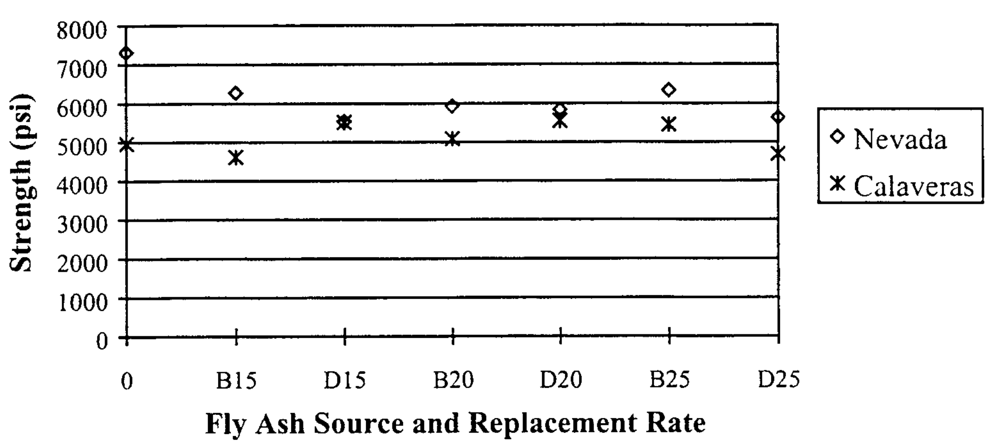

Figure 4.4 - 56-Day Strengths for All-Lite Aggregate Concrete

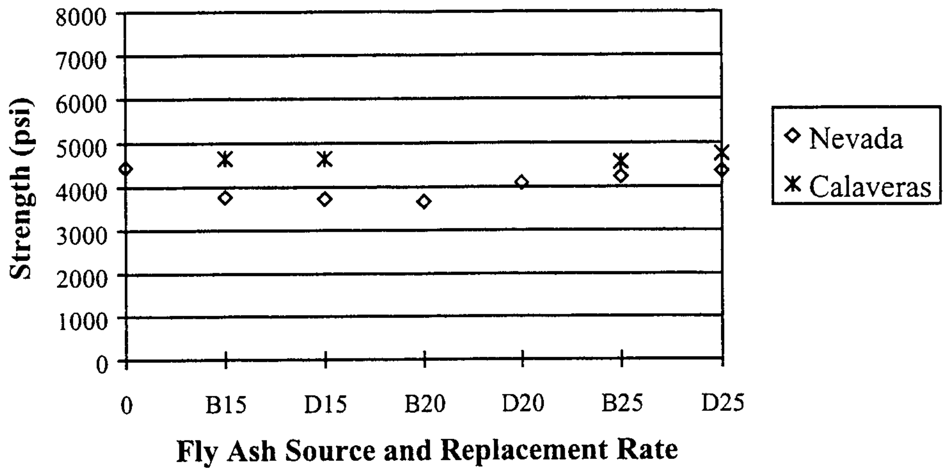

Figure 4.5 - 28-Day Strengths of Paiute Aggregate Concrete

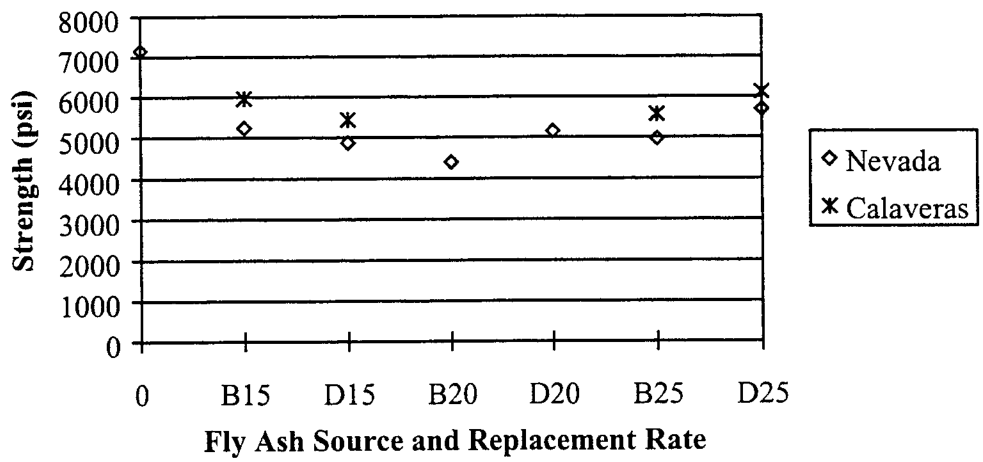

Figure 4.6 - 56-Day* Strengths for Paiute Aggregate Concrete
***Actual test age is greater than or equal to 56 days.**

The reason for this behavior is unclear. It would be expected that for aggregates with similar gradations, the relative effect of different cement sources would be similar, but this is not the case. The results of this research seem to indicate that there is some type of interaction between the coarse aggregate and the cementitious materials which influences the concrete strength. It is normally assumed that there is no chemical interaction between the aggregates and the cement which affect strength. However, the literature indicates that there may be some impact on strength due to such a reaction (44).

The results of this research do not yield any definite trends which relate fly ash source to strength. The literature indicates that pozzolans, such as fly ash, may cause lower rates of strength gain at early ages as well as higher strengths at later ages (44). Based on the data collected for this project, it is difficult to establish any trends between fly ash and strength. It is shown that no strength is lost due to replacement of portland cement with fly ash.

4.2.2 The Effect of Coarse Aggregate Source on Strength

Figures 4.7, 4.8, 4.9, and 4.10 compare how the coarse aggregate source effects strength for different combinations of cement and fly ash. Figures 4.7 and 4.8 give the 28-day and 56-day strengths for Nevada cement combinations. These figures show that, at 28 and 56 days, the Paiute aggregate concrete tends to have the lowest strength. At 28 days, there is no definite trend between the Vega and All-Lite aggregate mixes but, at 56 days, Vega aggregate mixes appear to have the highest strength regardless of the amount and source of fly ash used.

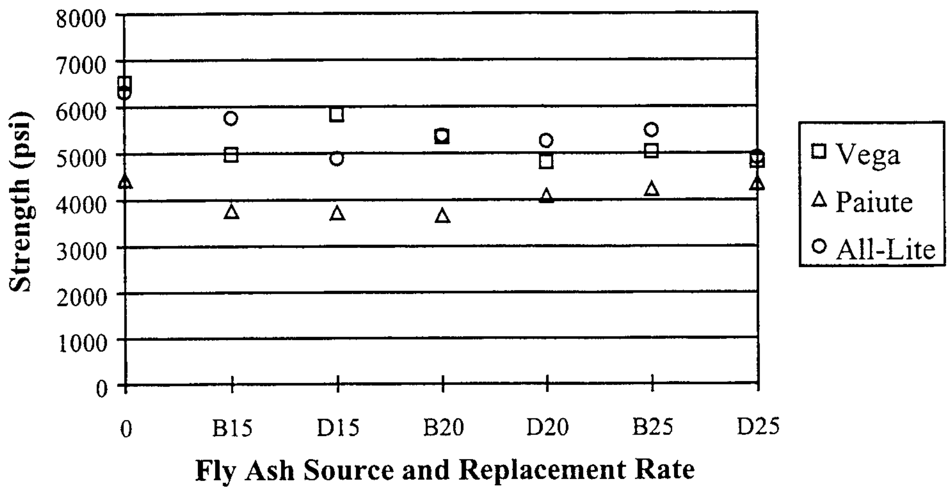

Figure 4.7 - 28-Day Strengths for Nevada Cement Concrete

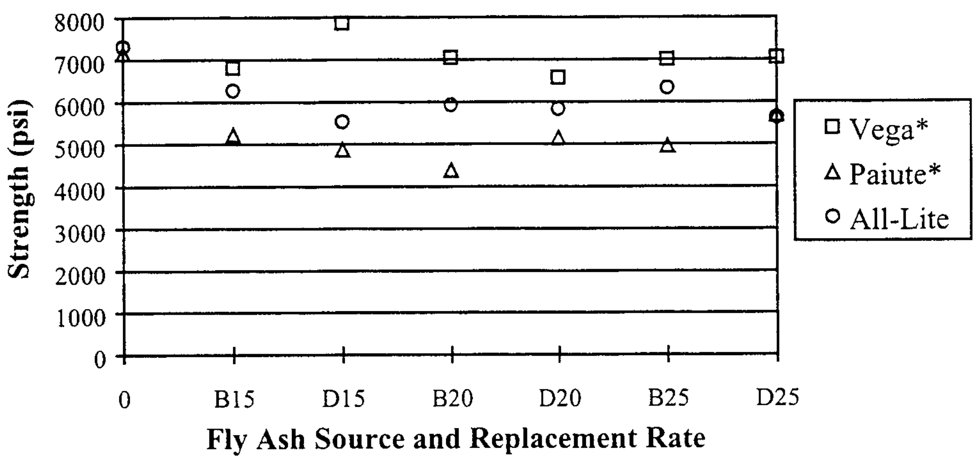

Figure 4.8 - 56-Day Strengths of Nevada Cement Concrete
***Actual test age is greater than 56 days.**

It should be noted that the majority of the 56-day strength values for Vega and Paiute coarse aggregate mixes were actually determined at an age greater than 56 days. Originally, no 56-day tests were scheduled, however, it was later determined that 56-day strength values might be useful, so all of the trial batches were tested at an age greater than or equal to 56 days. The test age for Vega aggregate mixes ranges from 191 to 217 days and the average test age is 203 days. The test age for Paiute aggregate mixes ranges from 110 to 194 days and the average test age is 160 days.

This may account for the reason that, at 28 days, there is no definite trend between Vega and All-Lite mixes but the "56-day" values appear to show that the Vega aggregate mixes are stronger than the All-Lite mixes. However, if there is significant strength gain after 56 days, one would expect that, in Figure 4.8, the Paiute aggregate mixes, which have lower strengths than All-Lite at 28 days, would have higher strengths than the All-Lite mixes. This is not the case, so it may be reasonable to conclude that, for the combinations of raw materials investigated in this project, there is not a significant amount of strength gain after 56 days.

Figures 4.9 and 4.10 show the 28 and 56-day strengths for Calaveras cement combinations. Any trends shown in these figures are much less defined than those shown by the data for Nevada cement mixes. With the exception of the 25% replacement with Bridger fly ash combinations, mixes which contain Vega coarse aggregate tend to be stronger at both the 28 and 56-day ages. Although the data shows several exceptions to this trend, it would also appear that, for Calaveras cement combinations at 28 and 56 days, All-Lite aggregate mixes tend to have lower strengths than mixes containing aggregates from the other two sources. Again, it should be noted that, for the "56-day" strength tests for Vega and Paiute coarse aggregate mixes, the actual test age is greater than 56 days so the values shown are probably greater than the actual strength at 56 days.

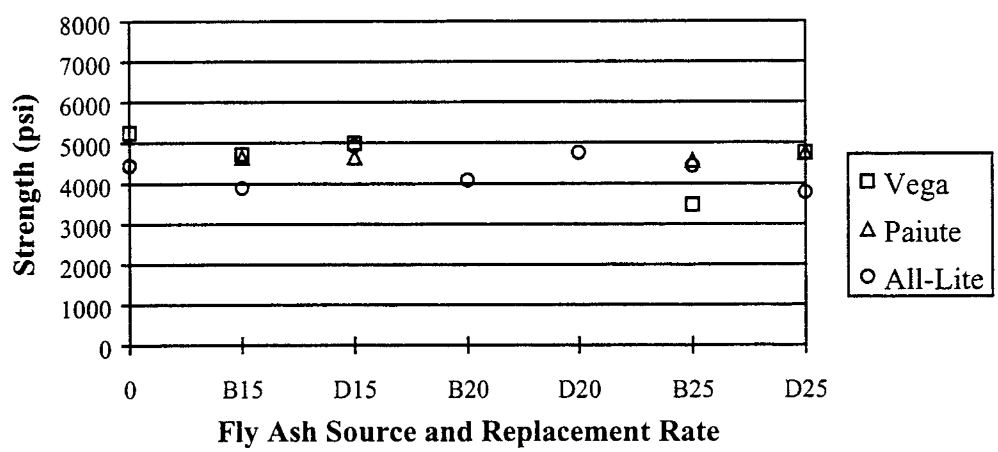

Figure 4.9 - 28-Day Strengths of Calaveras Cement Concrete

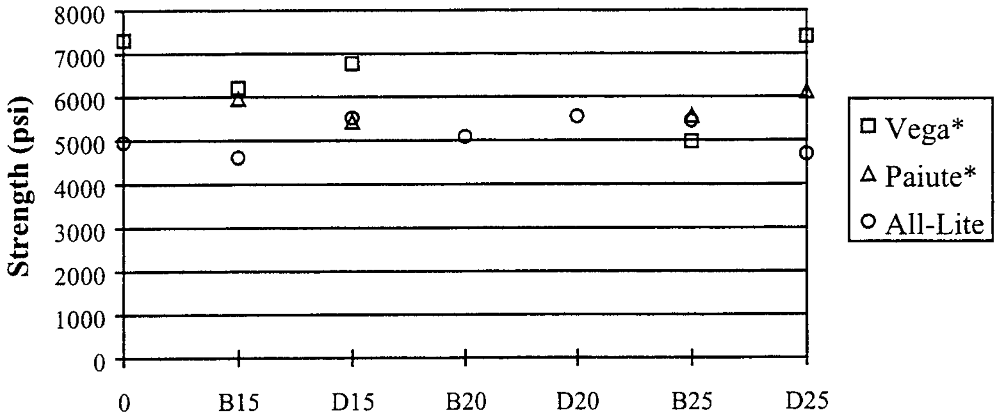

Figure 4.10 - 56-Day Strength for Calaveras Cement Concrete
***Actual test age is greater than 56 days.**

Figures 4.7, 4.8, 4.9, and 4.10 do not reveal any consistent trends relating combinations of fly ash with coarse aggregate to strength. Cementitious material such as fly ash are not expected to undergo any chemical interaction with aggregate which would affect strength. The data presented here supports this statement.

4.2.3 Strength in Terms of HPC

The HPC compressive strength performance grades suggested by FHWA are based on 56-day strengths. FHWA's strength grades for HPC are given in Table 2.8 in chapter 2. The mix designs created for this project were based on a 28-day compressive strength of 5000 psi. FHWA's HPC definition does not specify a performance grade which includes 5000 psi and does not account for the 28-day test age. At 28 and 56 days, the majority of trial batches developed for this project had a strength of less than 6000 psi. Based on FHWA's definition, it is not possible to assign HPC strength grades to these mixes. As will be discussed in Chapter 5, if NDOT chooses to adopt a grading system similar to FHWA's HPC definition, a method to specify strengths less than 6000 psi should be included.

4.3 Modulus of Elasticity (ASTM C 469) Results

The results for all of the 28-day age modulus of elasticity tests are given in Table A4.3 in the appendix. The main trend found in this data relates to the coarse aggregate source. Figure 4.11 is a plot of the square root of the 28-day compressive strength vs. the elastic modulus for all of the test data.

As the figure shows, for a given compressive strength, the All-Lite aggregate concrete has a lower elastic modulus than concrete made with the other two aggregate sources. ACI 318 (5), ACI 363 (6), and the AASHTO LRFD Bridge Design Specifications (1) provide equations to estimate the modulus of elasticity based on compressive strength and unit weight.

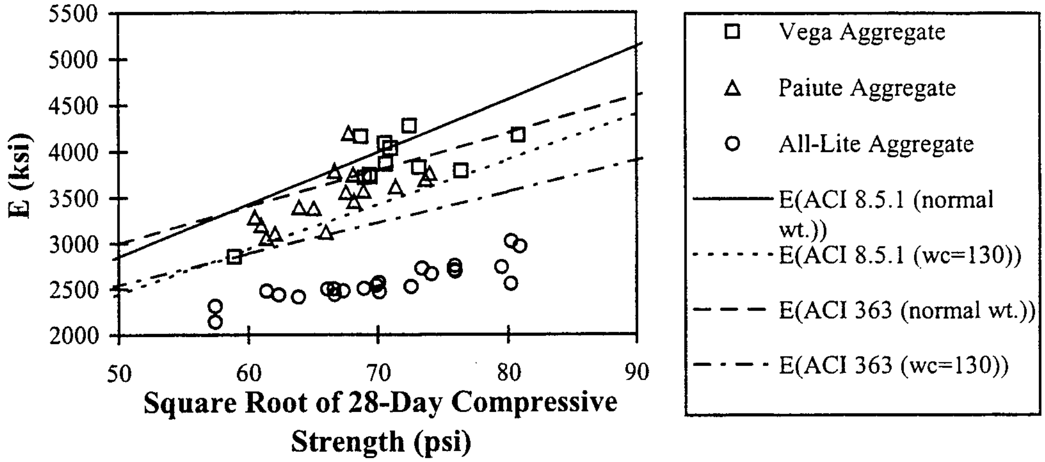

Figure 4.11 - Effect of Coarse Aggregate Source on Elastic Modulus

ACI 318.8.5.1 and AASHTO LRFD 5.4.2.4 provide the following equation:

$$\mathbf{E = w_c^{1.5}\, 33\sqrt{f'c}} \text{ (for unit weights between 90 and 155 lb/ft}^3\text{).}$$

ACI 318 also provides the equation:

$$\mathbf{E = 57000\sqrt{f'c}} \text{ (for normal weight concrete).}$$

ACI 363 gives the equation:

$$\mathbf{E = (40000\sqrt{f'c} + 1.0 \times 10^6)(w_c/145)^{1.5}}$$

In these equations, E is the modulus of elasticity in psi, w_c is the unit weight in lb/ft^3, and f'c is the compressive strength in psi. These equations are shown graphically in Figure 4.11. In order to graph the ACI 318 equation which includes unit weight, the unit weight was taken to be 130 lb/ft^3. The ACI 363 equation is graphed for both normal weight concrete (unit weight = 145 lb/ft^3) and for concrete with a unit weight of 130 lb/ft^3. A unit weight of 130 lb/ft^3 was used to compare the ACI approximations with the data for All-Lite aggregate concrete, which has a unit weight of approximately 130 lb/ft^3.

