CLASS C FLY ASH IN PAVEMENTS

PROJECT: SPR (501)

July 2010

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PROJECT: SPR (501)

FINAL REPORT

PRINCIPAL INVESTIGATORS

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13. Abstract

Portland cement is the most dominant material used in concrete pavements in the state of Nebraska. In order to improve performance, reduce cost, and advance sustainability, a percentage of the Portland cement is replaced with a recycled material known as fly ash. In recent years, Nebraska Department of Roads (NDOR) began noticing premature deterioration in many Portland cement concrete pavements (PCCP). A preliminary investigation into these pavements led NDOR to identify Class C fly ash used as a supplementary cementious material (SCM) in PCCP as one of the possible causes of the distress. As a result, NDOR changed their specifications banning the use of Class C fly ash in PCCP. This research project was conducted to investigate the cause of the PCCP deterioration and propose methods of mitigation while allowing the use of Class C fly ash. A thorough review of all relevant literature was conducted and potential mixes using Class C fly ash were identified. A testing program was established to determine which potential mixes meet the expected performance criteria. The first phase of testing was to assess the potential for Alkali Silica Reactivity (ASR) using ASTM C 1567. The testing was carried out on 14 potential mixes as well as the mix used in deteriorated PCCP and the reference mix currently used by NDOR in PCCP. Based on ASR testing results, four mixes were chosen to undergo overall performance testing, which includes strength and durability properties as well as fresh concrete properties. The testing comprised ASTM C666, C1202, C157, C403, C39, C78, and NDOR's wet-dry test. Testing results have indicated that three mixes have superior performance over the reference mix with 25% Class F fly ash as the only SCM. The three mixes were used in two field applications and specimens were taken for further laboratory testing to ensure their overall performance. The three proposed mixes have the same aggregate composition of the reference mix (70% 47B sand and gravel + 30% limestone), while containing different percentages of Class C fly ash and other SCM: 16% Class C fly ash + 20% Class F fly ash; 20% Class C fly ash + 20% Class F fly ash; and 15% Class C fly ash + 18% Class F fly ash + 15% Slag.

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ABSTRACT

Portland cement is the most dominant material used in concrete pavements in the state of Nebraska. In order to improve performance, reduce cost, and advance sustainability, a percentage of the Portland cement is replaced with a recycled material known as fly ash. In recent years, Nebraska Department of Roads (NDOR) began noticing premature deterioration in many Portland cement concrete pavements (PCCP). A preliminary investigation into these pavements led NDOR to identify Class C fly ash used as a supplementary cementious material (SCM) in PCCP as one of the possible causes of the distress .

As a result, NDOR changed their specifications banning the use of Class C fly ash in PCCP until further investigation. This research project was conducted to investigate the cause of the PCCP deterioration and propose methods of mitigation while allowing the use of Class C fly ash. A thorough review of all relevant literature was conducted and potential mixes using Class C fly ash were identified. A testing program was established to determine which potential mixes meet the expected performance criteria. The first phase of testing was to assess the potential for Alkali Silica Reactivity (ASR) using ASTM C 1567. The testing was carried out on 14 potential mixes as well as the mix used in deteriorated PCCP and the reference mix currently used by NDOR in PCCP. Based on ASR testing results, four mixes were chosen to undergo overall performance testing, which includes strength and durability properties as well as fresh concrete properties. The testing comprised ASTM C666, C1202, C157, C403, C39, C78, and NDOR's wet-dry test. Testing results have indicated that three mixes have superior performance over the reference mix with 25% Class F fly ash as the only SCM. The three mixes were used in two field applications and specimens were taken for further laboratory testing to ensure their overall performance. The three proposed mixes have the same aggregate composition of the reference mix (70% 47B sand and gravel + 30% limestone), while containing different percentages of Class C fly ash and other SCM: 16% Class C fly ash + 20% Class F fly ash; 20% Class C fly ash + 20% Class F fly ash; and 15% Class C fly ash + 18% Class F fly ash + 15% Slag.

Acronyms

AASHTO	American Association of State and Highway Transportation Officials
ACI	American Concrete Institute
ASR	Alkali Silica Reactivity
ASTM	American Society for Testing and Materials
FBC	Fluidized-Bed Combustion
GGBFS	Ground Granulated Blast Furnace Slag
LON	Loss On Ignition
NDOR	Nebraska Department of Roads
NPPD	Nebraska Public Power District
OPPD	Omaha Public Power District
SCM	Supplementary Cementious Material
TAC	Technical Advisory Committee
PC	Pulverized Coal
PCCP	Portland Cement Concrete Pavements

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PART 1 – THEORTICAL INVESTIGATION

1.0 INTRODUCTION

1.1 BACKGROUND INFORMATION

Concrete pavements consume approximately 70% of the Portland cement used in the United States. It is the most dominant pavement material on major roads in Nebraska. Generally, a portion of Portland cement is replaced with one or more supplementary cementitious materials or SCM's. These materials commonly include fly ash, ground granulated blast furnace slag (GGBFS) and silica fume. The addition of such materials have been shown to offer numerous benefits to Portland cement concrete such as increased resistance to alkali-silica reaction, sulfate attack and freeze thaw deterioration. In addition, fly ash, being a coal power plant by-product, is a material that can be used effectively to simultaneously improve performance and reduce cost. In Nebraska, the most available SCM is Class C fly ash which is the result of the combustion of subbituminous or lignite coal generally coming from the Powder River Basin in Wyoming. Therefore, Nebraska power companies only produce Class C fly ash which had been the foremost SCM used in Nebraska pavements.

The durability of hydraulic-cement concrete can be defined as its ability to resist weathering action, chemical attack, abrasion or any other process of abrasion (ACI 201.2R-92). The deterioration of Portland cement concrete pavements (PCCP) is rarely the cause of a single mechanism. Often, the presence of one deterioration mechanism will allow the development of additional deterioration mechanisms and will produce a synergistic effect resulting in a rapid degradation of the structure. Therefore, specifying mix designs and construction practices that properly account for all of the potential deleterious effects is essential in ensuring that PCCP will reach its expected service life.

Although the processes as well as effective preventive measures involved in many of the common deterioration mechanisms are well understand, the continual development of new materials and wide spread use of recycled materials requires a constant evaluation of a concrete's durability. While it is true that the use of SCM's can improve concrete durability, the high variability of the SCM's eliminates the possibility of a one-size-fits-all approach in specifying mix proportions. Therefore, the factors influencing the effect a given SCM will have on specific deterioration mechanisms must be carefully studied and extensive testing procedures are required to verify these effects.

1.2 RESEARCH SIGNIFICANCE

In recent years Nebraska Department of Roads (NDOR) began noticing premature deterioration in many Portland cement concrete pavements. An investigation into these pavements led NDOR to identify Class C fly ash as one of the possible causes of the distress. As a result, NDOR changed their specifications, no longer allowing Class C fly ash in pavements. Since the exact mechanism(s) of the distress is still unknown, the University of Nebraska in a joint effort with NDOR, Omaha Public Power District (OPPD) and Nebraska Public Power District (NPPD) began a research project in January 2005 to determine both the cause of the distress and ways in which Class C fly ash can be used to mitigate the problem(s).

In addition to the importance of ensuring the adequate performance of the PCCP in Nebraska, the millions of tons of fly ash generated in Nebraska have become a large liability for the power companies. Their disposal into landfills has become a significant economical concern due to both the cost of disposal and the wasting of a useful commodity as well as a lost opportunity to lower emissions from cement manufacturing facilities. In the meantime, the Class F fly ash being specified for pavements in current projects has to be imported at an additional cost. The price of the blended cement used in Nebraska is the same price as type I/II cement.

2.0 LITERATURE REVIEW

As previously discussed, a through understanding of the effect of any materials used in a PCCP mix design is essential before any potential mix proportions can be identified. Therefore, the study began with extensive review of relevant literature regarding concrete durability and more specifically the issues being faced in Nebraska.

2.1 FLY ASH CHARACTERISTICS

The most available and cost-effective SCM is fly ash. Therefore, the characteristics of this material were carefully investigated. Since a significant difference is seen between the performance of PCCP using Class C fly ash and Class F fly ash, the differences between these materials were of interest.

2.1.1 OVERVIEW

Fly ash is the result of the combustion of pulverized coal. It is produced by coal-fired electric and steam generating plants. Once the coal is pulverized it is placed into the combustion chamber where it is immediately ignited, creating heat and a molten coal residue. Boiler tubes extract the heat from the boiler causing the flue gas to cool. The molten coal residue then hardens and forms ash. The coarse ash particles fall to the bottom of the chamber and are referred to as bottom ash. The lighter ash particles remain suspended in the flue. These particles are known as fly ash. (FHWA-IF-03-019)

There are four basic types of coal-fired boilers: pulverized coal (PC), stoker-fired, cyclone and fluidized-bed combustion (FBC). From the boiler, fly ash is collected from the flue using electrostatic precipitators (ESP) or bag houses and is typically conveyed to storage silos where it is kept dry. This is referred to as dry transfer. The method of fly ash transfer can also be wet which is a system where the dry ash is mixed with water and conveyed to an onsite storage pond. The physical and chemical characteristics of fly ash vary greatly among different methods of combustion and transfer as well as the coal source. (FHWA-IF-03-019)

Currently over 25 million tons of fly ash are used annually in a variety of engineering applications. The largest user of fly ash is Portland cement concrete with other uses including soil and road base stabilization, flowable fills, grouts and asphalt filler. (FHWA-IF-03-019)

2.1.2 CHEMICAL COMPOSITION

When fly ash is used as a mineral admixture in concrete, it is classified as either Class C or Class F ash based on its chemical composition. Class C ashes are generally derived from sub-bituminous or lignite coals and consist primarily of calcium alumino-sulfate and are usually referred to as high calcium fly ash since they usually contain more than 20 percent CaO. Class F fly ashes come from bituminous and anthracite coals and consist mainly of alumino-silicate glass. Table 2.1.2.1 shows the chemical analysis of Class C and Class F fly ash samples used in Nebraska.

Table 2.1.2.1 Chemical Analysis of Typical Fly Ash Used in Nebraska

Compounds	**Class F Fly Ash**	**Class C Fly Ash**
	Wt. %	Wt. %
SiO_2	54.24	30.84
Al_2O_3	14.20	16.21
Fe_2O_3	6.55	6.09
Sum of Oxides	74.99	53.14
CaO	13.67	27.70
MgO	3.55	4.85
SO_3	0.74	2.13
Na2O	1.66	1.38
K2O	2.05	0.30
Eq. Alkali	3.01	1.58

2.1.3 CHARACTERISTICS RELATED TO QUALITY

The quality and consistency of the fly ash being used is very important, especially when its intended use is in Portland cement concrete (PCC). The quality of a certain fly ash is affected by numerous things such as the coal source, and the method of combustion, transfer and storage. The four most relevant characteristics of fly ash in connection with quality are loss on ignition (LOI), fineness, chemical composition and uniformity.

Loss on Ignition (LOI) is the amount of unburned carbon that remains within the ash. It is an extremely important characteristic of fly ash, especially for concrete applications. High carbon levels, as well as the type of carbon and variability of the carbon content can cause substantial air entrainment problems in fresh concrete. Consequently, AASHTO and ASTM have specified limits for LOI, although some state DOTs require a lower value for LOI.

The fineness of fly ash is almost directly related to the operating conditions of the coal crusher as well as the grindability of the coal itself. Fineness is defined as the percent by weight of the material retained on the 0.044 mm (No. 325) sieve. Finer fly ash samples are more reactive and typically contain lower carbon levels. AASHTO and ASTM specify limits for fineness of 34 percent for both Class C and Class F fly ash. (FHWA-IF-03-019)

As discussed earlier, the chemical composition of fly ash is related directly to the mineral chemistry of the coal source. The chemical composition can also be affected by the use of additives during the combustion process as well and any pollution control technology that is used. The chemistry of the coal should be checked constantly and evaluated for compliance with its intended use. (FHWA-IF-03-019)

Since fly ash characteristics are usually known in advance, appropriate concrete mixes are designed. Therefore, it is extremely important to have uniformity of characteristics from shipment to shipment. If any changes are made to the operating methods of the plant, fly ash characteristics should be re-evaluated. In addition, on the job site each concrete delivery should be checked for the desired characteristics, especially air content. (FHWA-IF-03-019)

2.1.4 STANDARD SPECIFICATIONS FOR FLY ASH

The American Association of State and Highway Transportation Officials (AASHTO) and American Society for Testing and Materials (ASTM) specify requirements for Class C and Class F fly ash. These requirements are related to the chemical and physical characteristics of fly ash. Table 2.1.4.1 shows the specifications for fly ash in PCC.

Table 2.1.4.1 Specifications for fly ash in PCC (FHWA-IF-03-019)

AASHTO M 295 (ASTM C 618) – Classes F and C			
Chemical Requirements		**Class F**	**Class C**
SiO_2 + Al2O_3 + Fe_2O_3	min %	70	50
SiO_3	max %	5	5
Moisture Content	max %	3	3
Loss on Ignition (LOI)[1]	max %	5[1]	5[1]
Optional Chemical Requirements			
Available Alkalies	max %	1.5	1.5
Physical Requirements			
Fineness (+325 Mesh)	max %	34	34
Pozzolanic Activity/Cement (7 days)	min %	75	75
Pozzolanic Activity/Cement (28 days)	min %	75	75
Water Requirement	max %	105	105
Autoclave Expansion	max %	0.8	0.8
Uniformity Requirements: Density Fineness	max % max %	5 5	5 5
Optional Physical Requirements			
Multiple Factor (LOI x Fineness)		255	-

Increase in Drying Shrinkage	max %	0.03	0.03
Uniformity Requirements: Air Entraining Agent	max %	20	20
Cement/Alkali Reaction: Mortar Expansion (14 days)	max %	0.020	-
Notes:			
1. ASTM requirements are six percent			

2.2 ISSUES OF DURABILITY

Many factors influence the overall durability of a given concrete. Many of common deterioration mechanisms are affected by the same concrete properties such as strength, air content, and permeability. The presence of water is required for almost of the deterioration mechanisms and the ability of the concrete to limit the ingress of water is closely related to the concrete's long term durability. The following presents a detailed description of deterioration mechanisms common in Nebraska as well as the material and concrete characteristics which are responsible for preventing such mechanisms.

2.2.1 STRENGTH

When SCM's are used as a replacement for Portland cement on a one to one basis, the typical result is a reduced early strength and higher long term strength (Thomas, 2004). It is well established that the strength of concrete is a function of the relative quantity of water compared to amount of cement, which is referred to as the water cement ratio (w/cm ratio). However, when SCM's are used, the relationship between w/cm ratio and strength can change significantly due to several factors such as the characteristics of the SCM, the level of replacement and the age of the concrete at testing. Figure 2.2.1.1 shows typical strength gains of various SCM concretes with a constant w/cm ratio.

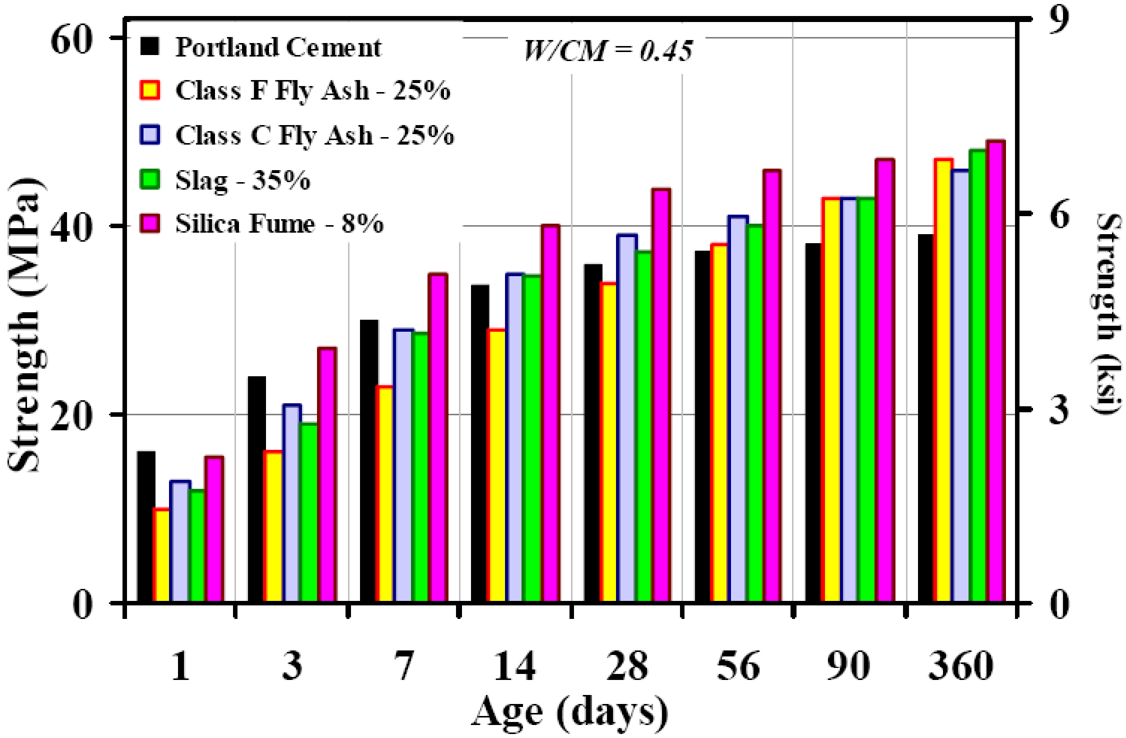


Figure 2.2.1.1 Effect of SCM's on Strength (Thomas 2004)

2.2.2 SET TIME

The use of SCM's in concrete will generally increase the set time. This is true for all Class F fly ashes and the majority of Class C fly ashes. The use of slag can increase or decrease the set time compared to fly ash concretes, depending on the source of Portland cement being used, particularly the type of clinker being used (Wang and Ge, 2003). SCM's actual quantitative effect on set time can depend on several factors including the composition of the SCM's, level of replacement and curing temperature.

A study was done by Wang et. al. (2003) to investigate the setting properties of blended cements. The research looked at concrete containing various amounts of slag and Class C fly ash. The mix water was adjusted to produce a given consistency and resulted in w/cm ratios between 0.245 and 0.252 (referred to as W/B in figure 2.1.2.4.1). The binary mixes containing 15% Class C fly ash increased the set time in all cases. The ternary mixes using Class C fly ash combined with slag also had higher set times than the ordinary Portland cement (OPC) concrete, but in some cases had shorter set times than the binary mix using 15% Class C fly ash. Figure 2.2.2.1 show the paste set times of these blends.

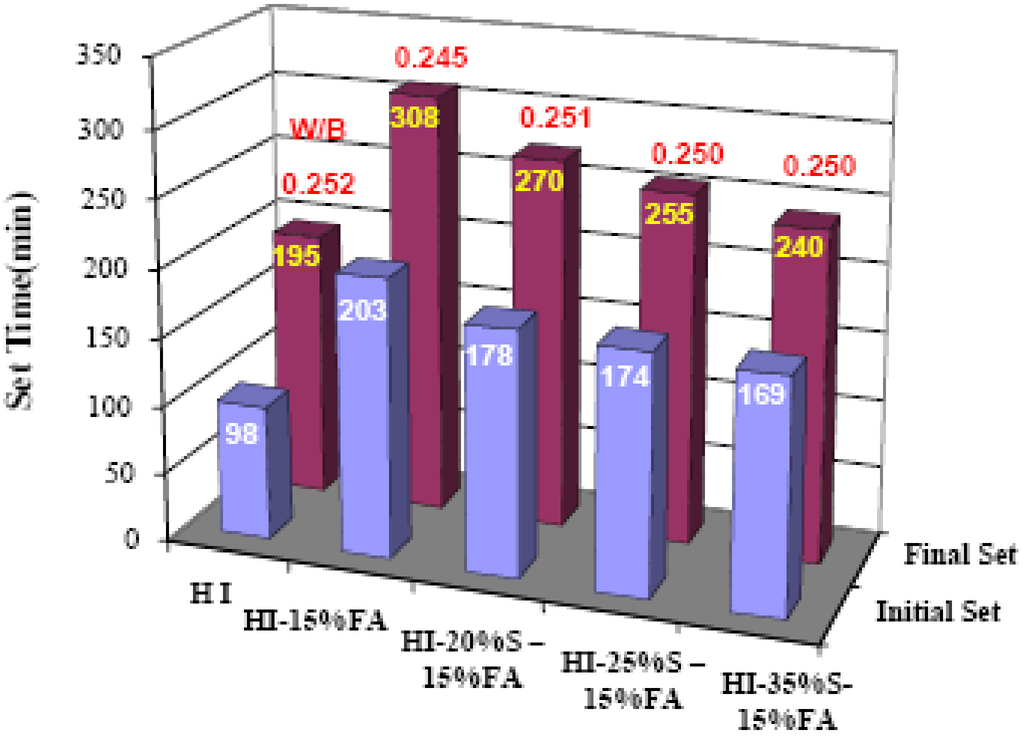


Figure 2.2.2.1 Set time for Holcim Cement (Wang and Ge, 2003)

The set times of concrete made with similar mixes were investigated, as well as the effect of curing temperature. As shown in Figure 2.2.2.2, the temperature of curing had a significant effect on the set times. This was especially pronounced with the blended cement using 15% Class C fly ash.

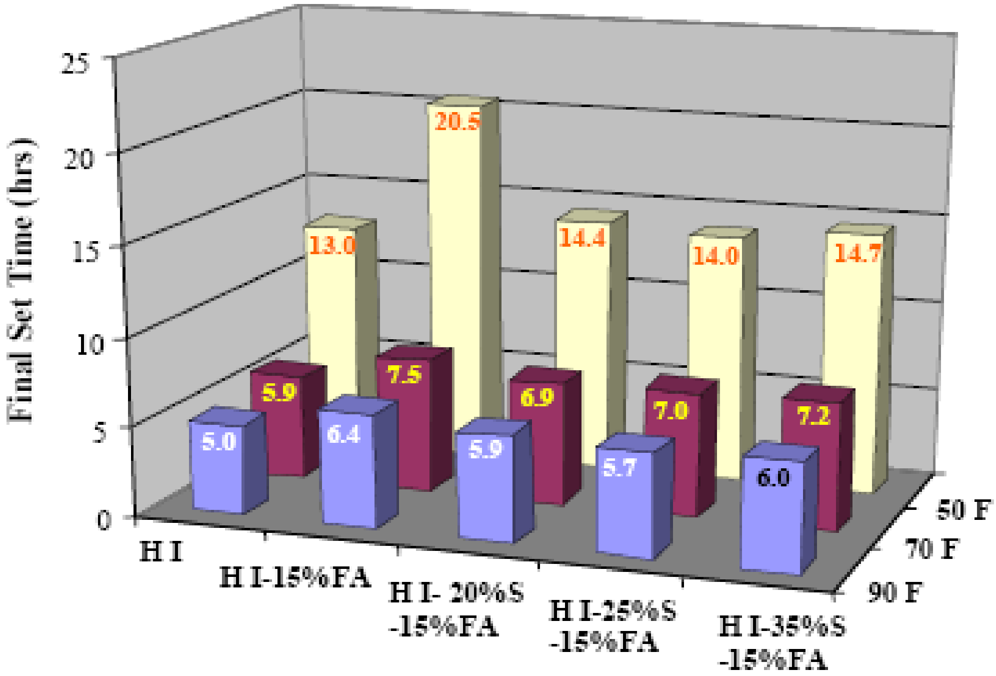


Figure 2.2.2.2 Final Set Times for Holcim Concrete (Wang and Ge, 2003)

In addition, Figure 2.2.2.3 shows the initial and final set times of various mixes used in Nebraska.

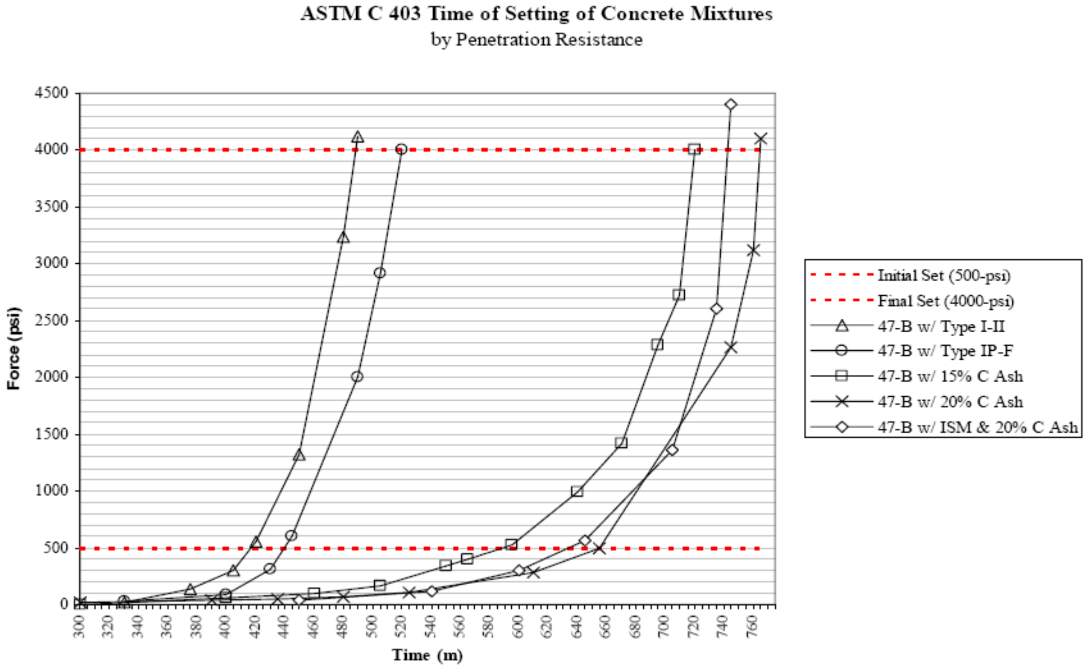


Figure 2.2.2.3 Time of Setting of 47-B Concrete Mixes (Hanson, 2005)

From the previous figures it is clear that the set time of a given concrete is very dependent on many factors including the type and amount of SCM's and the temperature during curing. It is very important to accurately characterize the setting properties of any binary or ternary mixes that are used, especially for contractors. When mixes with SCM's are used, the time frame for saw-cutting joints must be adjusted. In addition, because of the significant effect of curing temperature on pozzolanic reaction, covering slabs to trap heat or extending curing time will be necessary for SCM concrete in cold weather conditions. Due to this fact, some agencies restrict the use of fly ash or slag during the early spring or late fall. However, guidelines or specifications for curing management may be more appropriate for the use of SCM concrete in pavement. (Wang and Ge, 2003)

2.2.3 FREEZING AND THAWING

Expansions from freezing and thawing are often a major cause of damage in concrete structures such as pavements. Especially in cold climates, it is critical for concrete to have a high resistance to freeze thaw cycling. The porosity of concrete is an interconnected pore system formed by the space occupied by the mix water that did not participate in hydration. This is generally referred to as capillary pores. When the concrete is exposed to moisture, water travels through the concrete in these pores. If temperatures fall below freezing, the water in these pores freezes and can expand up to 9% of its original volume. The expanding ice exerts pressure on the concrete and can cause cracking. (Whiting and Nagi, 1998)

The resistance of concrete to freeze thaw cycling is a direct function of the air entrainment and air-void system of the concrete (Malhotra and Mehta, 2002). Therefore, the concrete’s durability under freeze thaw cycling is not directly related to the amount or type of fly ash in it. However, fly ash can create difficulties in obtaining proper air content and could indirectly affect freezing and thawing durability. A detailed discussion on achieving proper air content can be found in following section.

Numerous studies have been done to evaluate the effect of fly ash and other SCM’s on the performance of concrete under freezing and thawing. Naik et al., as well as others, have shown acceptable performance when the concrete possessed adequate strength and appropriate air-void systems. Adequately proportioned concrete should have 4 to 7 percent air content with an air bubble spacing factor less than 200µm and specific surface greater than 24 mm^2/mm^3 (Naik, 1997). According to Mather (1989), concrete with or without fly ash will be durable against freezing and thawing if: (1) it is properly air entrained, (2) it has attained approximately 4000 psi compressive stress when it is subjected to freezing and thawing, (3) it is made with sound aggregates, and (4) proper construction, in particular surface finishing, operations are followed correctly.

In addition, the effect of freeze thaw cycling is greatly influenced by the moisture in the concrete. Areas within the structure that are exposed to saturated conditions will experience greater deterioration due to freezing and thawing. Therefore, for a given air void system, concrete with fly ash will generally show higher freeze thaw resistance due to its lower permeability.

2.2.3.1 Air Content

As discussed, for PCCP located in cold weather regions proper air content is essential in achieving a durable concrete and is therefore an aspect of mix proportioning that needs careful consideration. For over 50 years, entrained air has been deliberately incorporated in concrete mixes to reduce damage from freezing and thawing. This process of entraining air is done through the use of chemical admixtures known as air-entraining agents. In properly air-entrained concrete there are more than 1 million air bubbles in one cubic inch. These bubbles provide voids within the concrete which act as reservoirs for the water moving through the capillary pores, relieving the pressure caused during the freezing of that water. (Whiting and Nagi. 1998)

When fly ash is used as a cement replacement material in PCC, air content requires close monitoring. It is generally accepted that the use of fly ash in concrete will increase the amount of air entrainment agent (AEA) needed to obtain the specified air content. This is a result of the unburned carbon present in the fly ash. The carbon absorbs the air-entraining agent and prevents it from stabilizing the air bubbles in fresh concrete (Malvar, 2001). Therefore, fly ashes containing high LOI values will typically require higher dosages of AEA. However, the LOI value alone is not always sufficient in indicating the necessary amount of air-entraining agent. Some types of fly ashes contain carbon particles with a higher surface area and therefore absorb more AEA. In addition, with some combinations of fly ash, cement and air-entraining agents, the air-void system may be unstable and trial mixes should be used to evaluate this effect.

When proper consideration is given to the potential problems, there is generally no difficulty in entraining adequate air in fly ash mixtures. If a problem is encountered with obtaining proper air content, changing the source of fly ash or the brand of air-entraining admixture should be considered. Therefore it is essential to prepare trial mixes and to regularly check the air content of incoming loads at the job site. (FHWA-RD-03-047)

Although the incoming loads of concrete may contain adequate air content, a significant degradation of the air void system may occur during placement and consolidation. Problems with workability, often due to material incompatibility, hot weather, or poor aggregate gradation, can force contractors to over-vibrate the concrete, effectively ruining the air void system. It is recommended that vibration frequency be limited to 8,000 vpm for paver speeds greater than 3 ft/min, and should be further reduced for slower paving speeds.

The state of Iowa began observing early deterioration of pavements built between 1984 and 1994. The deterioration being observed was very similar to that in Nebraska pavements. After further investigation into these pavements, it was seen that they possessed air contents well below the recommend values. The petrographic analysis revealed spacing factors almost double the recommend value of 0.2 mm. A significant difference in the air void system was seen for the top and bottom of the cores and cracking was generally confined to the top 6” where air content was the lowest. Therefore, the problem was believed to be the result of a poor air void system. (IDOT - MLR-98-06)

In 1994, the Iowa DOT implemented changes in their specifications. Previously, specifications called for a target air content of 6 ± 1 % and a minimum vibration of 7000 vpm’s with no maximum value. After 1994 a new target air content of 7 ± 1% was specified. In addition, vibration specifications were changed to a range of 5000 to 8000 vpm’s. Since adoption of the new specifications, pavements experiencing the premature deterioration have been greatly reduced. In addition, cores of the good performing pavements were analyzed. These cores showed the air content in the mortar was above the recommend value of 9% and the air content of the concrete was above the specification of 6%. It was also observed that good performing pavements all had spacing factors below 0.200 mm, while the poor performing pavements had spacing factors above 0.25 mm. These results confirmed their belief that the cause of the deterioration was an inadequate air void system. (IDOT -MLR-98-06)

Recently, Iowa DOT has adjusted their specification for air content to further ensure an adequate air void system. The specifications require an air content of 6% plus the loss from the paver, +1.5/-1 %. Therefore, once a day the air content is measured in front of and behind the paver. The calculated loss of air is then added to the 6% and that is the value for the target air content +1.5/-1 %.