As Figure 4.11 shows, the ACI equations for normal weight concrete both slightly overestimate the modulus of elasticity for the majority of the Vega and Paiute aggregate concrete data. The results of the equations calculated with a unit weight of 130 lb/ft^3 are significantly greater than the All-Lite data and are therefore a poor representation of the actual elastic modulus. A trend line for the All-Lite data yields the following equation:

$$\mathbf{E = 36500\sqrt{f'c}} \text{ (for All-Lite aggregate concrete in this project).}$$

For a given compressive strength, this trend line equation yields elastic modulus values which are about 65% of the values calculated using the ACI 318 normal weight concrete equation. This means that the actual elastic displacements in a concrete member made with All-Lite aggregate would have a magnitude which is 150% of the values that would be expected based on the ACI 318 normal weight concrete equation. In applications in which deflections are a controlling factor in design, this deviation in actual elastic modulus from the accepted approximation must be recognized and provided for in the design.

4.3.1 Elastic Modulus in Terms of HPC

FHWA's performance grades for elastic modulus are given in Table 2.9 in chapter 2. FHWA's performance grades are based on the 56-day elastic modulus. All of the elastic modulus tests for this research project were performed at an age of 28 days. The lowest elastic modulus included in FHWA's grading system is $4x(10^6)$ psi. The majority of the data obtained for this project falls below this value so it is not possible to assign it an HPC grade. Similar to compressive strength, if a grading system similar to FHWA's is adopted by NDOT, a way to specify lower values of elastic modulus should be included. Recommendations relating to the application of elastic modulus grades will be further discussed in chapter 5.

In specifying HPC parameters for a given job, it must be remembered that strength and elastic modulus are related properties. Concrete with a higher strength also tends to have higher values for elastic modulus. Therefore, in situations where both of these parameters are specified, it is important to make sure that the required levels of performance are compatible with one another and that both can be achieved in the same mix design.

4.4 Freeze-Thaw (ASTM C 666) Test Results

Due to limitations on the available freeze-thaw test equipment, the freeze-thaw test was only performed on a very limited number of specimens. Table 4.1 gives the freeze thaw data obtained in this research project.

Table 4.1 - Freeze-Thaw Data

Batch #	Mix I.D.	Air Content (%)	# of Cycles	Length Change (microstrains)	Relative Dynamic Elastic Modulus (%)
3	N-B20-V	5.125	322	190	95.3
11	C-D15-V	5.25	311	175	98.1
12	C-D25-V	5.75	294	115	98.2
13	N-0-P	4	299	150	98.4
14	N-B15-P	6.25	306	85	99.7
27	N-D15-P	6.5	394	70	100.7
28	N-B20-P	6	375	135	94.3
29*	N-0-A	4.25	164	2870	74.2
30*	N-0-A	4.25	319	1850	65.0
31**	N-B15-A	6	319	3605	34.5
32**	N-B20-A	5.75	319	3125	27.6
34	N-D15-A	6.5	297	670	79.8

*Due to severe cracking during test, results from only one specimen are shown.
**Both specimens severely fractured during test.

Unless otherwise noted, the values in Table 4.1 are the average results from two specimens. The freeze-thaw test method, ASTM C 666, specifies that the test be run for 300 cycles, however, as can be seen in Table 4.1, there is some variation as to the actual duration that the test was performed.

The two factors that can be used to judge freeze-thaw performance are the relative dynamic modulus of elasticity, in percent, and the length change, in microstrains. Both of these factors are measures of the amount of damage that the concrete experienced. Higher values of length change indicate that the concrete has experienced significant expansion due to the formation of micro-cracks. Higher values of the relative dynamic modulus of elasticity indicate good performance and that the concrete has maintained a higher percentage of its original dynamic modulus.

The main trend revealed by the freeze-thaw data is related to the coarse aggregate source. Even though there is some variation in air content, the concrete batches made with Vega and Paiute aggregate have very good freeze-thaw resistance. All of these batches have a length change of less than 200 microstrains and a relative dynamic modulus of elasticity greater than 94%. One of these mix designs, N-D15-P, even achieved a relative dynamic modulus greater than 100%. This means that even while the N-D15-P specimens were in the freeze-thaw environment, they continued to cure and experience a gain in dynamic modulus of elasticity.

The concrete made with All-Lite aggregate had very poor freeze-thaw resistance. All of these specimens experienced high amounts of expansion and have relative dynamic modulus of elasticity values which are less than 80%. Most of the All-Lite specimens had visible fractures and some of them even broke in half.

The results of this research indicate that coarse aggregate can have a significant effect on freeze-thaw resistance. The data obtained from this test is very limited so, based on this research alone, it is difficult to determine any effect that fly ash and cement source may have on freeze-thaw resistance. The literature indicates that for concrete with the same strength and air content, concrete with fly ash has the same freeze-thaw resistance as concrete without fly ash (39). Air entrainment is the main method used to ensure good freeze-thaw performance. Among the All-Lite specimens tested, the best performance was obtained from the batch with the highest air content (6.5%). This may indicate that air contents higher than 6% would improve the freeze-thaw performance of concrete made with aggregates such as All-Lite. However, the data obtained in this research is not sufficient to verify this hypothesis so further research in this area would be useful.

4.4.1 Freeze-Thaw Results in Terms of HPC

FHWA's performance grades for freeze-thaw resistance are given in Table 2.3 in chapter 2. All of the concrete specimens made with Vega and Paiute aggregates have an HPC performance grade of 2 for freeze-thaw resistance. This is the best freeze-thaw performance grade specified in FHWA's definition of HPC. The All-Lite Aggregate concrete specimens either have a freeze-thaw grade of 1 or have such poor freeze-thaw resistance that they do not correspond to a FHWA performance grade. A freeze-thaw performance grade of 1 is the lowest performance grade specified in FHWA's HPC definition.

4.5 Scaling Resistance (ASTM C 672) Test Results

All of the results from the scaling resistance tests are given in Table A4.4 in the appendix. Each of the results given in Table A4.4 is the average result from two test specimens. The results of this test are based on a qualitative rating of the surface damage on the specimen after 50 cycles as defined by ASTM C 672. As stated in chapter 2, a surface with no damage receives a rating of "0" and a severely damaged surface receives a scaling rating of "5." Several of the results in Table A4.4 show a scaling rating in decimal form, such as "2.5" or "4.5." ASTM C 672 does not specify ratings which fall between whole numbers on the rating scale. Unless otherwise stated, all of the results in Table A4.4 are the average results from two specimens, so two specimens with ratings of "2" and "3" would yield an average result of "2.5."

Scaling damage takes place on the surface of concrete so scaling resistance is highly sensitive to the surface finishing procedure. As chapter 1 stated, the HPC bridge deck projects in New York have experienced more scaling damage in areas that were finished by hand and possibly overworked. In this project, all of the scaling resistance specimens were finished by hand. Due to variations in workability, the specimens in this project received variable finishes and it is very likely that some of them were over finished. This inconsistency casts doubt on the applicability of the results.

4.5.1 Effect of Cement Source on Scaling Resistance

Figures 4.12, 4.13, and 4.14 compare the effect of cement source and fly ash for each of the aggregate sources. Figure 4.12 shows that, for mixes made with Vega Aggregate, Calaveras cement combinations tend to have slightly better scaling resistance than Nevada cement mixes. Figure 4.13 seems to show the opposite trend for All-Lite aggregate mixes. With the exception of the mix without fly ash, All-Lite mixes with Nevada cement have scaling resistance which is equal to or better than that of Calaveras cement mixes. Figure 4.14 shows the data for Paiute aggregate mixes. There is no obvious trend relating cement source to scaling resistance in the Paiute aggregate concrete data.

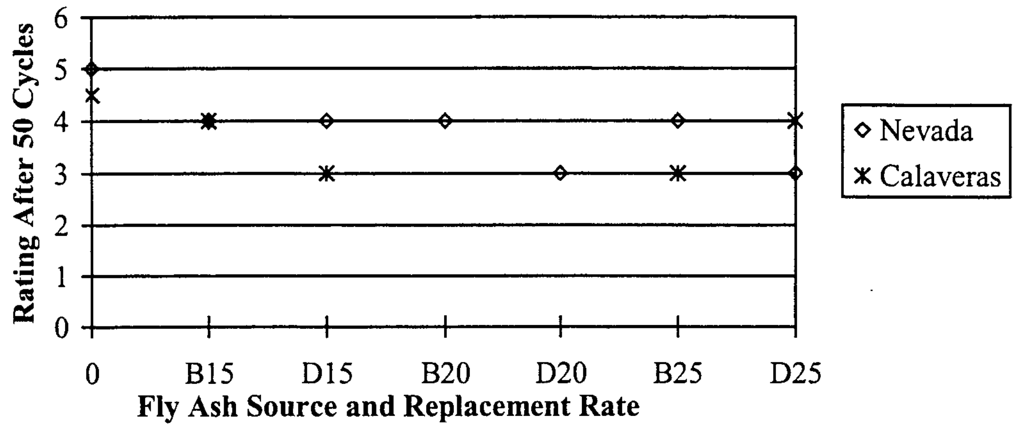

Figure 4.12 - Scaling Resistance: Effect of Cement Source with Vega Aggregate

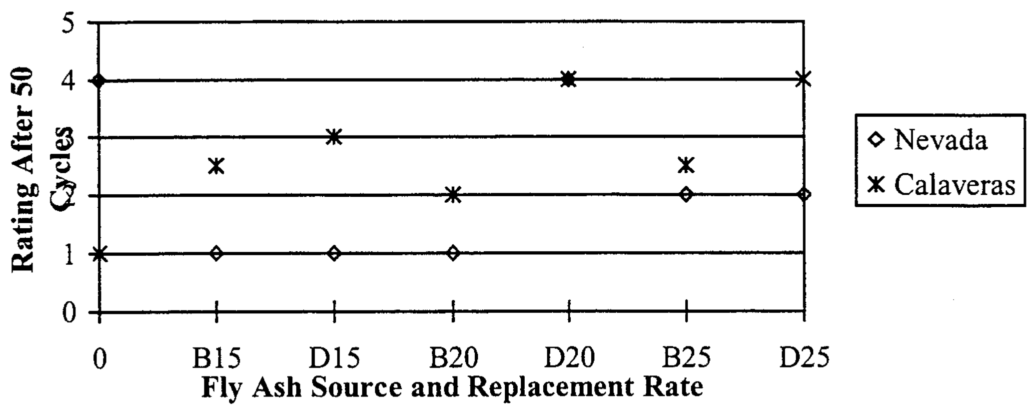

Figure 4.13 - Scaling Resistance: Effect of Cement Source with All-Lite Aggregate

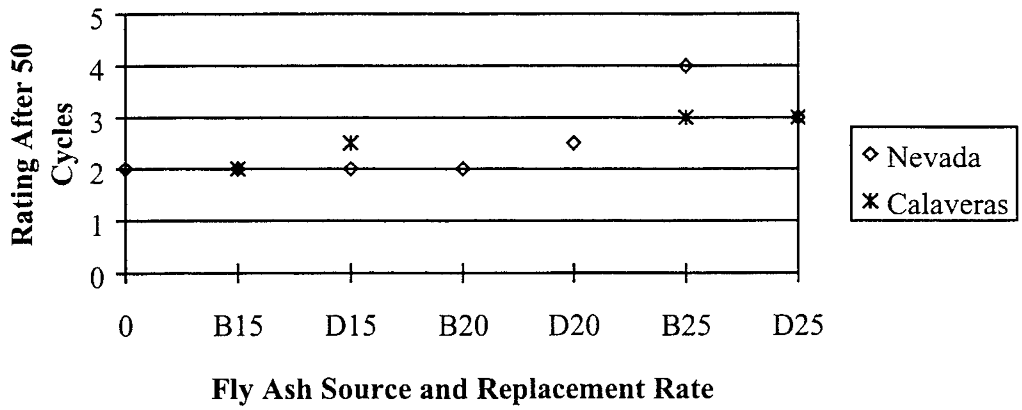

Figure 4.14 - Scaling Resistance: Effect of Cement Source with Paiute Aggregate

Based on the data obtained in this research, it is difficult to establish any definite relationship between cement source and scaling resistance because the data from concrete made with different aggregate sources seems to be contradictory. It is normally assumed that there is no chemical interaction between cement and aggregate. However, aside from variations in the finishing process, this may be the only way to explain the seemingly contradictory data.

Based on the data obtained in this project, there does not seem to be any correlation between fly ash and scaling resistance. Since the scaling mechanism involves deicer salts penetrating a concrete surface, it would be expected that using fly ash, which reduces permeability, would improve scaling resistance. However, the data does not provide evidence to support this type of behavior.

4.5.2 Effect of Coarse Aggregate Source on Scaling Resistance

Figures 4.15 and 4.16 compare aggregate source and fly ash for the two cement sources. Figure 4.15 show the scaling resistance results for concrete made with Nevada cement. With the exception of the mixes with 20% replacement of Nevada cement with Delta fly ash and the 0% fly ash mixes, All-Lite aggregate concrete tends to have the best scaling resistance and Vega aggregate concrete tends to have scaling resistance which is worse than or equal to that of Paiute aggregate. This trend is only partially duplicated in the Calaveras cement concrete data, given in Figure 4.16. In Calaveras cement mixes with Vega aggregate, Vega aggregate concrete tends to have scaling resistance worse than or equal to that of the other aggregate sources. There is no definite trend showing which of the other two aggregate sources, Paiute or All-Lite, leads to the best scaling resistance. The reason that the Calaveras and Nevada cement concrete data do not completely agree on the effect of aggregate source is unknown. Again, this is probably due to finishing variations or possibly some type of chemical interaction between the aggregate and cement. Due to these variations, it is difficult to draw any conclusive results.

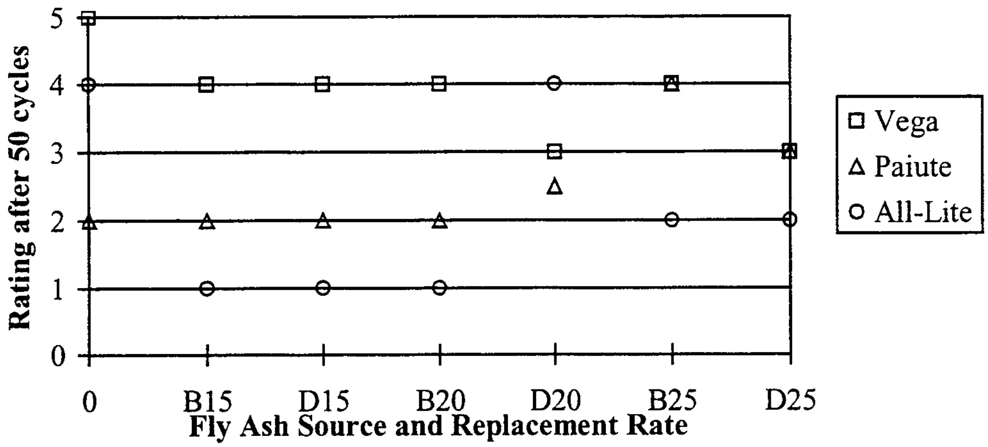

Figure 4.15 - Scaling Resistance: Effect of Aggregate Source with Nevada Cement

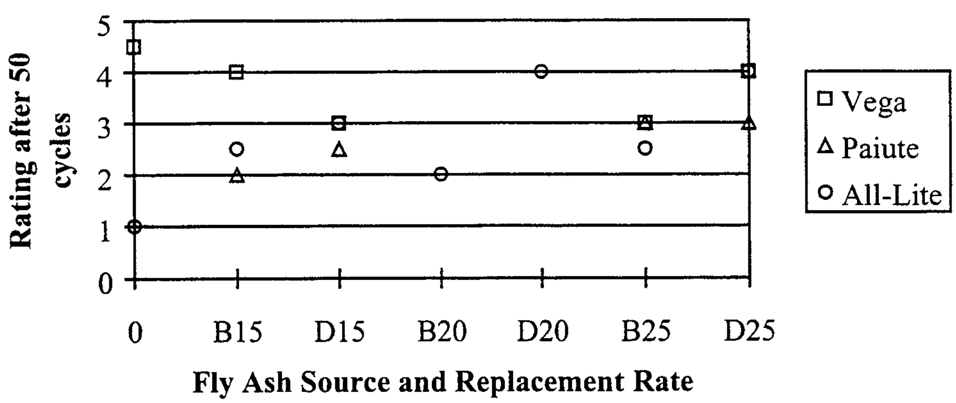

Figure 4.16 - Scaling Resistance: Effect of Aggregate Source with Calaveras Cement

4.5.3 Scaling Resistance in Terms of HPC

Table 2.4 in chapter two gives FHWA's performance grades for scaling resistance. The scaling resistance grades of individual specimens tested in this project range from 0, the best grade, to 5, the worst grade. There is a wide variability in the scaling resistance grades obtained in this project and a lack of a consistent trend relating to materials or mix proportions. The specimens were all exposed to the same curing procedure but, due to variations in workability, the amount of hand finishing work required to achieve a smooth surface varied between trial batches. As described in chapter 1, HPC bridge projects in New York have experienced poor scaling resistance in areas where the surface was hand finished and overworked. Based on these facts, it seems very likely that the scaling resistance variability of the specimens made for this project is a result of variations in the finishing procedure. While the data does provide some evidence that cement and aggregate source effect scaling resistance, it would seem that the finishing procedure is at least as important as the materials and mix proportions to achieve good performance.

4.6 Chloride Ion Penetration (ASTM C 1202) Test Results

All of the results of the chloride ion penetration tests are given in Table A4.5 in the appendix. Each values given in Table A4.5 is the average result from tests on 3 specimens. The chloride ion permeability test was performed at 56 days and at 120 days in order to account for the hydration reactions involving fly ash which take longer to reach completion.

4.6.1 Effect of Fly Ash on Permeability

In general, concrete permeability decreases with increasing amounts of cement replacement with fly ash. This trend is visible at 56 days and becomes more pronounced by 120 days. With very few exceptions, this behavior is shown consistently throughout the data. Figures 4.17, 4.18, 4.19, and 4.20 are good examples of this trend. As these figures show, for a given coarse aggregate source, even the lowest rate of fly ash addition (15% replacement of cement) can yield permeability which is significantly better than mixes without fly ash. At 56 days, concrete with 15% fly ash has a permeability which is close to 1000 coulombs less than concrete with no fly ash. At 120 days, 15% fly ash concrete has a permeability which is 1500 to 2000 coulombs less than concrete with no fly ash. At 56 days, concrete with 25% fly ash achieves permeability values which are 1000 to 2000 coulombs less than concrete without fly ash. At 120 days, 25% fly ash concrete has a permeability which is 2000 to 3000 coulombs less than concrete with no fly ash.

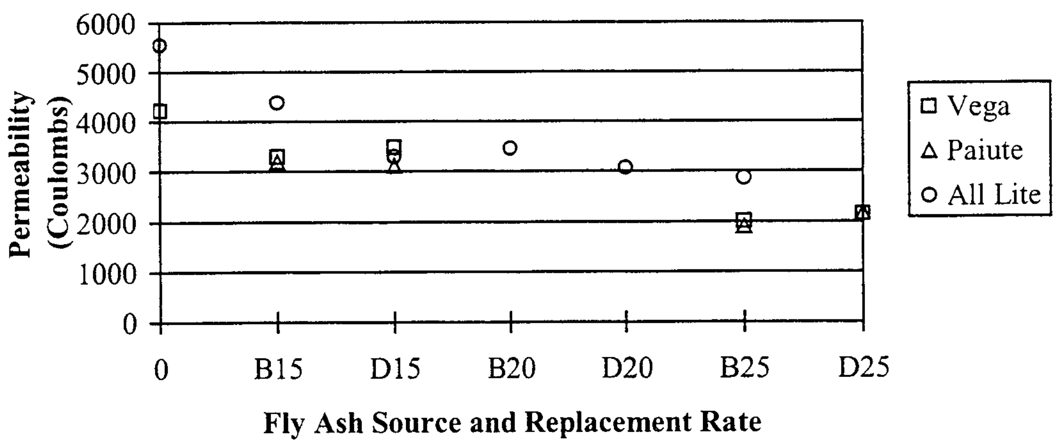

Figure 4.17 - Effect of Fly Ash on the 56-Day Permeability of Calaveras Cement Concrete

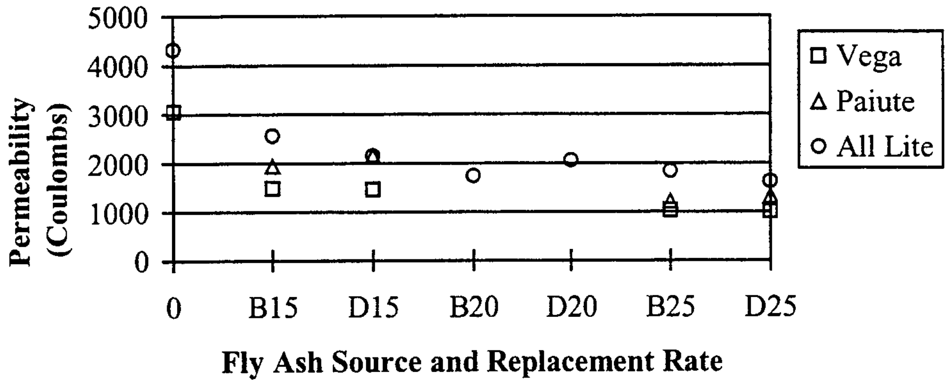

Figure 4.18 - Effect of Fly Ash on the 120-Day Permeability of Calaveras Cement Concrete

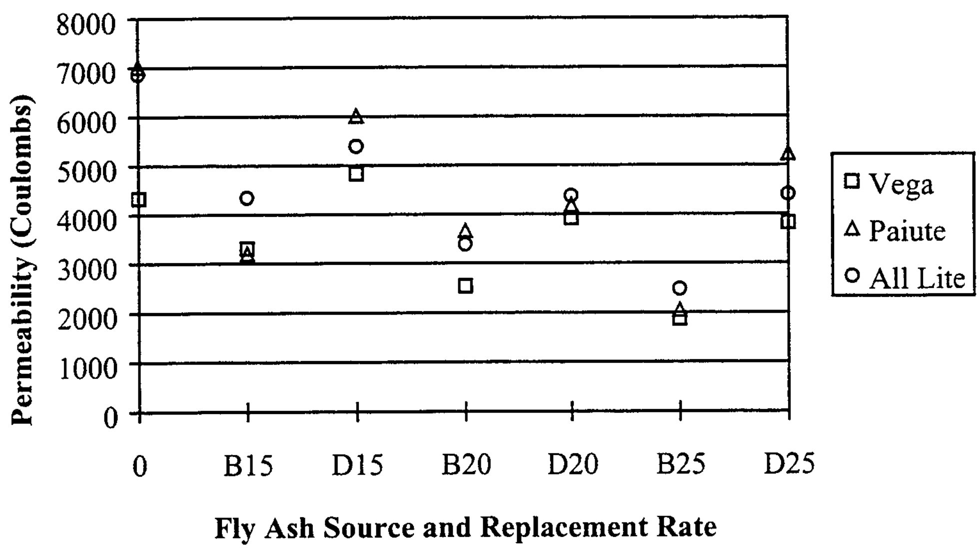

Figure 4.19 - Effect of Fly Ash on the 56-Day Permeability of Nevada Cement Concrete

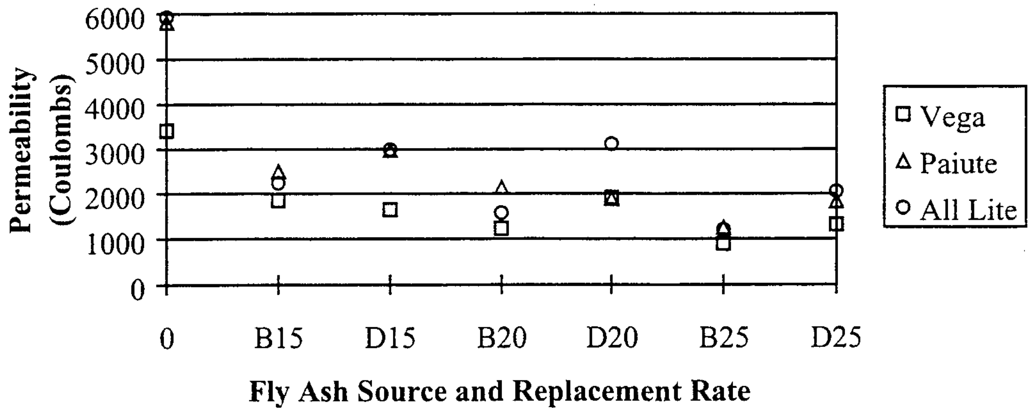

Figure 4.20 - Effect of Fly Ash on the 120-Day Permeability of Nevada Cement Concrete

The ASTM C 1202 rating system for permeability is given in Table 2.5 in chapter 2. ASTM's ratings are "high," "moderate," "low," "very low," and "negligible." Of the 51 concrete batches tested in this project, only one achieved a permeability rating of "very low" at 120 days. In general, the ratings range between "low" and "high." With the exception of two "low" permeability 25% fly ash batches, at 56 days, all of the concrete batches tested had a permeability rating of "high" or "moderate." Table 4.2 shows that 100% of the batches made with no fly ash had a "high" permeability rating while only 21% of the batches made with 25% fly ash had "high" permeability at 56 days. Table 4.3 shows that, at 120 days, 78% of the mixes with no fly ash had "high" permeability and 79% of the mixes with 25% fly ash had low permeability.

Table 4.2 - Relative Distribution of 56-Day Permeability Ratings in Relation to Cement Replacement with Fly Ash (# of batches shown in parenthesis)

Rate of Cement Replacement with Fly Ash	"High" Permeability	"Moderate" Permeability	"Low" Permeability
0% Fly Ash (9)	100% (9)	0%	0%
15% Fly Ash (16)	44% (7)	56% (9)	0%
20% Fly Ash (12)	33% (4)	67% (8)	0%
25% Fly Ash (14)	21% (3)	64% (9)	14% (2)

Table 4.3 - Relative Distribution of 120-Day Permeability Ratings in Relation to Cement Replacement with Fly Ash (# of batches shown in parenthesis)

Rate of Cement Replacement with Fly Ash	"High" Permeability	"Moderate" Permeabilit y	"Low" Permeabilit y	"Very Low" Permeability
0% Fly Ash (9)	78% (7)	22% (2)	0%	0%
15% Fly Ash (16)	0%	63% (10)	37% (6)	0%
20% Fly Ash (12)	0%	50% (6)	50% (6)	0%
25% Fly Ash (14)	0%	14% (2)	79% (11)	7% (1)

Tables 4.2 and 4.3 clearly show that cement replacement with fly ash can improve the permeability rating significantly. Based on this research, it can be seen that higher rates of fly ash addition result in lower permeability values. Also, regardless of fly ash replacement, permeability decreases with time.

4.6.2 Effect of Cement Source on Permeability

Figures 4.21, 4.22, and 4.23 compare how the 56-day permeability is affected by the two cement sources over the range of combinations with fly ash for each of the coarse aggregate sources. As these figures show, at 56 days, concrete made with the two cement sources exhibits very similar behavior except for combinations of Nevada Cement with Delta fly ash. The Nevada cement-Delta fly ash combinations tend to have higher permeability than the other combinations of cement and fly ash. Figures 4.21 and 4.23, for Vega and All-Lite aggregate concrete, show this trend most clearly.

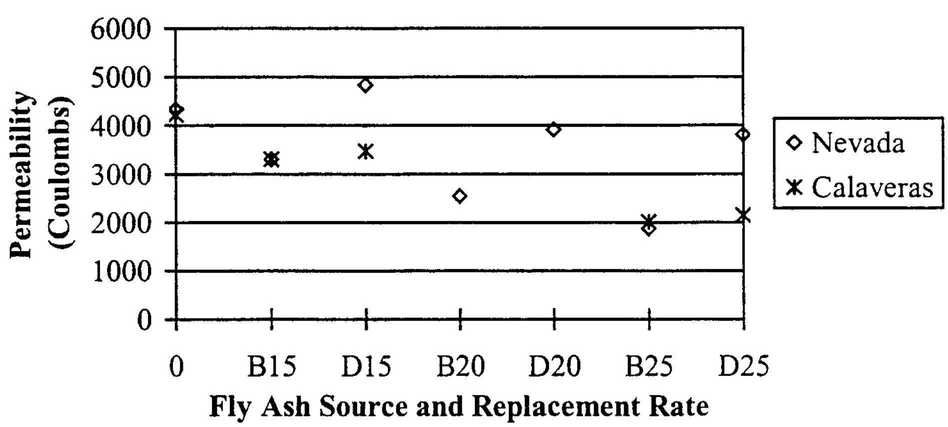

Figure 4.21 - Effect of Cement Source on 56-Day Permeability of Vega Aggregate Concrete

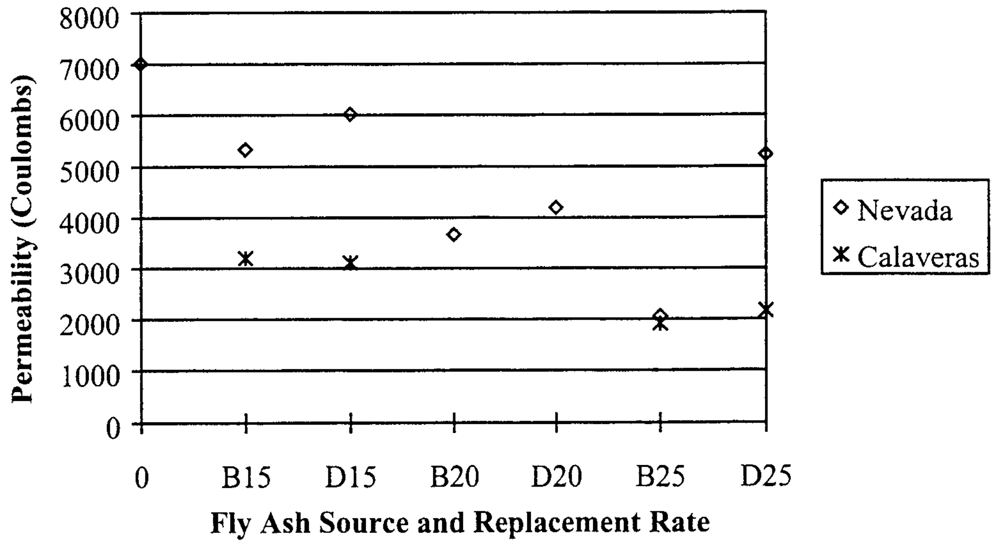

Figure 4.22 - Effect of Cement Source on 56-Day Permeability of Paiute Aggregate Concrete

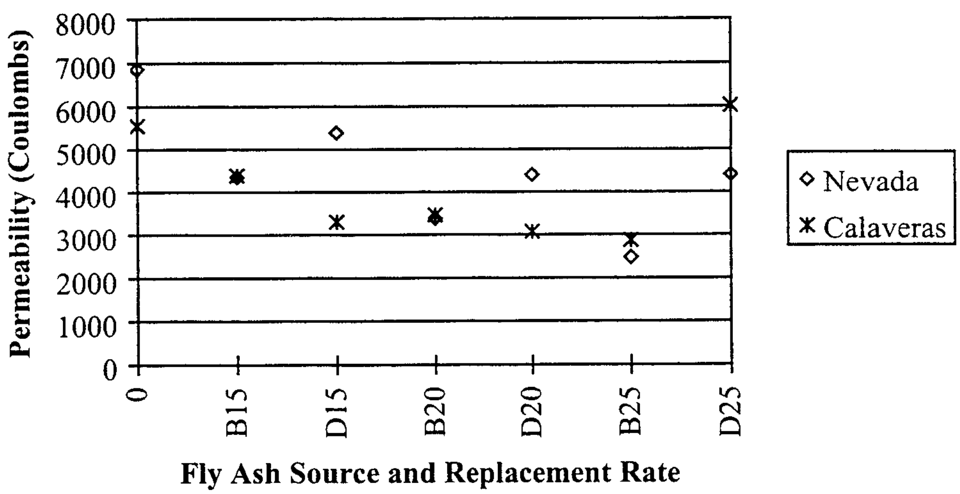

Figure 4.23 - Effect of Cement Source on 56-Day Permeability of All-Lite Aggregate Concrete

This trend is much less pronounced in the 120-day data. Figures 4.24, 4.25, and 4.26 compare how the 120-day permeability is affected by the two cement sources over the range of combinations with fly ash for each of the coarse aggregate sources.

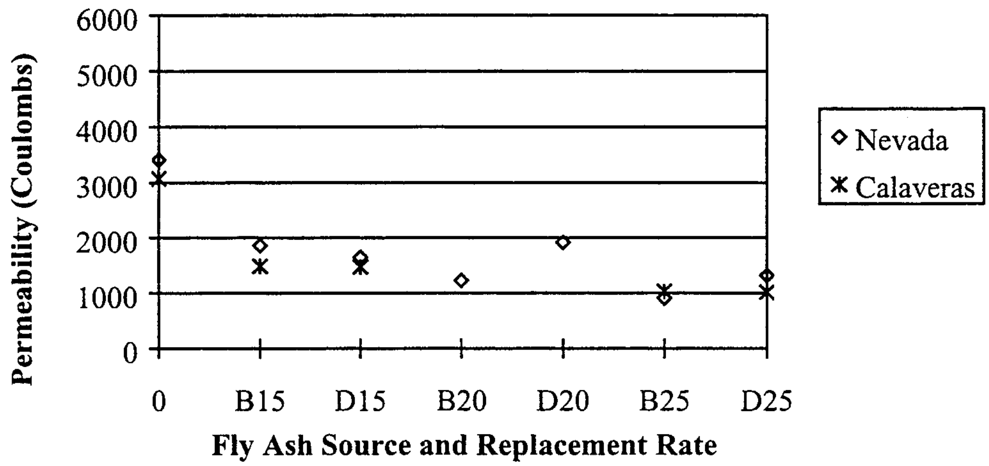

Figure 4.24 - Effect of Cement Source on 120-Day Permeability of Vega Aggregate Concrete

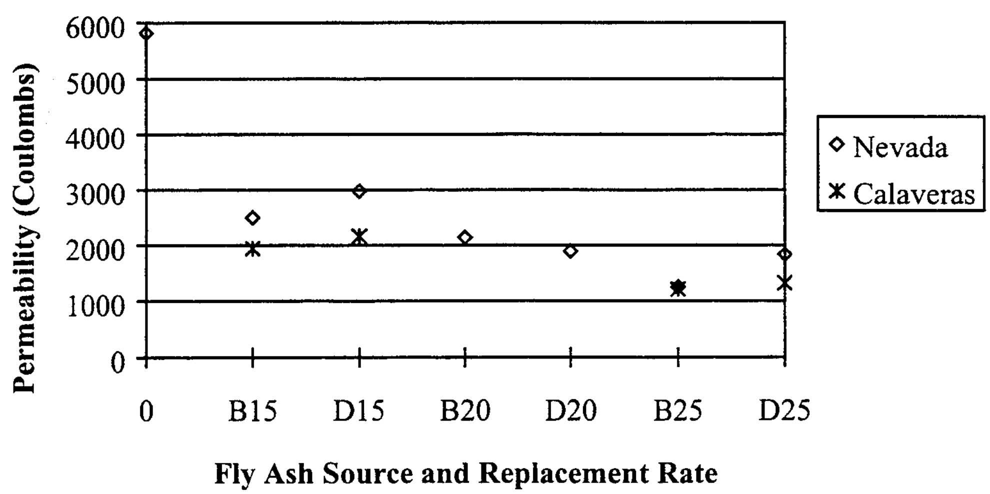

Figure 4.25 - Effect of Cement Source on 120-Day Permeability of Paiute Aggregate Concrete

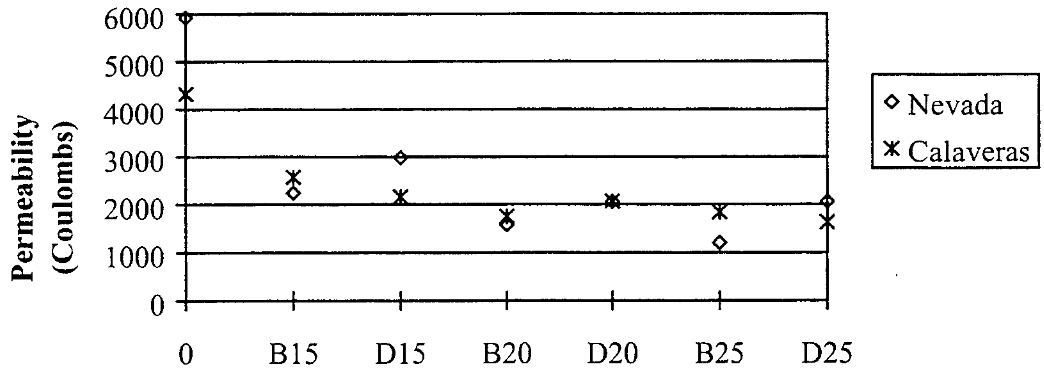

Figure 4.26 - Effect of Cement Source on 120-Day Permeability of All-Lite Aggregate Concrete

At 120 days, for a given percent of fly ash, concrete batches made with the two cement sources have very similar behavior, regardless of the fly ash source. This cement-fly ash interaction can also be seen in Figure 4.17, 4.18, 4.19, and 4.20. Figure 4.19 clearly shows that, for Nevada cement concrete at 56 days, mix combinations with Delta fly ash have a consistently higher permeability than Bridger fly ash combinations at the same rate of addition. As Figure 4.20 shows, at 120 days, this trend is still present, but it is much less pronounced. Figures 4.17 and 4.18 show that this trend does not exist for Calaveras cement at 56 or 120 days.