A recent development has the potential for accurate analysis of the air system in fresh concrete. Unlike conventional tests that only measure the total volume of air, the Air Void Analyzer or AVA can reportedly be used to evaluate the air void system including the volume of entrained air, size of air voids and distribution of air voids within fresh concrete (CPCCP, 2004). Using this information, real-time adjustments can be made during paving to ensure an adequate air void system, greatly increasing the durability of the pavement. However, this testing procedure is very new and can have problems with repeatability. In addition, the test is very sensitive to any movement or vibrations such as wind or road vibrations and can lead to unreliable results.

2.2.4 SALT SCALING

In cold weather regions, freezing and thawing is a major cause of damage to concrete structures. The use of deicing chemicals further aggravates this problem. When freezing and thawing occurs with the presence of moisture and deicing chemicals, concrete surfaces can experience scaling. Scaling of concrete in the presence of a deicing chemical can be described as a delamination of surface layers of concrete (Mehta, Olek, Weiss and Nantung, 2005). It has been shown that salt-scaling damage can occur by both chemical and physical mechanisms (Naik, Krause, Ramme, and Chun, 2003).

The physical mechanism occurs when small flakes of concrete, break away from the surface of the pavement. This mechanism is probably due to the deicer concentration gradient and the degree of saturation. Due to hygroscopic character of deicer salts, the degree of saturation of concrete increases with the use of deicers and differential stresses are generated as a result of layer-by layer freezing, resulting from the salt concentration gradients (Naik, Krause, Ramme, and Chun, 2003). The chemical mechanism through which salt-scaling occurs was described by Neville (1969). It was shown that saturated solutions of certain deicing chemicals can cause deleterious effects, even with out freezing and thawing. This effect was greatly dependent on the water-cement ratio. For concrete with high w/cm ratios, the deicing chemical resulted in increased expansions, mass gains and loss in dynamic modulus of elasticity (Naik, Krause, Ramme, and Chun, 2003). Even for a low w/cm ratio, large mass losses and a degradation of the dynamic modulus of elasticity was seen.

Many factors influence the salt-scaling resistance of concrete such as the level of saturation, compressive strength, entrained air, w/cm ratio, type and amount of SCM, aggregate type and gradation, finishing and curing procedures and exposure conditions. Typically, high replacement levels of SCM's will lower the salt-scaling resistance of a concrete. For this reason, the ACI 318 building code places a limit on the maximum amount of SCM's allowed in concrete exposed to deicing chemicals as follows:

Slag ≤ 50%

Fly Ash ≤ 25%

Silica Fume ≤ 10%

Total SCM in Concrete with Slag ≤ 50%

Total SCM in Concrete without Slag ≤ 35%

Although pavements are not required to follow these limits, most state agencies specify similar limits for PCCP.

The most commonly used test method to assess salt-scaling resistance is ASTM C 672: *Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals*. In this test method, specimens are cast and covered with ¼” of a calcium chloride and water solution. The specimens are then subjected to 50 freezing and thawing cycles. During these cycles, the proper depth of solution is maintained on the top surface of the specimen. Visual ratings and mass loss are recorded throughout the test. Visual ratings are done by evaluating the specimens with ratings of 1-5, with 1 being no scaling and 5 being severe scaling.

Although scaling resistance is dependent on many factors other than the amount of SCM’s used, studies have shown concrete with high volumes of fly ash can have satisfactory scaling resistance. Naik et al. (2003) showed that in some cases, concrete containing 0-50% fly ash showed equal or greater resistance to salt-scaling when compared to the control mix made with no fly ash. However, another series of mixes showed the opposite trend with the salt-scaling resistance decreasing as the fly ash content increases.

According to ASTM C 672, salt scaling resistance of concrete containing fly ash should be a major durability concern in cold regions. However, unlike laboratory tests which often show scaling, very few states have reported scaling problems in concrete with fly ash (Mehta, Olek, Weiss and Nantung, 2005). Often, if scaling is observed it is attributed to issues either with finishing or curing. Therefore, it has been suggested that ASTM C 672 does not correlate well with actual field performance and often shows concrete containing SCM's to be at a large disadvantage. Several experts including Thomas and Naik agree that ASTM C 672 is too severe (Thomas, 2004). Figure 2.2.4.1 shows field performances with respect to salt scaling of various fly ash mixes.

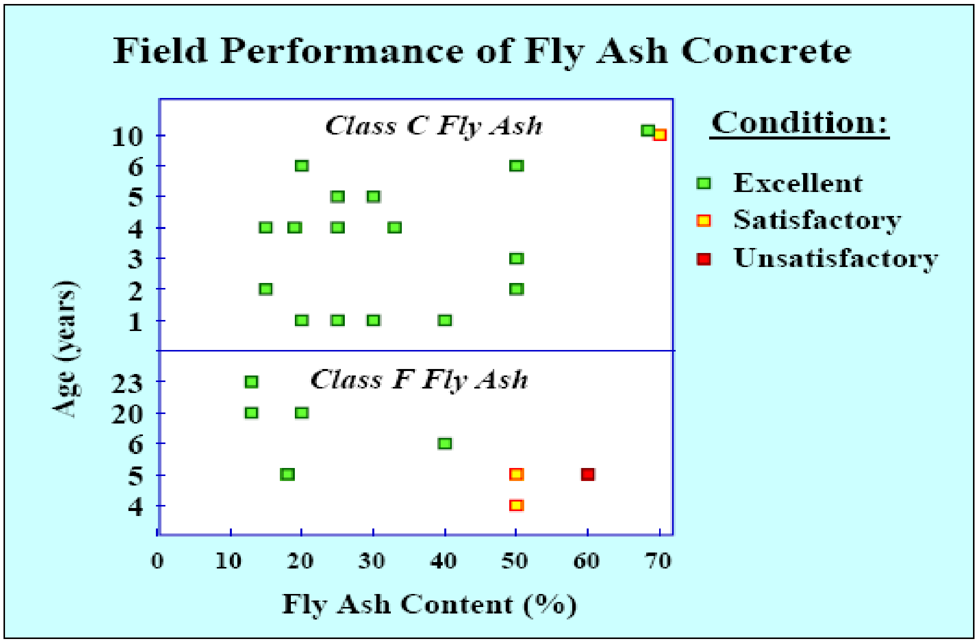


Figure 2.2.4.1 Scaling Field Performance of Fly Ash Concrete (Thomas, 2005)

A study was conducted by Mehta et al. (2005) to attempt to identify the potential reasons for the apparent discrepancy between ASTM C 672 and actual field performance. It was concluded that surface evaporation and slab thickness have a significant effect on salt scaling resistance. This indicates that ASTM C 672 is too severe because it does not account for several important conditions that are likely to be present in the field.

In addition, it has been suggested by other experts that ASTM C 679 may not be rigorous enough in evaluating the scaling resistance of concrete. They have suggested increasing the total number of freezing and thawing cycles as well as increasing the number of cycles per day. Due to the immense disagreement concerning the accuracy of ASTM C 679, any results obtained from this test should be analyzed very carefully before any conclusions are made concerning the scaling resistance of a specific mix.

2.2.5 ALKALI-SILICA REACTION

Throughout the state of Nebraska, PCCP use Platte River Sand and Gravel exclusively as a fine aggregate. Additionally, the limited supply of limestone in the state results in mix designs heavy weighted towards the abundant Sand and Gravel aggregate. It has been well established that Platte River Sand and Gravel is a reactive aggregate which can potentially lead to Alkali-Silica Reaction or ASR. Therefore, ASR has been identified as one of the leading deterioration mechanism in Nebraska. Currently, ASR mitigation is the driving factor in the mix designs allowed in the state**.**

2.2.5.1 Mechanism of ASR

Alkali-silica reaction was first recognized by Stanton (1940,1942) of the California State Division of Highways (ACI 221.1R-98). Since that time, alkali-silica reaction or (ASR) has been identified as a major durability problem in the Unitd States, resulting in premature deterioration of many types of concrete structures. Extensive research has been done in recent years to both develop test methods to determine expansive reactivity as well as effective means to control ASR-induced expansion.

Alkali-silica reaction is the reaction between alkali hydroxide and certain siliceous rocks and minerals in the aggregates (Malvar, 2001). The product of this reaction is a gel that absorbs water creating expansive stresses on the concrete. It is widely acknowledged that the three essential components needed for ASR are reactive silica, sufficient alkalies and sufficient moisture.

Reactive aggregates refer to aggregates that tend to breakdown under exposure to a highly alkaline pore solution in concrete and react with the alkali-hydroxides to produce ASR gel (FHWA-RD-03-047). It was originally believed that there was a limited group of reactive aggregates. However, it is now recognized that ASR can occur with a wide range of siliceous aggregates and cases of ASR have been reported throughout all parts of the US and much of the world. Table 2.2.5.1.1 list rock types and minerals that are susceptible to ASR.

Table 2.2.5.1.1 Rocks and Minerals susceptible to ASR (FHWA-RD-03-047)

Rocks	**Minerals**
Arenite	Crisobalite
Argillite	Cryptocrystalline (or microcrystalline) quartz opal
Arkose	Strained quartz tridymite
Chert	Volcanic glass
Flint	
Gneiss	
Granite	
Greywacke	
Hornfels	
Quartz-arenite	
Quartzite	
Sandstone	
Shale	
Silicified Carbonate	
Siltstone	

A study was done by David W. Hadley in 1968 on the sand and gravel aggregate used widely throughout Nebraska and Kansas. These sand and gravel aggregates are poorly graded siliceous stream gravels deposited in the high plains region as the result of the erosion of the Rocky Mountains. The gravel sizes of the aggregate are composed primarily of particles of coarse-grained granite and similar rock types. The sand sizes are predominately individual grains of quartz and potassium feldspar. Many sand-gravels also contain a significant amount of volcanic and opaline materials which are also known to be reactive.

Another required component for ASR is sufficient alkalies. Generally, Portland cement is the dominant source of available alkalies, but other sources such as SCM's (e.g. fly ash), chemical admixtures and deicing salts can be contributors to the available alkalies in the system. The two alkali constituents present in Portland cement are sodium oxide and potassium oxide. The total equivalent alkali is described as equivalent $Na_2O(Na_2O_e)$ and is calculated as percent Na_2O plus 0.658 * percent K_2O (ACI 221.1R-98). Although the percentage of alkalies in Portland cement is relatively low compared to other oxides, the bulk of the alkalies reside in the pore solution of the concrete, and it is the associated hydroxyl (OH^-) concentration that produces the inherent high pH in the pore solution (FHWA-RD-03-047). Higher percentages of sodium and potassium alkalies, produce higher concentrations of hydroxyl ions and raises the pH. The higher the pH of the pore solution, the more readily it attacks the reactive silica. Once the silica is in the pore solution, it reacts with the alkalies and forms alkali silica gel. (ACI 221.1R-98)

Sufficient moisture is also required for ASR to occur. An internal relative humidity of about 80% is required for the reaction. In most concretes, some of the mixing water is usually available for a long period. For low w/cm ratios, all the water may be used up during the hydration of the cement. However, for structures exposed to external moisture such as pavement, it is very difficult to reduce the moisture below the critical threshold value. Areas within pavements that tend to accumulate and retain moisture, such as joints, will be particularly susceptible to ASR. (FHWA-RD-03-047)

2.2.5.2 Symptoms of ASR

The most common symptoms of deleterious ASR are map cracking, closed joints, spalled surfaces and relative displacements of the structure. Deterioration due to ASR is relatively slow. However, ASR cracking can lead to serviceability problems and can also magnify other deterioration mechanisms such as freeze-thaw and sulfate attack.

In pavements and slabs on grade, cracking from ASR usually begins near free edges and joints, where sufficient moisture is available. The cracks are usually perpendicular to the transverse joints and parallel to the free edges or other areas of minimum restraint. The appearance and intensity of ASR in pavements is usually highly variable. The variable concentration of the necessary components, as well as the ambient temperature produces significant differences in the rate and effect of the reaction. Popouts and discoloration may indicate ASR, although these symptoms can occur for other reasons. The presence of ASR gel on the surface indicates evidence of alkali-silica reaction, but does not by itself mean that ASR is causing excessive expansion of the concrete. (ACI 221.1R-98)

It is very difficult to absolutely identify distress caused by ASR based only on visual examination. Commonly, the first step in evaluating a deteriorated pavement is to test the mechanical properties of the concrete, particularly the tensile strength. In addition, it is usually necessary to run ASTM C 856 *Standard Practice for Petrographic Examination of Hardened Concrete*. With this procedure, in most cases with excessive expansion from ASR, deposits of the reaction product or ASR gel can be found. However, the amount of gel present is not always related to the extent of the cracking due to ASR. In some cases, with high cracking due to ASR, the amount of gel can be limited. The opposite can also be true, where a large amount of gel can produce no significant cracking (ACI 221.1R-98). Therefore, it is important to distinguish between the ASR reaction and the damage caused from the reaction (FHWA-IF-03-019).

Another method for identifying the presence of alkali silica gel in concrete is the *uranyl-acetate treatment procedure* as outlined in the annex of ASTM C 856. The concrete surface is sprayed with a solution of uranyl acetate, rinsed with water and viewed under an ultraviolet light. Reacted particles and gel will appear as yellow or green. However, the test does not separate between a non-detrimental presence of gel and that which is detrimental. (FHWA-IF-03-019)

Another test method used to evaluate the chemical durability of a mix, particularly ASR, is the NDOR wet-dry test. In this test method, specimens are cast and after 28 days of moist curing they are placed in a wetting and drying chamber. The chamber is kept at approximately 70^{o}F and exposes the specimens to cycles of wetting and drying. Every 28 days measurements are taken on the specimens including length change, sonic modulus and weight. The test is run for a total of 548 days or approximately 18 months. The repeated wetting and drying will accelerate any potential chemical reactions within concrete, such as alkali-aggregate reaction.

A more advanced method for examining hardened concrete is x-ray image analysis. In this procedure, electron beams are sent into the sample. X-ray photons are then produced from the incoming electrons knocking inner shell electrons out of atoms in the sample. Since each element has its own unique set of energy levels, the emitted photons are indicative of the element that produced them. Analyzers are then used to characterize the x-ray photons for their energy and abundance to determine the chemistry of sample. In addition, due to the small size of the electron probe, it is possible to obtain elemental analyses for volumes as small as 1 um in diameter. This capability is useful for determining the presence of contaminants, investigating phase differences, and locating inclusions, as well as determining qualitative or quantitative elemental composition.

2.2.5.3 Methods of Mitigation

As discussed previously, the three components necessary for alkali-silica reaction are sufficient alkalies, reactive siliceous aggregates, and sufficient moisture. Methods for preventing ASR expansion focus on controlling one or more of these components.

Limiting Alkalies

Limiting the total alkalies in the pore solution and in effect lowering the pH will control ASR. For many years it was thought that ASR is unlikely to occur in concrete with cements that have an alkali content below 0.6%. This is because studies showed that the hydroxyl ion concentration is related to the alkali content of the Portland cement (ACI 221.1R-98). Therefore, cements with higher alkali contents produce higher expansions. However, it is now recognized that by itself, limiting the alkali content of the Portland cement is not an effective way of controlling ASR. This is because it does not control the total alkali content of the concrete. Laboratory tests have shown that total alkali contents below 5 lb/yd^3 is generally successful in controlling ASR. However, field results have shown ASR expansion in structures with much lower alkali contents. This may be due to the addition of alkalies from external sources such as deicing salts (FHWA-RD-03-047). In addition, environmental regulations may make it difficult for cement manufacturers to effectively limit the alkali content of the cement (ACI 221.1R-98).

Limiting Moisture

As discussed earlier, it is very difficult to limit the available moisture in structures exposed to external moisture such as pavements. However, any reduction in available moisture through good mix design, drainage design or joint design will help to control ASR and improve long term durability. Mixes that have low permeability will reduce the ingress of moisture and deicing salts, which can reduce the potential for ASR expansion. Reducing the amount of external moisture entering the concrete will also reduce the water that is absorbed by the ASR gel and reduce expansion. One way to reduce permeability is to use a low w/cm ratio. This will reduce the ion mobility and delay the reaction (ACI 221.1R-98). However, lowering the w/cm ratio will also increase the alkali concentrations of the pore solution. Therefore, it is better to use SCM's to lower the permeability than to reduce the w/cm ratio.

Aggregate Selection

For controlling alkali-silica reaction, using nonreactive aggregates is most effective. However, this is not always practical in many regions. Field performance is extremely important in predicting the reactivity of aggregates. When reactive aggregates are used it may be possible to improve the aggregate quality with several beneficiation strategies (Farny, and Kosmatka, 1997).

By blending reactive aggregates with non-reactive aggregates it is possible to dilute the reactive silica concentration. Limestone sweetening or replacing a percentage of the reactive sand and gravel with crushed limestone has been shown effective in some parts of the country. Heavy media separation or selective quarry can be used to avoid certain reactive materials although these processes can be expensive. In addition, washing or scrubbing the aggregates can remove some of the reactive fines and reactive coatings. (ACI 221.1R-98)

Using Supplementary Cementitious Materials

The most common method used to mitigate ASR expansion in concrete is the use of supplementary cementitious materials. The use of SCM's has many benefits both chemically and physically. The most widely used SCM is fly ash due to its wide range of economical and technical benefits.

a) Pozzolans/Fly Ash

Many issues affect the ability of a given fly ash to control expansion from ASR. These include the dosage of fly ash, the chemical composition of the fly ash, the reactivity of the aggregate and the alkali content of the concrete (FHWA-RD-03-047). Fly ash can reduce ASR expansion by first reducing the permeability of the concrete which limits the available moisture. The pozzolanic reaction created by the fly ash will consume alkalies and reduce the mobility of the alkalies. In addition, by replacing Portland cement, which has a relatively high concentration of alkalies, with fly ash containing low levels of alkalies, it will reduce the total alkali content of the concrete.

The effectiveness of fly ash to control ASR varies significantly for different fly ashes. The most important factor affecting the efficiency of a given fly ash is the CaO content. Typically low lime fly ashes are more effective in reducing ASR. This is because low CaO ashes produce a calcium silicate hydrate structure with a lower calcium to silicon ratio which can absorb cations, particularly alkalies. High lime fly ashes may react to a greater extent and release a large portion of its alkalies into the concrete. (FHWA-RD-03-047)

Since Class C fly ashes have 10-40% CaO by mass, they are less effective in controlling ASR than Class F fly ash which typically has less than 15% CaO (ACI 221.1R-98). Class F fly ash can usually control expansion with replacement levels from 15 to 30%. Class C fly ash replacement levels often exceed 30% and for use with reactive aggregates necessary replacement levels may not be appropriate due to the effect on other properties (ACI 221.1R-98).

Studies have shown that for fly ashes with high alkali content, there is a pessimum limit (Naik, Krause, Ramme, B. and Chun, 2003). The pessimum limit is the percent replacement for which the expansive reaction is the greatest and is usually very pronounced for Class C fly ash. Any replacement levels below the pessimum limit will cause equal or greater expansion than if fly ash was not used. Pessimum limits vary for every fly ash, but are typically around 20% replacement by mass. There is no one size fits all for using fly ash to mitigate ASR. It is generally required to use testing such as ASTM C 1567 to determine the necessary replacement needed to control ASR.

Natural pozzolans, identified as Class N pozzolans, include naturally occurring amorphous siliceous material (ACI 221.1R-98). Historically volcanic ash has been one of the most commonly used natural pozzolans. In addition, calcining some siliceous material to temperatures of 1000^{o}C can produce a pozzolanic material. The pozzolanic reactions caused by these materials can be very effective in limiting the amount and mobility of the alkalies in the concrete. However, the use of natural pozzolans is relatively limited in the Unitd States due to cost and availability.

b) Ground Granulated Blast Furnace Slag (GGBFS)

Ground granulated blast furnace slag is a by-product from manufacturing iron. Slag can be effective in controlling ASR, but is needed in higher dosages. Slag is not a pozzolan, but rather a latent hydraulic cement. However, unlike Portland cement, the hydration of slag does not produce calcium hydroxide, which is weak and porous and may contribute to ASR. Effective amounts of GGBFS to control ASR expansion range from 25 to 50%. Although the alkalies in slag will contribute to the alkalinity of the concrete, the alkalies in slag are released at a much slower rate than Portland cement (ACI 221.1R-98).

2.2.5.4 Testing for ASR Potential

There are numerous test methods available to assess the ASR potential of aggregates as well as specific concrete mixes. The three most widely used and accepted tests are ASTM C 1293: *Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction,* ASTM C 1260: *Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar Bar Method) and* ASTM C 1567: *Standard Test Method for Determining the Potential Alkali-Silica Reactivity of combinations of Cementitious materials and Aggregate (Accelerated Mortar-Bar Method)..*

ASTM C 1293

Commonly referred to as the concrete prism test, ASTM C 1293, is generally considered an accurate way of predicting field performance of aggregates. The cement used is required to have an equivalent alkali content between 0.8 and 1.0 percent. Additional alkalies are then added to the mixing water to obtain a total alkali content of 1.25 percent. Concrete prisms are cast and stored at 38°C with 100% relative humidity. The test is generally run for 1 year, however, if evaluating SCM's, the test may be run for up to 2 years. Typically, maximum expansions of 0.04% are used with this test. (FHWA-RD-03-047)

ASTM C 1293 is generally thought to be the most realistic in representing field performance. However, this test is less conservative than ASTM C 1567 and can potentially allow the use of some deleterious aggregates. The relatively long testing period required for ASTM C 1293 is one of the major drawbacks of the test and has limited its use recently.

ASTM C 1260

ASTM C 1260, or the accelerated mortar bar test, was developed by Oberholster and Davies in 1986 at the National Building Research Institute in South Africa (FHWA-RD-03-047). It is used to assess the potential for ASR of aggregates. In this test method, mortar bars are cast with aggregates processed to a standard gradation. After curing, the mortar bars are submersed in a 1 N NaOH solution at 80°C where they are stored for 14 to 56 days. Length change measurements are taken periodically during the storage period. Typically, expansions at 14 days are used in specifications, although limits for different agencies vary. The expansion criteria provided by ASTM, indicate that expansions less than 0.1% are considered innocuous, 0.1 to 0.2% are considered potentially reactive and greater than 0.2% is reactive. (FHWA-RD-03-047)

ASTM C 1567

ASTM C 1567 is probably the most widely used test method to assess the ASR potential of specific mixes. This test method is very similar to that of ASTM C 1260. However, this test method allows for the use of combinations of SCM's in the mortar bars to allow for the evaluation of specific concrete mixes. As with ASTM C 1260, these mortar bars use aggregates processed to a specific gradation and are immersed in 1 N NaOH solution. ASTM C 1567 is generally recognized as a conservative test since it provides excess NaOH in the 1 N solution in which the specimen is immersed, as well using high temperatures (Malvar, 2001). By providing essentially unlimited alkalies, this test method is useful for identifying slowly reactive aggregates, however, the test can also identify sound aggregates as being reactive. Because of the severity of the test, aggregates found to be innocuous are very likely to perform well in the field. ASTM recommends an expansion limit of 0.1% at 14 days (FHWA-RD-03-047). However, the state of Nebraska due to its slowly reactive aggregates, requires an expansion limit of 0.1% at 28 days.

2.2.5.5 Effect of Deicing Chemicals on ASR

Recent research by the American Concrete Pavement Association or ACPA (2005) has shown that common screening tests, such as ASTM C 1567, that are being used to identify ASR potential may not be accurate if the materials are subjected to certain deicing chemicals. In some airfields premature deterioration has been observed, often in pavements less than 10 years old. In many of these cases, the surface deteriorations can be correlated to the application patterns of the deicing chemicals. Therefore, deicing agents commonly used on airfields are being investigated. These chemicals include Potassium Acetate (KA) and Sodium Acetate (NaA). It was seen that many aggregates or mixes that were determined to be innocuous with expansion under 0.1%, exhibited far greater expansions when these deicing chemicals were used as the soak solution. Figures 2.2.5.5.1 and 2.2.5.5.2 show the differences in expansions between using the standard NaOH soak solution and using the deicing chemical Potassium Acetate as the soak solution.

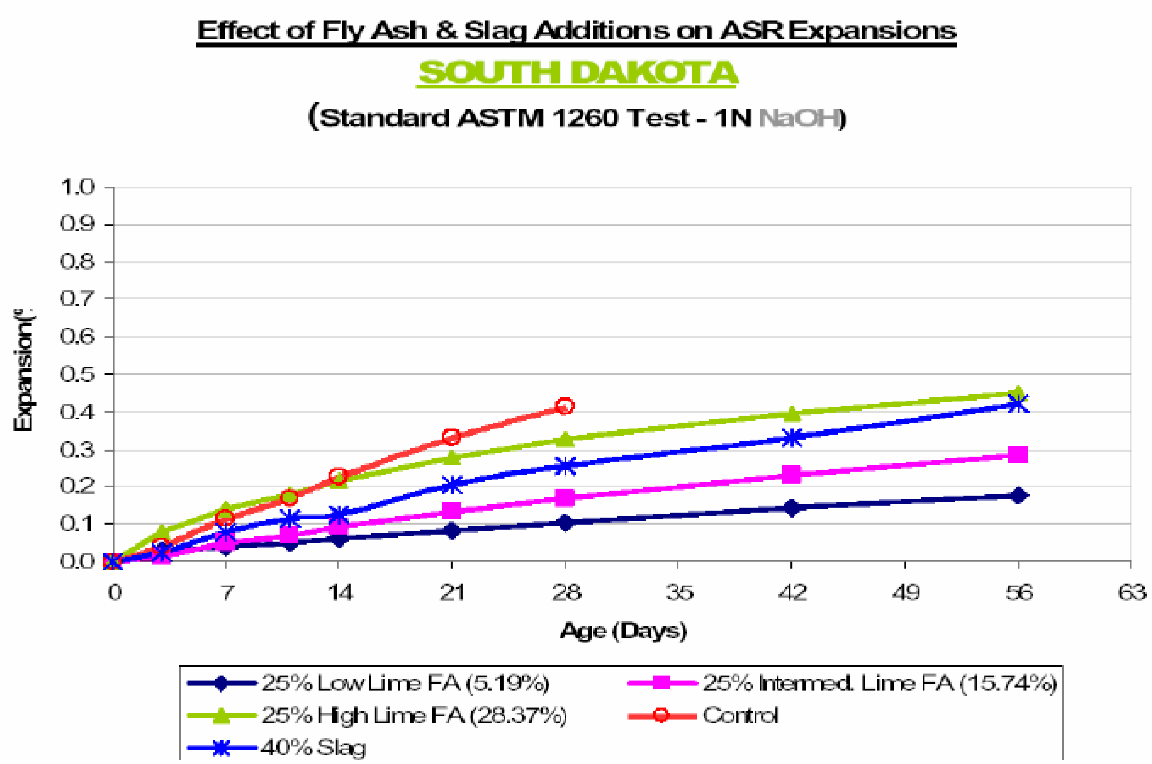


Figure 2.2.5.5.1 ASTM C 1260/1567 Expansions for 1N NaOH (Donovan, 2005)

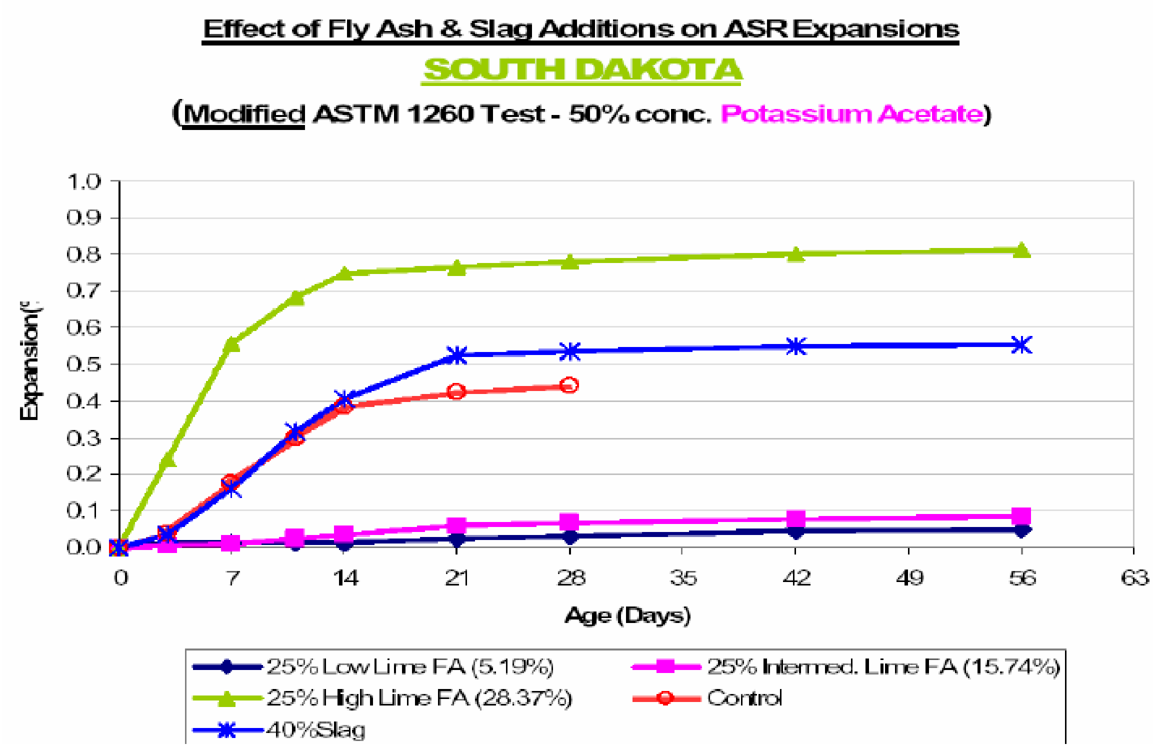


Figure 2.2.5.5.2 ASTM C 1260/1567 Expansions for Potassium Acetate (Donovan, 2005)

Due to these findings, a new testing procedure has been proposed. In this new testing procedure, the standard NaOH soak solution is substituted with the deicing chemical that the materials will be exposed to. Experience has shown that if expansion results are adequate with Potassium Acetate as the soak solution, then other forms of chemical acceleration should be satisfied. If expansion of the mortar bars is greater than 0.1% after 28 days then mitigation steps are required such as using non reactive aggregates, using a higher dosage or different type of SCM, limiting the total alkalies in the concrete, etc. (ACPA, 2005)

2.2.6 PERMEABILITY

Permeability is the property that defines the ease with which fluids can penetrate a material. The ability of concrete to prevent the ingress of fluids has significant effects on the concrete's resistance to many durability issues such as ASR and freezing and thawing. Therefore, permeability can be considered a primary factor in connection with the durability of concrete. The primary factors influencing the permeability of concrete are water/cementitious material ratio, water content, use of SCM's and method of curing. As with many other properties, the permeability of concrete containing SCM's is very dependent on the type and dosage of SCM's used. Generally, concrete containing SCM's will have slightly higher permeability at early ages. However, after 14 to 28 days, SCM concrete will generally have superior permeability compared to OPC concrete. This is mainly due to the increased amount of Calcium Silicate Hydrate (C-S-H) phase which is the "glue" that is responsible for the concrete's strength and impermeability. In concretes containing fly ash, during the pozzolanic reaction the fly ash reacts with the weak and porous Calcium Hydroxide (CH) formed during cement hydration to form additional C-S-H, lowering the permeability. The use of slag creates minimal pozzolanic reactions, however, unlike Portland cement the hydration of slag produces no CH, also lowering the permeability.

The permeability of concrete is often measured using ASTM C 1202: *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*. This test measures the ability of concrete to resist the penetration of chloride ions and the results are measured in coulombs. The following is the rating system per ASTM C 1202.

Charged Passed	Chloride Permeability
> 4000	High
2000-4000	Moderate
1000-2000	Low

100-1000	Very Low
<100	Negligible

Due to a pronounced synergistic effect, using ternary mixes with two types of SCM's will often produce concrete with extremely low permeability. Figure 2.2.6.1 shows the rapid chloride permeability results for various ternary blends using Class F fly ash and slag.