The reason that Nevada cement-Delta fly ash concrete has higher permeability at 56 days is most likely related to the rate of reaction of the chemical interaction between the cement and fly ash. Due to a slower rate of reaction, fly ash has been known to cause slower rates of early strength gain as well as higher strengths at later ages (44). This type of behavior could also explain why certain combinations of cement and fly ash experience higher permeability values at early ages but have comparable permeability values at later ages. The early rate of reaction between Nevada cement and Delta fly ash is slow but, with time, it reaches the same state as that of the other cement and fly ash combinations.

4.6.3 Chloride Ion Penetration in Terms of HPC

FHWA's performance grades for HPC are given in Table 2.6 in chapter 2. None of the trial batches achieved the highest permeability rating, "3," at 56 or 120 days. At 56 days, the majority of the trial batches had permeability values greater than 3000 coulombs so they do not fall within the range of FHWA's grading system. A few batches with higher fly ash addition rates had permeability grades of "1" or "2" at 56 days. At 120 days, all of the batches without fly ash had permeability values greater than 3000 coulombs. With the exception of one trial batch, all of the batches with fly ash had a performance grade of at least "1" at 120 days. Close to 60% of the batches made with fly ash had permeability grades of "2" at 120 days. This demonstrates that the use of fly ash yields concrete with better permeability values.

4.7 Drying Shrinkage (ASTM C 157) Test Results

All of the results from the shrinkage tests are given in Table A4.6 in the appendix. Shrinkage readings were taken at 7 days, 14 days, 28 days, 8 weeks, 16 weeks, and 32 weeks after the specimens were placed in air storage. Due to time limitations, the 32 week data is not complete. However, the 16 week data should be sufficient to establish trends regarding the raw materials and to evaluate the shrinkage of each trial batch.

4.7.1 Effect of Coarse Aggregate Source on Shrinkage

The driving mechanism behind shrinkage is a volumetric change in the paste portion of concrete. The aggregates do not experience a volume change, however, they do affect shrinkage because they act as a constraint against volume changes in the paste. The mechanical properties of the aggregates and the bonds they form with the paste, therefore, affect shrinkage.

Figures 4.27 and 4.28 compare the effects of the three aggregate sources for both cement sources over the range of fly ash sources and rates of addition. Both of these figures show that, for a given source and rate of addition of fly ash, batches with Paiute coarse aggregate have the highest amount of shrinkage and batches with Vega coarse aggregate have the lowest amount of shrinkage.

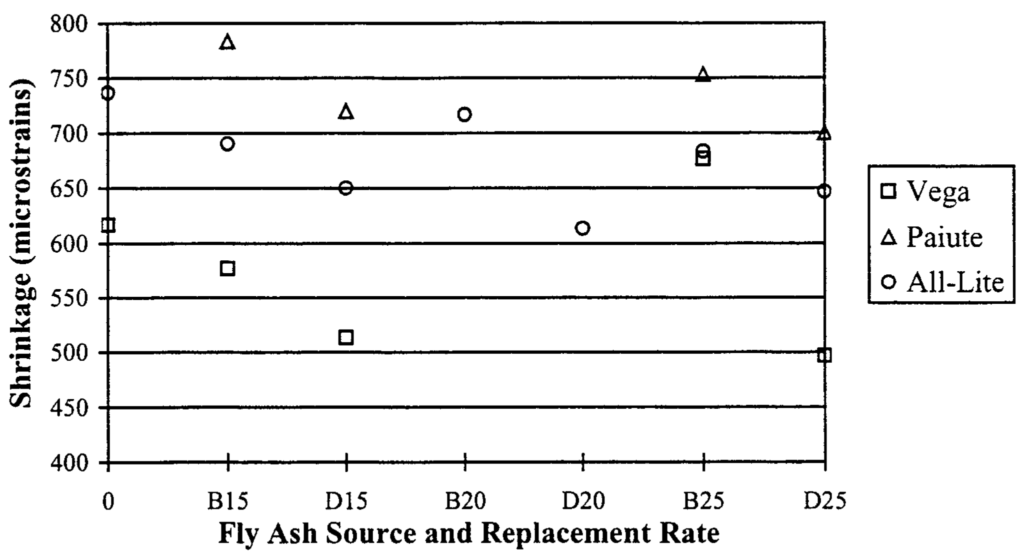

Figure 4.27 - Shrinkage of Calaveras Cement Concrete (16 weeks)

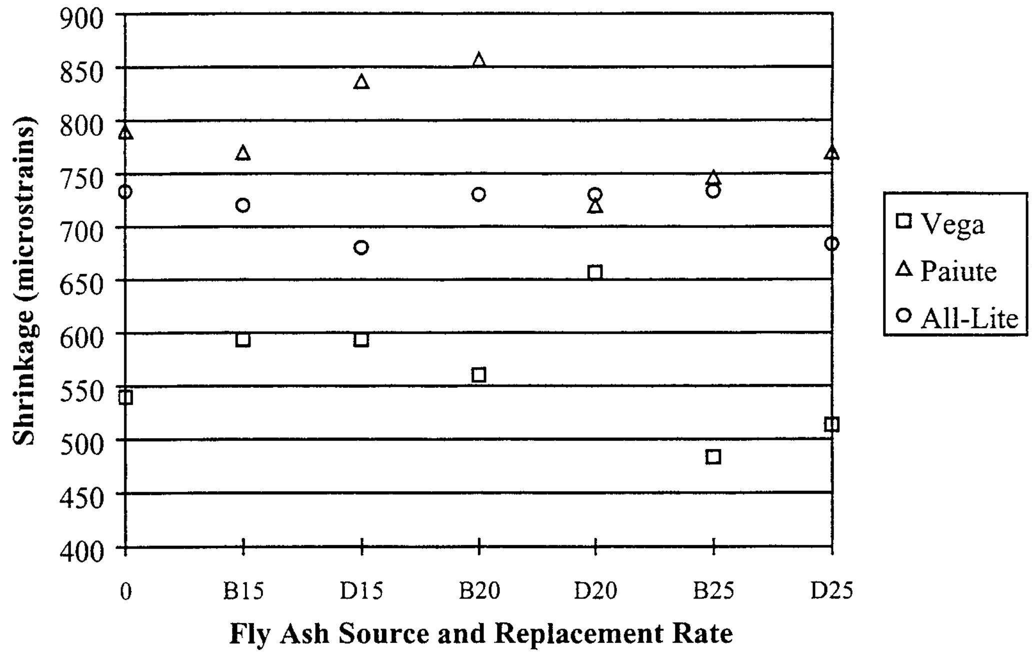

Figure 4.28 - Shrinkage of Nevada Cement Concrete (16 weeks)

4.7.2 Effect of Cement Source on Shrinkage

Figures 4.29, 4.30 and 4.31 compare the effect of the two cement sources for each of the coarse aggregate sources over the range of fly ash sources and rates of addition. There does not appear to be a consistent trend relating to cement source. Figure 4.31 shows that, with the exception of the 0 % fly ash mix, for All-Lite aggregate concrete at a given fly ash source and rate of replacement, Nevada cement batches have shrinkage which is consistently greater than Calaveras cement batches. This trend is not duplicated in the Vega and Paiute aggregate concrete data.

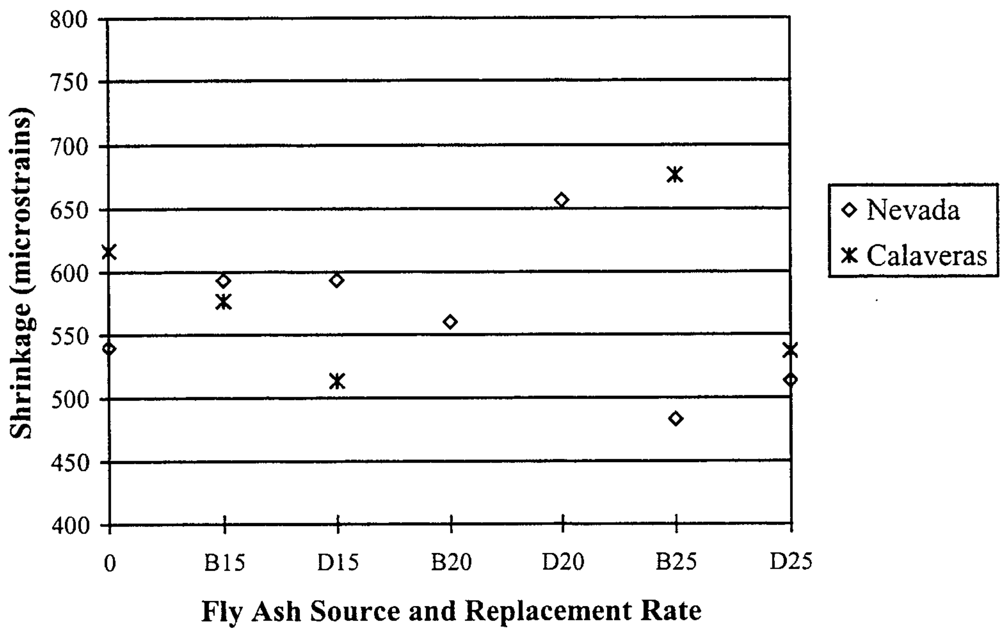

Figure 4.29 - Shrinkage of Vega Aggregate Concrete (16 weeks)

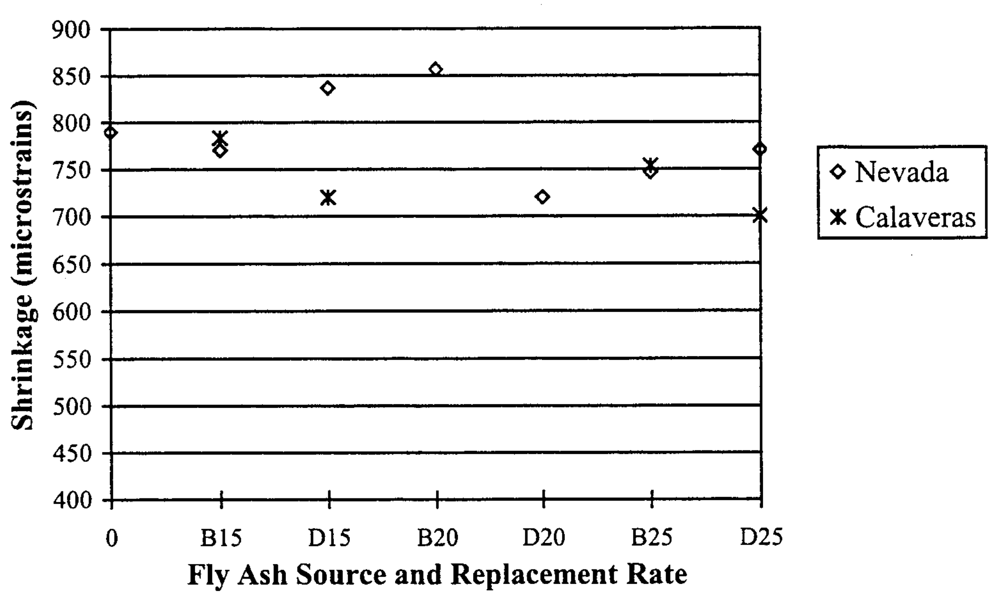

Figure 4.30 - Shrinkage of Paiute Aggregate Concrete (16 weeks)

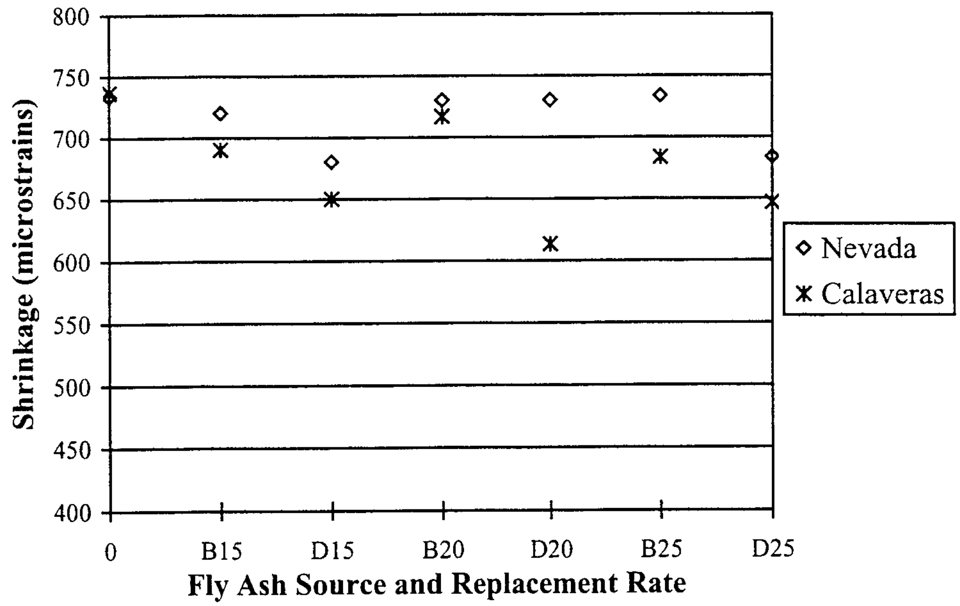

Figure 4.31 - Shrinkage of All-Lite Aggregate Concrete (16 weeks)

4.7.3 Effect of Fly Ash on Shrinkage

Pozzolans have been known to increase drying shrinkage in situations where they increase the water requirements of the concrete (44). The mix designs developed for this research project all had the same maximum water/cementitious materials ratio so varying water requirements was not a factor. There are no definite trends relating fly ash to shrinkage which are consistently visible throughout the data. However, certain portions of the data do reveal some trends. Figure 4.31 shows that for All-Lite aggregate at a given rate of fly ash addition, Delta fly ash mixes have lower shrinkage than Bridger fly ash mixes, regardless of cement source. Figure 4.27 shows that for Calaveras cement concrete, Delta fly ash concrete has lower shrinkage than Bridger fly ash concrete, regardless of coarse aggregate source.

In some cases, Delta fly ash seems to slightly increase the volumetric stability of concrete. The reason that Delta fly ash causes lower shrinkage has to be related to how it effects the paste portion of the concrete. However, the exact cause or mechanism behind this behavior is unknown.

4.7.4 Shrinkage in Terms of HPC

FHWA's performance grades for shrinkage are given in Table 2.2 in chapter 2. The majority of the Vega aggregate concrete batches have a shrinkage performance grade of "2" at 16 weeks. Most of the All-Lite and Paiute aggregate concrete batches have a shrinkage grade of "1" at 16 weeks. At 32 weeks, the data shows that most of the Vega aggregate batches have a shrinkage grade of "1." Although 32-week data from the Paiute aggregate batches is incomplete, most of the batches that were tested have shrinkage values above 800 microstrains and, therefore, do not fall within the range of FHWA's grading system.

Overall, the trial batches tested in this research had high values of shrinkage. This could represent a significant problem if any of these mix designs were used in a bridge deck. Some measures which might improve drying shrinkage performance are using a lower water/cementitious materials ratio or increasing the aggregate content. Lowering the water/cementitious materials ratio reduces the amount of water in the concrete thereby decreasing the amount of drying that will take place over time. Aggregate acts as a restraint against volume changes in the cement. Increasing the aggregate content increases the amount of constraint against shrinkage within the concrete.

4.8 Results of Alkali-Silica Reactivity Test (ASTM C 1260)

The alkali-silica reactivity test results are given in Table 4.4. Table 4.4 gives the average expansion for mortar bars made with aggregate from each of the three coarse aggregate sources as well as the fine aggregate source.

Table 4.4 - Alkali-Silica Reactivity Test Results

Aggregate	Average Expansion (%)
Vega Coarse	0.450
All-Lite Coarse	0.078
Paiute Coarse	0.585
Paiute Fine	0.448

AASHTO T 303, which is essentially the same as ASTM C 1260, indicates that when the expansion from this test is greater than 0.10 %, "it is indicative of potentially deleterious expansion" (2). When the expansion is less than 0.10 %, "it is indicative of innocuous behavior" (2).

With the exception of the All-Lite coarse aggregate, all of the mortar bars made with aggregates used in this project had expansions greater than 0.10 %. This means that these aggregate sources have the potential to participate in alkali-silica reactions and ultimately cause severe concrete damage. This fact should be recognized by engineers developing mix designs with these aggregates. Based on the results of this research, measures should be taken to improve the ASR resistance of concrete made with these aggregates. Recommendations as to how this might be accomplished are given in chapter 6.

4.9 Impact of Results

The test results presented in this chapter show that concrete behavior is dependent on the raw materials as well as the interaction of those materials. The trial batches made and tested for this project use materials which are available in Northern Nevada so the test results have the greatest relevance in Nevada.

Cement replacement with fly ash is one of the main factors explored in this research. The test results show that the use of up to 25 % replacement of cement with fly ash yields decreased chloride ion permeability. The use of fly ash does not result in any detrimental effects on the other long-term properties of concrete. With the exception of cases in which pozzolans are used as a means to control alkali-silica reactivity, NDOT does not currently allow greater than 17 % cement replacement with fly ash. Based on the data obtained in this project, NDOT should allow the use of cement replacement with fly ash at rates up to 25 % by mass. The use of fly ash will result in concrete structures which are less permeable and, therefore, more resistant to damage from corrosion of reinforcing steel.