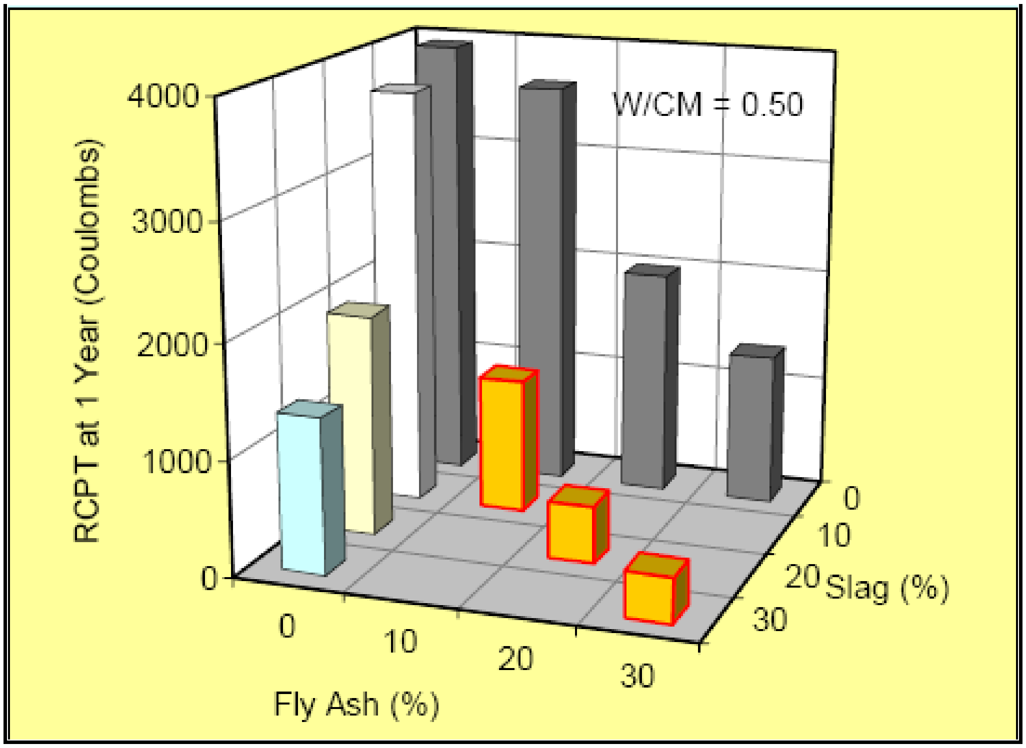


Figure 2.2.6.1 Chloride Penetration of Class F Fly Ash and Slag Mixes (Thomas, 2004)

Studies have also shown that ternary mixes using both Class C and Class F fly ash can produce a very low permeable concrete. Naik et. al. (1996) showed that concrete using 20% Class C fly ash and 20% Class F fly ash had far superior permeability than OPC concrete or binary mixes containing 35% Class C fly ash. The ternary mixes using C and F ash had penetrations of 1330 coulombs at 28 days and 225 coulombs at 365 days.

Such mixes would significantly limit the ingress of fluids, particularly water, and should provide an increased resistance to many durability issues.

2.2.7 MATERIAL INCOMPATIBILITY

At the 8th International Conference on Concrete Pavements a presentation was given by Dr. Peter Taylor emphasizing that with the recent complexity of concrete mixes, using a number of SCM's and admixtures, special attention should be paid to material incompatibility. Material incompatibility is the combination of acceptable materials interacting in an undesirable or unexpected way (Taylor, 2005). Such interactions can cause unusual stiffening or setting, cracking and problems with the air void system.

Early stiffening and setting can be seen when the concrete sets in the mixer or stiffens on the way through the paver. This will cause contractors to over-vibrate the concrete and often the concrete will set before finishing is completed. Unusual stiffening and setting can also include retardation which can lead to plastic shrinkage cracks and cracking before saw cuts can be done. Proper setting requires a careful balance between many of the chemical components present in the cementitious material as well as admixtures. The most important of these are Gypsum, Aluminates and Silicates. The following are reactions that contribute to stiffening and setting (Taylor, 2005).

Gypsum

$CS \rightarrow$ Gypsum deposition (Causes False Set)

Aluminates

$3C_3A + CH + 12H \rightarrow CAH$ (Causes Flash Set)

$3C_3A + 3CS + 32H \rightarrow C_3A\cdot 3CS\cdot H_{32}$ (Ettringite)

$2C_3A + C_3A\cdot 3CS\cdot H_{32} \rightarrow C_3A\cdot 3CS\cdot H_{12}$ (Monosulfate)

Silicates

$3C_3S + 6H \rightarrow 3CSH + 6CH$

$3C_2S + 4H \rightarrow 3CSH + CH$

Many factors can lead to early stiffening such as not enough or the wrong kind of Sulfates, Class C fly ash with high C_3A, very fine cement, some admixtures, w/cm ratio or hot weather (Taylor, 2005). Cement manufactures carefully balance the Gypsum, Aluminates and Silicates. However, when SCM's are used, they can sometimes contain some of these compounds, throwing off that balance. Therefore, it is recommended to have Portland cement pre-blended with SCM's so that cement manufacturers can obtain a proper balance of the reactions controlling setting.

Other results of material incompatibility can be cracking and a poor air void system. Cracking is a function of many things including shrinkage, restraint, stiffness, strength and creep (Taylor, 2005). Shrinkage effects are always cumulative and cracking is usually never due to a single mechanism. Problems with the air void system can usually be categorized as the wrong amount of air or the wrong kind of air. Material incompatibility can cause the wrong amount of air by making the air bubbles unstable causing the air content to be unpredictable. The wrong kind of air can include entrapped air where the bubbles are too coarse or coalescence where bubbles are clustered around the outside of the aggregate.

2.**2.8 AGGREGRATE GRADATION**

Recently, aggregate optimization has received increasing attention due to higher use of manufactured aggregates as well as new classes of concrete such as high performance concrete and self compacting concrete. Optimizing aggregates can have many beneficial results such as lower cost due to less cement, less segregation, less shrinkage, less porosity, improved workability and improved finishability. (Fowler, 2005)

The most commonly used method for optimizing aggregates is the Shilstone method. Shilstone’s method focuses on the combined aggregate gradation. It divides the aggregates into coarse, intermediate and fine. This method proposes using the “coarseness chart”, the “18-8 criteria chart” and the “0.45 power chart” to optimize the aggregate gradation.

The “coarseness chart” shown in Figure 2.2.8.1 uses the Coarseness factor and Workability factor.

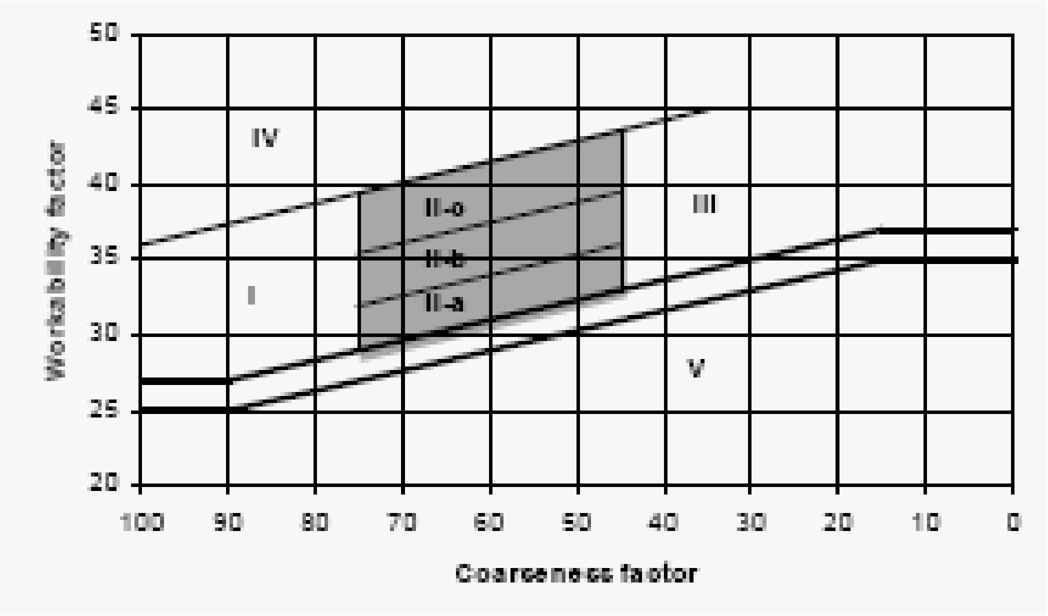


Figure 2.2.8.1 Coarseness Chart (Fowler, 2005)

The Coarseness factor is the combined percentage retained above the 3/8” sieve divided by the combined percentage retained above the #8 sieve. The workability factor is the particles passing the #8 sieve. In addition, the workability factor is increased by 2.5% for each increase of 94 pounds of cement over 564 pounds per cubic yard. The Coarseness Chart places aggregate gradations into 5 Zones (Shilstone, 2005).

Zone I: Mixtures have the tendency to segregate during placement and contribute to edge slump or edge slough and, during vibration lead to an excess of mortar rising to the surface creating spalling and scaling.

Zone II: Mixes in this zone provide good workability and finishability

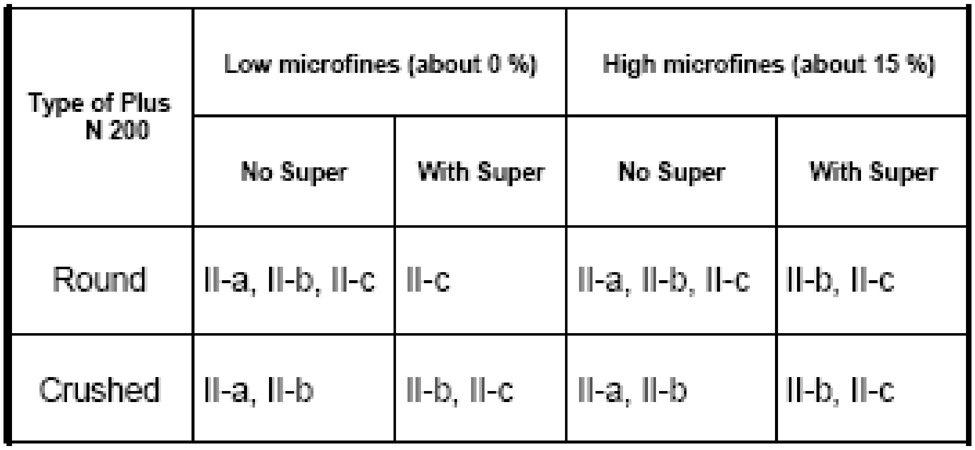
Zone III: Mixes in this zone are an extension of Zone II, but have a nominal max. aggregate size of ½”.

Zone IV: Mixes in this zone are over-sanded requiring more water and air-entraining agents.

Zone V: Mixes in this zone have too much coarse aggregate to be plastic and workable

Table 2.2.8.1 shows the suggested zones in the “Coarseness Chart”

Table 2.2.8.1 Desired Zones in Coarseness Chart (Fowler, 2005)



Type of Plus N 200	Low microfines (about 0 %)		High microfines (about 15 %)	
	No Super	With Super	No Super	With Super
Round	II-a, II-b, II-c	II-c	II-a, II-b, II-c	II-b, II-c
Crushed	II-a, II-b	II-b, II-c	II-a, II-b	II-b, II-c

The 0.45 power curve, shown in figure 2.2.8.2, is based on the percent passing on each sieve. Historically, the 0.45 power curve has been used to develop uniform gradations for asphalt mix designs, however, it is increasingly being used to develop

uniform gradations for Portland cement concrete mix designs (IDOT I.M. 532). The plotted line should follow the trendline as close as possible.

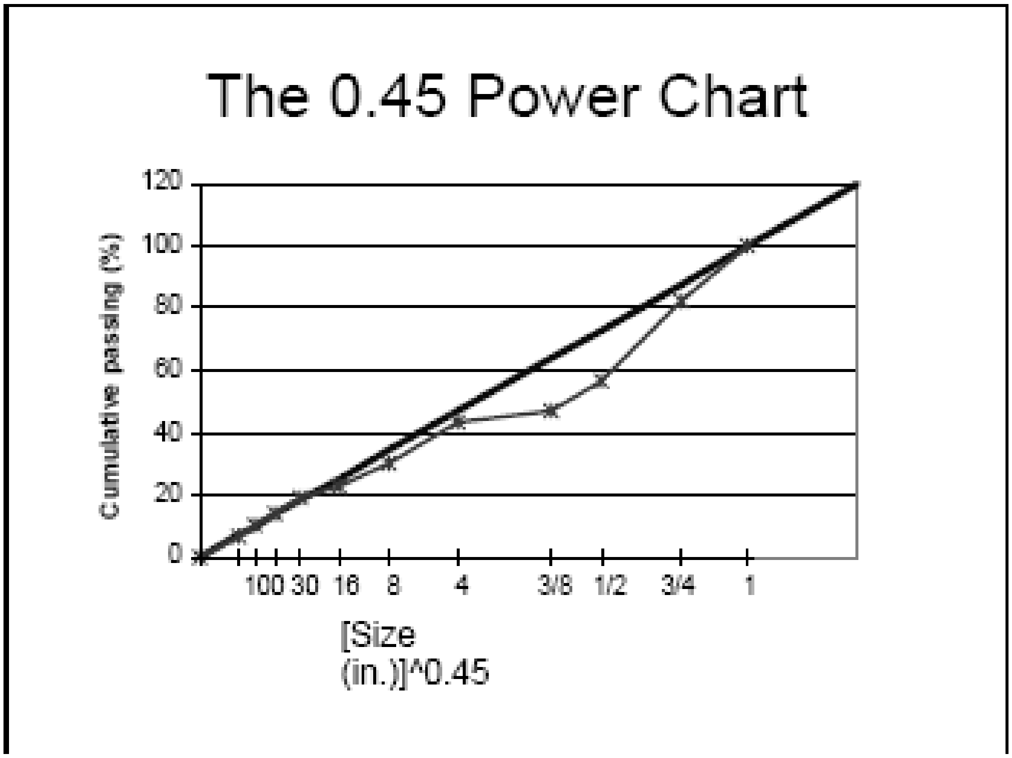


Figure 2.2.8.2 0.45 Power Chart (Fowler, 2005)

The third tool used in the Shilstone method is the percent retained chart. In this chart the combined percentage retained for each sieve is plotted. This chart attempts to produce uniform grading and limits the amounts retained in most sieves between 8% and 18%. A well graded aggregate combination should have no major peaks or valleys and should remain between the boundary lines shown in the chart.

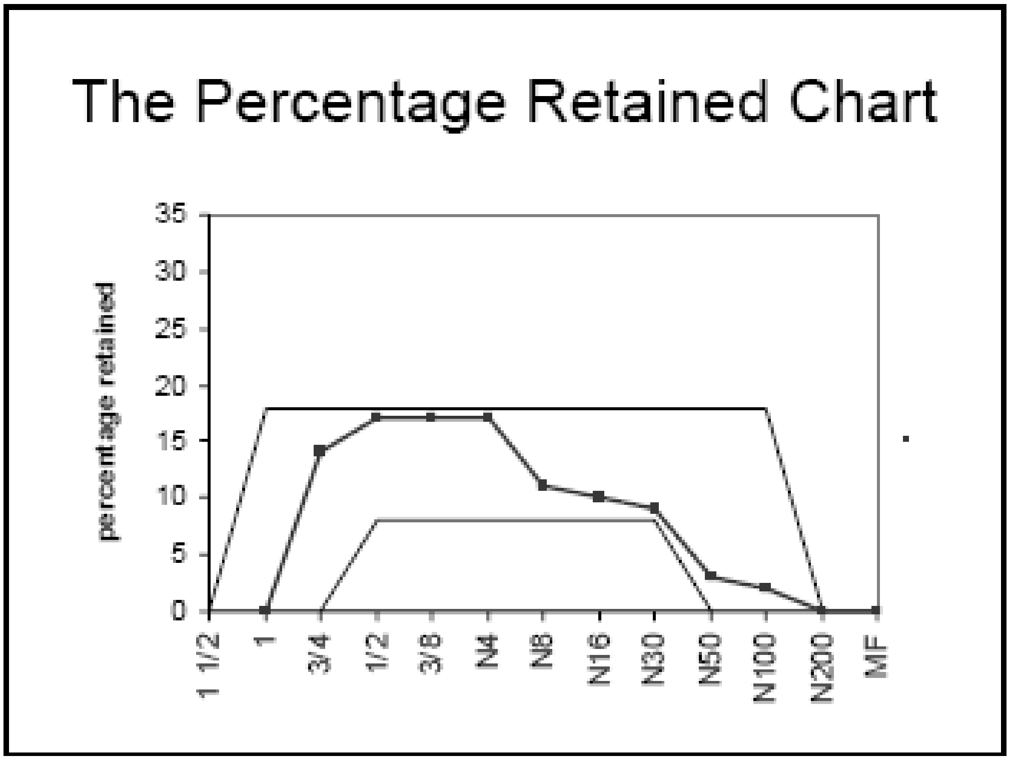


Figure 2.2.8.3 Percentage Retained Chart (Fowler, 2005)

It is important to use all three charts together to determine the optimum aggregate gradation. Although this method does not guarantee a good mix, using all these charts will be very effective in identifying mixtures that may have potential problems. In addition, it is important to realize that the “Coarseness Chart” does not account for the shape or texture of the aggregate. The shape and texture of aggregate particles affect the volume of paste needed to coat particles as well as the interactions during placement. The ideal aggregate shape for workability is smooth and round. Smooth and round particles, such as gravels, have a low surface to volume ratio and require less paste to coat the surfaces of each particle. Crushed limestone aggregates, which usually tend to be more angular and rough than gravel aggregates, have a higher surface to volume ratio, and may require more paste to reduce particle interactions. (IDOT I.M. 532)

Due to a downward trend in concrete pavement performance, the state of Iowa began a program called Quality Management Concrete or QM-C. The initiative was a cooperation between industry and agency to develop a better performing concrete. Today, the basis of QM-C is to have the contractor, with agency oversight, develop their own concrete mix design using optimum aggregate gradation (Smith, 2005). This provides contractors with the control and responsibility to ensure a quality product. These new specifications eliminated many of the problems with past pavements such as highly sanded and gap graded mixes. The new QM-C mixes exhibited much higher workability, strength, consistency and decreased permeability.

Due to concerns from contractors, the new QM-C program started out as a developmental specification with contractors analyzing mixes on pilot projects. As QM-C gained support it moved through the Iowa DOT districts and was used primarily on major projects. In addition, during the early years of implementation, a partnering workshop was required for all QM-C projects. Today, over 60 projects have used the QM-C specifications and the Iowa DOT now offers incentives based on aggregate gradation, primarily the Shilstone principles. Although in order to meet the new specifications most projects require the use of three sources of aggregates, the majority of contractors have accepted the new policies due to better control of mixes and higher quality concrete. (Smith, 2005)

A study was done by Todd Hanson of the Iowa Department of Transportation to evaluate Nebraska’s aggregates using the Shilstone method. Figures 2.2.8.4 – 2.2.8.6 show the results of this analysis using 30% limestone and 70% sand and gravel. Figure 2.2.8.6 also shows the results for mixes using different percentages of limestone. From the figures it is clear that mixes using 30% limestone do not produce a well-graded aggregate structure. Such mixes are gap graded with too much being retained on the #4 sieve and not enough on the #8. In addition, the Coarseness chart shows this mix to be in zone IV, indicating an over-sanded mix having a high water demand. It can also be seen that mixes using a higher percentage of limestone will fall in the desired zone.

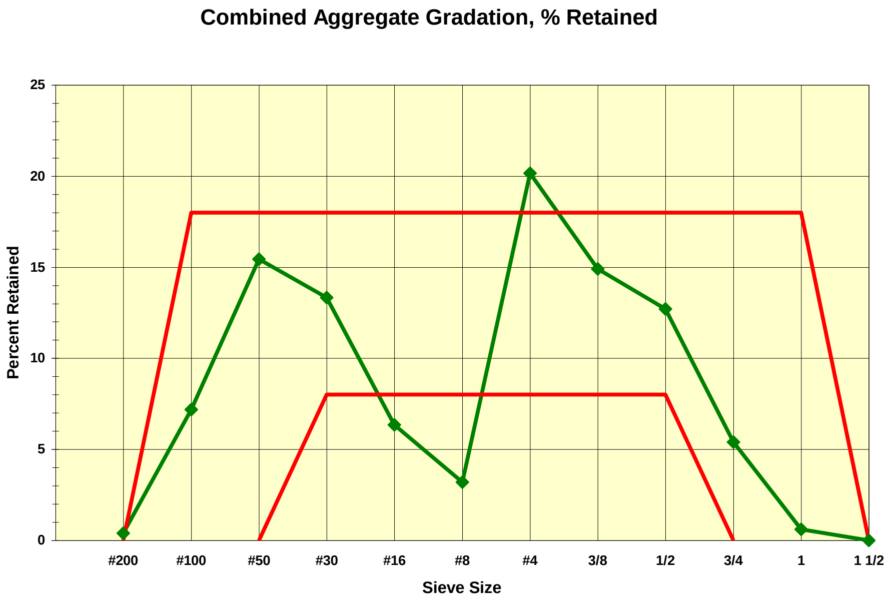


Figure 2.2.8.4 Percent Retained Chart with NE Aggregates
(Hanson, Hart, and Schram ,2004)

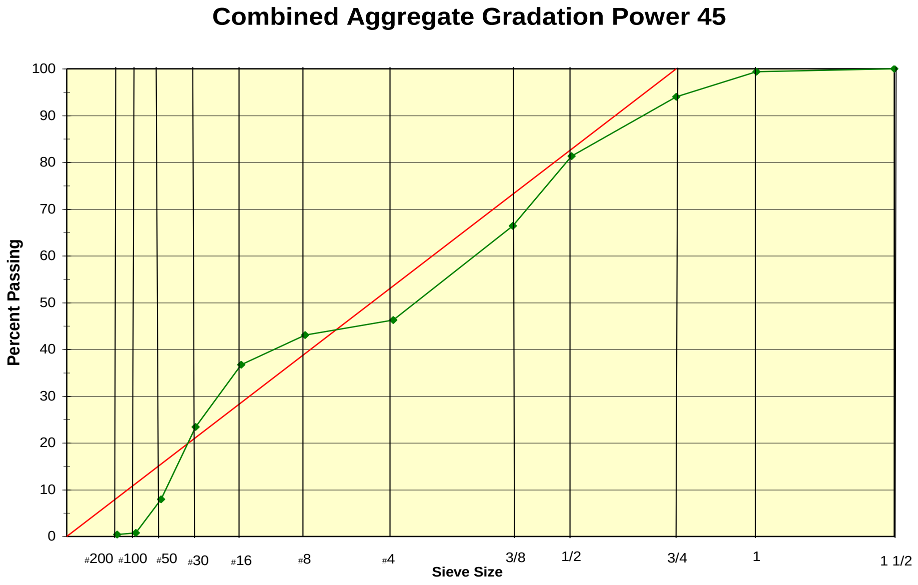


Figure 2.2.8.5 0.45 Power Chart with NE Aggregates (Hanson, Hart and Schram, 2004)

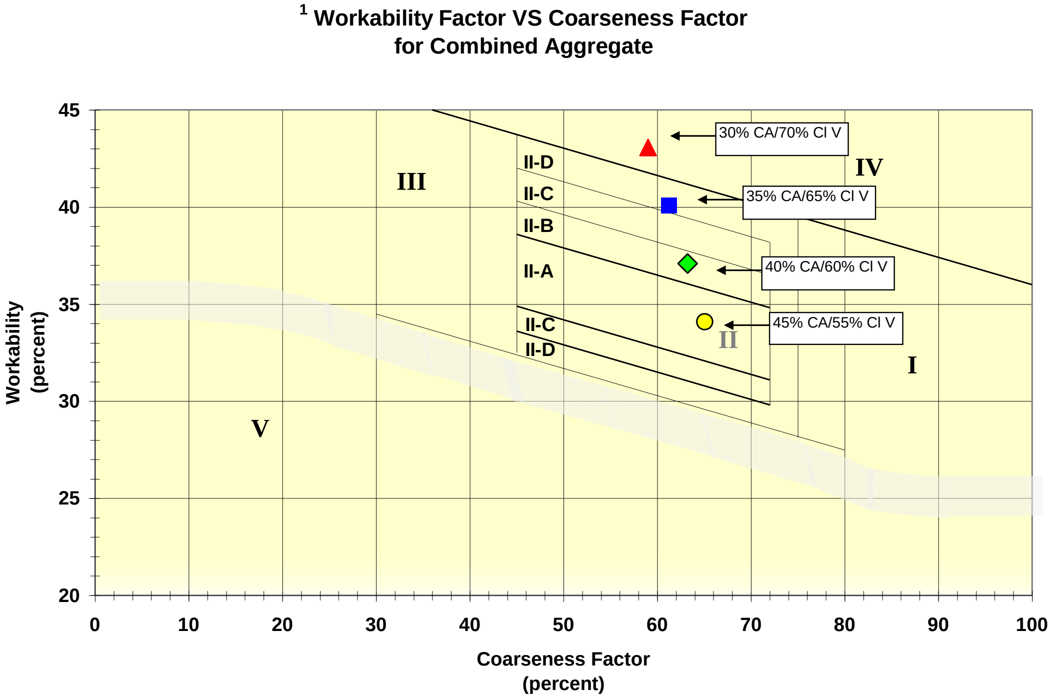


Figure 2.2.8.6 Coarseness Chart with NE Aggregates (Hanson, Hart and Schram, 2004)

2.2.9 JOINT CONSTRUCTION

Jointed plain concrete pavement (JPCP) is the most commonly constructed type of rigid pavement. In this type of pavement, joints are used to permit expansion and contraction of concrete due to temperature and moisture changes, to relieve warping from temperature gradients within the slab and to minimize uncontrolled cracking from frost action (USACE, 1984). In addition, joints can be used to separate areas of concrete placed at different times. The three general types of joints used in rigid pavements are contraction, expansion and construction.

Longitudinal contraction joints are required continuously along the centerline of non-reinforced rigid pavement. These joints are typically formed in the concrete by sawing, creating a weakened plane. In order ensure that cracking will occur along the joint, it is important to saw cut the joints to an adequate depth. This is typically 1/3 the depth of the slab. The timing of the sawing operation is very important, especially with the use of SCM’s. The sawing should be done late enough to avoid raveling of the new concrete, but soon enough so that random cracking does not begin to occur (ACI, 1998). The saw-cut joint is then filled with backer rod and sealed. Figure 2.2.9.1 shows a cross section of a typical longitudinal joint

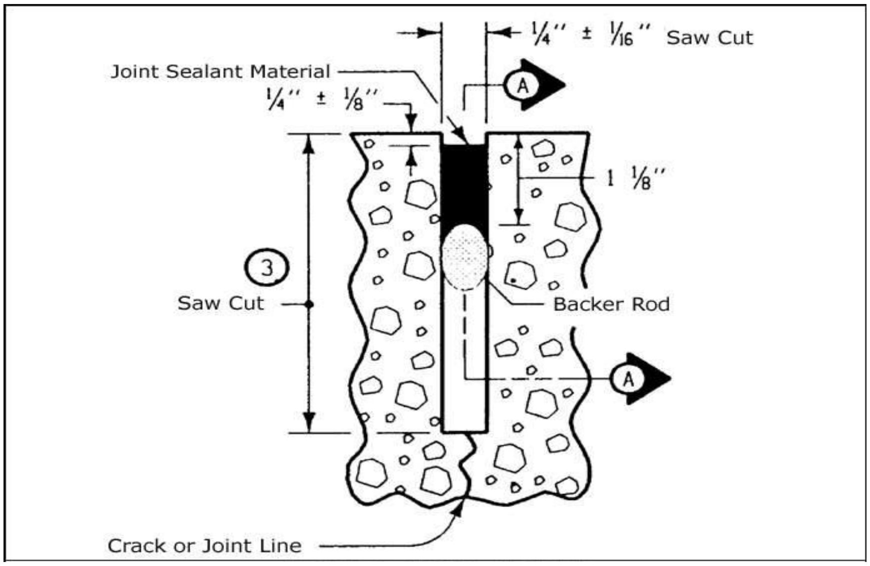


Figure 2.2.9.1 Typical Longitudinal Joint (Steffes, 2005)

Transverse contraction joints should be constructed in the same manner as longitudinal joints. The spacing for transverse joints depends on the pavement thickness and subbase type. It is recommended that joint spacing be between 12 ft and 20 ft [29]. Currently, the state of Nebraska uses a joint spacing of 16.5 ft. Unlike longitudinal joints, transverse joints often require some type of load transfer to prevent joint faulting. The most common method of load transfer is the use of dowel bars. Dowel bars typically have a diameter of 1/8 the thickness of the slab and are generally 18” long. They should be placed at mid-depth of slab, perpendicular to the joint at a spacing of 12”. To ensure adequate performance it is critical to prevent horizontal or vertical misalignment of the dowel bars. Therefore, agencies will specify tolerances for vertical and horizontal translational alignment. The tolerances vary but are typically around 0.25 to 0.375 in/ft. (Rufino et. al., 2005)

Expansion joints are used for the relief of forces resulting from thermal expansion of the pavement and to permit unrestrained differential horizontal movement of adjoining pavement or structures (USACE, 2004). They are placed between any structure projecting through, into or against the pavement (ACI, 1998). The joints should be a minimum of ¼" and an expansion joint filler is placed in the joint. Construction joints are formed at the end of each day or any time interruptions occur in the concreting operation. These can be longitudinal joints (between lanes) or transverse joints (when paving is stopped each day).

In addition, it is known that the current method of joint construction takes time and money as well as creating large amounts of dust. These joints have a tendency to accumulate and retain moisture, accelerating deterioration of the pavement. Therefore, a new method for constructing longitudinal joints is being developed where the joints are formed during paving. This method involves placing joint-forming knives in front and behind the paver as shown in figures 2.2.9.2 and 2.2.9.3.

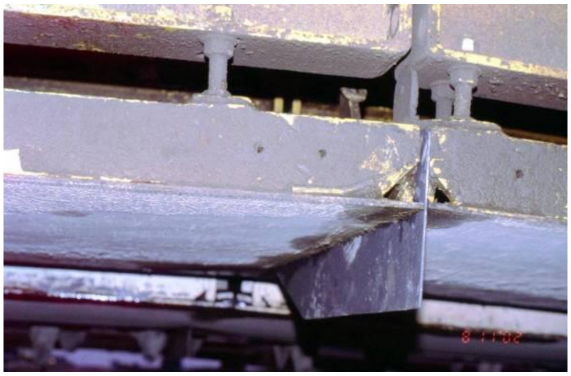


Figure 2.2.9.2 Knife Mounted on Paver (Steffes, 2005)

Figure 2.2.9.3 Knife Formed Joint (Steffes, 2005)

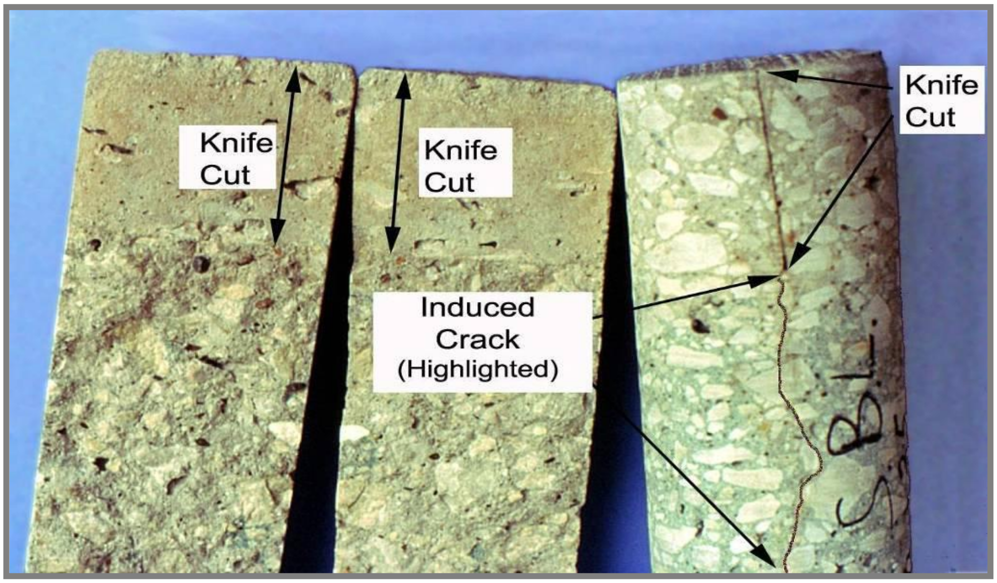


Figure 2.2.9.4 Core of Knife Formed Joint (Steffes, 2005)

This method of forming longitudinal joints is done with one pass during the paving operation. It requires no joint sealing or joint cutting and results in a virtually cost free longitudinal joint. However, further research still needs to be done to investigate the optimum size and shape of the knife and to develop specifications for its use.

2.3 MEETING AT IOWA STATE UNIVERSITY

On May 9, 2005 researchers from the University of Nebraska traveled to Ames, Iowa to meet with personal from the Iowa DOT and Iowa State University. The objective of the meeting was to gain knowledge of research that has been done in Iowa that could assist in the investigation of premature deterioration being observed in Nebraska roads. The discussion was very helpful to the progress of the study because of the comparable existing conditions in Iowa and the similar deterioration problems that they have seen and mitigated. In their opinion, the deterioration being seen in Nebraska roads is most likely due to the same type of problems that Iowa had been seeing. They suggested that Class C fly ash is probably not directly responsible for the deterioration although it may be leading to some of the problems causing the deterioration. A substandard air void system could be mostly responsible for the deterioration. Several steps were suggested to ensure an adequate air void system such as increasing the specifications for target air content as well as putting limitations on vibrator frequencies. A change in the mix design would also be very helpful. In addition, other factors such as joint cutting, joint sealing and deicing chemicals need to be evaluated. The minutes of the meeting at Iowa State University are included in Appendix A.

2.3 SURVEY OF STATE DOT'S

A survey was conducted with experts from various DOTs throughout the Unitd States and Canada. The purpose of the survey was to determine what SCMs were allowed in other states and if other states were experiencing similar problems with their pavements. In addition, the survey requested recommendations and comments for identifying and mitigating the deterioration being seen in Nebraska. The responses of that survey are included in Appendix D.

2.4 UNIVERSITY OF NEBRASKA WORKSHOP

A workshop was held in Nebraska and was attended by various experts in concrete pavement technology and concrete chemistry. The attendees included people from academia, DOTs as well as the project sponsors. The two day workshop included site visits in Lincoln, Nebraska to show examples of the premature deterioration being seen. Presentations were given by various experts on their experience with pavement durability and mitigation measures that are effective in controlling deterioration mechanism. In addition, a discussion was held regarding the information presented as well as recommendations for potential mixes. The minutes of the roundtable discussion can be found in Appendix B.

3.0 POTENTIAL MIXES

Using the information gathered through the literature review, potential mixes using Class C fly ash were identified. The potential mixes were carefully selected by the UNL team along with NDOR personnel with the goal of producing a concrete that would meet expected performance criteria. These mixes are both binary and ternary blends using the appropriate replacement levels needed to satisfy performance criteria. Some of the mixes also use alternative aggregate gradations instead of the standard 30% limestone – 70% sand and gravel. Since the chemical durability of concrete is most effectively increased by proper mix design, the foremost factor in determining the mix proportions was the expected resistance to ASR. In addition, since the exact mix proportions that are needed to effectively control ASR while meeting other performance criteria are unknown, the mix proportions may be adjusted during throughout the testing procedures to produce the best overall results. Any adjusted mixes must undergo all durability and performance testing.

3.1 MIX DESCRIPTION

In order to effectively track and label each potential mix, a labeling system was developed. The labeling system gives an overall description of each mix enabling the mix design associated with each mix to be easily identified. The definitions of the symbols used in the labeling system are given below.

CM = Total Cementitious Material
C = Class C Fly Ash
F = Class F Fly Ash
S = Ground Granulated Blast Furnace Slag

L = Limestone

The following table shows a summary of the potential mixes.

Table 3.1.1 Summary of Potential Mixes

Mix	**Total Cementitious Material (lb/yd^3)**	**Class C Fly Ash (%)**	**Class F Fly Ash (%)**	**Slag (%)**	**Limestone (% of Total Agg.)**	**Sand & Gravel (% of Total Agg.**
1A	564	40	0	0	30	70
1B	564	40	0	0	45	55
2	564	20	20	0	30	70
3	564	20	0	30	30	70
4A	650	35	0	0	30	70
4B	650	35	0	0	45	55
Reference Mixes						
5	564	17	0	0	30	70
6	564	0	25	0	30	70

3.2 COST ESTIMATE

Although the most important aspect of the potential mixes is the performance, the cost of each mix must be considered. Since the current standard mix has shown excellent performance, any proposed mixes must have an equal or lesser unit cost. Ideally, since the standard 1PF mix uses Class F fly ash which must be imported at a cost equal to Portland cement, the proposed mixes using Class C fly ash should have a significantly reduced cost. In addition, as mentioned earlier, due to the limited availability of limestone in the western parts of the state, a cost analysis must be performed for both eastern and western Nebraska. Table 3.3.1 shows the estimated unit cost of each material. In addition, Tables 3.1.2 and 3.1.3 show the concrete cost per cubic yard using estimated material quantities assuming w/cm of 0.45 and an air content of 7%.

Table 3.2.1 Typical Material Cost as of 2006

	Cost per Ton	
Material	Eastern Nebraska	Western Nebraska
TYPE I/II Cement	$90	$90
TYPE 1PF Cement	$90	$90
Class C Fly Ash	$15	$15
GGBFS	$90	$90
47B Sand and Gravel	$5	$5
Limestone	$10	$25

Table 3.2.2 Estimated Concrete Cost of Potential Mixes

		Cementitious Material					**Aggregates**		**[1]Materials Cost Per yd^3**	
	Mix	***Type I/II*** **lbs/yd^3**	***C ash*** **lbs./yd^3**	***1PF*** **lbs/yd^3**	***Slag*** **lbs/yd^3**	***Total*** **lbs/yd^3**	***LS*** **lbs/yd3**	***47B S & G*** **lbs/yd^3**	***Eastern Nebraska***	**Western Nebreska**
1A	40% C 30% Limestone	338	226	0	0	564	882	2074	$26.50	$33.12
1B	40% C 45% Limestone	338	226	0	0	564	1323	1630	$27.61	$37.53
2	20% C, 20% F 30% Limestone	0	113	451	0	564	880	2068	$30.71	$37.31
3	20% C, 30%S 30% Limestone	282	113	0	169	564	883	2077	$30.76	$37.39
4A	35% C 30% Limestone	423	228	0	0	650	830	1952	$29.75	$35.98
4B	35% C 45% Limestone	423	228	0	0	650	1245	1534	$30.78	$40.12
5	17%C 30% Limestone	468	96	0	0	564	887	2086	$31.44	$38.09
6	25% F 30% Limestone	0	0	564	0	564	882	2074	$34.98	$41.59

From the tables it can be seen that all potential mixes, even in western Nebraska with an increased cost for limestone, would have a lower material cost than the current standard mix (Mix #6) due to the premium cost of importing Class F fly ash.

PART 2 – EXPERIMENTAL INVESTIGATION

4.0 MATERIALS

Due to the longevity of the project and large testing program, only the results of the ASTM C 1567 testing have completed to date. The results of that testing are presented herein.

4.1 MATERIAL CHARACTERISTICS

As discussed previously, the resistance of a given concrete to Alkali-Silica reaction is a very dependent on the amount and type of cementitious materials that are used. However, the chemical composition of the materials and as well as the physical properties can have a significant effect on the expansions caused by ASR. Therefore, it was important to accurately identify the chemical and physical characteristics of any of the materials used in the mixes. All of the physical and chemical analysis of the cementitious material as well as the aggregates was conducted by NDOR. In addition, an adequate amount of materials were obtained for the entire testing program. Therefore, the mixes throughout the entire project will contain identical materials, making a comparison of the results easier and more valuable.

4.1.1 SOURCE OF MATERIALS

For the scope of this project it was decided to use only one source of each material. Due to the large testing program that was proposed as well as the number of potential mixes, testing for effect of the different material sources would significantly increase the number of mixes that must be tested. The fine aggregate that was chosen was identified as being the most reactive aggregate that is commonly used in Nebraska. Therefore, any mixes that produce acceptable expansion, would perform adequately with any other source of aggregate that is used in Nebraska. The source of the cementitious materials was chosen to represent the materials that would most commonly be used in PCCP in Nebraska. Table 4.1.1.1 shows the source of each material that was used in the testing.

Table 4.1.1.1 Source of Materials

Material	Producer	Source	Location
TYPE 1/II Cement	Ashgrove	Louisville NE Plant	Louisville, NE
TYPE 1PF Cement	Ashgrove	Louisville NE Plant	Louisville, NE
Class C Fly Ash	NE Ash	Gerald Gentlemen Station	Sutherland, NE
GGBFS	Lafarge	South Chicago Plant	Chicago, IL
47B Sand & Gravel	Lyman Richey	Plant 40	Waterloo, NE
Limestone	Martin Marietta	Weeping Water	Weeping Water, NE

4.1.2 ANALYSIS OF AGGREGATE

The following figures present the results of the physical analysis aggregate. The analysis was done by NDOR personnel to ensure that the material complied with NDOR standards for aggregate used in 47B concrete. The analysis included testing for bulk specific gravity, percent absorption, percent of soft particles, soundness and well as a sieve analysis. Figure 4.1.2.1 shows the physical analysis of the 47B sand and gravel which will be used as the Class B fine aggregate.

Physical Analysis of Aggregate

Project No.: *UNL Fly Ash Research Project*
Material: *Fine Aggregate - Class B (47B Sand & Gravel)*
For Use in: *Class 47B Concrete*
Source: *Plant 40*
Producer: *Lyman Richey*
Laboratory: *NDOR - Lincoln, NE*

Lab Identification: *FA06 - 257*

Sampled: *9/21/2006*
Received: *9/22/2006*
Reported: *10/6/2006*

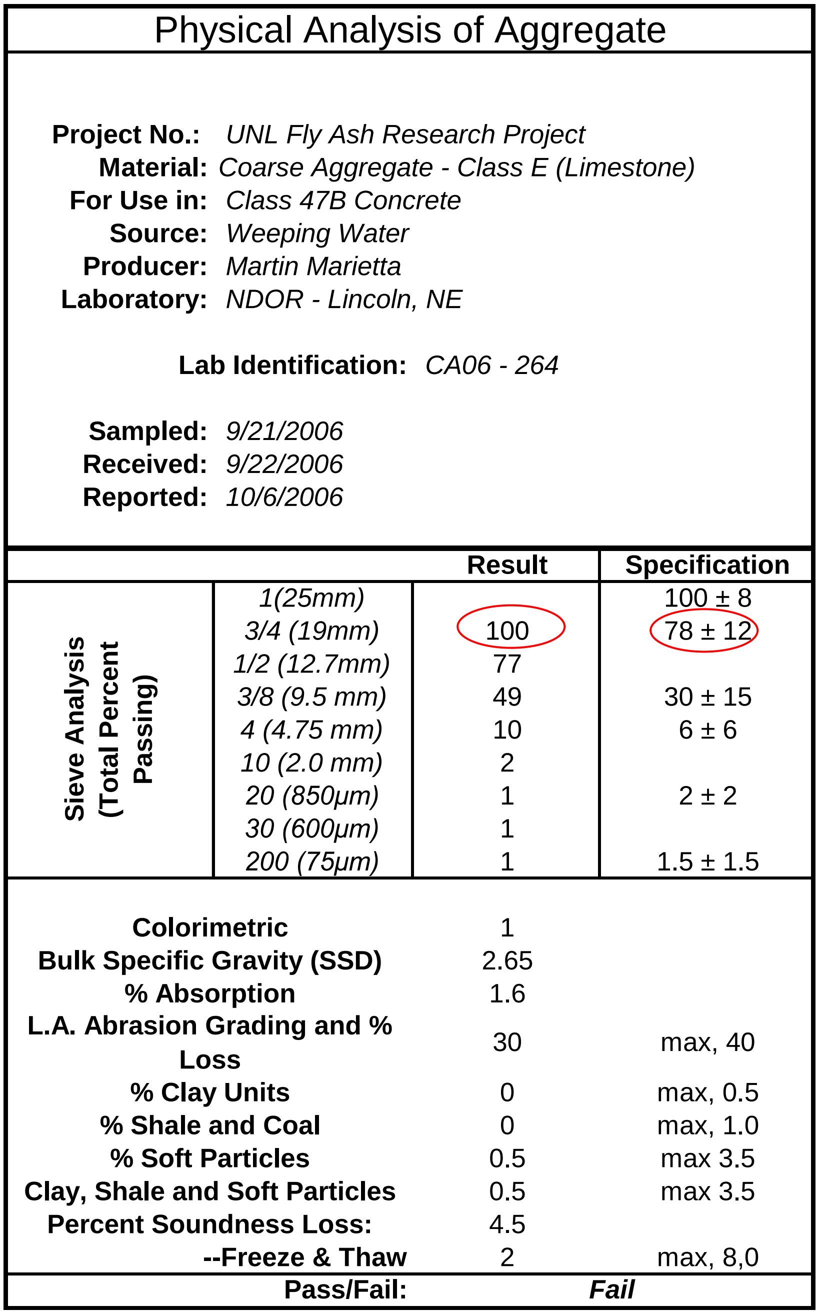
		Result	**Specification**
Sieve Analysis (Total Percent Passing)	*1½ (37.5mm)*		
	1 (25.0mm)		100
	¾ (19.00mm)	100	
	3/8 (9.5 mm)	96	
	4 (4.75 mm)	88	87 ± 10
	10 (2.0 mm)	65	60 ± 10
	20 (850μm)	42	
	30 (600μm)	31	28 ± 12
	200 (75μm)	0	1.5 ± 1.5

Colorimetric	1	
Bulk Specific Gravity (SSD)	2.62	
% Absorption	0.5	
% Clay Units	0	max, 0.5
Percent Soundness Loss:		
--Sodium Sulfate	2	max, 10
Pass/Fail:	***Pass***	

Figure 4.1.2.1 Physical Analysis of Class B Aggregate - 47B Sand & Gravel

As discussed previously, the coarse aggregate to be used in the testing was Limestone. The specifications for coarse aggregate used in Nebraska's 47B pavements are provided in NDOR specifications for Class E aggregate. Although the material was ordered to meet these specifications, test results showed that the limestone did not pass the specifications for the sieve analysis. The material composition was identical to that of Class E limestone, however, it was graded to a smaller particle size.

Although it was shown that the limestone did not meet NDOR specifications, it was decided to continue with the ASTM C 1567 testing using the not compliant aggregate. The only aspect of the physical analysis that did not meet the specification was the sieve analysis. However, the ASTM C 1567 standard specifications require that all the aggregate be sieved to an exact gradation. Since the limestone had to be crushed to meet the grading requirements, the particle gradation of the original limestone does not have any effect on the testing. In addition, limestone is known to be a non-reactive aggregate and the ASTM C 1567 test results will not be affected by the source of limestone. However, for all further performance tests, where the limestone will not be sieved, a new source of limestone will be used that complies with all NDOR standards for Class E coarse aggregate used in 47B concrete. Figure 4.1.2.2 shows the physical analysis for the non-compliant limestone that was used in the ASTM C 1567 testing. Figure 4.1.2.3 presents the physical analysis of the limestone that will be used for all further testing.



Physical Analysis of Aggregate

Project No.: *UNL Fly Ash Research Project*
Material: *Coarse Aggregate - Class E (Limestone)*
For Use in: *Class 47B Concrete*
Source: *Weeping Water*
Producer: *Martin Marietta*
Laboratory: *NDOR - Lincoln, NE*

Lab Identification: *CA06 - 264*

Sampled: *9/21/2006*
Received: *9/22/2006*
Reported: *10/6/2006*

		Result	**Specification**
Sieve Analysis (Total Percent Passing)	*1(25mm)*		100 ± 8
	3/4 (19mm)	100	78 ± 12
	1/2 (12.7mm)	77	
	3/8 (9.5 mm)	49	30 ± 15
	4 (4.75 mm)	10	6 ± 6
	10 (2.0 mm)	2	
	20 (850μm)	1	2 ± 2
	30 (600μm)	1	
	200 (75μm)	1	1.5 ± 1.5

Colorimetric	1	
Bulk Specific Gravity (SSD)	2.65	
% Absorption	1.6	
L.A. Abrasion Grading and % Loss	30	max, 40
% Clay Units	0	max, 0.5
% Shale and Coal	0	max, 1.0
% Soft Particles	0.5	max 3.5
Clay, Shale and Soft Particles	0.5	max 3.5
Percent Soundness Loss:	4.5	
--Freeze & Thaw	2	max, 8,0
Pass/Fail:	***Fail***	

Figure 4.1.2.2 Physical Analysis of Class E Aggregate - Limestone

Physical Analysis of Aggregate

Project No.: *UNL Fly Ash Research Project*
Material: *Coarse Aggregate - Class E (Limestone)*
For Use in: *Class 47B Concrete*
Source: *Weeping Water*
Producer: *Martin Marietta*
Laboratory: *NDOR - Lincoln, NE*

Lab Identification: *CA06 - 311*

Sampled: *10/27/2006*
Received: *10/30/2006*
Reported: *11/3/2006*

		Result	**Specification**
Sieve Analysis (Total Percent Passing)	*1(25mm)*	99	100 ± 8
	3/4 (19mm)	87	78 ± 12
	1/2 (12.7mm)		
	3/8 (9.5 mm)	29	30 ± 15
	4 (4.75 mm)	6	6 ± 6
	10 (2.0 mm)		
	20 (850μm)	3	2 ± 2
	30 (600μm)		
	200 (75μm)	2.5	1.5 ± 1.5
Colorimetric		1	
Bulk Specific Gravity (SSD)		2.66	
% Absorption		1.3	
L.A. Abrasion Grading and % Loss		28	max, 40
% Clay Units		0	max, 0.5
% Shale and Coal		0.2	max, 1.0
% Soft Particles		0	max 3.5
Clay, Shale and Soft Particles		0.2	max 3.5
Percent Soundness Loss:			
--Freeze & Thaw		3.7	max, 8,0
Pass/Fail:		***Pass***	

Figure 4.1.2.3 Physical Analysis of Class E Aggregate – Limestone Sample 2

4.1.3 CHEMICAL AND PHYSICAL ANALYSES OF CEMENTITIOUS MATERIAL

All cementitious material was analyzed for chemical composition. This information will be critical in the interpretation of the results since the chemical composition of the cementitious material has a significant effect of the performance of the concrete. In addition, an analyses of the physical characteristics was performed. The analyses showed that all the cementitious material met NDOR specifications. The following figures show the results of the chemical and physical analyses.

Cement Analysis

Brand: *Ash Grove Cement*
Type: *Type IPF*
Mill Location: *Louisville, NE*
Field I.D.: *1567 Tests*

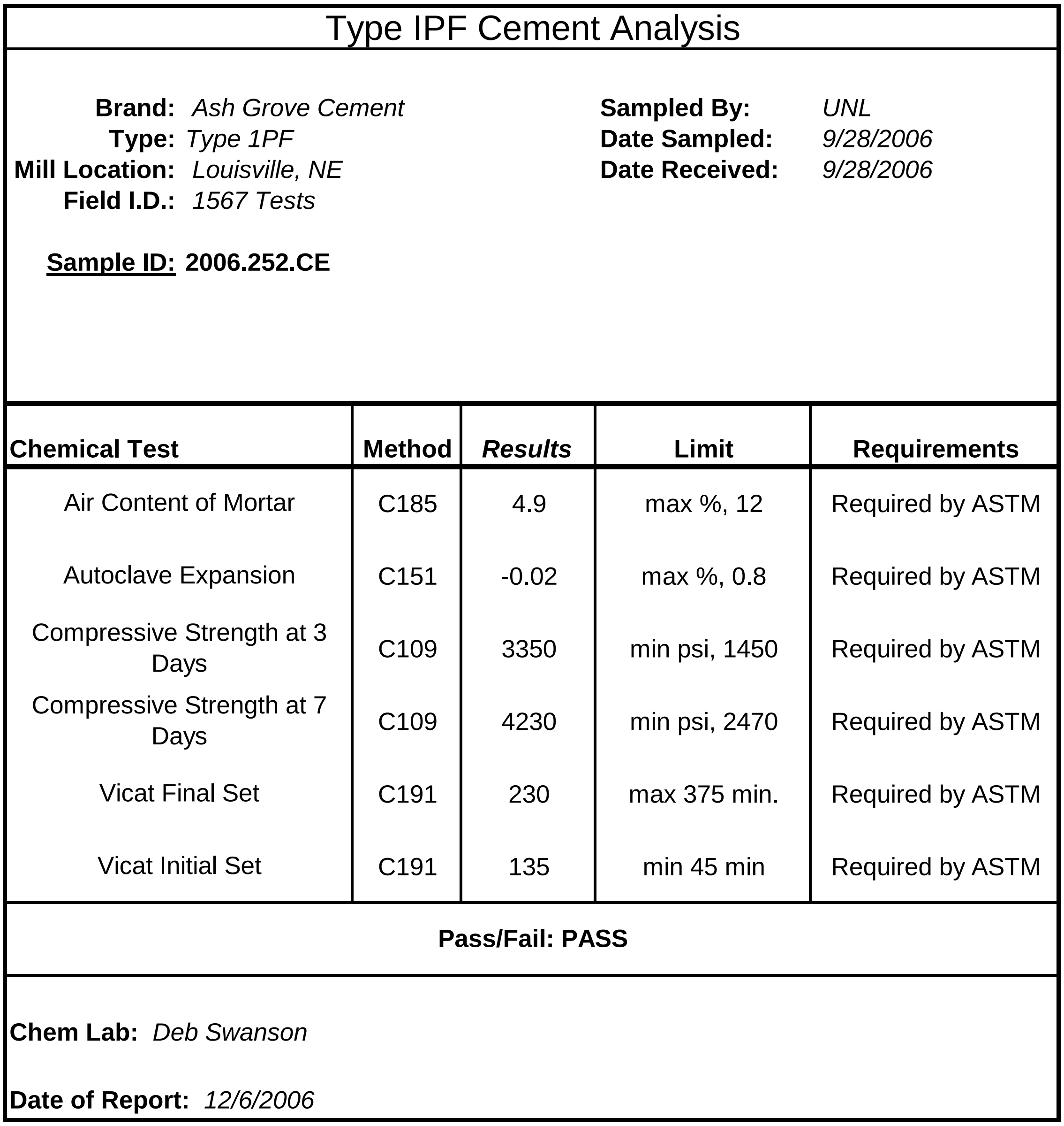
Sampled By: *UNL*
Date Sampled: *9/28/2006*
Date Received: *9/28/2006*

Sample ID
Sample 1: *2006.252.CE*
Sample 2: *2006.253.CE*
Sample 3: *2006.254.CE*

Chemical Test	**Method**	**Results for Each Sample**			**Limit**	**Requirements**
		Sample 1	***Sample 2***	***Sample 3***		
Aluminum Oxide (Al_2O_3)		8.32	8.33	8.29	n/a	Required by NDOR
Calcium Oxide (CaO)		51.83	51.69	51.8	n/a	Required by NDOR
Ferric Oxide (Fe_2O_3)		3.22	3.21	3.21	n/a	Required by NDOR
Free Lime (CaO)		1.15	-	-	n/a	Required by NDOR
Loss on Ignition	C114	1.14	1.24	1.25	max, 5%	Required by ASTM
Magnesium Oxide (MgO)	C114	2.79	2.85	2.83	max, 6%	Required by ASTM
Potassium Oxide (K_2O)		0.66	0.65	0.66	n/a	Required by NDOR
Silicon Dioxide (SiO_2)		28.05	28.13	27.99	n/a	Required by NDOR
Sodium Oxide (Na_2O)		0.2	0.21	0.21	n/a	Required by NDOR
Sulfur Trioxide (SO_3)	C114	3.39	3.33	3.36	max, 4%	Required by ASTM
Equivalent Alkalies	C114	0.64	0.64	0.64	n/a	Optional for NDOR
	Pass/Fail:	***Pass***	***Pass***	***Pass***		

Chem Lab: *Tanya Freeman and Jasmine Lee*
Date of Report: *10/16/2006*

Figure 4.1.3.1 Chemical Analysis of Ash Grove Type 1PF Cement



Type IPF Cement Analysis

Brand: *Ash Grove Cement*
Type: *Type 1PF*
Mill Location: *Louisville, NE*
Field I.D.: *1567 Tests*

Sampled By: *UNL*
Date Sampled: *9/28/2006*
Date Received: *9/28/2006*

Sample ID: **2006.252.CE**

Chemical Test	**Method**	***Results***	**Limit**	**Requirements**
Air Content of Mortar	C185	4.9	max %, 12	Required by ASTM
Autoclave Expansion	C151	-0.02	max %, 0.8	Required by ASTM
Compressive Strength at 3 Days	C109	3350	min psi, 1450	Required by ASTM
Compressive Strength at 7 Days	C109	4230	min psi, 2470	Required by ASTM
Vicat Final Set	C191	230	max 375 min.	Required by ASTM
Vicat Initial Set	C191	135	min 45 min	Required by ASTM

Pass/Fail: PASS

Chem Lab: *Deb Swanson*

Date of Report: *12/6/2006*

Figure 4.1.3.2 Physical Analysis of Ash Grove Type 1PF Cement

Cement Analysis

Brand:	*Ash Grove Cement*	**Sampled By:**	*UNL*
Type:	*Type I/II*	**Date Sampled:**	*9/28/2006*
Mill Location:	*Louisville, NE*	**Date Received:**	*9/28/2006*
Field I.D.:	*1567 Tests*		

Sample ID

Sample 1:	*2006.249.CE*
Sample 2:	*2006.250.CE*
Sample 3:	*2006.251.CE*

Chemical Test	**Method**	**Results for Each Sample**			**Limit**	**Requirements**
		Sample 1	***Sample 2***	***Sample 3***		
Aluminum Oxide (Al_2O_3)	C114	4.27	4.21	4.21	max, 6%	Required by ASTM
Calcium Oxide (CaO)		61.84	61.93	61.84	n/a	Required by NDOR
Ferric Oxide (Fe_2O_3)	C114	3.02	3.02	3	max, 6%	Required by ASTM
Free Lime (CaO)		1.17	-	-	n/a	Required by NDOR
Insoluble Residue	C114	0.54	-	-	max, 0.75%	Required by ASTM
Loss on Ignition	C114	2.91	2.95	2.97	max, 3%	Required by ASTM
Magnesium Oxide (MgO)	C114	3.12	3.1	3.14	max, 6%	Required by ASTM
Potassium Oxide (K_2O)		0.58	0.58	0.59	n/a	Required by NDOR
Silicon Dioxide (SiO_2)	C114	20.37	20.27	20.29	n/a	Required by ASTM
Sodium Oxide (Na_2O)		0.12	0.12	0.12	n/a	Required by NDOR
Sulfur Trioxide (SO_3)	C114	2.86	2.86	2.86	max, 3%	Required by ASTM
Tricalcium Aluminate (C_3A)	C114	6.2	6	6.1	max, 8.5%	Required by ASTM
Equivalent Alkalies	C114	0.51	0.51	0.51	max, 0.6%	Optional for ASTM
	Pass/Fail:	***Pass***	***Pass***	***Pass***		

Chem Lab: *Jasmine Lee*
Date of Report: *10/16/2006*

Figure 4.1.3.3 Chemical Analysis of Ash Grove Type I/II Cement

Type I/II Cement Analysis

Brand:	*Ash Grove Cement*	**Sampled By:**	*UNL*
Type:	*Type I/II*	**Date Sampled:**	*9/28/2006*
Mill Location:	*Louisville, NE*	**Date Received:**	*9/28/2006*
Field I.D.:	*1567 Tests*		

Sample ID: **2006.249.CE**

Chemical Test	**Method**	***Results***	**Limit**	**Requirements**
Air Content of Mortar	C185	7	max %, 12	Required by ASTM
Autoclave Expansion	C151	0.09	max %, 0.8	Required by ASTM
Compressive Strength at 3 Days	C109	3650	min psi, 1450	Required by ASTM
Compressive Strength at 7 Days	C109	4330	min psi, 2470	Required by ASTM
Fineness, Air Permeability	C204	440	min m^2/kg, 280	Required by ASTM
Vicat Final Set	C191	225	max 375 min.	Required by ASTM
Vicat Initial Set	C191	145	min 45 min	Required by ASTM

Pass/Fail: PASS

Chem Lab: *Deb Swanson*

Date of Report: *12/6/2006*

Figure 4.1.3.4 Physical Analysis of Ashgrove Type I/II Cement

Fly Ash Analysis

Brand: *Plains Pozzolanic*
Type: *Class C Fly Ash*
Mill Location: *Gerald Gentlemen Station*
Field I.D.: *1567 Tests*

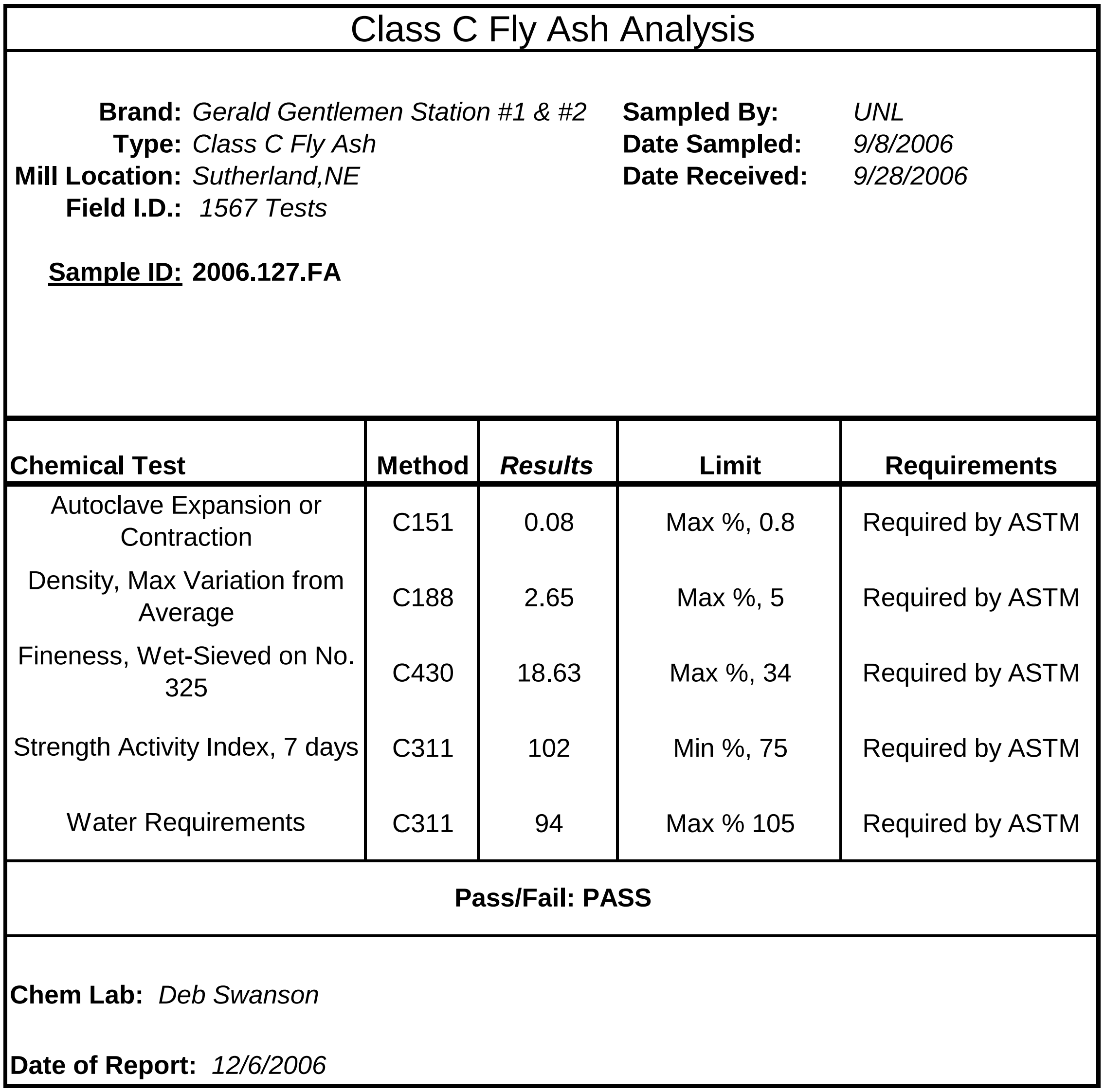
Sampled By: *UNL*
Date Sampled: *9/8/2006*
Date Received: *9/28/2006*