Coarse aggregate source is another factor which is shown to have significant impact on the long-term behavior of concrete. In this project, it is shown that coarse aggregate source has a significant impact on the elastic modulus, alkali-silica reactivity, and freeze-thaw resistance of concrete. To ensure that durable, long-lasting concrete is achieved in the field, the various effects of the coarse aggregate and the other raw materials must be taken into consideration. The use of a performance based specification system would provide a means to ensure the use of concrete with the desired properties. Chapter 5 presents a proposed performance based specification system for NDOT.

Of the mix designs produced and tested for this project, mix designs which use Paiute or Vega coarse aggregate and 25 % replacement of cement with fly ash are the most suitable for bridge decks in Nevada. With the exception of the use of 25 % fly ash, these mix designs are very similar to Modified EA class concrete currently used by NDOT. All-Lite coarse aggregate is not recommended for this application because test results indicate that it yields concrete with poor freeze-thaw resistance. Paiute and Vega coarse aggregates have the potential to participate in alkali-silica reactivity, however, the use of 25 % fly ash should help mitigate this reaction. The use of 25 % fly ash also reduces permeability significantly. At 120 days, concrete without fly ash has a chloride ion permeability of approximately 5000 coulombs while concrete with 25 % fly ash has a permeability of approximately 1500 coulombs.

CHAPTER 5
Proposed Concrete Performance Grading System

5.1 Introduction

The results of trial batch testing show that there are significant effects on the long-term properties of concrete due to the use of different sources of raw materials as well as the interaction of those materials. The various effects of different raw materials, different combinations of those materials, and their mix proportions must be realized and accounted for to ensure long lasting, durable concrete. The use of a concrete performance grading system will ensure that concrete with known performance properties is utilized.

Table 5.1 - Suggested Concrete Performance Grades for NDOT[1]

Performance Parameter	Test Method	Concrete Performance Grade			
		0	1	2	3
Freeze-Thaw Durability (x = relative dynamic modulus of elasticity)	ASTM C 666	NA	60% ≤ x<80%	80% ≤ x<90%	x ≤ 90%
Scaling Resistance (x = visual rating of surface after 50 cycles)	ASTM C 672	NA	x = 2, 3	x = 1	x = 0
Abrasion Resistance (x = avg. depth of wear in mm)	ASTM C 944	NA	2.0>x ≥ 1.0	1.0>x ≥ 0.5	0.5>x
Chloride Penetration (x=coulombs)	ASTM C 1202	NA	4000 ≥ x>2000	2000 ≥ x>1000	1000 ≥ x
Elasticity (x = modulus of elasticity)	ASTM C 469	NA	2.5 ≤ x<4.5(10^6) psi	4.5 ≤ x<6.5 (10^6) psi	x ≥ 6.5 (10^6) psi
Sulfate Resistance (x = expansion at given age)	ASTM C1012	NA	x ≥ 0.10% at 6 months	x ≥ 0.10% at 12 months	x ≥ 0.10% at 18 months
Shrinkage (x = microstrain)	ASTM C 157	NA	800>x ≥ 600	600>x ≥ 400	400>x
Creep (x=microstrain/pressure unit)	ASTM C 512	NA	0.5 ≥ x>0.3/psi	0.3 ≥ x>0.2/psi	0.2/psi ≥ x

1. Compressive strength should be specified based on the desired value at a given age. Strengths lower than 2500 psi at 28 days shall not be specified.
Desired fresh concrete properties such as slump and unit weight should be specified.
Maximum aggregate size should be specified to ensure that concrete can flow into forms without segregation or void formation.
All aggregates proposed for use shall be tested for alkali silica reactivity according to ASTM C 1260. Aggregates yielding expansion less than 0.10 % at 16 days shall be considered to have acceptable ASR performance. Aggregates yielding expansion greater than 0.10% shall only be used in combination with materials which have been shown to mitigate the alkali silica reaction according to ASTM C 227. Combination of materials yielding expansions less than 0.05 % at 3 months and 0.10 % at 6 months shall be considered acceptable.

5.2 Proposed Concrete Performance Grades

FHWA's method for grading concrete performance is a useful and practical system. However, based on the results of trial batch testing and research done for this project, some modifications to FHWA's system would yield a grading system which is more suitable to the specific needs of NDOT. Table 5.1 is a suggested concrete performance grading system for NDOT. Table 5.1 differs from FHWA's HPC grades in several respects. FHWA's performance grades are given in Table 5.2. Each of the performance parameters suggested for NDOT, as well as the reasoning behind the performance grades will be briefly described. It should be noted that some of the parameters given in Table 5.1 are based on the performance grades suggested for the Pennsylvania Department of Transportation (PennDOT) in a report by Paul J. Tikalsky (53).

Table 5.2 - FHWA Performance Grades for HPC[1]

Performance Parameter	High Performance Concrete Grade 1	2	3	4
Freeze-Thaw Durability (x = relative dynamic modulus of elasticity)	$60\% \leq x < 80\%$	$x \geq 80\%$		
Scaling Resistance (x = visual rating of surface after 50 cycles)	x=4, 5	x=2, 3	x=0, 1	
Abrasion Resistance (x = avg. depth of wear in mm)	$2.0 > x \geq 1.0$	$1.0 > x \geq 0.5$	$0.5 > x$	
Chloride Penetration (x=coulombs)	$3000 \geq x > 2000$	$2000 \geq x > 800$	$800 \geq x$	
Strength (x = compressive strength)	$6 \leq x < 8$ ksi	$8 \leq x < 10$ ksi	$10 \leq x < 14$ ksi	$x \geq 14$ ksi
Elasticity (x = modulus of elasticity)	$4 \leq x < 6(10^6)$ psi	$6 \leq x < 7.5(10^6)$ psi	$x \geq 7.5(10^6)$ psi	
Shrinkage (x = microstrain)	$800 > x \geq 600$	$600 > x \geq 400$	$400 > x$	
Creep (x=microstrain/pressure unit)	$0.52 \geq x > 0.41$/psi	$0.41 \geq x > 0.31$/psi	$0.31 \geq x > 0.21$/psi	$0.21/\text{psi} \geq x$

[1]Table based on Reference (37).

5.2.1 Grade "0"

There are very few applications in which it would be required for the engineer to specify performance grades for all of the parameters. In most situations, some of the parameters would require higher levels of performance and the other parameters would not be important. For these situations, the engineer would specify a performance grade of "0" for the parameters which are not critical to the application. Since the FHWA grading system focuses on high-performance concrete they do not have a grade "0."

5.2.2 Freeze-Thaw Criteria

FHWA's highest freeze-thaw performance grade, given in Table 5.2, is "2" for a relative dynamic modulus of elasticity of 80%. Grade "2" for freeze-thaw resistance in Table 5.1 is for a relative dynamic modulus between 80 and 90%. Table 5.1 also includes a grade of "3" for a relative dynamic modulus greater than 90%. A freeze-thaw grade of "3" has been included in Table 5.1 because FHWA's highest grade, "2," covers too broad a range of possible performance. There is a significant difference in the freeze-thaw resistance between concrete with relative dynamic modulus of elasticity values of 80% and 100%. However, both were included in grade "2" by FHWA. Including grade "3" for freeze-thaw resistance in Table 5.1 ensures relatively consistent performance within the range of values covered by each grade.

The use of a freeze-thaw performance grade would make it no longer necessary to have an air content specification within the code. Currently the specification for air content is in place to ensure good freeze-thaw resistance. The air content test would still need to be performed on fresh concrete to make sure that it meets mix design requirements, however, the actual freeze-thaw resistance of each mix design would be evaluated using ASTM C 666. In the code, a minimum air content could be specified but this would not be consistent with a performance based specification system.

5.2.3 Scaling Resistance

FHWA's scaling resistance grade of "1," given in Table 5.2, corresponds to a visual surface rating of 4 or 5. Visual ratings of 4 and 5 indicate very poor scaling resistance. For Table 5.1, visual ratings of 4 and 5 were not included under any grade. The lowest scaling resistance grade in Table 5.1 is for visual surface ratings of 2 and 3 which indicate moderate scaling resistance. Visual ratings of 4 and 5 were not included in Table 5.1 because there is no case in which it would be necessary or desired to specify poor scaling resistance. In applications where the scaling resistance is not an important factor, rather than specifying poor scaling resistance, the design engineer would specify a grade of "0" for scaling resistance, indicating that it is not an important parameter for the application in question..

5.2.4 Abrasion Resistance

Table 5.1 uses the same grading scale for abrasion resistance as FHWA. The abrasion resistance of concrete is influenced by the hardness of the paste as well as the hardness of the aggregates. Concrete which is very resistant to abrasion will experience little or no surface wear. No surface wear indicates that the mortar alone is hard enough to resist abrasion. In less resistant concrete the mortar will wear off, exposing the aggregates. When the aggregates are exposed, they are directly subjected to the abrasive action and, therefore, begin to play a role in abrasion resistance. The highest grade for abrasion resistance, "3," corresponds to a depth of wear less than 0.5 mm. This indicates a situation in which the paste/mortar layer at the surface is very hard and sufficient to resist abrasion. Grades "1" and "2" are for situations in which the surface mortar is less resistant to abrasion and will wear off so that the aggregates are exposed.

5.2.5 Chloride Ion Penetration

In FHWA's grading system for chloride ion penetration, 800 coulombs is the dividing line between grade "2" and grade "3." In Table 5.1, the dividing line between grades "2" and "3" has been shifted to 1000 coulombs. Also, FHWA's grade "1" is for values between 2000 and 3000 coulombs but grade "1" in Table 5.1 is for a range of 2000 to 4000 coulombs. These changes were made so that the range of each performance grade would correspond to the penetrability description ranges set forth in ASTM C 1202 as given in Table 2.5. In Table 5.1, the chloride ion penetration grade of "3" corresponds to the ASTM C 1202 range for "negligible" or "very low" penetrability, grade "2" corresponds to "low" penetrability, and grade "1" corresponds to "moderate" penetrability.

5.2.6 Modulus of Elasticity

The modulus of elasticity grades given in Table 5.1 are significantly different from those given by FHWA. Comparison of Table 5.1 with the FHWA grades in Table 5.2 shows that the suggested grading scale for NDOT includes lower values than FHWA. Also, FHWA's grade "3" for elasticity requires a higher level of performance than that suggested for NDOT.

The grade scale suggested for NDOT is meant to cover elasticity values which are representative of normal and high strength concrete. This grading scale is based on the results of testing from this research project as well as modulus of elasticity test results from Texas (45). The majority of the elastic modulus data from this research project is between 2500 and 4500 ksi. This was taken to be representative of elasticity values for normal strength concrete. The range of values obtained from tests in this project roughly corresponds to the suggested range for grade "1." The Texas data contains concrete made from different aggregate types with 56 day strengths ranging from about 6500 to 16500 psi. The modulus of elasticity values from Texas range from about 4500 to 8000 ksi. The Texas data was used to represent concrete with strengths and elasticity values higher than those obtained in this project. The combination of the data from Texas and the data from this project show that modulus of elasticity values tend to range between 2500 and 8000 ksi. The grading scale suggested for NDOT divides this range into 3 performance grades.

5.2.7 Shrinkage

The suggested performance grades for shrinkage coincide with the values suggested by FHWA. The range of values included in the rating scale is from 0 to 800 microstrains. The amount of shrinkage for plain, normal weight concrete due to moisture loss from a saturated state to equilibrium with an ambient 50 % relative humidity is from 400 to 800 microstrains (40). This corresponds with the suggested range for grades "1" and "2." Grade "3" is for shrinkage values less than 400 microstrains which are not usually achieved in normal concrete.

5.2.8 Creep

Since there were no creep tests performed for this test, the creep grades suggested in Table 5.1 are based on FHWA's suggestions rather than experimental data. The range of creep values given in Table 5.1 is essentially the same as those given by FHWA. However the creep grades in Table 5.1 are slightly different than FHWA's. Grade "1" in Table 5.1 is for creep values from 0.3 to 0.5 microstrains/psi. In FHWA's grading system, this roughly corresponds to the range described by grades "1" and "2." The range for grades "2" and "3" for creep in Table 5.1 were determined by rounding FHWA's grades of "3" and "4" to the nearest tenth.

5.2.9 Compressive Strength

Compressive strength has not been included as a graded performance characteristic in Table 5.1. FHWA's performance grades of "1" and "2" each cover a range of 2000 psi. FHWA grade "3" covers a range of 4000 psi. Also FHWA's grades for strength make no allowance for strengths lower than 6000 psi.

Because strength is such a fundamental property in concrete design, it should not be specified by a performance grade which covers a broad range of possible values. Rather, compressive strength should continue to be specified in terms of the actual strength desired at a given age. The lowest 28-day compressive strength allowed in Table 1 of NDOT 501.03.04 is 2500 psi. This limitation has been included in the proposed grading system. The footnotes to Table 5.1 state that a strength of less than 2500 psi at 28 days shall not be specified.

In this research project, compressive strength tests were performed at both 28 and 56 days. As discussed in chapter 4, the data shows that a significant strength gain can occur after 28 days. Currently, NDOT uses the 28-day strength as a basis for acceptance, however, certain applications may not require the full design strength at 28 days. For such cases, the designer might choose to specify that the design strength be obtained at 56 days. As part of the acceptance process, the designer could also require that the concrete provider submit a strength curve demonstrating the rate of strength gain for a given mix design over time. For such a concrete batch to be accepted, it would have to meet or exceed the 28 day strength given on the strength curve and it would also be required to meet the required compressive strength at 56 days. At 28 days, any concrete batch which did not demonstrate that it had gained adequate strength according to the strength curve would be rejected.

5.2.10 Alkali-Silica Reactivity

NDOT's current policy regarding alkali-silica reactivity is given in Section 706 of the Standard Specifications. All aggregate sources must be tested according to ASTM C 289. If the aggregate fails this test requirement then it must be used in accordance with an approved Type N or Type F pozzolan or a Type IP cement. When a pozzolan is used for this purpose, it is used with cement at a rate of 1 part pozzolan to 4 parts cement by mass.

FHWA does not have performance grades for ASR. However, it does recommend that HPC aggregates should be tested for alkali-silica reactivity according to ASTM C227. FHWA recommends that, for this test, mortar bars should have less than 0.05% expansion at 3 months and less than 0.10% expansion at 6 months. The test method used for ASR in this project, ASTM C 1260, specifies that it should only be used to determine the reactivity of aggregates and not combinations of aggregates with different cementitious materials. AASHTO T 303, which is essentially the same as ASTM C 1260, states that mortar bars with an expansion of more than 0.10% at 16 days indicate potentially deleterious expansion.

For the suggested performance criteria for NDOT, it is recommended that all aggregate sources be tested according to ASTM C1260. Aggregates which yield expansions less than 0.10% at 16 days should be considered to have acceptable ASR performance. Aggregates which yield expansions greater than 0.10% at 16 days should be considered at high risk to participate in alkali-silica reactions. These high-risk aggregates should then be tested in combination with proposed materials for ASR mitigation, such as pozzolans, according to ASTM C 227. High-risk aggregates would only be accepted for use when used in combinations with materials which produced expansions less than 0.05 % at 3 months and 0.10 % at 6 months according to ASTM C 227.

5.2.11 Sulfate Resistance

FHWA does not include sulfate resistance as a graded performance parameter but it is an important factor relating to durability. Sulfates present in soil and water that is in contact with concrete may cause sulfate attack. Sulfate attack occurs when sulfates penetrate into the concrete and cause an expansive reaction which can lead to structural problems and deterioration. Sulfate attack was not included as a research parameter for this project and it is not needed for bridge deck applications. However, sulfate resistance has been included in the suggested performance criteria for NDOT because, depending on the sulfate exposure in a given application, it can have a significant effect on durability.

The test specified for sulfate resistance is ASTM C1012. For this test, mortar bars are placed in a sulfate solution and the amount of expansion is measured over time. Studies of sulfate attack have "found that the time required to develop an expansion of 0.10% correlates quite accurately with ultimate resistance performance" (54). Research has shown that certain combinations of cement and fly ash can yield relatively good sulfate resistance with expansions significantly less than 0.10% at one year (41).
The grades suggested for NDOT correspond to the age at which an expansion of 0.10% is reached. Specimens which take longer to reach this level of expansion have higher levels of sulfate resistance.

5.2.12 Unit Weight

Unit weight is an important concrete property but it is not directly related to long-term durability or mechanical performance of concrete. The main effect of unit weight is that it governs the dead load of the structure. Unit weight has not been included as one of the performance criteria in Table 5.1. However, in situations where a specified unit weight is desired, as in the case of lightweight concrete, the design engineer must specify the required unit weight on the plans.

5.2.13 Maximum Aggregate Size

The maximum coarse aggregate size must be specified by the engineer to ensure that concrete can flow into the forms and between the reinforcement without experiencing segregation and forming voids. In practice, mix designs using aggregates smaller than or equal to the maximum size specified would be acceptable for use.

5.2.14 Slump

The footnotes to Table 5.1 state that the desired slump must be specified by the engineer. The nature of slump variability and the use of superplasticizers present some issues regarding long-term performance and mix design acceptance. Because of these issues, there are some different methods which may be used in the treatment of slump specification and mix design acceptance. These slump issues as well as some different methods which may be used for slump specification are presented here.

The required slump values for different applications can have wide variability. Through the application of water reducers, a single mix design with a constant water/cementitious materials ratio can achieve almost any slump within the range of normal applications. However, this requires that varying amounts of water reducer are used for different slump values.

If it is assumed that the use of water reducer has no effect on the long-term properties of the concrete, then the design engineer would only be required to specify the desired slump and the concrete supplier would adjust the water reducer addition rate to meet the requirement. However, there is some evidence that higher dosages of superplasticizers can cause reductions in compressive strength (46). In this case, adjustments to the water reducer dosage rates may constitute significant changes to the mix design because the long-term behavior is effected.

There are two safe possibilities for the application of slump specifications. First, the engineer could specify the desired slump with an acceptable range of variation. It would then be the responsibility of the concrete supplier to submit a mix design with a set superplasticizer dosage rate. Mix design acceptance would be determined based on documentation of trial batch testing showing that the mix design in question meets all long-term performance requirements as well as achieving a slump within the acceptable range of variability. This option would require trial batch testing of every mix design with a different superplasticizer dosage rate. This method also presents a problem in terms of slump adjustments in the field. For instance, if the set superplasticizer dosage has already been applied but the desired slump has not been attained, then no additional admixture addition would be allowed and the batch would have to be rejected.