Sample ID
Sample 1: *2006.127.FA*
Sample 2: *2006.128.FA*
Sample 3: -

Chemical Test	Method	Results for Each Sample			Limit	Requirements
		Sample 1	***Sample 2***	***Sample 3***		
Aluminum Oxide (Al_2O_3)		18.72	18.7	-	n/a	Required by NDOR
Calcium Oxide (CaO)		28.15	28.15	-	n/a	Required by NDOR
Equivalent Alkalies		1.8	1.8		max, 1.9%	Required by NDOR
Ferric Oxide (Fe_2O_3)		5.1	5.12	-	n/a	Required by NDOR
Free Lime (CaO)		0.02	-	-	n/a	Required by NDOR
Magnesium Oxide (MgO)		4.84	4.82	-	n/a	Required by NDOR
Potassium Oxide (K_2O)		0.4	0.4	-	n/a	Required by NDOR
Silicon Dioxide (SiO_2)		33.99	34.12	-	n/a	Required by NDOR
$SiO_2 + Al_2O_3 + Fe_2O_3$	C311	57.81	57.94	-	min, 49.5%	Required by ASTM
Sodium Oxide (Na_2O)		1.54	1.54	-	n/a	Required by NDOR
Sulfur Trioxide (SO_3)	C311	1.86	1.84	-	max, 5%	Required by ASTM
Loss on Ignition	C311	0.4	0.35	-	max, 6%	Optional for ASTM
	Pass/Fail:	***Pass***	***Pass***	**-**		

Chem Lab: *Jasmine Lee*
Date of Report: *10/16/2006*

Figure 4.1.3.5 Chemical Analysis of Class C Fly Ash



Class C Fly Ash Analysis

Brand:	*Gerald Gentlemen Station #1 & #2*	**Sampled By:**	*UNL*
Type:	*Class C Fly Ash*	**Date Sampled:**	*9/8/2006*
Mill Location:	*Sutherland,NE*	**Date Received:**	*9/28/2006*
Field I.D.:	*1567 Tests*		

Sample ID: **2006.127.FA**

Chemical Test	**Method**	***Results***	**Limit**	**Requirements**
Autoclave Expansion or Contraction	C151	0.08	Max %, 0.8	Required by ASTM
Density, Max Variation from Average	C188	2.65	Max %, 5	Required by ASTM
Fineness, Wet-Sieved on No. 325	C430	18.63	Max %, 34	Required by ASTM
Strength Activity Index, 7 days	C311	102	Min %, 75	Required by ASTM
Water Requirements	C311	94	Max % 105	Required by ASTM

Pass/Fail: PASS

Chem Lab: *Deb Swanson*

Date of Report: *12/6/2006*

Figure 4.1.3.6 Physical Analysis of Class C Fly Ash

Ground Granulated Blast Furnace Slag Analysis

Brand: *Lafarge North America*
Type: *GGBFS (Slag)*
Mill Location: *South Chicago Plant*
Field I.D.: *1567 Tests*

Sampled By: *UNL*
Date Sampled: *9/12/2006*
Date Received: *9/28/2006*

Sample ID
Sample 1: *2006.129.FA*
Sample 2: *2006.130.FA*
Sample 3: *2006.131.FA*

Chemical Test	**Method**	**Results for Each Sample**			**Limit**	**Requirements**
		Sample 1	***Sample 2***	***Sample 3***		
Sulfide Sulfur (S)	C989	1.3	1.3	1.3	max, 2.5%	Required by ASTM
Total Sulfate ion as SO_3	C989	3.3	3.3	3.3	n/a	Required by ASTM
Aluminum Oxide (Al_2O_3)		11.25	11.31	11.31	n/a	Optional for NDOR
Calcium Oxide (CaO)		43.31	43.27	43.3	n/a	Optional for NDOR
Ferric Oxide (Fe_2O_3)		0.36	0.35	0.34	n/a	Optional for NDOR
Magnesium Oxide (MgO)		10.6	10.74	10.73	n/a	Optional for NDOR
Manganic Oxide (Mn_2O_3)		0.43	0.43	0.42	n/a	Optional for NDOR
Phosphorus Pentoxide (P_2O_5)		0.02	0.02	0.02	n/a	Optional for NDOR
Potassium Oxide (K_2O)		0.36	0.36	0.36	n/a	Optional for NDOR
Silicon Dioxide (SiO_2)		31.63	31.69	31.83	n/a	Optional for NDOR
Sodium Oxide (Na_2O)		0.19	0.19	0.19	n/a	Optional for NDOR
Strontium Oxide (SrO)		0.04	0.04	0.04	n/a	Optional for NDOR
Titanium Dioxide (TiO_2)		0.55	0.56	0.56	n/a	Optional for NDOR
	Pass/Fail:	***Pass***	***Pass***	***Pass***		

Chem Lab: *Jasmine Lee*
Date of Report: *10/16/2006*

Figure 4.1.3.7 Chemical Analysis of Ground Granulated Blast Furnace Slag

GGBFS Analysis

Brand: *Lafarge North America*
Type: *Chicago, IL*
Mill Location: *South Chicago Plant*
Field I.D.: *1567 Tests*

Sampled By: *UNL*
Date Sampled: *9/12/2006*
Date Received: *9/28/2006*

Sample ID: **2006.129.FA**

Chemical Test	**Method**	***Results***	**Limit**	**Requirements**
Air Content of Mortar	C185	5.7	Max %, 12	Required by ASTM
Density, Max Variation from Average	C188	3.18	Max %, 5	Required by ASTM
Fineness, Wet-Sieved on No. 325	C430	0.53	Max %, 20	Required by ASTM
Strength Activity Index, 28 days	C311	127	Min %, 110	Required by ASTM
Stength Activity Index, 7 days	C311	97	n/a	Required by ASTM
Water Requirements	C311	98	Max % 105	Required by ASTM

Pass/Fail: PASS

Chem Lab: *Deb Swanson*

Date of Report: *12/6/2006*

Figure 4.1.3.8 Physical Analysis of Ground Granulated Blast Furnace Slag

5.0 ASR TESTING USING ASTM C 1567

5.1 TESTING PROCEDURE

The testing was conducting in strict accordance with ASTM C 1567 *Standard Test method for Determining the Potential Alkali-Silica Reactivity of combinations of Cementitious materials and Aggregate (Accelerated Mortar-Bar Method).* As previously discussed, the test was carried out to 28 days per Nebraska Department of Roads specifications requiring a maximum expansion of 0.1% at 28 days. The following figures demonstrate the mixing, casting, curing and subsequent storage and readings throughout the ASTM C 1567 testing.

Figure 5.2.1 Weighed Materials and Work Area

Figure 5.2.2 Casting of Mortar Bars

Figure 5.2.3 Initial Curing of Cast Mortar Bars

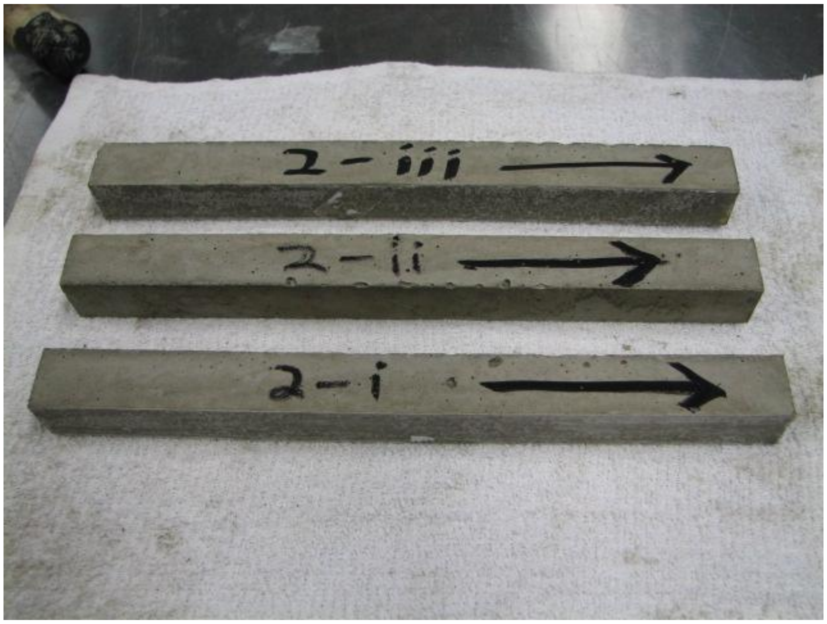


Figure 5.2.4 Hardened Mortar Bars with Mix Identification

Figure 5.2.5 Taking Comparator Reading of Mortar Bar

Figure 5.2.6 Storage Containers for Mortar Bars

Figure 5.2.7 Storage Containers in 80^{o}C Oven

Figure 5.2.8 Mortar Bars following 28 day Testing

5.2 EXPANSION RESULTS

The following are the expansion results from the ASTM C 1567 testing. The expansions were measured at 3, 7, 11, 14, 18, 21, 25 and 28 days. The results for each mix are shown as well as a comparison of the various mixes throughout the 28 day testing period. The actual data sheets containing all readings can be found in Appendix E.

**MIX 1A: 40%CLASS C FLY ASH
30% LIMESTONE**

Table 5.2.1 Summary of ASTM C 1567 Expansions for Mix 1A

Summary of Expansion Results		
Mix #1A CM564-C40_L30	Time, days	Average Change in Length, %
	0	0
	4	0.027
	7	0.051
	11	0.069
	14	0.083
	18	0.095
	21	0.106
	25	0.116
	28	0.125

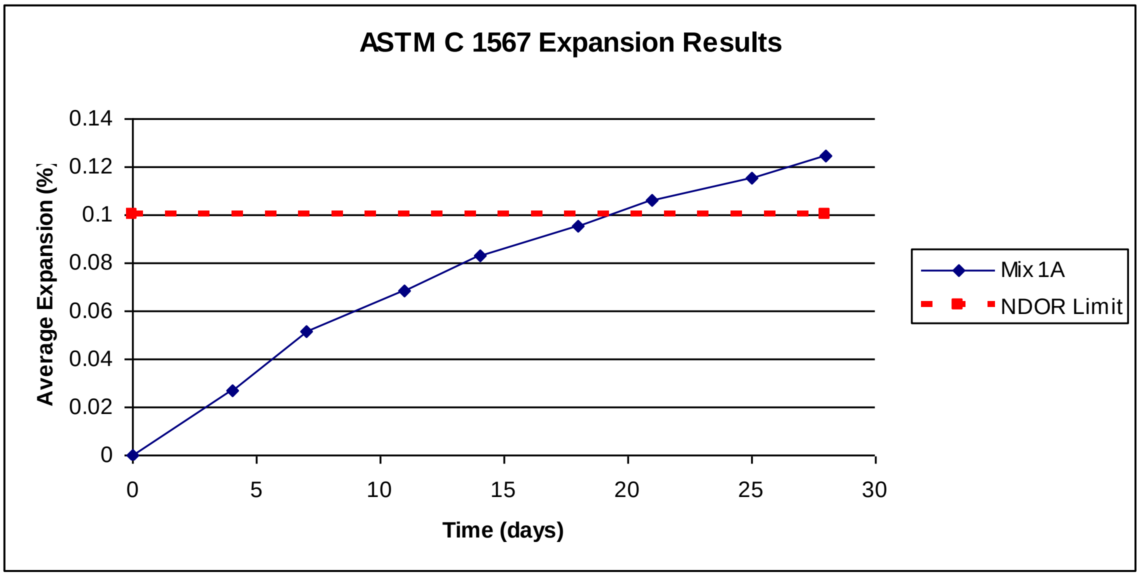


Figure 5.2.1 ASTM C 1567 Expansion Results for Mix 1A

MIX 1B: 40% CLASS C FLY ASH
45% LIMESTONE

Table 5.2.2 Summary of ASTM C 1567 Expansions for Mix 1B

Summary of Expansion Results		
Mix #1B CM564-C40_L45		Average Change in
	Time, days	Length, %
	0	0
	4	0.026
	7	0.036
	11	0.051
	14	0.062
	18	0.074
	21	0.083
	25	0.092
	28	0.101

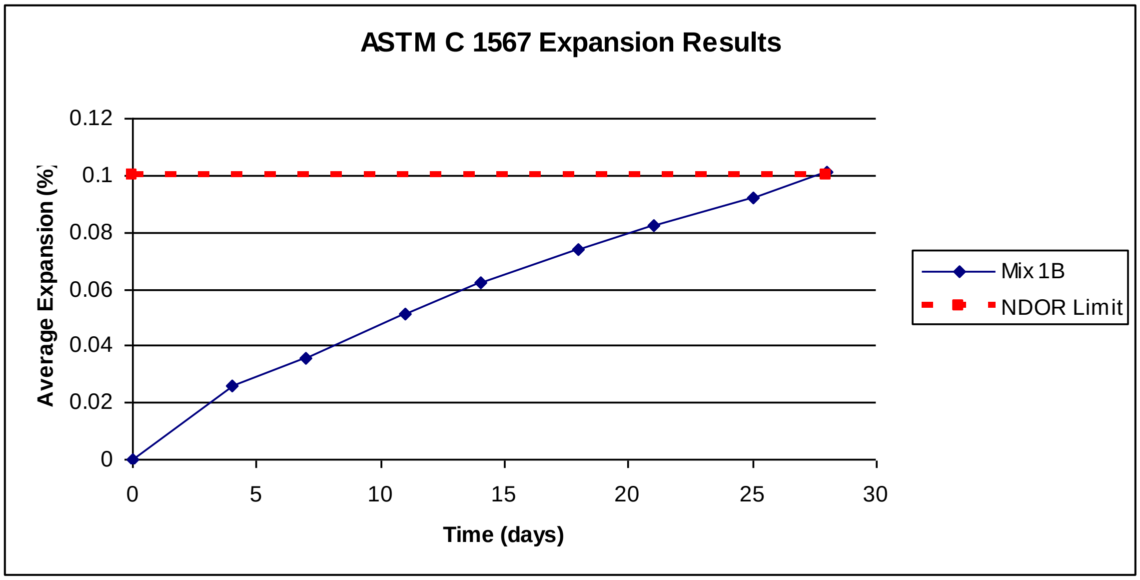


Figure 5.2.2 ASTM C 1567 Expansion Results for Mix 1B

MIX 2: 20% CLASS C FLY ASH
20% CLASS F FLY ASH
30% LIMESTONE

Table 5.2.3 Summary of ASTM C 1567 Expansions for Mix 2

Summary of Expansion Results		
Mix #2 CM564-C20F20_L30		Average Change in
	Time, days	Length, %
	0	0
	4	0.008
	7	0.013
	11	0.021
	14	0.026
	18	0.028
	21	0.035
	25	0.040
	28	0.051

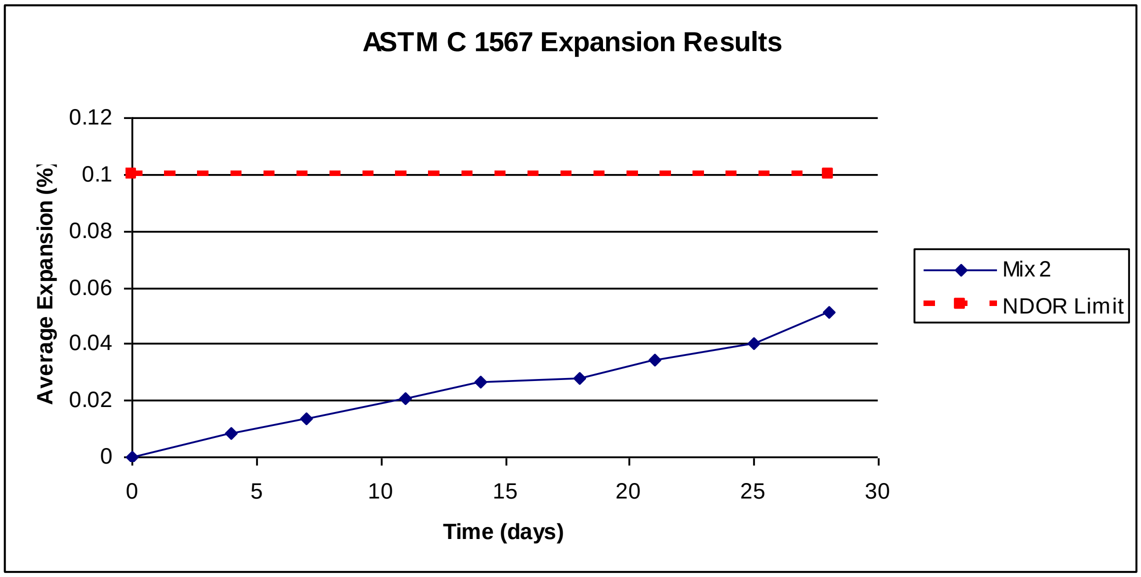


Figure 5.2.3 ASTM C 1567 Expansion Results for Mix 2

MIX 3: 20% CLASS C FLY ASH
30% GGBFS
30% LIMESTONE

Table 5.2.4 Summary of ASTM C 1567 Expansions for Mix 3

Summary of Expansion Results		
Mix #3 CM564-C20S30_L30		Average Change in
	Time, days	Length, %
	0	0
	4	0.018
	7	0.030
	11	0.044
	14	0.054
	18	0.068
	21	0.084
	25	0.101
	28	0.118

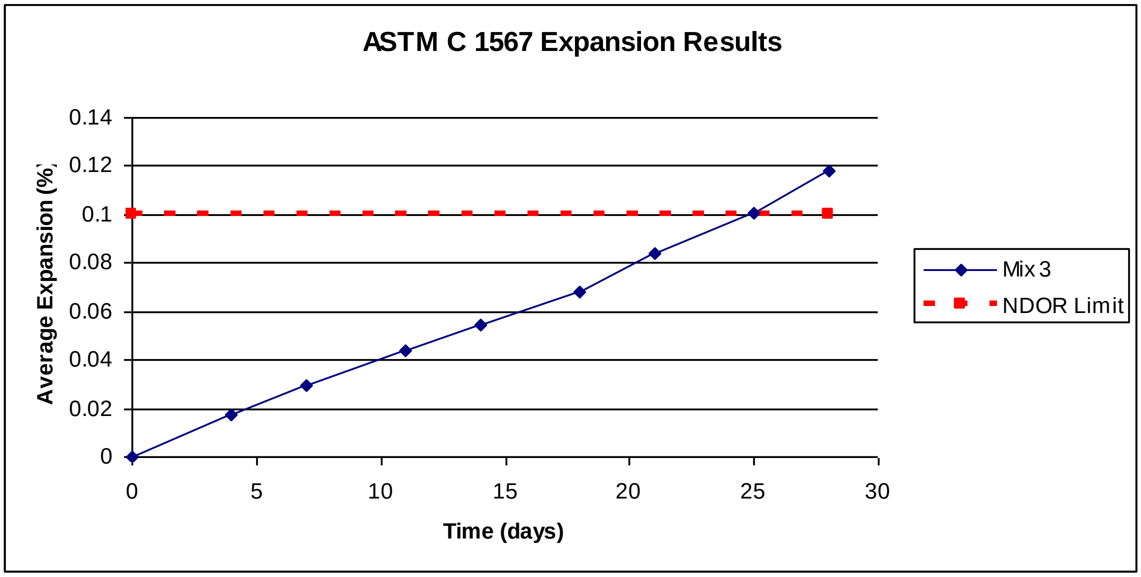


Figure 5.2.4 ASTM C 1567 Expansion Results for Mix 3

MIX 4A: 35% CLASS C FLY ASH
30% LIMESTONE

Table 5.2.5 Summary of ASTM C 1567 Expansions for Mix 4A

Summary of Expansion Results		
Mix #4A CM650-C35_L30		Average Change in
	Time, days	Length, %
	0	0
	4	0.046
	7	0.077
	11	0.101
	14	0.119
	18	0.135
	21	0.150
	25	0.164
	28	0.180

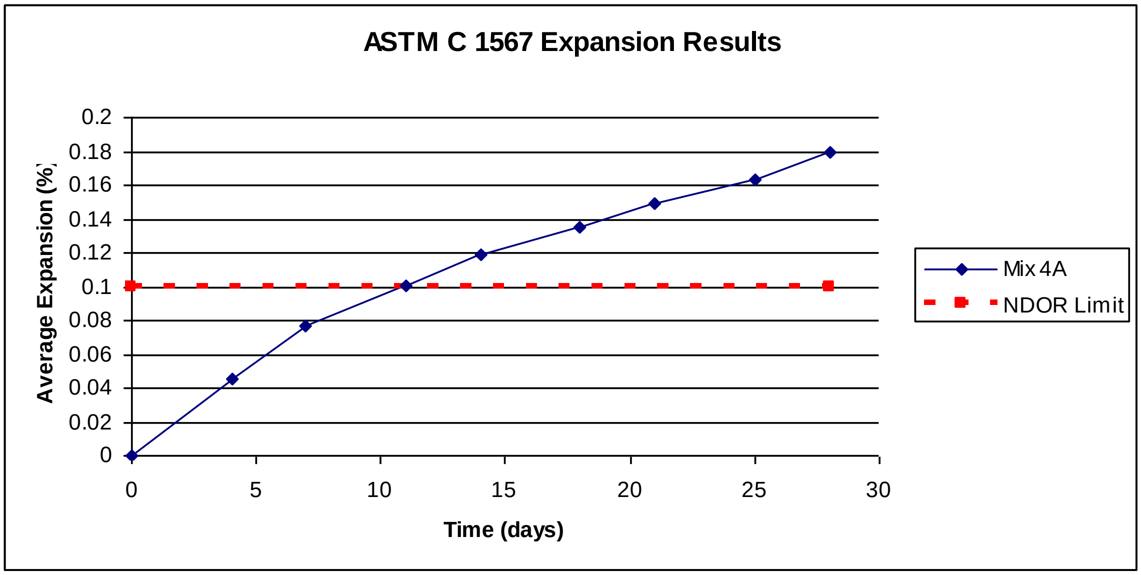


Figure 5.2.5 ASTM C 1567 Expansion Results for Mix 4A

MIX 4B: 35% CLASS C FLY ASH
45% LIMESTONE

Table 5.2.6 Summary of ASTM C 1567 Expansions for Mix 4B

Summary of Expansion Results Mix #4B CM650-C35_L45	Time, days	Average Change in Length, %
	0	0
	4	0.034
	7	0.051
	11	0.061
	14	0.073
	18	0.088
	21	0.097
	25	0.105
	28	0.119

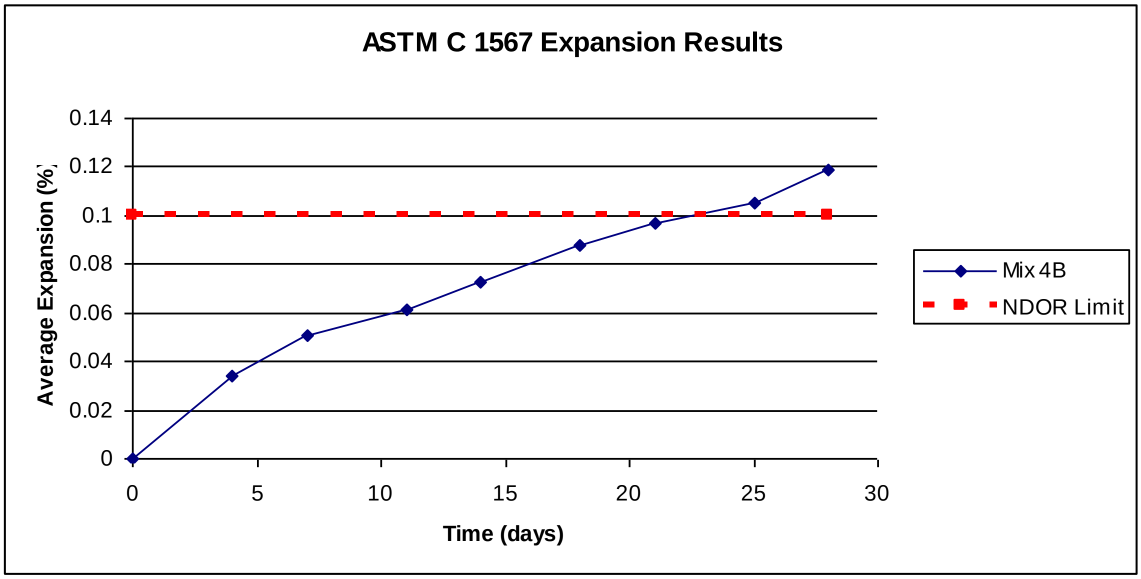


Figure 5.2.6 ASTM C 1567 Expansion Results for Mix 4B

MIX 5: 17% CLASS C FLY ASH
30% LIMESTONE

Table 5.2.7 Summary of ASTM C 1567 Expansions for Mix 5

Summary of Expansion Results		
Mix #5 CM564-C17_L30		Average Change in
	Time, days	Length, %
	0	0
	4	0.057
	7	0.106
	11	0.159
	14	0.189
	18	0.220
	21	0.240
	25	0.262
	28	0.277

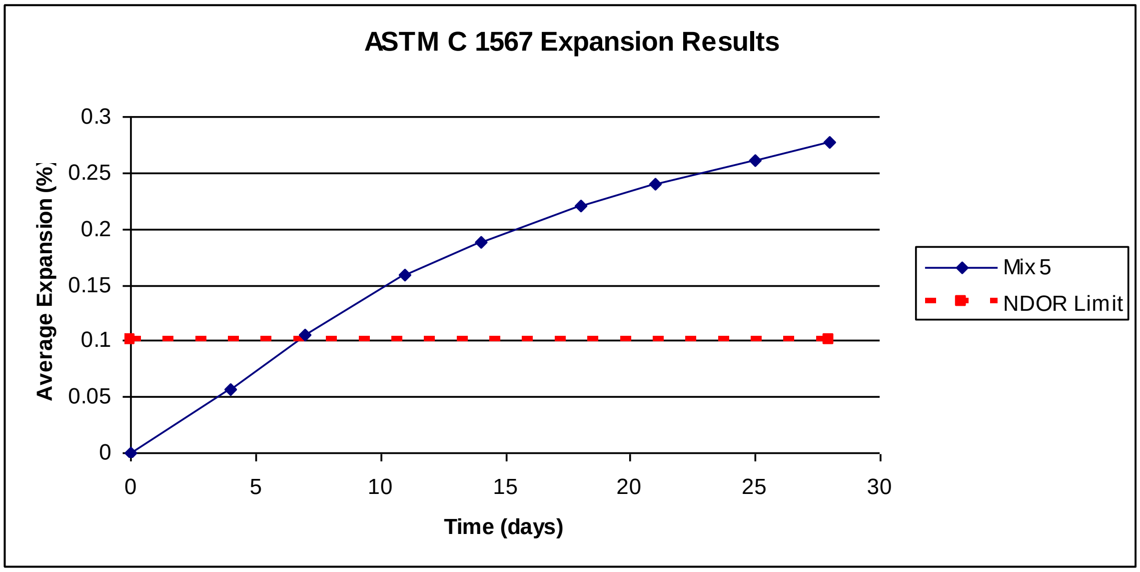


Figure 5.2.7 ASTM C 1567 Expansion Results for Mix 5

MIX 6: 25% CLASS F FLY ASH
30% LIMESTONE

Table 5.2.8 Summary of ASTM C 1567 Expansions for Mix 6

Summary of Expansion Results		
Mix #6 CM564-F25_L30		Average Change in
	Time, days	Length, %
	0	0
	4	0.008
	7	0.011
	11	0.017
	14	0.021
	18	0.027
	21	0.029
	25	0.032
	28	0.038

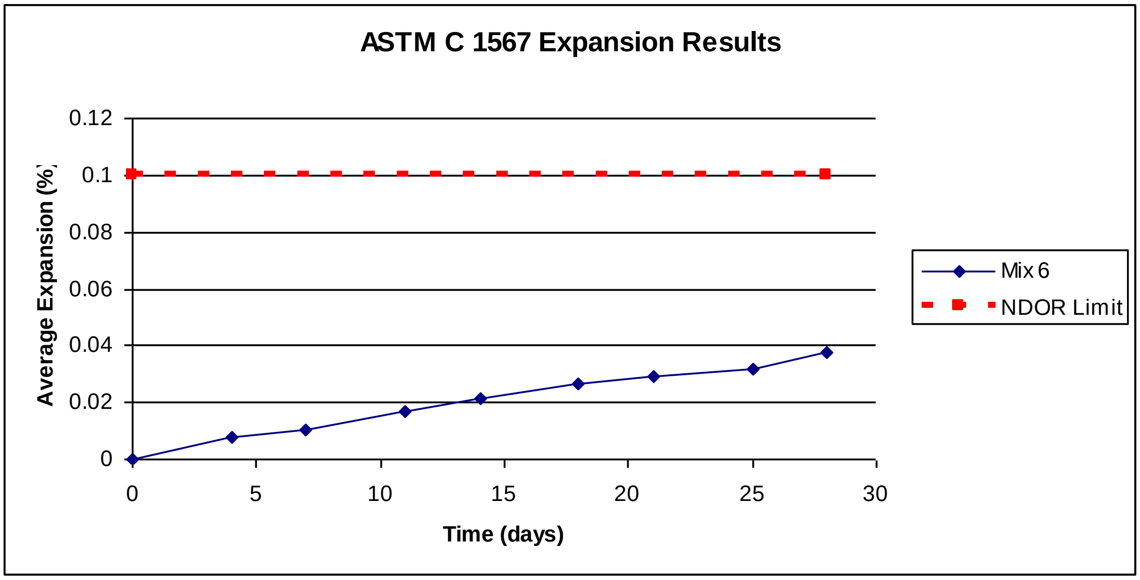


Figure 5.2.8 ASTM C 1567 Expansion Results for Mix 6

The following is a comparison of the expansion results for all mixes. The results are presented in Table 5.2.9.1 as well as in Figure 5.2.9.1 in graphical form. A discussion of these results can be found in the Conclusion and Recommendations.

Table 5.2.9 ASTM C 1567 Expansion Results for All Mixes (%)

Day	Mix															
	1A		***1B***		***2***		***3***		***4A***		***4B***		***5***		***6***	
	UNL	***NDOR***	***UNL***	***NDOR***	***UNL***	***NDOR***	***UNL***	***NDOR***	***UNL***	***NDOR***	***UNL***	***NDOR***	***UNL***	***NDOR***	***UNL***	***NDOR***
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0
4	0.03	0.04	0.03	0.03	0.01	0.01	0.02	0.03	0.05	0.04	0.03	0.03	0.06	0.05	0.01	0.01
7	0.05	0.05	0.04	0.04	0.01	0.01	0.03	0.04	0.08	0.06	0.05	0.05	0.11	0.09	0.01	0.01
11	0.07	0.07	0.05	0.06	0.02	0.02	0.04	0.06	0.10	0.08	0.06	0.06	0.16	0.14	0.02	0.02
14	0.08	0.08	0.06	0.07	0.03	0.02	0.05	0.08	0.12	0.10	0.07	0.08	0.19	0.16	0.02	0.02
18	0.10	0.10	0.07	0.08	0.03	-	0.07	0.10	0.14	0.12	0.09	0.09	0.22	0.19	0.03	0.03
21	0.11	0.11	0.08	0.09	0.03	-	0.08	0.11	0.15	0.14	0.10	0.11	0.24	0.21	0.03	0.04
25	0.12	-	0.09	-	0.04	-	0.10	-	0.16	-	0.11	-	0.26	-	0.03	-
28	0.13	-	0.10	-	0.05	-	0.12	-	0.18	-	0.12	-	0.28	-	0.04	-

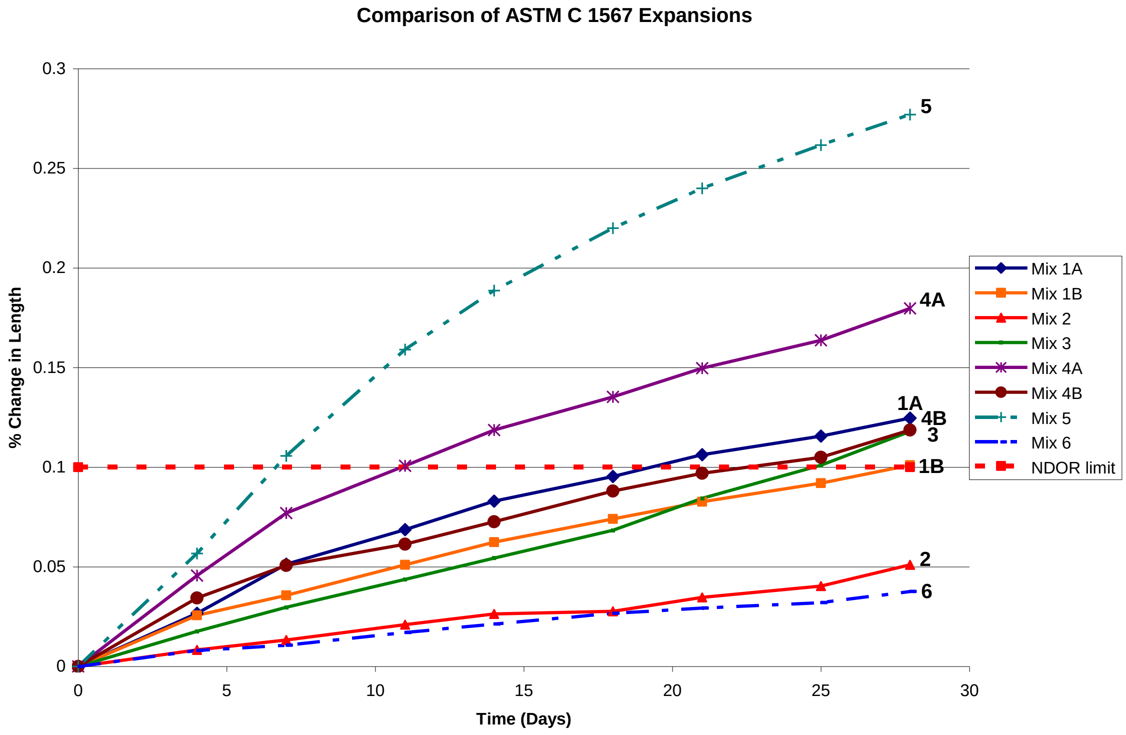


Figure 5.2.9 Comparison of ASTM C 1567 Expansion Results

5.3 ADDITIONAL POTENTIAL MIXES

Based on the results obtained from the ASTM C 1567 testing of 6 proposed mixes, additional mixes were added to mix matrix. The testing of additional mixes will provide a larger number of options when selecting the top mixes for additional performance testing. These mixes are similar to the original proposed mixes but have adjusted proportions. The additional mixes will be used to identify the optimal mix proportions to satisfy the ASTM C 1567 testing as well produce the most cost effective mix possible. The new mixes will once again include both binary and ternary blends but will focus more on ternary mixes containing both Class C and Class F fly ash. This class of mixes was shown to produce exceptional performance with regard to ASR resistance and it was desired to determine the optimum replacement levels for both the Class C and

Class F fly ash. The testing of these mixes will begin the second week of November, 2006 and will conclude in 28 days. Figure 5.4.1 presents the additional proposed mixes that will be tested using ASTM C 1567.

Table 5.3.1 Summary of Additional Proposed Mixes for ASTM C 1567 Testing

Mix	**Total Cementitious Material (lb/yd^3)**	**Class C Fly Ash (%)**	**Class F Fly Ash (%)**	**Slag (%)**	**Limestone (% of Total Agg.)**	**Sand & Gravel (% of Total Agg.**
1C	564	45	0	0	30	70
1D	564	45	0	0	45	55
2B	564	20	15	0	45	55
2C	564	15	15	0	45	55
2D	564	15	21	0	30	70
2E	564	15	21	0	45	55
2F	564	15	18	15	30	70
3B	564	20	0	30	45	55

.

Table 5.4.2 is a detailed mix matrix for all the potential mixes. The additional mixes are shown in yellow. Also included is a cost estimate as of 2005 for Eastern and Western Nebraska. The cost estimate uses the same material costs that were provided in Table 3.3.1.

Table 5.3.2 Detailed Mix Matrix and Cost Estimate for all Potential Mixes

		Cementitious Material					Aggregates		[1]Materials Cost Per yd^3	
	Mix	Type I/II lbs/yd^3	C ash lbs./yd^3	1PF lbs/yd^3	Slag lbs/yd^3	Total lbs/yd^3	LS lbs/yd3	47B S & G lbs/yd^3	Eastern Nebraska	Western Nebreska
1A	40% C 30% Limestone	338	226	0	0	564	882	2074	$26.50	$33.12
1B	40% C 45% Limestone	338	226	0	0	564	1323	1630	$27.61	$37.53
1C	**45% C** **30% Limestone**	**310**	**254**	**0**	**0**	**564**	**881**	**2072**	**$25.44**	**$32.05**
1D	**45% C** **45% Limestone**	**310**	**254**	**0**	**0**	**564**	**1322**	**1628**	**$26.54**	**$36.45**
2	20% C, 20% F 30% Limestone	0	113	451	0	564	880	2068	$30.71	$37.31
2B	**20% C, 15% F** **45% Limestone**	**113**	**113**	**338**	**0**	**564**	**1322**	**1628**	**$31.82**	**$41.74**
2C	**15% C, 15% F** **45% Limestone**	**141**	**85**	**338**	**0**	**564**	**1324**	**1630**	**$32.89**	**$42.82**
2D	**15% C, 21% F** **30% Limestone**	**0**	**85**	**479**	**0**	**564**	**880**	**2070**	**$31.77**	**$38.37**
2E	**15% C, 21% F** **45% Limestone**	**0**	**85**	**479**	**0**	**564**	**1320**	**1626**	**$32.86**	**$42.76**
2F	**15%C, 18%F, 15%S** **30% Limestone**	**0**	**85**	**395**	**85**	**564**	**880**	**2069**	**$31.81**	**$38.41**
3	20% C, 30%S 30% Limestone	282	113	0	169	564	883	2077	$30.76	$37.39
3B	**20% C, 30% S** **45% Limestone**	**282**	**113**	**0**	**169**	**564**	**1325**	**1632**	**$31.85**	**$41.79**
4A	35% C 30% Limestone	423	228	0	0	650	830	1952	$29.75	$35.98
4B	35% C 45% Limestone	423	228	0	0	650	1245	1534	$30.78	$40.12
5	17%C 30% Limestone	468	96	0	0	564	887	2086	$31.44	$38.09
6	25% F 30% Limestone	0	0	564	0	564	882	2074	$34.98	$41.59

5.4 EXPANSION RESULTS FOR ADDITIONAL MIXES

MIX 1C: 45% CLASS C FLY ASH
30% LIMESTONE

Table 5.4.1 Summary of ASTM C 1567 Expansions for Mix 1C

Summary of Expansion Results		
Mix #1C CM564-C45_L30		Average Change in
	Time, days	Length, %
	0	0
	4	0.027
	7	0.046
	11	0.055
	14	0.066
	18	0.076
	21	0.087
	25	0.095
	28	0.100

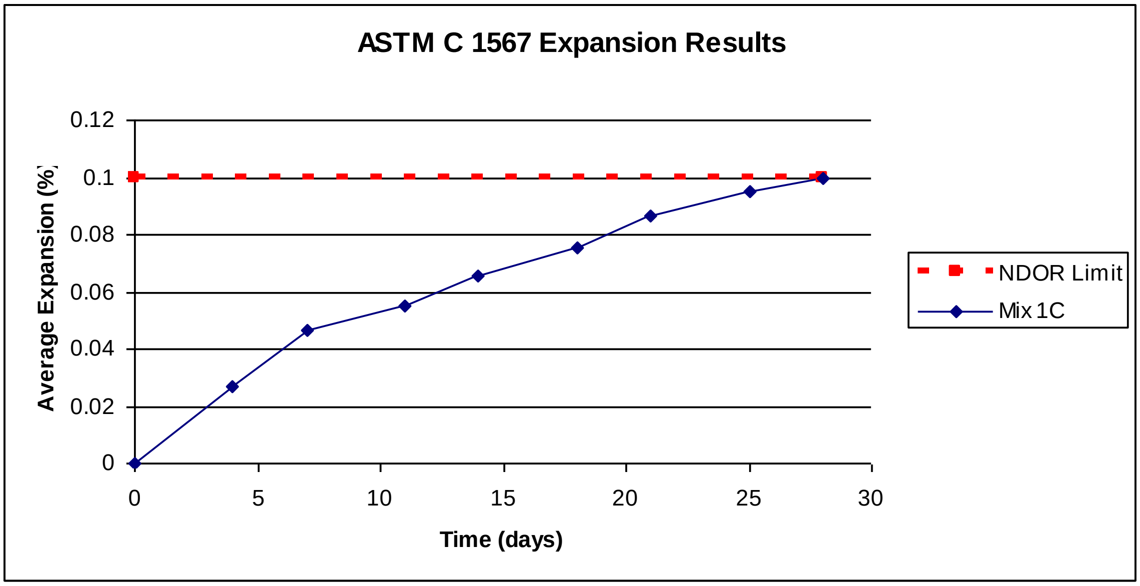


Figure 5.4.1 ASTM C 1567 Expansion Results for Mix 1C

MIX 1D: 45% CLASS C FLY ASH
45% LIMESTONE

Table 5.4.2 Summary of ASTM C 1567 Expansions for Mix 1D

Summary of Expansion Results		
Mix #1D CM564-C45_L45		Average Change in
	Time, days	Length, %
	0	0
	4	0.026
	7	0.042
	11	0.053
	14	0.063
	18	0.074
	21	0.085
	25	0.094
	28	0.101

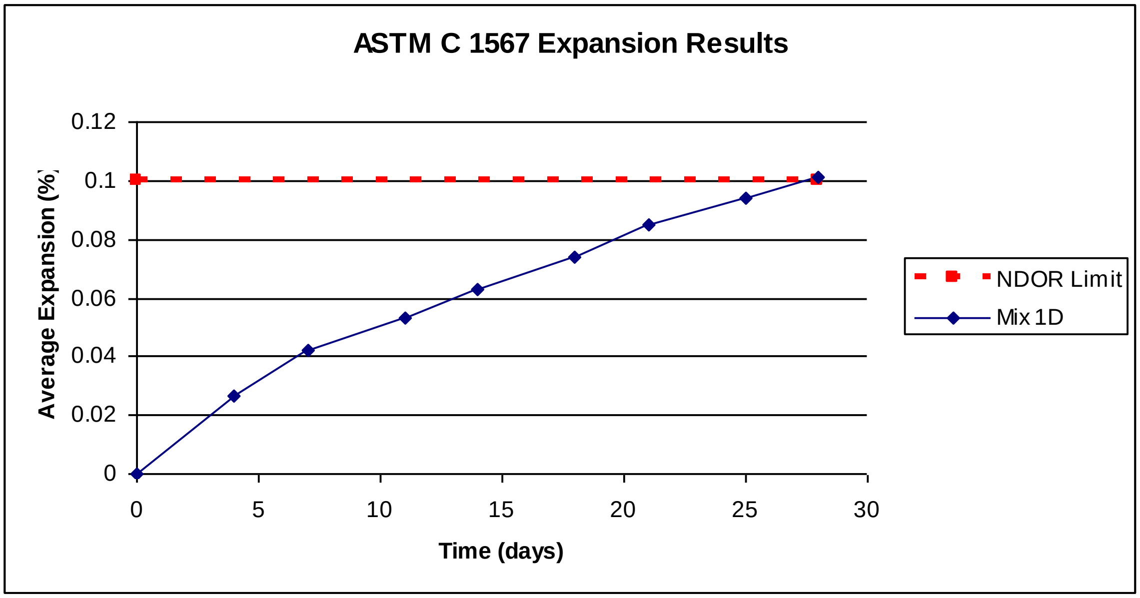


Figure 5.4.2 ASTM C 1567 Expansion Results for Mix 1D

MIX 2B: 20% CLASS C FLY ASH
15% CLASS F FLY ASH
45% LIMESTONE

Table 5.4.3 Summary of ASTM C 1567 Expansions for Mix 2B

Summary of Expansion Results		
Mix #2B		Average
CM564-C20F15_L45		Change in
	Time, days	Length, %
	0	0
	4	0.014
	7	0.023
	11	0.028
	14	0.039
	18	0.048
	21	0.053
	25	0.065
	28	0.073

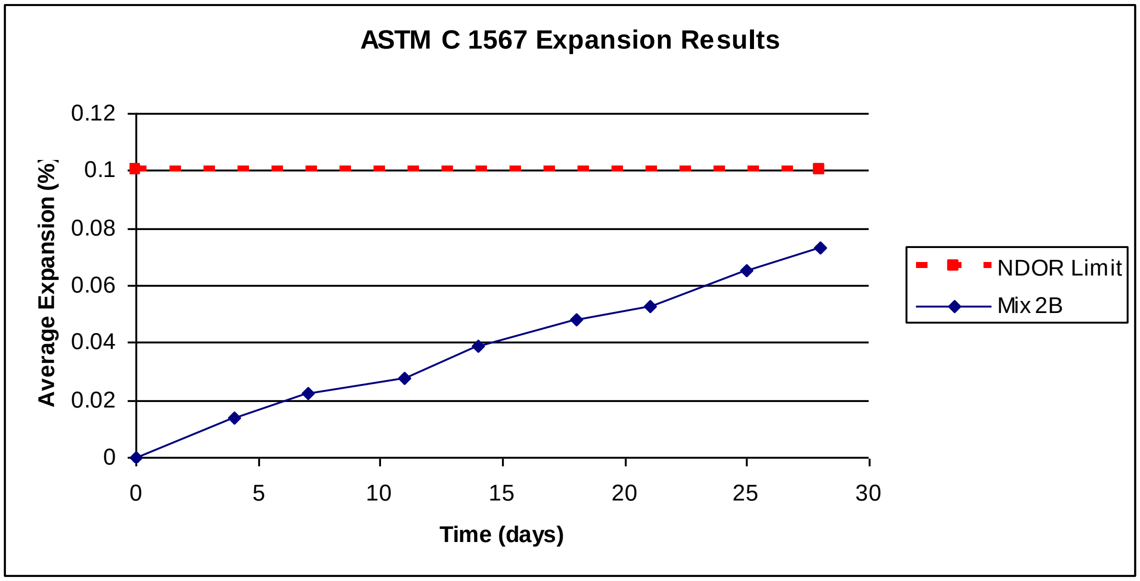


Figure 5.4.3 ASTM C 1567 Expansion Results for Mix 2B

MIX 2C: 15% CLASS C FLY ASH
15% CLASS F FLY ASH
45% LIMESTONE

Table 5.4.4 Summary of ASTM C 1567 Expansions for Mix 2C

Summary of Expansion Results		
Mix #2C CM564-C15F15_L45		Average Change in
	Time, days	Length, %
	0	0
	4	0.016
	7	0.024
	11	0.027
	14	0.036
	18	0.048
	21	0.060
	25	0.075
	28	0.086

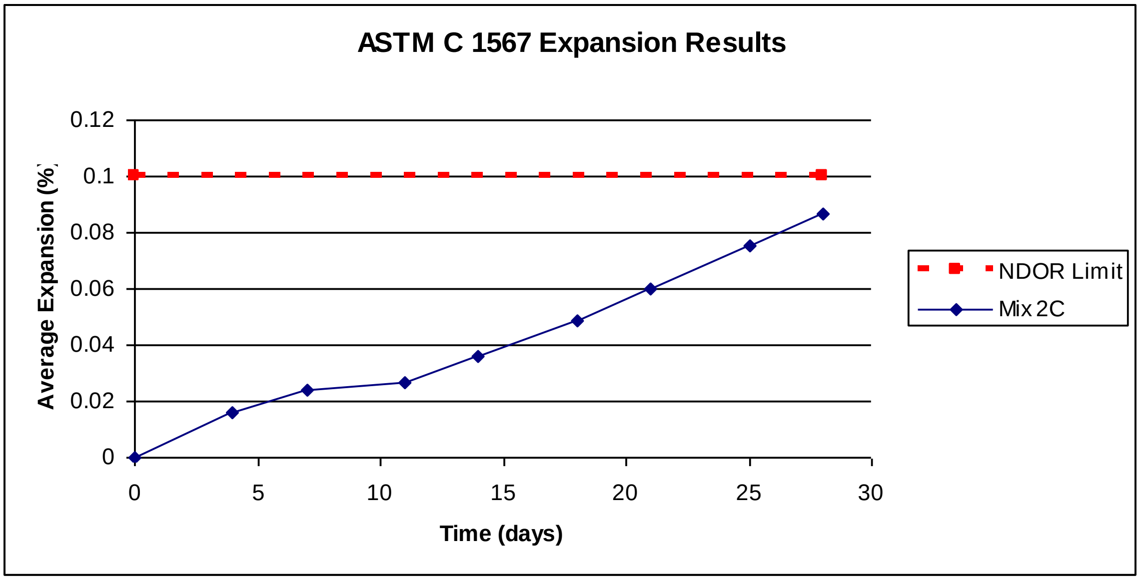


Figure 5.4.4 ASTM C 1567 Expansion Results for Mix 2C

MIX 2D: 15% CLASS C FLY ASH
21% CLASS F FLY ASH
30% LIMESTONE

Table 5.4.5 Summary of ASTM C 1567 Expansions for Mix 2D

Summary of Expansion Results		
Mix #2D CM564-C15F21_L30		Average Change in
	Time, days	Length, %
	0	0
	4	0.011
	7	0.015
	11	0.023
	14	0.028
	18	0.028
	21	0.041
	25	0.044
	28	0.045

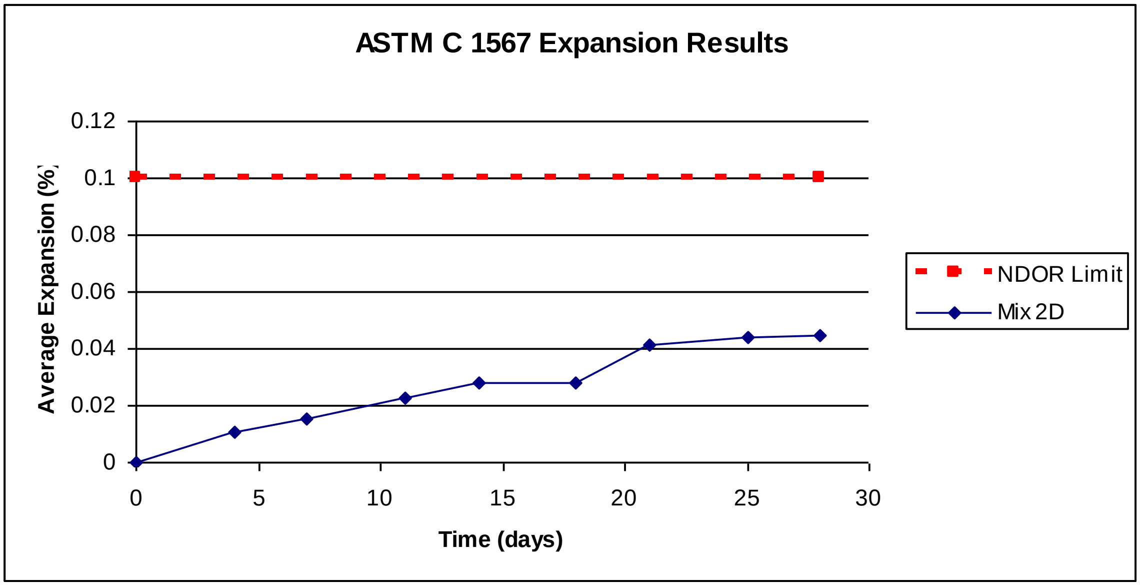


Figure 5.4.5 ASTM C 1567 Expansion Results for Mix 2D

MIX 2E: 15% CLASS C FLY ASH
21% CLASS F FLY ASH
45% LIMESTONE

Table 5.4.6 Summary of ASTM C 1567 Expansions for Mix 2E

Summary of Expansion Results		
Mix #2E CM564-C15F21_L45		Average Change in
	Time, days	Length, %
	0	0
	4	0.008
	7	0.013
	11	0.018
	14	0.019
	18	0.028
	21	0.034
	25	0.040
	28	0.043

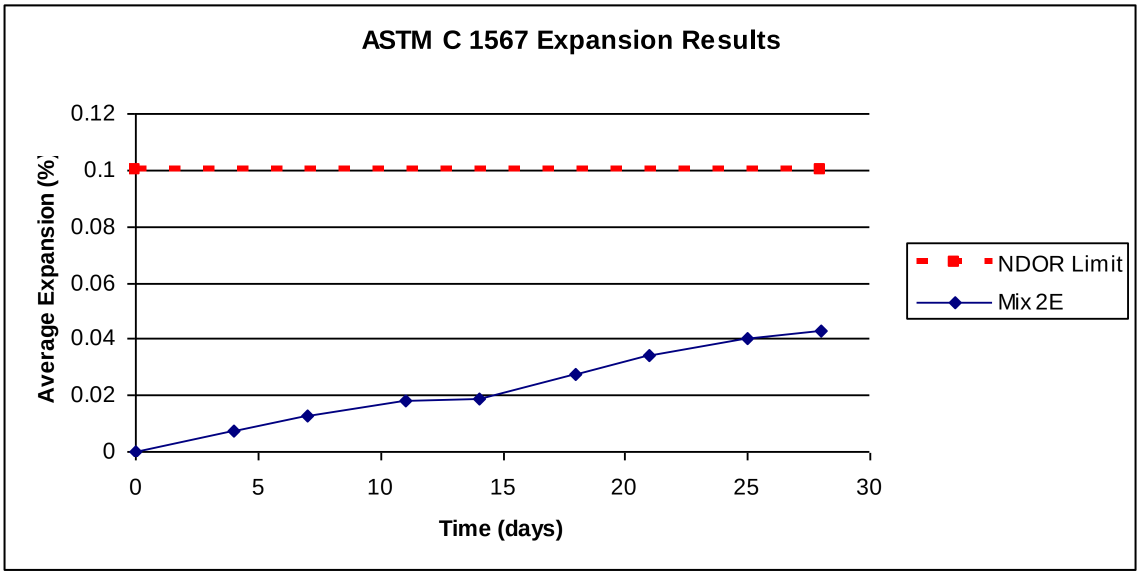


Figure 5.4.6 ASTM C 1567 Expansion Results for Mix 2E

MIX 2F: 15% CLASS C FLY ASH
18% CLASS F FLY ASH
15% GGBFS
30% LIMESTONE

Table 5.4.7 Summary of ASTM C 1567 Expansions for Mix 2F

Summary of Expansion Results		
Mix #2F CM564-C15F18S15_L45		Average Change in
	Time, days	Length, %
	0	0
	4	0.007
	7	0.013
	11	0.015
	14	0.021
	18	0.028
	21	0.030
	25	0.027
	28	0.041

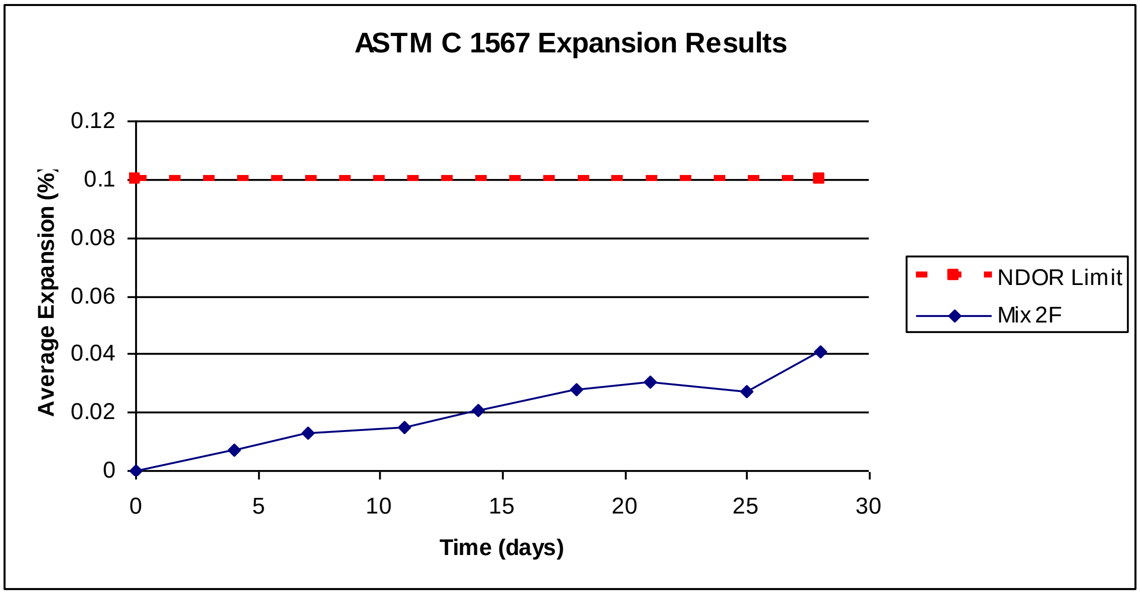


Figure 5.4.7 ASTM C 1567 Expansion Results for Mix 2F

MIX 3B: **20% CLASS C FLY ASH**
30% GGBFS
45% LIMESTONE

Table 5.4.8 Summary of ASTM C 1567 Expansions for Mix 3B

Mix #3B CM564-C20S30_L45	Time, days	Average Change in Length, %
	0	0.000
	4	0.018
	7	0.027
	11	0.040
	14	0.053
	18	0.070
	21	0.087
	25	0.095
	28	0.118

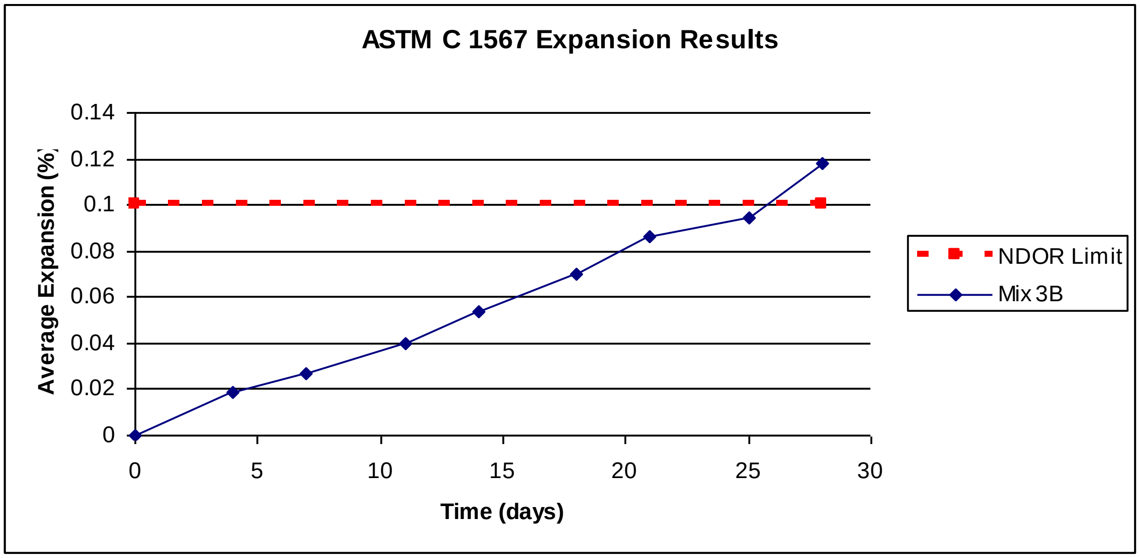


Figure 5.4.8 ASTM C 1567 Expansion Results for Mix 3B

Table 5.4.9 Summary of ASTM C 1567 Expansion Results for New Mixes

Average % Change in Length								
	Mix							
Day	***1C***	***1D***	***2B***	***2C***	***2D***	***2E***	***2F***	***3B***
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	0.03	0.03	0.01	0.02	0.01	0.01	0.01	0.02
7	0.05	0.04	0.02	0.02	0.02	0.01	0.01	0.03
11	0.06	0.05	0.03	0.03	0.02	0.02	0.01	0.04
14	0.07	0.06	0.04	0.04	0.03	0.02	0.02	0.05
18	0.08	0.07	0.05	0.05	0.03	0.03	0.03	0.07
21	0.09	0.08	0.05	0.06	0.04	0.03	0.03	0.09
25	0.10	0.09	0.07	0.08	0.04	0.04	0.03	0.09
28	0.10	0.10	0.07	0.09	0.04	0.04	0.04	0.12

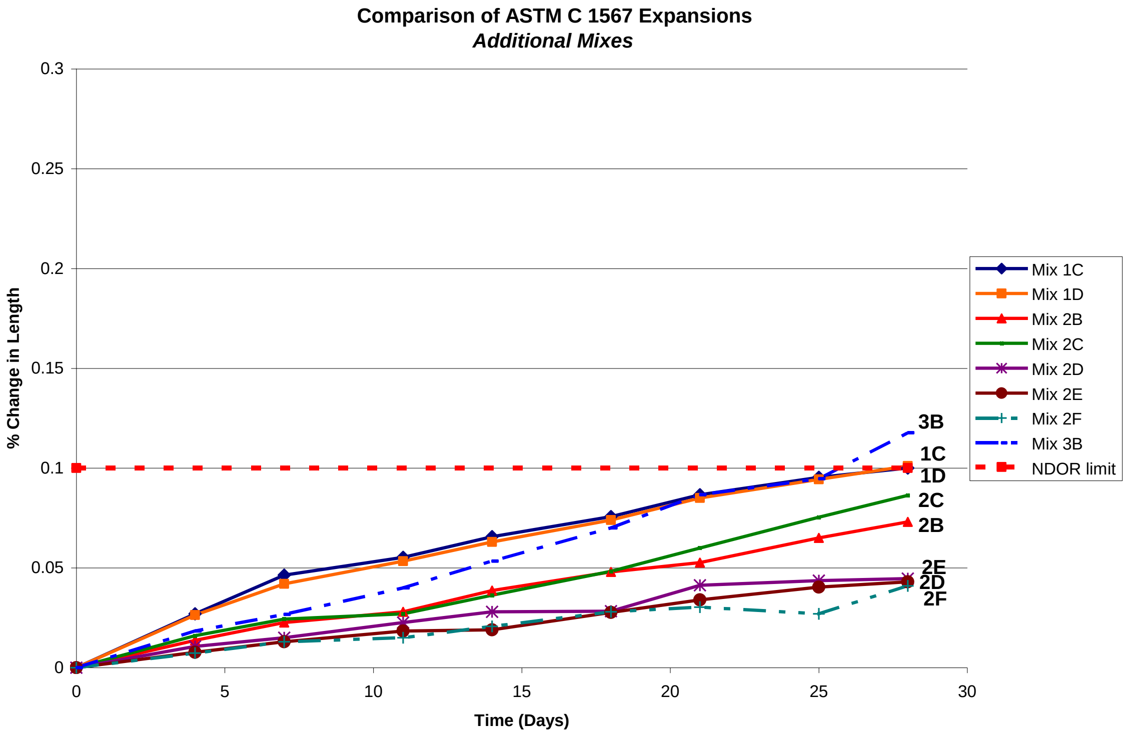


Figure 5.4.9 Comparison of ASTM C 1567 Expansion Results for New Mixes

5.5 ANALYSIS OF ASTM C 1567 RESULTS

Table 5.5.1 presents a summary of the 28 –day expansions for all mixes tested using ASTM C 1567. Within the table, the mixes highlighted in green represent those mixes that passed ASTM C 1567. The mixes highlighted in yellow are those whose expansions were very close to the limit of 0.1% at 28 days. All mixes shown in red failed the testing.