The other option would be to include slump as a graded performance criteria. Each slump grade would cover a range of values just like all of the other performance parameters. Table 5.3 is a suggested grading system for slump.

Table 5.3 - Suggested Slump Grades[1]

Performance Parameter	Test Method	Grade 0	Grade 1	Grade 2	Grade 3
Slump x = slump (inches)	ASTM C 143	$0" \leq x < 2"$	$2" \leq x < 5"$	$5" \leq x < 8"$	$x \geq 8"$

1. Based on Reference (53).

From the design engineer's point of view, the specification of slump would be no different from the current method. The desired slump would be specified with some acceptable range of variability. The concrete supplier would then be required to submit a mix design with a slump grade which includes the specified slump value.

The main benefit of using a slump grade is that it provides a way to allow for some water reducer adjustments to be made in the mix design without presenting a significant change in the mix design. As an example consider slump grade "2." Any slump value between 5" and 8" would be specified as grade "2." The amount of water reducer required to achieve different slump values within this range is variable. For the same mix design, it would require more superplasticizer to achieve a 7" slump than a 5" slump. However, under the slump grading system, it is assumed that, within the range of slump values specified by one grade, there is little effect on long-term performance due to varying rates of superplasticizer dosage. This means that if a slump of 7" was specified by the engineer, then a mix design that had a 5" slump in trial batch tests could be modified to have a slightly higher superplasticizer dosage.

Some issues arise from the possible use of slump grades. All of the other performance parameters given in Table 5.1 are long-term properties of concrete in which higher performance grades indicate better performance. For the parameters in Table 5.1, a certified mix design which has performance grades equal to or greater than those specified for a job would be acceptable. This would not be the case for slump. The acceptance of a mix design for slump criteria would require that it be certified through trial batch tests to have a slump range including the value specified. Also, for any parameter in Table 5.1, a required grade of "0" indicates that, in the specified application, no specific level of performance is required. The grade of "0" for slump suggested in Table 5.3, however, does not indicate that a slump is not an important criteria for the application in question. Rather, the suggested grade "0" for slump specifies low slumps in the range of 0 to 2 inches.

5.3 Application of Performance Grades

The adoption of a concrete performance grading system would require a system to ensure that concrete with the desired properties is used for each job. It would be very costly to test all of concrete produced in the field for each of the performance parameters. Rather than testing every batch produced for all of performance properties, it is more efficient to evaluate the performance of proposed mix designs based on trial batch testing. To accomplish this, it is necessary to establish a system in which mix designs are certified for their level of performance in each of the performance parameters.

5.3.1 Mix Design Submittal and Certification

Every mix design submitted for approval would require certification through trial batch testing. Initially, testing each mix design for all of the performance parameters will allow them to be classified using the performance grades. After a mix design has been accepted for use by NDOT and tested for performance, it would require re-certification based on trial batch testing every two years. The properties of raw materials from the same supplier can vary over time. Re-certification tests will ensure that the performance properties of a mix design are known even though variations in material properties may occur.

Any mix design modification which relates to mix proportions or materials would require new certification through trial batch testing to re-evaluate performance. Some examples of mix design modifications which would require re-certification tests are changing aggregate or cement source, changing aggregate gradation, changing the water/cementitious materials ratio, or changing the admixture source. Variations in water reducer addition rates would be allowed due to the variations in required slump as long as the slump stayed within the same performance range.

5.3.2 Specification and Acceptance of Concrete Mix Designs

When specifying concrete, the design engineer would specify the minimum required performance grades for all of the performance parameters. Parameters which are not critical for a given job would be specified with a grade of "0" indicating that they are not applicable. The required compressive strength at a given age and the slump would also be specified by the engineer. The concrete supplier would then be required to submit a mix design with current certification documentation. Mix design acceptance for a given job requires that it meets or exceeds the minimum performance requirements specified by the engineer.

5.3.3 Concrete Acceptance in the Field

Thus far, the discussion of concrete performance grades has only considered mix design certification tests and not any test of the actual concrete produced in the field. Under the proposed concrete grading system, the fresh concrete tests currently required by NDOT as well as the compressive strength test would still be required. Air content, unit weight, slump, and compressive strength would be used as a basis for evaluation and acceptance of concrete produced in the field.

5.3.4 - Example of Concrete Specification

Consider the following example: suppose an engineer specifies the desired performance criteria given in Table 5.4 and mix designs A and B have been submitted by the concrete supplier.

Mix design A is acceptable for the application in the example because it meets or exceeds all of the desired performance criteria. It should be noted that the aggregates used in mix design A do not pass the ASTM C 1260 alkali silica reactivity test requirements which are given in the footnotes of Table 5.1. However, the combination of aggregates and cementitious materials was tested according to ASTM C 227 and found to have acceptable performance. The materials used in mix design A are sufficient to mitigate ASR expansion so the mix design is acceptable.

Mix design B is not acceptable because it does not meet the minimum requirements for scaling resistance and chloride penetration. The aggregates used in mix design B were tested for ASR according to ASTM C 1260 so certification of this mix design did not require testing the combination of aggregates and cementitious materials according to ASTM C 227.

Table 5.4 - Concrete Specification Example

Performance Parameter	Desired Performance Criteria	Mix Design A - Certified Grades	Mix Design B - Certified Grades
Freeze-Thaw	Grade 2	Grade 3	Grade 2
Scaling Resistance	Grade 2	Grade 2	Grade 1
Abrasion Resistance	Grade 1	Grade 2	Grade 1
Chloride Penetration	Grade 3	Grade 3	Grade 2
Elasticity	Grade 1	Grade 1	Grade 2
Sulfate Resistance	Grade 0	Grade 1	Grade 2
Shrinkage	Grade 1	Grade 1	Grade 2
Creep	Grade 1	Grade 2	Grade 1
Strength	6000 psi at 28 days	6500 psi at 28 days	8000 psi at 28 days
Slump	5"	Grade 2	Grade 2
Maximum Aggregate Size	¾ inches	¾ inches	¾ inches
Aggregates pass ASTM C 1260 requirement		NO	YES
ASR mitigation acceptable by ASTM C 227		YES	NOT APPLICABLE

The treatment of slump in the previous example assumes the inclusion of slump as a graded performance parameter. The design engineer specified a slump of 5 inches which is within the range specified by grade "2." Mix designs A and B are both certified to have a grade of "2" in slump so they both pass the slump requirement.

5.4 Summary of Performance Grades Achieved by Concrete Produced in this Project

Table 5.5 shows the performance of the concrete produced for this project as rated by the grading system recommended for NDOT in Table 5.1. Table 5.5 also shows some factors which were shown to significantly effect the performance parameters. With the exception of higher levels of cement replacement with fly ash, concrete trial batches produced and tested for this research project are very similar NDOT Modified EA class concrete. Therefore it is reasonable to conclude that, with the exception of performance enhancements due to the use of fly ash, the performance achieved in concrete in this project is similar to the performance currently achieved in Modified EA concrete produced in the field. Table 5.5 shows that higher performance grades are attainable through the use of materials available in Nevada.

Table 5.5 - Performance Levels Achieved in This Project

Performance Parameter	Performance Grades Achieved in This Project	Notes and Factors Which Significantly Effect Performance as Determined from Trial Batch Testing
Freeze-Thaw	0, 1, 3	Concrete with All-Lite coarse aggregate yielded grades 0 and 1, Paiute and Vega aggregate concrete yielded grade 3.
Scaling Resistance	0, 1, 2, 3	No consistent trends were established from trial batches. It is believed that the method of surface finishing has a significant effect on scaling resistance.
Abrasion Resistance	Test Not Conducted	
Chloride Penetration	0, 1, 2, 3	Resistance to chloride penetration resistance improves with increasing amounts of fly ash addition. Significant improvements in permeability can occur over time.
Elasticity	0, 1	All-Lite aggregate yielded grades of 0 and 1. Paiute and Vega aggregates yielded grade 1.
Sulfate Resistance	Test Not Conducted	
Shrinkage at 32 weeks	0,1,2	Vega aggregate tends to have better shrinkage performance than the other two aggregate sources.
Creep	Test Not Conducted	
Strength	3500 - 6500 psi	Concrete may gain over 1000 psi after 28 days
ASTM C 1260	0.08 to 0.59 % expansion	All-Lite coarse aggregate does not experience deleterious expansion. Vega coarse aggregate and Paiute coarse and fine aggregates experience expansions indicative of ASR.
ASTM C 227	Test Not Conducted	Under the suggested grading system in Table 5.1, Vega and Paiute aggregate concrete would require this test before acceptance.

5.5 Suggestions for the Application of Concrete Performance Grades

Table 5.6 contains suggestions for which performance grades should be specified for different levels of environmental exposure and structural requirements. The applications of performance grades set forth in Table 5.6 are only suggestions. It is the responsibility of the design engineer to evaluate the levels of performance required for each job and specify performance grades accordingly. Table 1.4 in chapter 1 gives recommendations for the application of FHWA's high performance concrete grades for freeze-thaw durability, scaling resistance, abrasion resistance, and chloride ion penetration. This table uses criteria such as the number of freeze-thaw cycles per year, the amount of deicer salt applied each year, and the average daily traffic with studded tires. While the grades suggested for NDOT are different than those given by FHWA, the recommendations given in Table 1.4 may still be useful.

Table 5.6 - Suggestions for the Application of Performance Grades[1]

Performance Parameter	Concrete Performance Grade			
	0	1	2	3
Freeze-Thaw Durability	Concrete not exposed to freezing and thawing.	Exposed to freeze-thaw environment.	Exposed to freeze-thaw environment and deicer salts.	Concrete will be saturated with water and exposed to freeze-thaw environment and deicer salts
Scaling Resistance	Concrete not exposed to deicing salts.	Indirect exposure to deicer salts.	Exposed to direct application of deicer salts.	Exposed to direct application of deicer salts and surface loading (traffic).
Abrasion Resistance	Concrete not exposed to any surface abrasion.	Concrete only exposed to abrasion due to tire wear from traffic at constant speed.	Concrete exposed to tire wear in an area of accelerating or decelerating traffic (i.e. off-ramps and on-ramps).	Concrete exposed to abrasive action from tire studs or chains.
Chloride Penetration	Concrete not exposed to chloride salts or a soluble sulfate environment.	Concrete will be in a dry environment and exposed to chlorides or sulfates.	Concrete will be in a moist environment and exposed to chlorides or sulfates.	Concrete will be in a saturated condition when exposed to chloride salts.
Elasticity	Structural stiffness not critical.	Normal structural stiffness needed.	Higher than normal stiffness needed.	High stiffness is critical to structural performance.
Sulfate Resistance[2]	Exposed to soil containing less than 0.10% (by-weight) water-soluble sulfate.	Exposed to soil containing 0.10% to 0.20% (by-weight) water-soluble sulfate.	Exposed to soil containing 0.2% to 2% (by-weight) water-soluble sulfate.	Exposed to soil containing greater than 2% (by-weight) water-soluble sulfate in soil.
Shrinkage	Concrete is not exposed to moisture, chloride salts or soils containing soluble sulfates.	Concrete member is constructed with joints to allow some movement due to volumetric changes.	Concrete member is not constructed with joints and contains physical restraints which will not allow volume changes.	Concrete must be watertight and crack-free.
Creep	Long-term deformations are not an important design consideration.	Higher long-term deformations are acceptable.	Moderate long-term deformations are acceptable.	Small long-term deformations are critical to the structural performance.

1. Table is partially based on Reference (53).
2. Ranges of water soluble-sulfate concentrations correspond to "negligible," "moderate," "severe," and "very severe" sulfate exposure as defined in Reference (39).

5.6 Some Potential Effects of Adopting a Concrete Performance Grading System in Relation to Proportioning Limitations in the Standard Specifications

Using a concrete performance grading system would make it no longer necessary to include proportioning specifications and different concrete classes within the code. The current classes of concrete specified by the code have some basic mix proportioning guidelines and limitations. These specifications are in place to ensure good long-term performance. A performance grading system would ensure good long-term performance through certification and trial batch testing so the proportioning limitations set forth in the code would no longer be needed.

As an example, consider air content as specified for modified EA class concrete. The air content range specified by the code for modified EA concrete is from 5 to 7%. This air content specification is in place to ensure good freeze-thaw performance. Under the proposed concrete grading system, good freeze-thaw performance is ensured through trial batch testing of proposed mix designs. Consider a situation in which a freeze-thaw grade of "3" was specified by the design engineer and the concrete supplier submitted a certified mix design with a freeze-thaw grade of "3." Since the actual freeze-thaw performance of this mix design has been determined through direct test there is no need to meet an air content specification in the code. As long as the freeze-thaw performance is adequate, it doesn't matter if the air content meets code requirements. The specific proportions of the mix design don't technically have to meet any specified limitations since long-term performance is determined through direct test. However, it would still be important to do an air content test for batches produced in the field to ensure that they meet the air content requirements of the mix design.

5.7 Benefits of Using A Concrete Performance Grading System

The use of an concrete performance grading system has several benefits. A performance based grading system will allow design engineers to specify desired mechanical and durability properties to optimize concrete performance for each application. The use of a grading system will encourage materials engineers to develop mix designs with better durability properties. Mix designs which do not meet the specified performance grades will not be accepted. The process of developing and testing new mix designs to meet performance criteria will help materials engineers better understand how specific raw materials and mix proportions effect the various performance parameters.

CHAPTER 6
Conclusions and Recommendations

6.1 Summary of Report

To ensure long lasting and durable concrete for bridge decks in Northern Nevada, all of the loading conditions and environmental factors to which the concrete will be exposed must be considered. For concrete to perform as desired, it must be designed or engineered for the specific application and exposure conditions that it must withstand. In order to do this, it is necessary to understand how the durability and mechanical properties of the concrete will be affected by the properties of the raw materials and the interaction between those materials. Currently, strength, slump, air content, and maximum aggregate size are the main parameters used to specify concrete. While these are important properties, they are not sufficient to ensure long-term durability. Due to the fact that high compressive strengths do not guarantee good durability, obtaining high strengths was not an objective of this study.

Many states have begun to use the concept of high-performance concrete as a means to achieve the desired levels of concrete performance. FHWA's definition of high-performance concrete uses a system in which different levels of performance are specified by a grading system for each performance parameter. This system provides a way for engineers to specify the behavior of concrete over a wide range of durability and mechanical properties.

For this project, several mix designs were developed using raw materials available in Nevada. These mix designs incorporate various combinations of two cement sources, three coarse aggregate sources, one fine aggregate source, and two fly ash sources at three rates of addition. Trial batches were produced and tested according to the test methods set forth in FHWA's definition of high performance concrete. The results of these tests and the effects of the different combinations of raw materials are described in detail in chapter 4.

Chapter 5 proposes a concrete performance grading system for NDOT. The proposed grading system is very similar to FHWA's but it includes some additional parameters and modifications to the FHWA system. The proposed grading system was created to provide a concrete specification method for the range of environmental exposure and loading conditions that are critical to concrete in Nevada.

6.2 Conclusions

1) The different properties of the raw materials investigated in this project can have a significant impact on the long-term durability and mechanical properties of concrete.

Understanding how different raw materials affect the long-term performance of concrete is crucial to producing quality mix designs. Chapter 4 discusses the effects of the different raw materials in detail. As an example of how a single material can effect long-term concrete performance, consider coarse aggregate from All-Lite Pit. Compared to the other two coarse aggregate sources investigated in this project, All-Lite aggregate produces concrete with a lower unit weight, a lower elastic modulus, lower resistance to freeze-thaw action, and much lower levels of alkali-silica reactivity.

2) Replacement of cement with up to 25 % Class F fly ash is a reliable way to achieve chloride ion penetration resistance.

Trial batch testing shows that fly ash reduces permeability. An addition rate of 15% is sufficient to reduce permeability significantly. As the rate of addition is increased up to 25%, trial batch testing shows that chloride ion penetration resistance continues to increase. Cement replacement with fly ash is an effective way to protect bridge decks, pavements and other concrete elements from chloride ion penetration.

3) Of the mix designs tested in this project, mix designs with 25 % Class F fly ash and Paiute or Vega coarse aggregate are the most suitable for bridge decks in Nevada.

The results of trial batch testing indicates that 25 % cement replacement with fly ash yields the highest level of resistance to chloride ion penetration. Paiute and Vega coarse aggregates both produced concrete with good freeze-thaw resistance but they are highly likely to participate in alkali-silica reactions. However, the use of 25 % fly ash with these aggregates should be sufficient to mitigate the alkali-silica reactivity.

4) The use of an HPC specification system ensures that durable concrete is produced by allowing more freedom in the mix design process while still providing a means control the concrete's long-term performance.

Specifications which require a certain aggregate size, aggregate content, cement content, air content, and water cement ratio do little more than give a recipe which will ideally yield concrete with the desired long-term performance. The problem with this system is that a recipe cannot feasibly account for all of the different types of performance that may be required for different loading and exposure conditions. "Recipe" specifications focus on materials and proportions that make up the concrete mixture, while HPC specifications focus on the actual performance of the concrete. Both types of concrete specification are meant to result in concrete with certain levels of performance. The HPC specification system proposed in this paper provides a means to certify that mix designs reach the desired levels of performance. Under the HPC specification system, mix design issues such as maximum aggregate size and proportions would not need to be addressed by the general specifications. The criteria for acceptance is not that the mix design meets required proportions, but that the required levels of performance are met by the mix design provided.

6.3 Recommendations

1) NDOT should adopt the concrete performance grading system proposed in chapter 5 of this report.

Table 5.1 in chapter 5 shows the proposed grading system. The adoption and use of a grading system will allow design engineers to specify desired levels of performance relating to the long-term durability and mechanical behavior of concrete. This will ensure that concrete behavior is optimized for the specific exposure and loading conditions that will be experienced for different applications. Certification through trial batch testing of proposed mix designs according to the grading system criteria will ensure that the properties required in the design are realized in the field.

It has been suggested that the number of performance grades for each performance parameter could be reduced to two or three grades rather than the four grades which have been included in the grading system proposed for NDOT. The decision as to whether or not to reduce the number of performance grades should be based on the applications for which NDOT desires to use an HPC specification system. The system proposed in this report was meant to cover a wide range of possible applications. If HPC specifications are only adopted for use on certain applications, then the number of grades and the levels of performance covered by those grades could be adjusted according to desired performance levels and exposure conditions.

2) NDOT should change section 501.02.03 of the Standard Specifications to allow the replacement of cement with up to 25% fly ash. Also, cement replacement with fly ash at a 1:1 ratio based on mass should be allowed.

NDOT 501.02.03 currently allows a maximum cement replacement of 17% by mass with pozzolan. NDOT also requires that cement replacement with pozzolans "shall be at a rate of 1.2 kg of pozzolan for each kg of Portland Cement." The results of testing in this research project show that there are significant benefits from replacing cement with fly ash at rates up to 25%. Specifically, higher amounts of cement replacement with fly ash yield higher resistance to chloride ion penetration. Cement replacement with fly ash in this project was made at a rate of 1 lb. fly ash for each lb. of cement replaced. The test results showed that there are no adverse effects due to replacing cement at a 1:1 ratio with fly ash rather than at the 1.2:1 ratio required by NDOT.

3) Further research in the areas of abrasion resistance and creep should be performed.

Even though no creep or abrasion tests were performed for this project, these are still important properties relating to the long-term durability of concrete. Chains and snow tires are used in Northern Nevada during the winter months. The abrasive action caused by these factors presents a significant issue in the design of concrete for bridge decks. The issue of creep is important to the design of bridges because it has an impact on long-term pre-stress losses and deflection. Creep and abrasion resistance both merit further investigation.

4) NDOT should perform further research in the areas of drying shrinkage and scaling resistance.

All of the mix designs tested in this project yielded poor drying shrinkage. The scaling resistance tests yielded inconsistent results due to inconsistent surface finishing. These are both important properties and, in order to obtain durable concrete mixes, methods to ensure good performance in these two areas must be investigated.

5) NDOT should investigate the durability of high-performance concrete in the field through the construction and monitoring of highway test sections.

All of the tests for this project were performed in the lab. While this does provide useful information, it does not provide the same type of data that could be obtained through an actual trial section. The tests used in this project were meant to simulate an actual exposure condition or provide information relating to how concrete performs under different types of loads or exposure conditions which are found in Northern Nevada. However, the data provided by these tests gives little indication of how well the test results actually correlate to the performance of a given concrete mix design in the field. Investigating several mix designs through trial sections and laboratory testing would provide information which would allow the correlation of laboratory data to actual performance in the field. This, in turn, would give design engineers the information needed to understand how concrete specified using HPC parameters will actually perform under field conditions.

6) NDOT should further investigate the use of HPC by using HPC specifications for the construction of a trial bridge.

As stated in Chapter 1, many states have begun to use HPC specifications on selected projects. These projects provide an opportunity to investigate how successful high-performance concrete may be achieved and implemented in the field. A trial project would also provide valuable experience relating to how successful cooperation between the different professional disciplines might be achieved in order to provide durable concrete in an efficient manner.

6.4 Concerns

There are several concerns which were revealed through the results of trial batch testing in this project. These concerns relate to concrete behavior which may represent significant issues affecting the life-span and durability of concrete elements.

6.4.1 Drying Shrinkage

The majority of the concrete developed for this project had high levels of drying shrinkage. None of the mix design variables explored in this research seemed to cause significant reductions in shrinkage. Shrinkage is an important parameter in bridge decks because it can result in cracking of the concrete which may reduce the life-span of the bridge deck and necessitate extensive repair. Also, deicer chemicals can travel through cracks formed by shrinkage and come into contact with reinforcing steel. In such a case, the increased permeability due to cracking would override the reduced permeability benefits of adding fly ash.

6.4.2 Aggregates Which Yield Concrete With Poor Freeze-Thaw Resistance

The limited amount of freeze-thaw data obtained in this research project shows that All-Lite coarse aggregate tends to yield poor freeze-thaw resistance even with an air content as high as 6%. This is an important concern because freeze-thaw damage can cause severe cracking of concrete and reduce the life-span significantly.

6.4.3 Alkali-Silica Reactivity

Of the aggregate sources tested for potential alkali-silica reactivity in this project, All-Lite aggregate was the only one that exhibited good performance. Alkali-silica reactions can cause severe damage in the form of cracking. Effective methods of mitigating ASR must be used to ensure good long- term performance.

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Appendix

Table A3.1 - Mix Designs Included in Research

Mixture Designation	Cement Source	Fly Ash Source	Percent Replacement with Fly Ash	Coarse Aggregate Source
N-0-V	Nevada	None	0 %	Vega
N-B15-V	Nevada	Bridger, WY	15 %	Vega
N-B20-V	Nevada	Bridger, WY	20 %	Vega
N-B25-V	Nevada	Bridger, WY	25 %	Vega
N-D15-V	Nevada	Delta, UT	15 %	Vega
N-D20-V	Nevada	Delta, UT	20 %	Vega
N-D25-V	Nevada	Delta, UT	25 %	Vega
C-0-V	Calaveras	None	0 %	Vega
C-B15-V	Calaveras	Bridger, WY	15 %	Vega
C-B25-V	Calaveras	Bridger, WY	25 %	Vega
C-D15-V	Calaveras	Delta, UT	15 %	Vega
C-D25-V	Calaveras	Delta, UT	25 %	Vega
N-0-P	Nevada	None	0 %	Paiute
N-B15-P	Nevada	Bridger, WY	15 %	Paiute
N-B20-P	Nevada	Bridger, WY	20 %	Paiute
N-B25-P	Nevada	Bridger, WY	25 %	Paiute
N-D15-P	Nevada	Delta, UT	15 %	Paiute
N-D20-P	Nevada	Delta, UT	20 %	Paiute
N-D25-P	Nevada	Delta, UT	25 %	Paiute
C-B15-P	Calaveras	Bridger, WY	15 %	Paiute
C-B25-P	Calaveras	Bridger, WY	25 %	Paiute
C-D15-P	Calaveras	Delta, UT	15 %	Paiute
C-D25-P	Calaveras	Delta, UT	25 %	Paiute
N-0-A	Nevada	None	0 %	All-Lite
N-B15-A	Nevada	Bridger, WY	15 %	All-Lite
N-B20-A	Nevada	Bridger, WY	20 %	All-Lite
N-B25-A	Nevada	Bridger, WY	25 %	All-Lite
N-D15-A	Nevada	Delta, UT	15 %	All-Lite
N-D20-A	Nevada	Delta, UT	20 %	All-Lite
N-D25-A	Nevada	Delta, UT	25 %	All-Lite
C-0-A	Calaveras	None	0 %	All-Lite
C-B15-A	Calaveras	Bridger, WY	15 %	All-Lite
C-B20-A	Calaveras	Bridger, WY	20 %	All-Lite
C-B25-A	Calaveras	Bridger, WY	25 %	All-Lite
C-D15-A	Calaveras	Delta, UT	15 %	All-Lite
C-D20-A	Calaveras	Delta, UT	20 %	All-Lite
C-D25-A	Calaveras	Delta, UT	25 %	All-Lite

Table A3.2 - Mix Design Quantities For a 1 Cubic Yard Batch

Mixture	Cement (lb)	Fly Ash (lb)	Rock (lb)*	Sand (lb)*	Water (lb)
N-0-V	700	0	1707	1049	280
N-B15-V	595	105	1707	1019	280
N-B20-V	560	140	1707	1009	280
N-B25-V	525	175	1707	999	280
N-D15-V	595	105	1707	1022	280
N-D20-V	560	140	1707	1014	280
N-D25-V	525	175	1707	1005	280
C-0-V	700	0	1707	1049	280
C-B15-V	595	105	1707	1019	280
C-B25-V	525	175	1707	999	280
C-D15-V	595	105	1707	1022	280
C-D25-V	525	175	1707	1005	280
N-0-P	700	0	1734	1020	280
N-B15-P	595	105	1734	990	280
N-B20-P	560	140	1734	980	280
N-B25-P	525	175	1734	970	280
N-D15-P	595	105	1734	994	280
N-D20-P	560	140	1734	985	280
N-D25-P	525	175	1734	976	280
C-B15-P	595	105	1734	990	280
C-B25-P	525	175	1734	970	280
C-D15-P	595	105	1734	994	280
C-D25-P	525	175	1734	976	280
N-0-A	700	0	1498	1037	280
N-B15-A	595	105	1498	1007	280
N-B20-A	560	140	1498	997	280
N-B25-A	525	175	1498	987	280
N-D15-A	595	105	1498	1011	280
N-D20-A	560	140	1498	1002	280
N-D25-A	525	175	1498	993	280
C-0-A	700	0	1498	976	280
C-B15-A	595	105	1498	1007	280
C-B20-A	560	140	1498	997	280
C-B25-A	525	175	1498	987	280
C-D15-A	595	105	1498	1011	280
C-D20-A	560	140	1498	1002	280
C-D25-A	525	175	1498	993	280

*Note: Aggregate quantities are based on saturated-surface-dry condition.

Table A3.3 - Admixture Addition Rates and Fresh Concrete Properties

Trial Batch Number	Mix I.D.	HRWR Rate* (oz/100 lb cwt.**)	AE Rate*** (oz/100 lb cwt.**)	Slump (in.)		Air Content(%)	Temperature (degrees F)	Approx. Unit Wt. (lb./(cubic ft.))
				w/o HRWR	w/ HRWR			
1	N-0-V	8	1.5	1.375	4.5	4.5	unknown	unknown
2	N-B15-V	5.22	1.6	3	6.25	5.875	unknown	unknown
3	N-B20-V	5.54	1.6	2.875	6.125	5.125	unknown	unknown
4	N-B25-V	8	1.7	1.75	6.375	5	59.6	142.93
5	N-D20-V	5.45	1.8	2.25	5.75	5.25	53.3	139.95
6	N-D15-V	5.45	1.9	2.75	5.5	5.25	46.3	140.23
7	N-D25-V	5.38	1.9	2.625	5.625	5.25	49.7	140.18
8	C-0-V	10.92	1.5	1.5625	6.125	5.25	59.4	142.91
9	C-B15-V	7.83	1.6	1.875	5.875	4.375	66.6	unknown
10	C-B25-V	2.28	1.9	4.375	5.5	8.5	58.7	133.23
11	C-D15-V	6.19	1.6	1.6875	5.125	5.25	62.8	140.95
12	C-D25-V	7.83	1.6	1.375	6	5.75	unknown	141.17
13^	N-0-P	5.45	1.6	2.25	4.75	4	50	143.19
14	N-B15-P	1.92	1.7	4.25	5.625	6.25	68	138.01
15^	N-B20-P	1.79	1.7	4.25	5.125	6	55.8	138.2
16^	N-B25-P	5.45	1.7	2.375	5.5	3	66.1	144.18
17^	N-D15-P	6.19	2	1.75	5	4	53.1	143.34
18^	N-D20-P	11.58	2	2	6.25	3.25	58.3	144.25
19	N-D25-P	8	4	1.875	6	7	50.4	139.47
20	C-B15-P	9.68	3.5	1.75	5.0625	6.75	56.9	140.13
21	C-B25-P	9.94	3.25	1.125	6.125	6.25	74.2	143.14
22	C-D15-P	9.62	3.25	1.125	4.25	5	75.8	143.49
23	C-D25-P	12.06	3.25	0.625	4.625	4.75	73.3	144.59
24	N-B25-P	4.92	3.25	2.5	5.375	6.25	55.8	140.05
25	N-D20-P	8	3.25	2.25	7.5	6	80.3	141.92
26	N-0-P	8	3.25	1	5.25	5.25	82.1	133.64
27	N-D15-P	5.05	3.25	2.375	6	6.5	75.4	139.24
28	N-B20-P	4.56	3.25	2.75	6	6	77.2	139.52
29^	N-0-A	9.78	3.25	1	4.5	4.25	76.1	136.87
30^	N-0-A	5.05	4	1.625	4.625	4.25	70.7	135.77
31	N-B15-A	5.22	5	1.5	5	6	72.5	132.82
32	N-B20-A	5.22	5	1.625	4.875	5.75	74.5	133.82
33	N-B25-A	5.87	5	1.25	5.625	5.5	76.3	135.14
34	N-D15-A	8	5	0.9375	6.25	6.5	73.6	133.1
35^	N-D20-A	5.15	5	1.8125	5.75	7.5	77.4	129.84
36	N-D25-A	5.05	4	2	4.25	5.75	71.6	132.11
37	C-0-A	4.73	4	2.5	6	6.75	73.8	128.11
38^	C-B15-A	5.05	4	2.25	5.75	8.75	72.5	127.42
39	C-B20-A	5.05	3.25	2	5.5	7	83.2	129.59
40^	C-B25-A	3.26	3	3.25	5.5	8.5	85.3	126.87
41^	C-D15-A	8	2.5	1	4	3.25	73.6	136.59
42^	C-D20-A	3.1	3.25	3	5	7.25	82.1	126.71
43	C-B25-A	5.71	2.75	2	4.5	5.25	79.4	131.17
44	C-B15-A	4.83	2.75	2.5	5	7	81.5	128.65
45^	C-D15-A	9.78	2.75	1	5	4	69.7	135.81
46	C-D20-A	9.29	3	1.25	5.5	5.75	80.3	133.33
47	C-D15-A	8.97	3	1.25	6.5	6	71.8	132.6
48	C-D25-A	4.14	3	2.5	4.5	7	est. 60-70	128.29
49	N-D20-A	9.78	3	1	5.5	5	est. 65-75	133.51
50^	N-0-A	9.78	3	1	4.5	4.25	est. 65-75	136.21
51	N-0-A	8.97	4	1.25	5.75	7	est. 65-75	133.21

* "HRWR" stands for high range water reducer.
** "cwt" stands for cementitious weight which includes cement and fly ash
***"AE" stands for air entrainer.
"Unknown" values are due to equipment failure or human error.
Note: Estimated values ("est.") for temperature are based on air temperature and results from previous tests.
^Trial batch did not meet acceptable criteria for slump or air content.

Table A4.1 Acceptable Batches Based on Fresh Concrete Properties

Trial Batch Number	Mix I.D.	HRWR Rate* (oz/100 lb cwt.**)	AE Rate*** (oz/100 lb cwt.**)	Slump (in.) w/o HRWR	Slump (in.) w/ HRWR	Air Content(%)
1	N-0-V	8	1.5	1.375	4.5	4.5
2	N-B15-V	5.22	1.6	3	6.25	5.875
3	N-B20-V	5.54	1.6	2.875	6.125	5.125
4	N-B25-V	8	1.7	1.75	6.375	5
5	N-D20-V	5.45	1.8	2.25	5.75	5.25
6	N-D15-V	5.45	1.9	2.75	5.5	5.25
7	N-D25-V	5.38	1.9	2.625	5.625	5.25
8	C-0-V	10.92	1.5	1.5625	6.125	5.25
9	C-B15-V	7.83	1.6	1.875	5.875	4.375
10	C-B25-V	2.28	1.9	4.375	5.5	8.5
11	C-D15-V	6.19	1.6	1.6875	5.125	5.25
12	C-D25-V	7.83	1.6	1.375	6	5.75
13^	N-0-P	5.45	1.6	2.25	4.75	4
14	N-B15-P	1.92	1.7	4.25	5.625	6.25
15^	N-B20-P	1.79	1.7	4.25	5.125	6
16^	N-B25-P	5.45	1.7	2.375	5.5	3
17^	N-D15-P	6.19	2	1.75	5	4
18^	N-D20-P	11.58	2	2	6.25	3.25
19	N-D25-P	8	4	1.875	6	7
20	C-B15-P	9.68	3.5	1.75	5.0625	6.75
21	C-B25-P	9.94	3.25	1.125	6.125	6.25
22	C-D15-P	9.62	3.25	1.125	4.25	5
23	C-D25-P	12.06	3.25	0.625	4.625	4.75
24	N-B25-P	4.92	3.25	2.5	5.375	6.25
25	N-D20-P	8	3.25	2.25	7.5	6
26	N-0-P	8	3.25	1	5.25	5.25
27	N-D15-P	5.05	3.25	2.375	6	6.5
28	N-B20-P	4.56	3.25	2.75	6	6
29^	N-0-A	9.78	3.25	1	4.5	4.25
30^	N-0-A	5.05	4	1.625	4.625	4.25
31	N-B15-A	5.22	5	1.5	5	6
32	N-B20-A	5.22	5	1.625	4.875	5.75
33	N-B25-A	5.87	5	1.25	5.625	5.5
34	N-D15-A	8	5	0.9375	6.25	6.5
35^	N-D20-A	5.15	5	1.8125	5.75	7.5
36	N-D25-A	5.05	4	2	4.25	5.75
37	C-0-A	4.73	4	2.5	6	6.75
38^	C-B15-A	5.05	4	2.25	5.75	8.75
39	C-B20-A	5.05	3.25	2	5.5	7
40^	C-B25-A	3.26	3	3.25	5.5	8.5
41^	C-D15-A	8	2.5	1	4	3.25
42^	C-D20-A	3.1	3.25	3	5	7.25
43	C-B25-A	5.71	2.75	2	4.5	5.25
44	C-B15-A	4.83	2.75	2.5	5	7
45^	C-D15-A	9.78	2.75	1	5	4
46	C-D20-A	9.29	3	1.25	5.5	5.75
47	C-D15-A	8.97	3	1.25	6.5	6
48	C-D25-A	4.14	3	2.5	4.5	7
49	N-D20-A	9.78	3	1	5.5	5
50^	N-0-A	9.78	3	1	4.5	4.25
51	N-0-A	8.97	4	1.25	5.75	7

^Trial batch did not achieve acceptable air content or slump.
"Unknown" values are due to equipment failure or human error.
* "HRWR" stands for high range water reducer.
** "cwt" stands for cementitious weight which includes cement and fly ash
***"AE" stands for air entrainer.