Table 5.5.1 Pass/Fail Results for ASTM C 1567

Mix No.	Description	ASTM C 1567 28-day Expansion %
1A	40%C__30%LS	**0.12**
1B	40%C__45%LS	**0.10**
1C	45% C__30%LS	**0.10**
1D	45% C__45%LS	**0.10**
2	20%C - 20%F__30%LS	**0.05**
2B	20%C - 15%F__45%LS	**0.07**
2C	15%C - 15%F__45%LS	**0.09**
2D	15%C - 21%F__30%LS	**0.04**
2E	15%C - 21%F__45%LS	**0.04**
2F	15%C-18%F-15%S__30%LS	**0.04**
3	20%C - 30%S__30%LS	**0.12**
3B	20%C - 30%S__45%LS	**0.12**
4A	35%C__30% LS	**0.18**
4B	35%C__45%LS	**0.12**
5	Ref. Mix - 17% C__30%LS	**0.28**
6	Ref. Mix - 25% F__30%LS	**0.04**

The results of the ASTM C 1567 were as expected. The relative performance of each mix was identical to what would be predicted. This comparison of relative performances appears to validate the testing results. Although the relative performances were very much in line with what would be expected, the overall ASR resistance of the potential mixes was lower than anticipated. Out of the eight mixes tested, only 2 mixes successfully passed the ASTM C 1567 limit set by NDOR of 0.1%. These include mixes 2 and 6.

The use of Class C fly alone, as in mixes 1A, 1B, 4A and 4B, would have difficulties satisfying the ASTM C 1567 testing and therefore may not have adequate ASR resistance in the field. Up to 40% replacement levels along with the use of 45% limestone still produce 28-day expansions equal to 0.1%. This is mainly due to the extremely high CaO content found in the Class C fly ash produced in Nebraska. Such fly ash will behave more similarly to hydraulic cement than a pozzolan. The hydration that occurs creates a weak and porous matrix that allows for easy mobility of the alkalies. The pozzolanic reactions that do occur produce a C-S-H structure with high calcium to silicon ratio. This structure instead of absorbing cations, particularly alkalies, may actually release alkalies into the system. Therefore, much higher replacement levels will be needed control ASR, possibly in the order of 45 to 50 percent which will be investigated in further testing. Although this amount of replacement may control ASR, it will have difficulties satisfying other performance criteria such as flexural strength, set time and permeability. Therefore additional measures may be necessary such as additional cement content.

As was expected, the mixes containing Class F fly ash showed exceptional performance with regards to ASR resistance. Both mixes using Class F fly ash alone as well as the ternary mix using both Class C and Class F showed similar performance. Unlike the Class C fly ash, Class F fly ash behaves more closely to a true pozzolan. Its reactions are almost exclusively pozzolanic with limited cementing value of it own. This produces a very dense and impermeable matrix by converting the Calcium Hydroxide produced during cement hydration into additional C-S-H, reducing the mobility of the alkalies. In addition the low lime content creates a C-S-H structure with a very low calcium to silicon ratio which enables the absorption of alkalies. Both mixes tested to date used a minimum of 20% replacement of Class F fly ash. Further mixes will identify what is the minimum amount of Class F fly ash along with other SCM’s needed to control ASR. By adjusting the SCM quantities to maximize the amount of class C fly ash and minimize the amount of Class F fly, a very cost effective mix with extremely high performance is likely.

The mix containing GGBFS and Class C fly showed improved performance over using Class C fly alone, however, acceptable performance was still not achieved. The increased ASR resistance is mainly attributed to hydration of slag which does not produce the weak and porous Calcium Hydroxide phase. This reduces the mobility of the alkalies. To produce acceptable ASR resistance using slag, a higher replacement level may be needed. However, such a mix would mostly likely fail other performance tests. Therefore, further testing will investigate the use of 45% limestone used along with same replacement levels of 20% Class C fly ash and 30% slag.

As was expected, all mixes using 45% limestone and 55% sand and gravel showed superior performance over the companion mixes using the traditional 47B gradation of 30% limestone and 70% sand and gravel. These results are very predictable since a portion of the reactive sand and gravel is being replaced with the non-reactive limestone. This is a well established method of reducing ASR expansions and is known as “limestone sweetening”.

Although the exact progression of deterioration mechanism being seen previously in Nebraska pavements in unknown, the results from the ASTM C 1567 testing provides excellent evidence that the deterioration was largely due to Alkali-Silica reaction. The current standard mix using 1PF cement (25% Class F fly ash) showed the lowest expansion results of 0.04%. The previous standard mix, which was used in the pavements experiencing premature deterioration, had expansions nearly double that of any other mix. This mix had expansions exceeding the limit of 0.1% after only 7 days of testing.

5.6 SELECTION OF TOP MIXES

Following the conclusion of all ASTM C 1567 testing, a meeting was held at NDOR. A presentation was given by the UNL team to the project sponsors. A summary of the relevant Phase I finding as well as the results from the ASTM C 1567 testing were presented. A discussion was held to determine which of the mixes passing ASTM C 1567 would be the most attractive regarding cost as of 2006, performance and availability. The following 4 mixes plus the reference mix was selected by the group.

		Cementitious Material					Aggregates		[1]Materials Cost Per yd^3	
	Mix	*Type I/II* lbs/yd^3	*C ash* lbs./yd^3	*1PF* lbs/yd^3	*Slag* lbs/yd^3	*Total* lbs/yd^3	*LS* lbs/yd3	*47B S & G* lbs/yd^3	*Eastern Nebraska*	Western Nebreska
2	20% C, 20% F 30% Limestone	0	113	451	0	564	880	2068	$30.71	$37.31
2D	15% C, 21% F 30% Limestone	0	85	479	0	564	880	2070	$31.77	$38.37
2E	15% C, 21% F 45% Limestone	0	85	479	0	564	1320	1626	$32.86	$42.76
2F	15%C, 18%F, 15%S 30% Limestone	0	85	395	85	564	880	2069	$31.81	$38.41
6	25% F 30% Limestone	0	0	564	0	564	882	2074	$34.98	$41.59

The selected mixes were tested for ASR with 100 % Platte River Sand & Gravel according to the standard specification. Table 5.5.2 shows the results and shows that all the results meet NDOR requirements.

Table 5.5.2 ASR results using 100% Platte River Sand and Gravel

Mix No.	**Description**	**ASTM C 1567 28-day Expansion %**
2	20%C - 20%F	**0.04**
2D	15%C - 21%F	**0.04**
2F	15%C-18%F-15%S	**0.03**

6.0 OVERALL PERFORMANCE TESTING

Based on the results of ASTM C1567 testing, four mixes were selected, in addition to the reference mix, for overall performance testing. Table 6.1.1 lists the ten tests performed on plastic and hardened concrete to evaluate the overall performance. Two different w/cm ratios (0.4 and 0.45) were used for making the specimens for overall performance testing. Also, the two w/cm ratios correspond to the concrete pavement construction using two different methods: slipform paving (w/cm = 0.4), and conventional paving (w/cm = 0.45). Below is a summary of these tests and their results.

Table 6.1 List of Test Performed

Category	**ASTM #**	**Test Description**
Plastic Concrete Tests	C143	Standard Test Method for Slump of Hydraulic-Cement Concrete
	C138	Standard Test Method of Density (Unit Weight)
	C231	Standard Test Method for Air Content of Freshly Mixed Concrete by Pressure Method
	C403	Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance
Hardened Concrete Tests	C39	Standard Test Method for Compressive Strength of Cylindrical concrete Specimens
	C78	Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
	C666	Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing
	C157	Standard Test Method for Length Change of Hardened Hydraulic-Cement, Mortar and Concrete
	C1202	Standard Test Method for Electrical Indication of Concrete’s Ability to Resist Chloride Ion Penetration
	NDOR	NDOR Wet and Dry Test

6.1 Plastic Concrete Tests

6.1.1 Slump test

The slump testing was conducted in accordance to ASTM C143 Standard Test Method for Slump of Hydraulic-Cement Concrete as shown in Figure 6.1.1.1 . NDOR specifies a slump in the range of 1.5” - 2.0” for slip form paving and allows up to 4.5” for conventional construction. Tables 6.1.1 and 6.1.2 show that all the mixes satisfy these requirements in the two cases, respectively.

Figure 6.1.1.1 Slump test

6.1.2 Unit Weight

The unit weight testing was conducted in accordance to ASTM C138 Standard Test Method of Density as shown in Figure 6.1.1.2. NDOR does not have special requirements regarding the unit weight. Tables 6.1.1 and 6.1.2 indicate that all mixes have a unit weight in the range of normal weight concrete, which is from 140 lb/ft3 to 150 lb/ft3 .

Figure 6.1.2.1 Unit weight

6.1.3 Air Content

The air content testing was conducted in accordance to ASTM C231 Standard Test Method for of Freshly Mixed Concrete by Pressure Method as shown in figure 6.1.3.1. The air content of each mix was measured and reported to the nearest 0.25 %. NDOR specifies air content from 7.5% to 10% before the paver. Tables 6.1.1 and 6.1.2 show the air results using w/cm = 0.45 and 0.4 respectively.

Figure 6.1.3.1 Air Content

Table 6.1.1 Plastic Concrete Testing Results (w/c = 0.45)

Mix	Air (%)	Slump (in.)	Unit weight (lb/ft^3)	W/c
2 (20C-20F-30LS)	7	4.5	144	0.45
2D (15C-21F-30LS)	7.5	3.5	140	0.45
2E (15C-21F-45LS)	7.5	3.25	150	0.45
2F(15C-18F-15S-30LS)	7.75	3	143	0.45
6(25F-30LS)	**7.75**	**2.75**	**142**	**0.45**

Table 6.1.2 Plastic Concrete Testing Results (w/c 0.4 to 0.42)

Mix	Air (%)	Slump (in.)	Unit weight (lb/ft^3)	W/c
2 (20C-20F-30LS)	7.75	1.75	144	0.4
2D (15C-21F-30LS)	7.75	1.75	144	0.41
2E (15C-21F-45LS)	7.75	1.75	146	0.42
2F(15C-18F-15S-30LS)	7.5	1.75	144	0.4
6(25F-30LS)	**8**	**2**	**144**	**0.42**

6.1.4 Time of Setting

Time of setting testing was conducted in accordance to ASTM C403 Standard Test Method for Time Setting of Concrete Mixtures by Penetration Resistance as shown in Figure 6.1.4.1. Although NDOR does not specify limits for the time of setting, this test was performed for information and comparison purposes only. According to ASTM C403, stress requirements for initial set is 500 psi and for final set is 4000 psi. Figures 6.1.4.2 and 6.1.4.3 show the setting time versus stress for all the mixes using w/cm = 0.45 and 0.4 respectively based on average of three specimens. These figures confirm that effect of fly ash as a retarder that increases the initial and final setting times. The reference mix that has the lowest cement replacement with fly ash has the shortest setting time.

Figure 6.1.4.1 Set time test

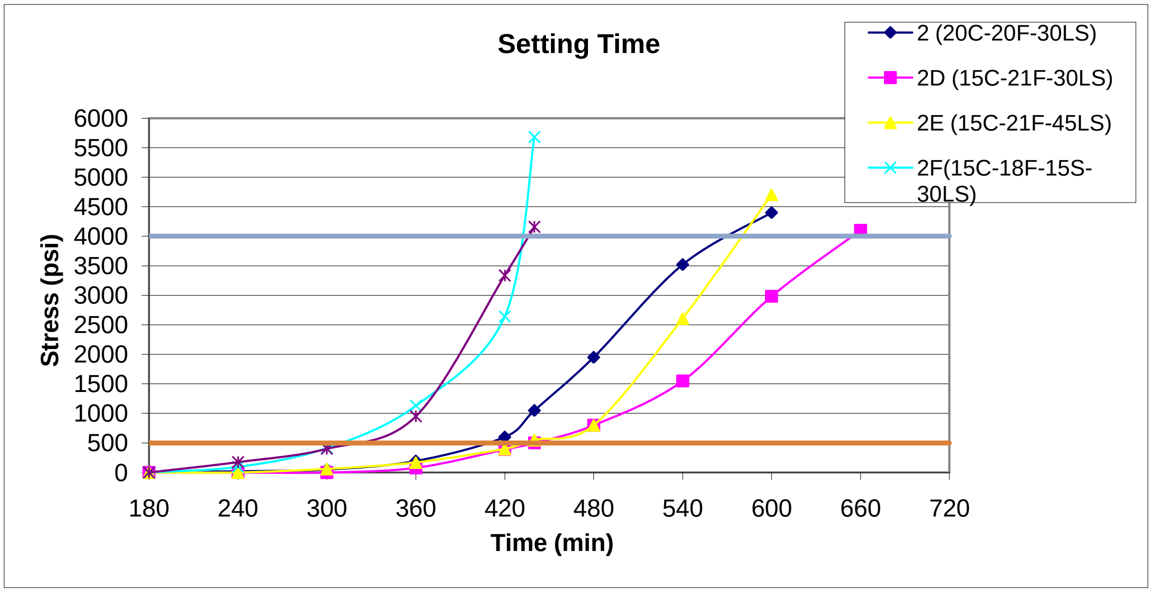


Figure 6.1.4.2 Setting Time (w/cm = 0.45)

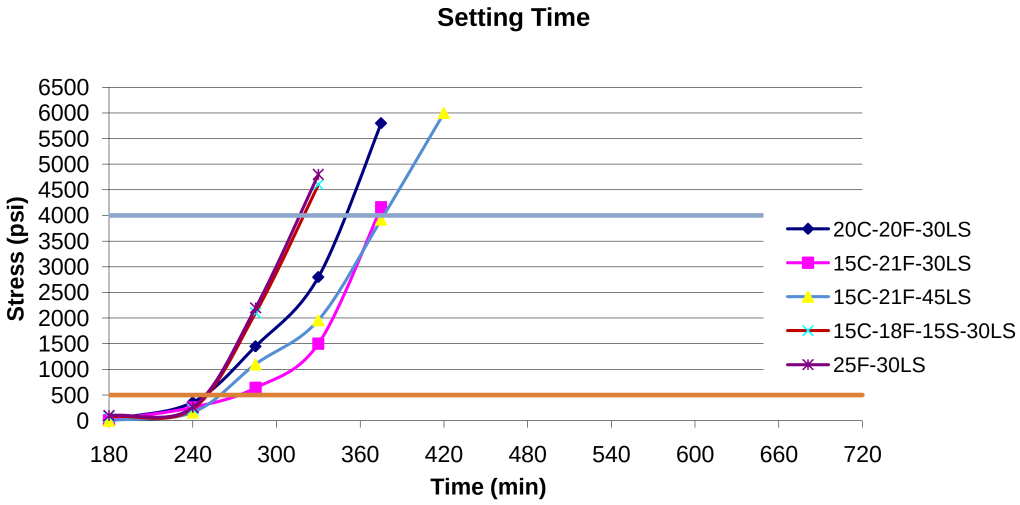


Figure 6.1.4.3 Setting Time (w/cm = 0.40 to 0.42)

6.2 Hardened Concrete Strength Tests

6.2.1 Compressive Strength

Compressive strength testing was conducted in accordance to ASTM C39 Standard Test Method for Compressive Strength of Cylindrical concrete Specimens as shown in figure 6.2.1.1. The compressive strength tests were conducted at 3, 7, 14 and 28 days using three 4”x8” cylinders. NDOR specifies a minimum compressive strength of 3,500 psi at 14 days for PCCP projects. Tables 6.2.1.1 and 6.2.1.2 lists test results for the five mixes with w/cm = 0.45 and 0.4 respectively. These results indicate that the chosen mixes outperform the current NDOR standard mix (Mix 6) in terms of compressive strength.

Figure 6.2.1.1 Compressive Strength Testing

Table 6.2.1.1 Compressive Strength (psi) (w/c 0.45)

Mix	**3 day**	**7 day**	**14 day**	**28 day**
2 (20C-20F-30LS)	2642	3474	4227	4728
2D (15C-21F-30LS)	2724	3642	4388	4973
2E (15C-21F-45LS)	3449	4054	4914	5816
2F(15C-18F-15S-30LS)	3134	4027	4944	5875
6(25F-30LS)	**1945**	**3069**	**3851**	**4377**

Table 6.2.1.2 Compressive Strength (psi) (w/c 0.4 to 0.42)

Mix	**3 day**	**7 day**	**14 day**	**28 day**
2 (20C-20F-30LS)	3259	3858	5186	6036
2D (15C-21F-30LS)	2773	3469	4437	5087
2E (15C-21F-45LS)	3245	3410	4739	5438
2F(15C-18F-15S-30LS)	3360	4082	4998	5693
6(25F-30LS)	**2693**	**3375**	**4362**	**4917**

6.2.2 Flexural Strength

Flexural strength testing was conducted in accordance to ASTM C 78 Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading) as shown in Figure 6.2.2.1. The testing was conducted at, 3,7,14 and 28 days using three prisms. NDOR specifies a minimum strength of 700 psi for PCCP projects. Tables 6.2.2.1 and 6.2.2.2 lists test results for the five mixes with w/cm = 0.45 and 0.4 respectively. These results indicate that the chosen mixes outperform the current NDOR standard mix (Mix 6) in terms of flexure strength.

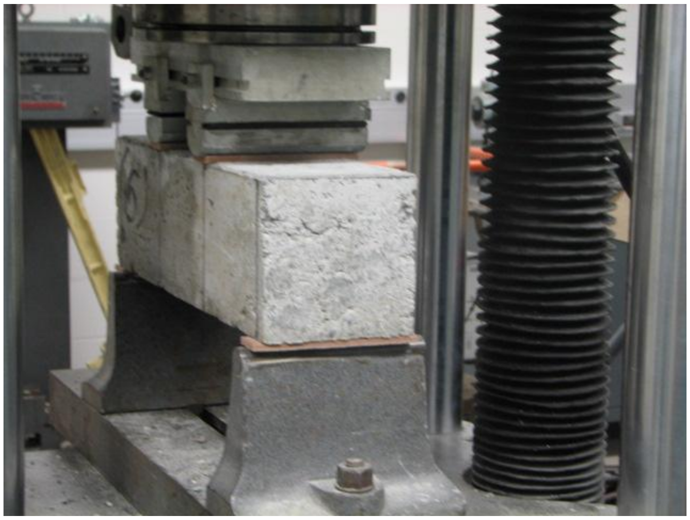


Figure 6.2.2.1 Flexural Strength Testing

Table 6.2.2.1 Flexural Strength (psi)(w/c 0.45)

Mix	3 day	7 day	14 day	28 day
2 (20C-20F-30LS)	439	495	591	740
2D (15C-21F-30LS)	491	564	638	745
2E (15C-21F-45LS)	509	578	668	830
2F(15C-18F-15S-30LS)	507	619	667	812
6(25F-30LS)	**436**	**518**	**596**	**687**

Table 6.2.2.2 Flexural Strength (psi) (w/c 0.40 to 0.42)

Mix	3 day	7 day	14 day	28 day
2 (20C-20F-30LS)	464	564	612	760
2D (15C-21F-30LS)	433	510	617	774
2E (15C-21F-45LS)	472	587	648	834
2F(15C-18F-15S-30LS)	492	625	640	805
6(25F-30LS)	**421**	**511**	**598**	**692**

6.3 Hardened Concrete Durability Tests

6.3.1 Freeze and Thaw

Freeze and thaw testing was conducted in accordance with the ASTM C666 Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing. Procedure A was followed using three beams of 3"x4"x16". Figure 6.3.1.1 shows the freeze and thaw cabinet and Figure 6.3.1.2 shows the test specimens of one of the mixes (Mix 2E). The tests started after the specimens had been cured for 14 days and lasted about seven weeks for 300 freeze-thaw cycles. A sonometer was used to measure the fundamental transverse frequencies of the specimens after about 35 freeze-thaw cycles. The relative dynamic modules of elasticity with time of specimens were calculated. The weight losses or gains of the specimens with time were measured. NDOR specifies a durability factor > 70 % and mass losses < 5%. Tables 6.3.1.1 show the test results for all the five mixes with w/cm = 0.4.

Figure 6.3.1.1 Freeze and Thaw cabinet

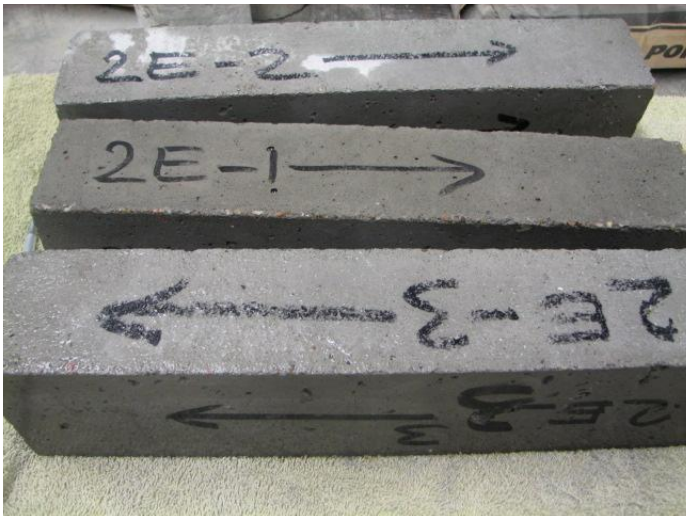


Figure 6.3.1.2 Freeze and Thaw test specimens (Mix 2E)

Table 6.3.1.1 Freeze and Thaw (w/c 0.40)

Mix	**Durability Factor**	**Mass Loss**
20C-20F-30LS	70%	0.93%
15C-21F-30LS	74%	0.23%
15C-21F-45LS	73%	0.54%
15C-18F-15S-30LS	70%	0.36%
25F-30L	**72%**	**0.10%**

6.3.2 Length change: Length change testing was conducted in accordance to ASTM C157 Standard Test Method for Length Change of Hardened Hydraulic-Cement, Mortar and Concrete. Three specimens from each mix were tested; the size of each specimens was 3”x3”x10”. The specimens were cured in a moist cabinet for 24 hours. The initial comparator reading was taken after placing the specimens in lime-saturated water for half hour. Specimens were then placed in lime saturated water until they have reached the age of 28 days and a second comparator reading was taken. Specimens were stored in chamber with 50% humidity and 73 F, where comparator readings were taken after 4, 7, 14 and 28 days of air curing and after 8, 16, 32 and 64 weeks. Figures 6.3.2.1 and 6.3.2.2 show the comparator readings and the chamber. , Tables 6.3.2.1 and 6.3.2.2 shows the readings for the five mixes with w/cm = 0.45 and 0.4 respectively. These results clearly indicate that, in spite of the variation of the length change values among the five mixes at 28 days, the length change values are almost the same after 64 weeks. Also, there is no significant difference between the values with w/cm = 0.45 and w/cm = 0.4.

Figure 6.3.2.1 length change, comparator reading

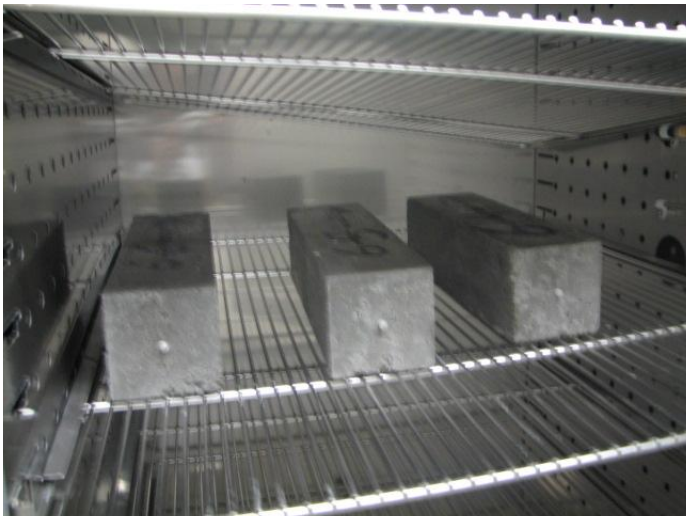


Figure 6.3.2.1 length change, Specimens in the chamber

Table 6.3.2.1 Length change (w/c 0.45)

Mix	Length change at 64 weeks (%) Chamber
20C-20F-30LS	-0.17
15C-21F-30LS	-0.15
15C-21F-45L	-0.15
15C-18F-15S-30LS	-0.16
25F-30LS	**-0.16**

Table 6.3.2.1 Length change (w/c 0.40)

Mix	Length change 64 weeks (%) Chamber
20C-20F-30LS	-0.15
15C-21F-30LS	-0.16
15C-21F-45L	-0.15
15C-18F-15S-30LS	-0.15
25F-30LS	**-0.16**

6.3.3 Chloride Ion Penetration:

Chloride ion penetration was conducted in accordance to ASTM C1202 Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration. Three 2" thick slices of 4" nominal diameter cylinder were used. The amount of electrical current passed through the specimens, in coulombs, was measured during a 6 hours period. A potential difference of 60 Volts DC was maintained across the ends of the specimen, one of which was immersed in a sodium chloride solution, the other in a sodium hydroxide solution. ASTM specify the following categories based on the electric charge measured in coulomb: very low (100-1,000), low (1,000 – 2,000), moderate (2,000 – 4,000), and high (> 4,000). NDOR does not have acceptance criteria for this test, but it recommends mixes that have very low or low chloride ion penetration. Tables 6.3.3.1 and 6.3.3.2 indicate that all the mixes fall in these categories.

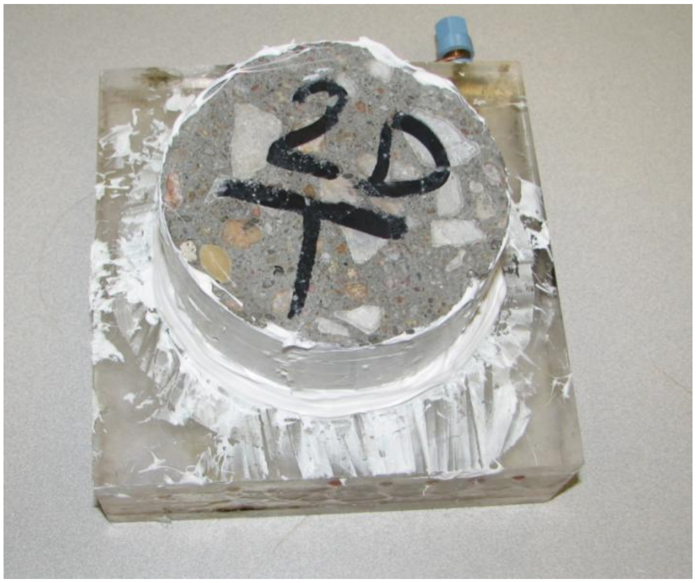


Figure 6.3.3.1 Chloride ion penetration, test Specimen mix 2D

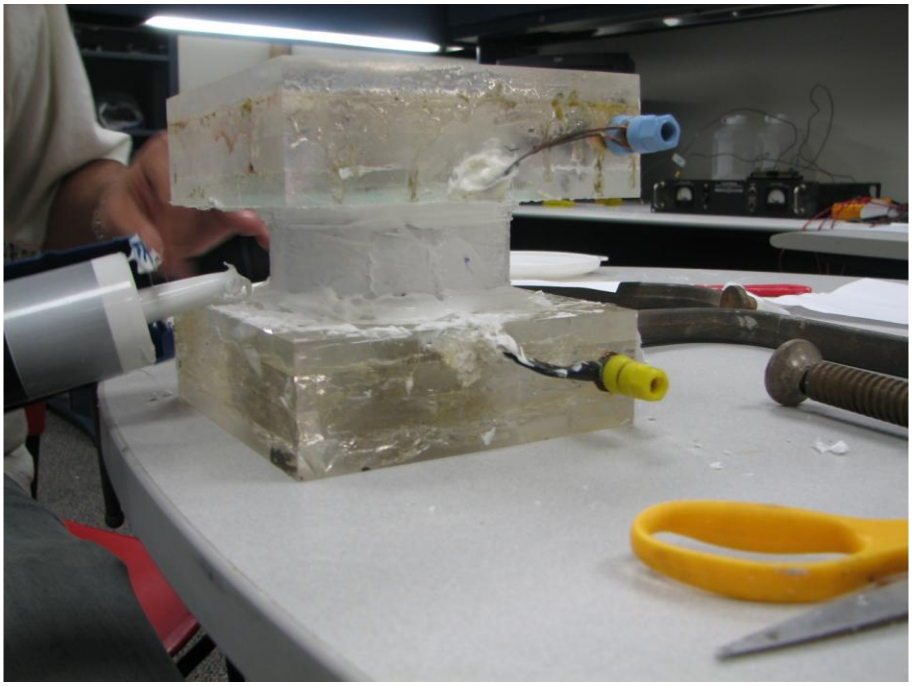


Figure 6.3.3.2 preparing test specimen

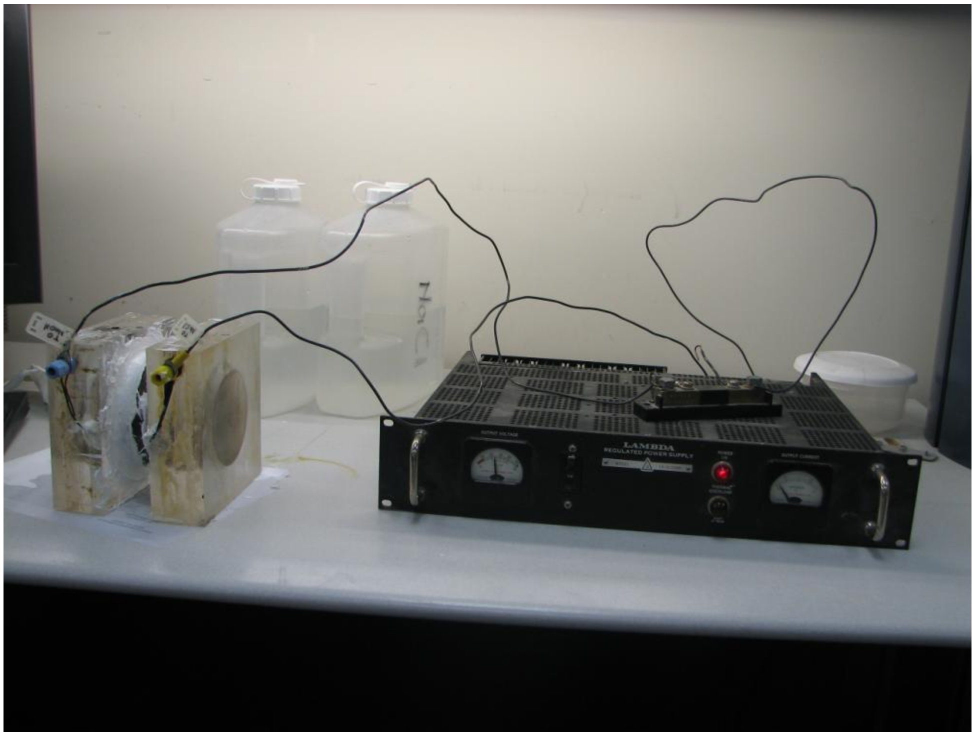


Figure 6.3.3.3 test running

Table 6.3.3.1 Chloride ion penetration (w/c 0.45)

Mix	Charge Coulombs
2 (20C-20F-30LS)	2300
2D (15C-21F-30LS)	1975
2E (15C-21F-45LS)	1359
2F(15C-18F-15S-30LS)	1442
6(25F-30LS)	**1950**

Table 6.3.3.1 Chloride ion penetration (w/c 0.45)

Mix	Charge Coulombs
2 (20C-20F-30LS)	891
2D (15C-21F-30LS)	702
2E (15C-21F-45LS)	774
2F(15C-18F-15S-30LS)	945
6(25F-30LS)	**873**

6.3.4 NDOR Wet and Dry Test

The NDOR Wet & Dry test evaluates the behavior of the mix under heating and cooling environment. It evaluates/indicates the amount of deterioration that may occur in the field, due to the reactivity of the aggregate.