Table A4.2 - Compressive Strength Test Results (psi)

Test Set	Mix I.D.	7 day	14 day	28 day	56 day *
1	N-0-V	5133	5909	6527	
2	N-B15-V	3929	4513	4975	6806
3	N-B20-V	3966	4578	5348	7041
4	N-B25-V	3760	4501	5038	7002
5	N-D20-V	3640	4579	4811	6576
6	N-D15-V	4271	4998	5830	7865
7	N-D25-V	3527	3902	4823	7045
8	C-0-V	4251	4709	5243	7302
9	C-B15-V	3694	4241	4718	6198
10	C-B25-V	2308	2921	3472	4967
11	C-D15-V	3512	4090	4985	6763
12	C-D25-V	3478	4112	4757	7393
13^	N-0-P	4147	4911	5461	7149**
14	N-B15-P	2727	3363	3768	5232
15^	N-B20-P	2803	3252	3854	4825
16^	N-B25-P	3339	3879	4589	6103
17^	N-D15-P	3701	4641	5091	6699
18^	N-D20-P	4043	4555	5418	6703
19	N-D25-P	3059	3795	4353	5683
20	C-B15-P	3424	4100	4647	5955
21	C-B25-P	3490	3833	4562	5563
22	C-D15-P	3768	4143	4640	5429
23	C-D25-P	3885	4319	4750	6111
24	N-B25-P	2709	3337	4229	4961
25	N-D20-P	3168	3592	4086	5145
26	N-0-P	3700	3949	4440	2664**
27	N-D15-P	2820	3217	3723	4869
28	N-B20-P	2672	3197	3661	4389
29^	N-0-A	5048	6192	6435	8142
30^	N-0-A	4376	5182	5760	6381
31	N-B15-A	4222	4723	5752	6266
32	N-B20-A	4106	4654	5378	5923
33	N-B25-A	4085	4676	5484	6328
34	N-D15-A	3695	4156	4884	5526
35^	N-D20-A	3257	3898	4534	5405
36	N-D25-A	3400	4071	4909	5626
37	C-0-A	3217	3918	4440	4956
38^	C-B15-A	3106	3770	4368	5014
39	C-B20-A	2732	3685	4079	5080
40^	C-B25-A	1883	2490	3301	3805
41^	C-D15-A	4977	5673	6548	7260
42^	C-D20-A	2160	2699	3301	3692
43	C-B25-A	3153	3771	4436	5444
44	C-B15-A	2913	3336	3887	4606
45^	C-D15-A	4520	4976	5748	6461
46	C-D20-A	3591	4021	4753	5543
47	C-D15-A	3709	4141	4916	5508
48	C-D25-A	2659	3239	3771	4682
49	N-D20-A	4013	4740	5257	5826
50^	N-0-A	5372	5939	6433	7291
51	N-0-A	5131	5892	6316	7309

*Actual test age for sets 1 to 29 is greater than 56 days.
**Due to severe voids in the 56-day concrete specimen for set 26, the 56-day data for set 13 was used in place of set 26 for analysis.
With the exception of the 56-day results for sets 2 - 37, all values given are the average result for three test specimens.
The 56-day results for sets 2 - 37 are from one specimen.
^Trial batch did not meet acceptable criteria for slump or air content.

Table A4.3 - Modulus of Elasticity Test Results

Test Set	Mix I.D.	E (ksi)
1	N-0-V	4175
2	N-B15-V	4087
3	N-B20-V	3828
4	N-B25-V	4033
5	N-D20-V	3750
6	N-D15-V	3790
7	N-D25-V	3727
8	C-0-V	4275
9	C-B15-V	4163
10	C-B25-V	2852
11	C-D15-V	3866
12	C-D25-V	3715
13	N-0-P	3760
14	N-B15-P	3055
15	N-B20-P	3102
16	N-B25-P	4196
17	N-D15-P	3619
18	N-D20-P	3699
19	N-D25-P	3121
20	C-B15-P	3458
21	C-B25-P	3558
22	C-D15-P	3745
23	C-D25-P	3569
24	N-B25-P	3381
25	N-D20-P	3393
26	N-0-P	3788
27	N-D15-P	3198
28	N-B20-P	3283
29	N-0-A	3022
30	N-0-A	2692
31	N-B15-A	2752
32	N-B20-A	2725
33	N-B25-A	2665
34	N-D15-A	2538
35	N-D20-A	2479
36	N-D25-A	2570
37	C-0-A	2438
38	C-B15-A	2497
39	C-B20-A	2409
40	C-B25-A	2312
41	C-D15-A	2965
42	C-D20-A	2137
43	C-B25-A	2501
44	C-B15-A	2434
45	C-D15-A	2706
46	C-D20-A	2505
47	C-D15-A	2467
48	C-D25-A	2478
49	N-D20-A	2524
50	N-0-A	2558
51	N-0-A	2738

With the exception of set 6, all results are the average from three specimens. Set 6 test results are from only one test specimen.

Table A4.4 - Scaling Resistance Test Results

Test Set	Mix I.D.	Avg. 50 day rating
1	N-0-V	5
2	N-B15-V	4
3	N-B20-V	4
4	N-B25-V	4
5	N-D20-V	3
6	N-D15-V	4
7	N-D25-V	3
8	C-0-V	4.5
9	C-B15-V	4
10	C-B25-V	3
11	C-D15-V	3
12	C-D25-V	4
13	N-0-P	3
14	N-B15-P	2
15	N-B20-P	2
16	N-B25-P	3
17	N-D15-P	2.5
18	N-D20-P	2
19	N-D25-P	3
20	C-B15-P	2
21	C-B25-P	3
22	C-D15-P	2.5
23	C-D25-P	3
24	N-B25-P	4
25	N-D20-P	2.5
26	N-0-P	2
27	N-D15-P	2
28	N-B20-P	2
29	N-0-A	0.5
30	N-0-A	0.5
31	N-B15-A	1
32	N-B20-A	1
33	N-B25-A	2
34	N-D15-A	1
35	N-D20-A	3
36	N-D25-A	2
37	C-0-A	1
38	C-B15-A	2
39	C-B20-A	2
40	C-B25-A	4
41	C-D15-A	1
42	C-D20-A	2.5
43	C-B25-A	2.5
44	C-B15-A	2.5
45	C-D15-A	2
46	C-D20-A	4
47	C-D15-A	3
48	C-D25-A	4
49	N-D20-A	4
50	N-0-A	4.5
51	N-0-A	4

Each value shown is the average result from two test specimens.

Table A4.5 - Chloride Ion Penetration Test Results

		Avg. Permeability (Coulombs)			
Set #	Mix I.D.	56 day	Rating	120 day	Rating
1	N-0-V	4333	High	3409	Moderate
2	N-B15-V	3302	Moderate	1855	Low
3	N-B20-V	2538	Moderate	1224	Low
4	N-B25-V	1869	Low	899	Very Low
5	N-D20-V	3914	Moderate	1913	Low
6	N-D15-V	4835	High	1640	Low
7	N-D25-V	3815	Moderate	1313	Low
8	C-0-V	4231	High	3066	Moderate
9	C-B15-V	3300	Moderate	1481	Low
10	C-B25-V	2011	Moderate	1026	Low
11	C-D15-V	3477	Moderate	1468	Low
12	C-D25-V	2147	Moderate	1010	Low
13	N-0-P	5297	High	4340	High
14	N-B15-P	5331	High	2497	Moderate
15	N-B20-P	5593	High	2408	Moderate
16	N-B25-P	2584	Moderate	1397	Low
17	N-D15-P	4534	High	2547	Moderate
18	N-D20-P	3056	Moderate	1833	Low
19	N-D25-P	5229	High	1833	Low
20	C-B15-P	3199	Moderate	1945	Low
21	C-B25-P	1906	Low	1213	Low
22	C-D15-P	3108	Moderate	2159	Moderate
23	C-D25-P	2162	Moderate	1315	Low
24	N-B25-P	2053	Moderate	1251	Low
25	N-D20-P	4188	High	1891	Low
26	N-0-P	7009	High	5816	High
27	N-D15-P	6013	High	2971	Moderate
28	N-B20-P	3665	Moderate	2137	Moderate
29	N-0-A	5652	High	4726	High
30	N-0-A	6579	High	5243	High
31	N-B15-A	4343	High	2235	Moderate
32	N-B20-A	3380	Moderate	1573	Low
33	N-B25-A	2472	Moderate	1198	Low
34	N-D15-A	5388	High	2983	Moderate
35	N-D20-A	5259	High	2393	Moderate
36	N-D25-A	4400	High	2052	Moderate
37	C-0-A	5548	High	4317	High
38	C-B15-A	3473	Moderate	1844	Low
39	C-B20-A	3459	Moderate	1746	Low
40	C-B25-A	3412	Moderate	2015	Moderate
41	C-D15-A	3755	Moderate	2082	Moderate
42	C-D20-A	3985	Moderate	2108	Moderate
43	C-B25-A	2872	Moderate	1835	Low
44	C-B15-A	4387	High	2570	Moderate
45	C-D15-A	3931	Moderate	2404	Moderate
46	C-D20-A	3069	Moderate	2058	Moderate
47	C-D15-A	3296	Moderate	2153	Moderate
48	C-D25-A	6014	High	1622	Low
49	N-D20-A	4377	High	3113	Moderate
50	N-0-A	5984	High	5024	High
51	N-0-A	6858	High	5925	High

Each value shown is the average result from three test specimens.

Table A4.6 - Drying Shrinkage Test Results

		Average Length change (microstrains)						
Set #	Mix I.D.	28 day (M*)	7 day (A**)	14 day (A**)	28 day (A**)	8 week (A**)	16 wk (A**)	32 wk (A**)
1	N-0-V	-60	200	313	423	497	540	613
2	N-B15-V	3	257	370	460	560	593	657
3	N-B20-V	-33	220	357	433	540	560	627
4	N-B25-V	-33	243	363	423	507	483	607
5	N-D20-V	33	297	400	537	610	657	757
6	N-D15-V	83	220	310	470	540	593	667
7	N-D25-V	-50	187	313	410	500	513	617
8	C-0-V	-3	220	350	450	513	617	653
9	C-B15-V	-13	243	330	423	510	577	627
10	C-B25-V	-37	290	420	517	627	677	723
11	C-D15-V	-60	197	280	383	473	513	577
12	C-D25-V	-73	177	280	373	447	497	537
13	N-0-P	0	273	423	603	717	830	880
14	N-B15-P	-33	250	393	557	673	770	830
15	N-B20-P	7	283	487	657	760	850	877
16	N-B25-P	7	190	353	553	587	727	793
17	N-D15-P	7	270	393	583	680	777	833
18	N-D20-P	13	223	317	520	610	747	800
19	N-D25-P	-33	230	393	553	623	770	807
20	C-B15-P	0	293	463	557	603	783	793
21	C-B25-P	13	240	410	527	653	753	790
22	C-D15-P	13	217	357	493	627	720	773
23	C-D25-P	-3	217	303	440	617	700	757
24	N-B25-P	-3	203	347	483	637	747	803
25	N-D20-P	-20	167	357	460	613	720	753
26	N-0-P	0	160	317	510	667	790	820
27	N-D15-P	-10	250	387	550	723	837	870
28	N-B20-P	3	267	440	590	780	857	
29	N-0-A	17	110	230	407	613	730	
30	N-0-A	-20	133	270	450	630	737	
31	N-B15-A	0	100	243	457	633	720	
32	N-B20-A	-20	147	267	500	640	730	
33	N-B25-A	-3	160	297	550	657	733	
34	N-D15-A	-10	123	250	463	590	680	
35	N-D20-A	-10	180	340	557	687	767	
36	N-D25-A	-60	73	230	427	587	683	
37	C-0-A	-53	113	267	447	643	737	
38	C-B15-A	-37	177	307	473	627	713	
39	C-B20-A	-23	127	247	420	603	717	
40	C-B25-A	13	180	283	483	643	723	
41	C-D15-A	-7	103	217	397	570	677	
42	C-D20-A	17	133	290	470	623	697	
43	C-B25-A	-23	97	213	427	607	683	
44	C-B15-A	-17	107	227	440	607	690	
45	C-D15-A	-90	17	163	350	507	620	
46	C-D20-A	-30	63	183	367	520	613	
47	C-D15-A	-30	90	243	403	577	650	
48	C-D25-A	-70	70	230	380	550	647	
49	N-D20-A	0	220	363	530	667	730	
50	N-0-A	0	157	270	500	677	770	
51	N-0-A	-10	137	267	513	660	733	

* "M" indicates amount of time in moist storage.
** "A" indicates amount of time in air storage.
Note: Negative values indicate expansion.
Each value shown is the average result from three test specimens.

Table A4.7 - Calaveras Cement Test Report Results

Data Source Information	
Source	Calaveras Cement Co. Cement Test Report
Report Date	1/11/00
Cement	Type I-II Low Alkali
Date Shipped	Jan-99
Lot #	800

Standard Chemical Requirements ASTM C 114	**Test Results**	**ASTM C 150 Specifications Type I**	**Type II**
Silicon Dioxide, %	21.3	----	20.0 Min
Aluminum Oxide, %	4.2	----	6.0 Max
Ferric Oxide, %	3.3	----	6.0 Max
Calcium Oxide, %	64.6	----	----
Magnesium Oxide, %	1.4	6.0 Max	6.0 Max
Sulfur Trioxide, %	3.1	3.0 Max	3.0 Max
Loss on Ingnition, %	1.2	3.0 Max	3.0 Max
Insoluble Residue, %	0.4	0.75 Max	0.75 Max
Alkalies, %	0.41	0.60 Max	0.60 Max
Tricalcium Silicate, %	59	----	----
Dicalcium Silicate, %	18	----	----
Tricalcium Aluminate, %	6	----	8 Max
Tetracalcium Aluminoferrite, %	10	----	----
Physical Requirements			
(ASTM C 1038) Expansion @ 14 days, %	0.007	0.020 Max	0.020 Max
(ASTM C 204) Blaine Fineness	390	280 Min	280 Min
(ASTM C 430) 325 Mesh, %	94.7	----	----
(ASTM C 191) Time of Setting - Initial (Vicat)	88	45 Min	45 Min
(ASTM C 451) False Set, %	88	50 Min	50 Min
(ASTM C 185) Air Content, %	6.9	12 Max	12 Max
(ASTM C 151) Autoclave Expansion, %	0.002	0.80 Max	0.80 Max
(ASTM C 187) Normal Consistency, %	24.3	----	----
(ASTM C 109) Compressive Strength, psi (Mpa)			
1 Day	1600 (11)	----	----
3 Day	3220 (22.2)	1800 (12.4) Min	1500 (10.3) Min
7 Day	4220 (29.1)	2800 (19.3) Min	2500 (17.2) Min
28 Day	6000 (41.2)	----	----

Table A4.8 - Nevada Cement Test Report Results

Data Source Information	
Source	Nevada Cement Co. Cement Test Report
Report Date	October-99
Cement	Type I-II Low Alkali

Standard Chemical Requirements	**Test**	**ASTM C 150 Specifications**	
ASTM C 114	**Results**	**Type I**	**Type II**
Silicon Dioxide, %	21.2	----	20.0 Min
Aluminum Oxide, %	4.5	----	6.0 Max
Ferric Oxide, %	3	----	6.0 Max
Magnesium Oxide, %	2.3	6.0 Max	6.0 Max
Sulfur Trioxide, %	2.7	3.0 Max	3.0 Max
Loss on Ingnition, %	1.4	3.0 Max	3.0 Max
Insoluble Residue, %	0.44	0.75 Max	0.75 Max
Alkalies, %	0.48	0.60 Max	0.60 Max
Tricalcium Aluminate, %	7	----	8 Max
Physical Requirements			
(ASTM C 204) Blaine Fineness	382	280 Min	280 Min
(ASTM C 191) Time of Setting - Initial (Vicat)	114	45 Min	45 Min
(ASTM C 191) Time of Setting - Final (Vicat)	279		
(ASTM C 185) Air Content, %	6	12 Max	12 Max
(ASTM C 151) Autoclave Expansion, %	0.00	0.80 Max	0.80 Max
(ASTM C 187) Normal Consistency, %	24.3	----	----
(ASTM C 109) Compressive Strength, psi (Mpa)			
3 Day	3920 (27.0)	1800 (12.4) Min	1500 (10.3) Min
7 Day	5290 (36.5)	2800 (19.3) Min	2500 (17.2) Min

Table A4.9 - Delta Fly Ash Test Report Results

Data Source Information	
Source	CTL Commercial Testing Laboratories Report
Report Date	8/12/98
Plant of Origin	IPSC/Delta
Sample ID	135T
Sample Date	6/8/98
Date Received	7/6/98

Chemical Composition	**Test Results**	**ASTM C 618-97 Specifications Class F**	**Class C**
Silicon Dioxide, %	62.41		
Aluminum Oxide, %	15.7		
Iron Oxide, %	4.95		
Total Silica, Aluminum, and Iron (%)	83.06	70 Min	50 Min
Sulfur Trioxide, %	0.47	5.0 Max	5.0 Max
Calcium Oxide, %	8.82		
Loss on Ingnition, %	0.52	6.0 Max	6.0 Max
Moisture Content, %	0.03	3.0 Max	3.0 Max
Physical Test Results			
Fineness, Retained On #325 Sieve	16.16	34 Max	34 Max
Soundness, Autoclave Expansion, %	0.052	0.8 Max	0.8 Max
Water Requirement, % of Control	95	105 Max	105 Max
Strength Activity Index with Portland Cement, %			
Ratio to Control @ 7 Days	85.8		
Ratio to Control @ 28 Days	89.5	75 Min	75 Min
Density	2.39		

Table A4.10 - Bridger Fly Ash Test Report Results

Data Source Information	
Source	CTL Commercial Testing Laboratories Report
Report Date	8/12/98
Plant of Origin	Bridger
Sample ID	29-98
Sample Date	6/6/98
Date Received	6/16/98

Chemical Composition	**Test Results**	**ASTM C 618-97 Specifications**	
		Class F	**Class C**
Silicon Dioxide, %	65.82		
Aluminum Oxide, %	17.08		
Iron Oxide, %	4.89		
Total Silica, Aluminum, and Iron (%)	87.79	70 Min	50 Min
Sulfur Trioxide, %	0.17	5.0 Max	5.0 Max
Calcium Oxide, %	5.12		
Loss on Ingnition, %	0.19	6.0 Max	6.0 Max
Moisture Content, %	0.01	3.0 Max	3.0 Max
Physical Test Results			
Fineness, Retained On #325 Sieve	26.34	34 Max	34 Max
Soundness, Autoclave Expansion, %	0.023	0.8 Max	0.8 Max
Water Requirement, % of Control	94.2	105 Max	105 Max
Strength Activity Index with Portland Cement, %			
Ratio to Control @ 7 Days	81.9		
Ratio to Control @ 28 Days	91.9	75 Min	75 Min
Density	2.31		

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