The wet and dry apparatus consists of a test chamber in which test specimens are placed and subjected to total submergence in water for a period of 8 hours and a drying in heated air for 16 hours. The chamber temperature is maintained at 70 -75 F during the wetting cycle and at 120 F during the dry cycle.

Heat is supplied during the drying cycle by a gas fired forced air furnace. Tap water is used during the test and is not reused after each cycle. Racks are used to hold the specimens in a horizontal position in the test chamber and also to prevent the test specimens from touching each other. The racks are inserted and removed from the test chamber by means of an overhead crane.

Six 3" x 3" x 16" beams are fabricated for this test from each mix design. Three of the beams will be tested for flexural strength after 28 days of normal curing. The remaining three beams are to be tested for flexure after 548 days of wet & dry cycle. Machined stainless studs will be embedded in the ends of three of the beams during casting to facilitate measuring during the test period.

After the beams have been removed from the molds, they are stored in the curing room for 26 days for the three beams for wet and dry testing. On the 26th day, the beams will be removed from the racks and placed in saturated lime water for 48 hours. This would complete the 28 days of required cure time. The remaining three beams will be stored in the curing room for 28 days for the flexure testing.

TEST PROCEDURE:

1. Record initial readings from sonic analysis.

   Sonic Analysis: are performed by the test method (Fundamental Transverse, Longitudinal, and Tensional Resonant Frequencies of Concrete Specimens) - ASTM C 215. The resonant frequencies are performed by the transverse mode. The transverse frequency of the specimen are recorded
2. Place specimens on a rack and insert into the test chamber. All racks in the test chamber must be full at all times. This can be accomplished by filling the racks with dummy specimens see Figure
3. The analysis are to be recorded every 28 days for the 365 days in the chamber
4. The reading will be recorded every 56 days after the 365 days in the chamber.

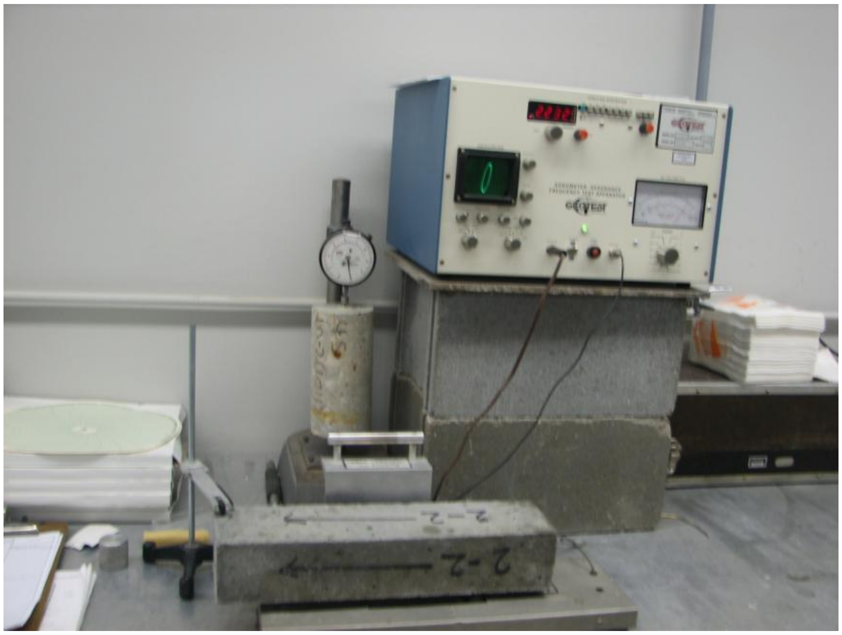


Figure 6.3.4.1 Wet and dry test, Sonic Analysis

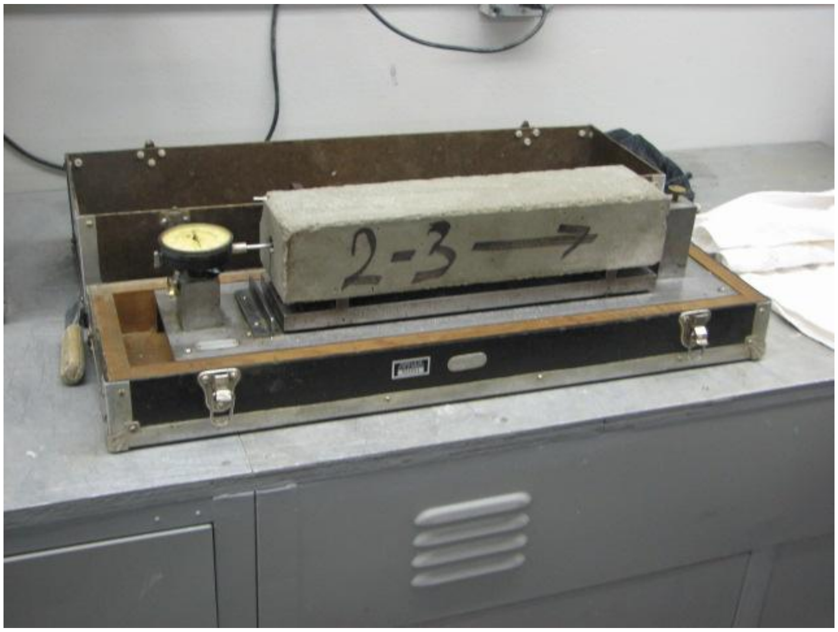


Figure 6.3.4.2 Wet and dry test, length change reading

Table 6.3.4.1 Wet and Dry, Durability Factor (w/c 0.45)

Mix	**Relative dynamic modulus of elasticity %**
20C-20F-30LS	94.1
15C-21F-30LS	92.3
15C-21F-45L	92.5
15C-18F-15S-30LS	94.8
25F-30LS	**92.1**

Table 6.3.4.2 Wet and Dry, Durability Factor (w/c 0.40)

Mix	**Relative dynamic modulus of elasticity %**
20C-20F-30LS	95.8
15C-21F-30LS	91.4
15C-21F-45L	93.7
15C-18F-15S-30LS	95.6
25F-30LS	**89.2**

6.4 SUMMARY OF TEST RESULTS

Table 6.4.1 shows summary of test results for all mixes using both w/c =0.45 and 0.4.

Table 6.4.1 Summary of Test Results

Test Number	**Mix ID**	**Plastic Concrete Properties**						**Hardened Concrete Properties**						
		Water-Cement Ratio	**Slump (in) ASTM C143**	**Density (lb/ft^3) ASTM C138**	**Time of Initial Setting (min)* ASTM C403**	**Time of Final Setting (min)* ASTMC403**	**Air Content (%) ASTM C231**	**Comp. Str. 14 Days (psi)** AASHTO T-22**	**Flex. Str. 28 Days (psi)** AASHTO T-97**	**Freeze-Thaw Durability Factor (%)** ASTM C666**	**Freeze-Thaw Mass Loss (%)** ASTM C666**	**Wet-Dry (%)** NDOR**	**Length Change (%)** ASTM C157**	**Permeability Charge (Coulomb)** ASTM C1202**
Duration		N/A	N/A	N/A	N/A	N/A	N/A	14 days	28 days	300 cycle	300 cycle	584 cycle	64 weeks	60 days
NDOR Limit		**N/A**	**1.5 to 2**	**N/A**	**N/A**	**N/A**	**7.5 to 10**	**> 3500**	**> 700**	**> 70**	**< 5**	**> 90**	**N/A**	**Low < 2000**
Lab Test #1	20C-20F-30L	0.45	4.50	144	405	572	7.0	4,227	740	Broken	Broken	94.1	-0.17	2,300
	15C-21F-30L	0.45	3.50	140	440	714	7.5	4,388	745	71%	1.10%	92.3	-0.15	1,975
	15C-21F-45L	0.45	3.25	150	433	579	7.5	4,914	830	76%	0.10%	92.5	-0.15	1,359
	15C-18F-15S-30L	0.45	3.00	143	307	428	7.8	4,944	812	Broken	Broken	94.8	-0.16	1,442
	25F-30L	**0.45**	**2.75**	**142**	**310**	**436**	**7.8**	**3,851**	**687**	**74%**	**0.65%**	**92.1**	**-0.15**	**1,950**
Lab Test #2	20C-20F-30L	0.4	1.75	144	245	342	7.8	5,186	760	70%	0.93%	95.8	-0.15	891
	15C-21F-30L	0.41	1.75	144	268	378	7.8	4,437	774	74%	0.23%	91.4	-0.16	702
	15C-21F-45L	0.42	1.75	146	256	377	7.8	4,739	834	73%	0.54%	93.7	-0.15	774
	15C-18F-15S-30L	0.4	1.75	144	246	319	7.5	4,998	805	70%	0.36%	95.6	-0.15	945
	25F-30L	**0.42**	**2.00**	**144**	**245**	**316**	**8.0**	**4,362**	**692**	**72%**	**0.11%**	**89.2**	**-0.16**	**873**

NOTES:

*** These results shows the average of two specimens**

**** These results shows the average of three specimens**

7.0 TESTING OF FIELD APPLICATIONS

7.1 Introduction

In addition to the reference mix (mix # 6), three from the four other mixes have been selected for test pavements (Mixes 2, 2D, and 2F). Mix 2E has not been selected because it uses 45% limestone. The test pavements have been constructed in two locations. The first location is located at Fremont, NE and the second is located at Lincoln, NE. The length of each constructed pavement is at least 50 ft. Three tests have been conducted in the field, slump test; air content and unit weight. Test specimens have been taken from a 10 cubic yard truck for laboratory testing by both, UNL research team and NDOR team. Figures 7.1.1, 7.1.2, 7.1.3, 7.1.4 show pictures during construction of test pavements.

Figure 7.1.1 Constructed Pavement

Figure 7.1.2 Constructed Pavement

Figure 7.1.3 Constructed Pavement

Figure 7.1.4 Constructed Pavement

Figure 7.1.5 Test Specimens at the field

7.2 First Location

The first location selected for test pavement was Fremont East Bypass Hwy 275 & Hwy 30. Figure 8.2.1 shows the location of the constructed pavements.

Four sections were constructed using four different mixes in two different days. Two test pavements were constructed each day. The percentage by weight of the total cementitious material did not meet the targeted mix designs. The mixes required for this project and what had received on the project is stated in Table 7.2.1. The only mix that has met the target mix is mix 6. The research team and the TAC member decided to use the mix 16C-20F-30LS as mix 15C-21F-30LS.

The locations of the two sections are:

1PF Station 36+88 to 36+65 Poured on 06/13/2008

21F-15C (Delivered to Project 20F-16C) Station 36+88 to 35+50 Poured on 06/13/2008

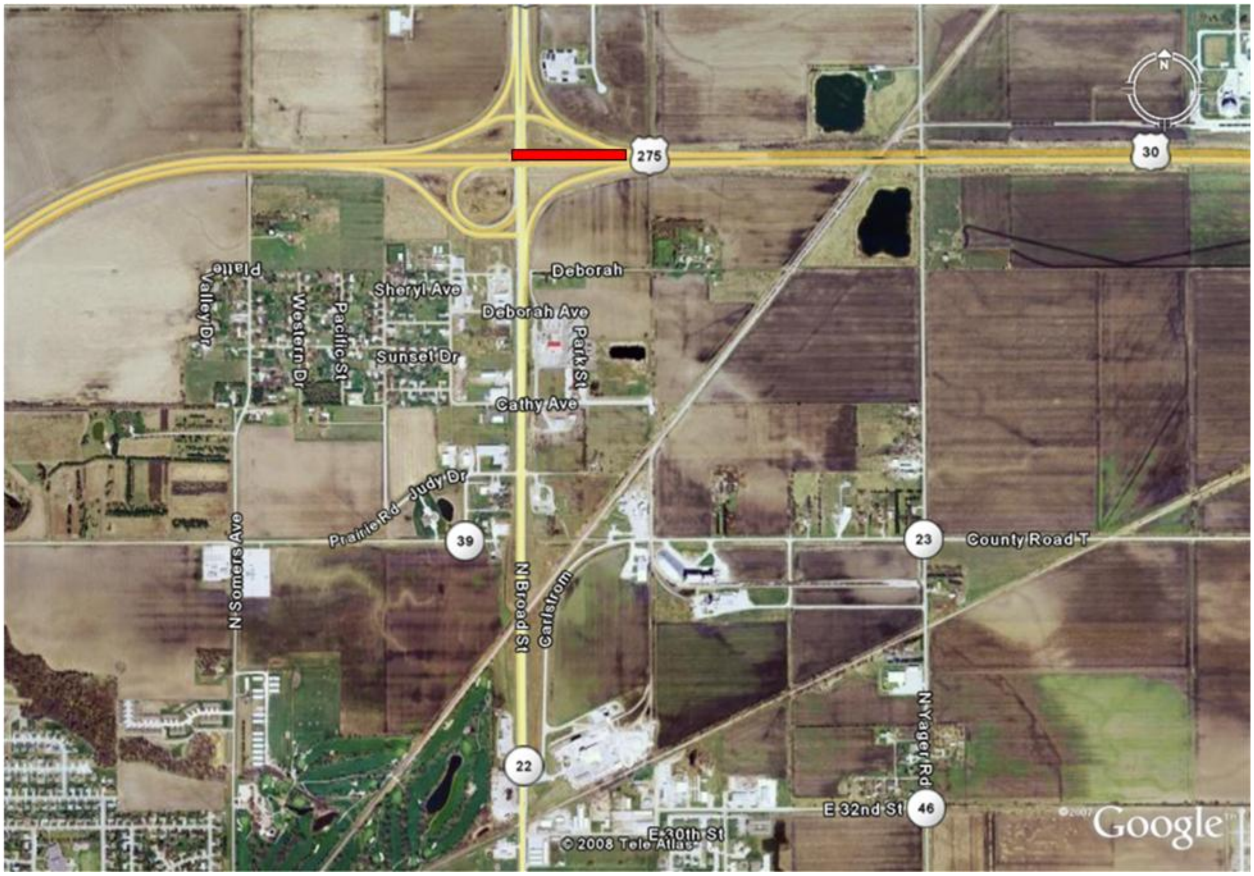


Figure 7.2.1 Constructed Pavement Location (Highway 275)

Table 7.2.1 Required and Provided Mixes

Required Mix	Provided Mix
20C-20F-30LS	16C-20F-30LS
15C-21F-30LS	12C-21F-30LS
15C-18F-15S-30LS	12C-18F-10S-30LS
25F-30LS	25F-30LS

7.3 Second Location

The second location were selected for test pavement is the East Bound I-80 on Ramp (27 St & I-80) Lincoln, Nebraska. Figure 7.3.1 shows the location. Two sections were constructed using two different mixes. One test pavement is constructed using mix 2 and the other is constructed using mix 2F.

The locations of the two mixes are:

18F-15C-15 Slag Station 7211+17 to 7210+36 Poured on 11/14/2008

20 F-20C Station 7209+80 to 7211+78 Poured on 11/14/2008

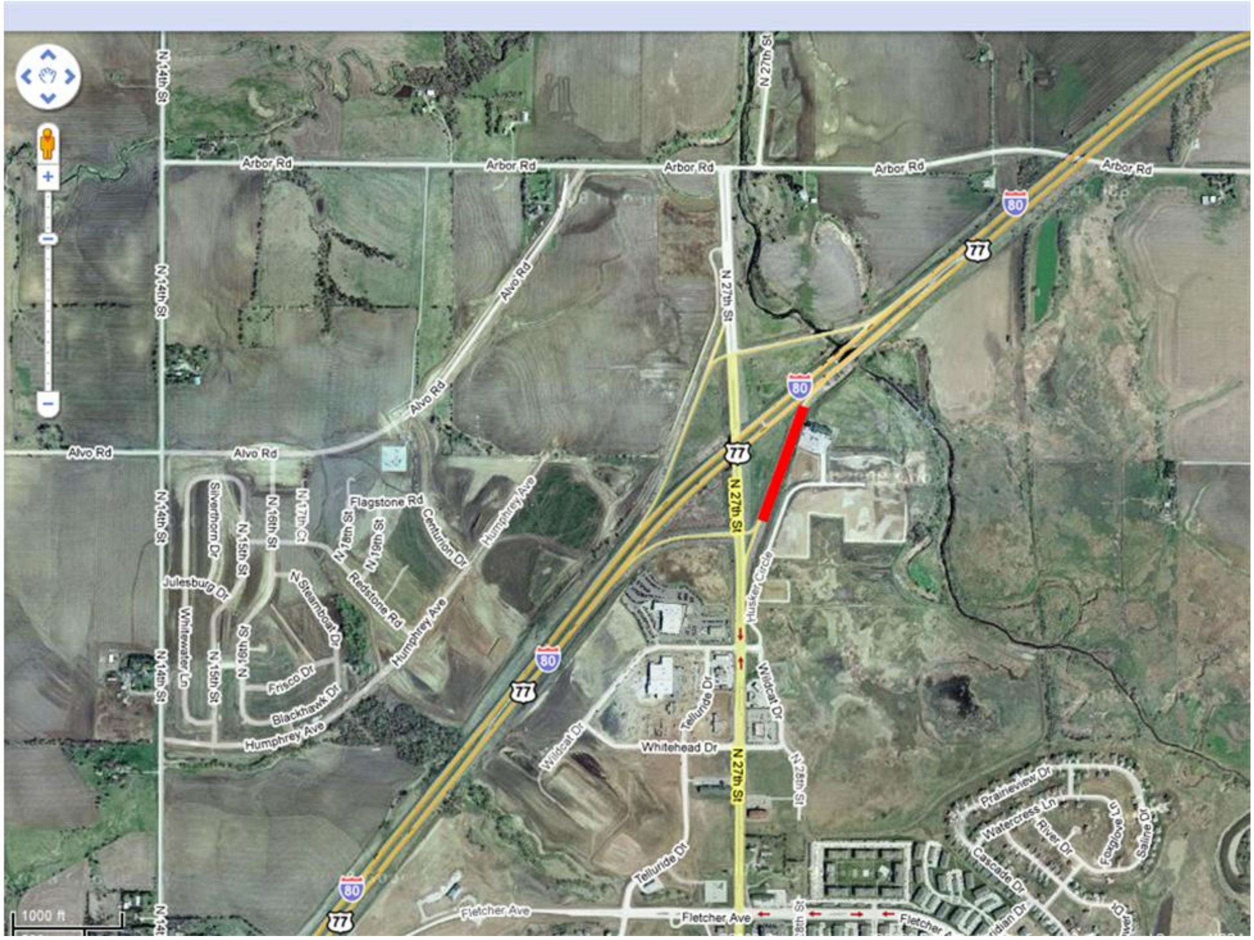


Figure 7.3.1 Constructed Pavement Location (Lincoln, NE)

7.4 Tests conducted in field

The following test are conducted at the laboratory

1. Slump
2. Air content
3. Unit weight

Table 7.4.1 shows test result for the four section using four different mixes .

Table 7.4.1 filed tests results

Mix	Air at plant (%)	Slump (in.)	Unit weight (lb/ft3)	W/C	Air in the field (%)	Hardened Air (%)
16C-20F-30LS	6.9	3	138	0.39	6.2	8.45
20C-20F-30LS	N/A	2	142	0.37	8.9	7.7
15C-18F-15S-30LS	N/A	1	143	0.38	8.8	9.66
25F-30LS	**5.5**	**2.5**	**144**	**0.38**	**5.5**	**9.27**

7.5 Tests conducted in laboratory

The following test are conducted at the laboratory

1. Compressive strength: Tables 7.5.1 and 7.5.2 show the test result at the UNL and NDOR lab respectively. NDOR specifies a minimum compressive strength of 3,500 psi at 14 days for PCCP projects. These results indicate that the chosen mixes outperform the current NDOR standard mix (Mix 6) in terms of compressive strength

Table 7.5.1 Compressive strength (psi) , UNL

Mix	4 day	7 day	14 day	28 day
16C-20F-30LS	3650	4256	5157	5652
20C-20F-30LS	2750	3352	4273	4549
15C-18F-15S-30LS	2860	3066	4338	6076
25F-30LS	**3118**	**3800**	**4375**	**5375**

Table 7.5.2 Compressive strength (psi), NDOR

Mix	4 day	7 day	14 day	28 day
16C-20F-30LS	3487	3863	4883	5647
20C-20F-30LS	2320	3770	4650	5420
15C-18F-15S-30LS	1730	3820	5430	6010
25F-30LS	**3460**	**3983**	**4530**	**5273**

2. Flexural strength: Tables 7.5.3 and 7.5.4 show the test result at the UNL and NDOR lab respectively.

Table 7.5.3 Flexural strength(psi), UNL

Mix	4 day	7 day	14 day	28 day
16C-20F-30LS	555	549	648	811
20C-20F-30LS	N/A	406	422	645
15C-18F-15S-30LS	N/A	538	667	736
25F-30LS	**441**	**540**	**665**	**739**

Table 7.5.4 Flexural strength (psi), NDOR

Mix	4 day	7 day	14 day	28 day
16C-20F-30LS	395	540	540	666
20C-20F-30LS	390	440	590	530
15C-18F-15S-30LS	350	495	670	650
25F-30LS	**395**	**380**	**555**	**625**

3. Chloride ion penetration: Tables 7.5.5 and 7.56 show the test result at the UNL and NDOR lab respectively. ASTM specify the following categories based on the electric charge measured in coulomb: very low (100-1,000), low (1,000 – 2,000), moderate (2,000 – 4,000), and high (> 4,000). NDOR does not have acceptance criteria for this test, but it recommends mixes that have very low or low chloride ion penetration.

Table 7.5.5 Chloride ion penetration, UNL

Mix	Charge Coulombs
16C-20F-30LS	1210
20C-20F-30LS	875
15C-18F-15S-30LS	1050
25F-30LS	**1050**

Table 7.5.6 Chloride ion penetration, NDOR

Mix	Charge Coulombs
16C-20F-30LS	768
20C-20F-30LS	528
15C-18F-15S-30LS	403
25F-30LS	**582**

4. Length change: Table 7.5.7 shows the test result at the UNL and NDOR lab. It can be noticed from the test results that all of the mixes have small length change.

Table 7.5.7 Length Change Results

Mix	**NDOR water (in./ft)**	**NDOR air(in./ft)**	**UNL air (in./ft)**
16C-20F-30LS	-0.002	-0.112	-0.140
20C-20F-30LS	-0.112	-0.112	-0.150
15C-18F-15S-30LS	-0.160	-0.070	-0.090
25F-30LS	0.010	0.004	-0.100

5. Freeze and thaw: Table 7.5.8 shows the test result at the UNL and NDOR lab. Mix 16C-20F-30LS did not meet NDOR limit and NDOR lab and mix 25F-30LS did not meet NDOR limit at both labs. That was due to the low amount of entrained air.

Table 7.5.8 Freeze and Thaw results

Mix	NDOR	UNL
16C-20F-30LS	65%	76%
20C-20F-30LS	79%	84%
15C-18F-15S-30LS	84%	81%
25F-30LS	61%	62%

6. NDOR wet and dry test: Table 7.5.9 shows the test result at the UNL and NDOR lab. The results show that mix 16C-20F-30LS and 25F-30LS met the required limit. However, mix 20C-20F-30LS and mix 15C-18F-15S-30LS did not meet the limit.

Table 7.5.8 Wet and Dry test results

Mix	Durability factor (NDOR)	Durability factor (UNL)
16C-20F-30LS	98%	95%
20C-20F-30LS	80%	87%
15C-18F-15S-30LS	88%	94%
25F-30LS	90%	91%

7.6 Recommendation for the test sections

The TAC member and the research team recommended that NDOR will continue monitoring these two sections to quantify how well these SCM's mix designs performed.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

The objective of the project was to develop Portland cement concrete pavements PCCP mixes containing Class C fly ash that have equal or better performance characteristics than the current NDOR's standard mix. It was found that Alkali Silica Reactivity (ASR) has been the primary contributor to pavement deterioration in Nebraska due to the widespread use of reactive aggregates. In this project, 16 mixes were identified to be tested for ASR according to the ASTM C1567. The results of the ASTM C 1567 were as expected. The use of Class C fly ash as the only supplementary cementitious material (SCM) with in practical replacement quantities results in concrete mixes that do not satisfy NDOR acceptance criteria for ASR (i.e. 28-day expansion less that 0.1% bad English). Mixes containing Class F fly ash with 25% cement replacement showed exceptional performance with regard to ASR resistance. The ternary mix using both Class C and Class F showed satisfactory performance. The mixes containing ground granulated blast furnace slag GGBFS and Class C fly showed improved performance over using Class C fly alone, however, acceptable performance was still not achieved for mitigating ASR. To produce acceptable ASR resistance using GGBFS, either a higher replacement level or quaternary mixes are used. All mixes using 45% limestone and 55% sand and gravel showed that using 45 % limestone replacement helps the ASR mitigation. These results are very predictable since a portion of the reactive sand and gravel is being replaced with the non-reactive limestone. This alternative aggregate gradation would not only produce superior results with regards to ASR, but would also improve the overall quality of the concrete. Based on the ASTM C1567 test results and material cost, four mixes have been identified for overall performance testing. Overall performance testing of these mixes has indicated that they are satisfactory in terms of slump, unit weight, air content, setting time, compressive strength, flexural strength, freeze and thaw resistance, length change, chloride ion penetration and wet and dry. Three of these mixes were used in field application. Specimens were tested in the lab and confirmed acceptance of the performance of these mixes. All three mixes had 70% Class 47B sand and gravel and 30% limestone. They had three different proportions of SCM: (1) 16% Class C fly ash + 20% Class F fly ash; (2) 20% Class C fly ash + 20% Class F fly ash; and (3) 15% Class C fly ash + 18% Class F fly ash + 15% GGBFS.

8.2 RECOMMENDATIONS

Based on the results of the investigation carried out in this project, the following recommendations are made:

1- Class C fly ash should not be used in relatively small quantities as the only SCM for PCCP construction in Nebraska because it results in concrete mixes that do not satisfy NDOR acceptance criteria for PCCP. A mix with 17% class C has historically not performed well, and was shown in experimental work in this project to have poor ASR performance. Mixes containing over 45% of C Ash would have acceptable, but not exceptional, ASR. In addition they possess properties that may not give the best overall pavement performance.

2- Aggregate composition of 45% limestone and 55% 47B sand and gravel was tried and shown to result in mixes with better performance than those with 30% limestone and 70% 47B sand and gravel. This is an expected finding as increased limestone reduced the amount of reactive sand and gravel. The team encourages use of larger quantities of limestone where economically feasible.

3- The two mixes recommended in this study are (1) 20C-20F-30LS (2)15C-21F-30LS. These mixes were shown to produce pavements of equal or better quality than the previously use 25F-30LS. Mixes that have 15% - 20% cement replacement with Class C fly ash and a 20% -25% cement replacement with Class F fly ash have better performance than the standard mix with 25% cement replacement with Class F fly ash only. Mixes 1 and 2 are shown to be less expensive than the base mix based on 2006 cost estimates.

4- The mix that had cement replacement of 15% with Class C fly ash, 18% with Class F fly ash; and 15% with GGBFS is also recommended, as it results in the highest compressive strength and lowest permeability. However, cost and availability of storage silos may prohibit its use, unless a cement manufacturer pre-blends the supplementary materials with during cement manufacturing.

IMPLEMENTATION

By Wally Heyen

(NDOR Material and Research)

In the late 1990's, Nebraska Department of Roads found concrete pavement was deteriorating years before the pavement reached its design life. Materials and Research began an in-depth investigation of existing pavements and found extensive map cracking (ASR), staining at the joints, low entrained air, large entrapped voids, and segregation. The Department did a visual inspection of cores, a hardened concrete air count, ASR Reactivity (Uranium Acetate Test) and ran Petrographic Examination. The Department found the failure of the pavement was due to freeze-thaw, poor construction and the possible use of a Supplement Cementitious materials (SCM) Class C Fly Ash, which did not mitigate the ASR at 17% replacement. The Department stopped using C-Ash in 2004 and made changes to the specifications including higher air content, curing application and consolidation of concrete. The Department also started placing new mix designs in the field for evaluation before making any permanent changes to the specifications. Materials and Research has monitored the three mixes (20%C-20%F, 15%C-21%F and 15%C-18%F-15%S) for approximately two years after placement of concrete in the field. We will continue to monitor for an additional three years for a total time of five years. Materials and Research will be looking for map cracking, staining along the cracks and any other deterioration of the pavement. At the end of the observation, a decision will be made whether to add the research mixes to the Department's approved Class of Concrete Mix Table.

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APPENDIX A: MEETING AT IOWA STATE UNIVERSITY

Minutes of Meeting at Iowa State University

Date: May 9, 2005

Location: Center for Transportation Research and Education (CTRE), Ames, Iowa

Attending:

Dr. Kejin Wang, assistant professor at Iowa State University, Mr. Jim Grove, PCC Paving Engineer at CTRE, Mr. Todd Hanson, a PCC Engineer with the Iowa Department of Transportation. Dr. Amgad Girgis, Mr. Matt Kleymann, Dr. George Morcous, Dr. Maher Tadros, UNL.

Objective:

The objective of the meeting was to gain knowledge of research that has been done in Iowa that could assist in the investigation of premature deterioration being observed in Nebraska roads.

Summary of Information Presented

To begin the meeting, a brief introduction was given explaining the purpose and scope of the research project. It was explained that a number of relatively new pavement installations in Nebraska were experiencing premature deterioration and NDOR has identified the use of Class C fly ash as a possible cause. Consequently, NDOR has banned the use of Class C fly ash. The University of Nebraska is currently involved in a research project to determine the cause(s) and possible solutions to this problem. In addition to explaining the scope of the project, a PowerPoint presentation was also shown. The PowerPoint was created by the Nebraska Department of Roads. It showed examples of pavements showing premature deterioration, and of pavements not showing such deterioration. The presentation also summarized NDOR's findings which led them to the decision to ban the use of Class C fly ash.

Research Done by Iowa

Iowa representatives discussed their knowledge on the subject. It was indicated that Iowa had experienced similar deterioration on pavements constructed between 1984 and 1994. Just as with Nebraska roads, the deterioration started with discoloration at the joints which is where the cracking first began. Iowa's investigation into this problem led them to several conclusions. It was their opinion that that there was no inherent chemical problem with Class C fly ash. However, they found that the Class C fly ash potentially could lead to deterioration problems such as lowered resistance to ASR attack and ettringite formation. However, the most significant of these problems was a poor entrained air void system which they believed was the underlying cause of the premature deterioration. Other factors contributing to possible deterioration included paving machine vibration, use of certain deicing chemicals, method and timing of cutting control joints, and sealing of control joints.

The use of Class C fly ash can create a relatively stiff mix with low workability. This is attributed to high initial water demand and cementitious properties of Class C fly ash. Therefore, in order to place this concrete, contractors had to increase water content and amount of vibration, negatively impacting the air void system. An analysis was done on cores taken from poor performing pavements in Iowa. It was found they did not possess adequate air content. The mortar air content in the cores was well below the recommended value. In addition, the concrete air content was well below the Iowa DOT specifications. When the spacing factor was analyzed it was observed many of the pavements had spacing factors well above the recommended value of 0.200 mm. It was also observed that the air void systems were significantly worse at the top of the core and that the cracking was commonly confined to the top 6 inches. Since the cracking was commonly found in the top of the core where the air content was much lower, it was believed that the deterioration was a cause of a substandard air system.

In 1994, the Iowa DOT implemented changes in their specifications. Previously, specifications called for a target air content of 6 ± 1 % and a minimum vibration of 7000 vpm’s with no maximum value. After 1994 a new target air content of 7 ± 1% was specified. In addition, vibration specifications were changed to a range of 5000 to 8000 vpm’s. Since adoption of the new specifications, pavements experiencing the premature deterioration have been greatly reduced. In addition, cores of the good performing pavements were analyzed. These cores showed the air content in the mortar was above the recommend value of 9% and the air content of the concrete was above the specification of 6%. It was also observed that good performing pavements all had spacing factors below 0.200 mm, while the poor performing pavements had spacing factors above 0.25 mm. These results confirmed their belief that the cause of the deterioration was an inadequate air void system.

Recently, Iowa DOT has adjusted their specification for air content to further ensure an adequate air void system. The specifications require an air content of 6% plus the loss from the paver, +1.5/-1 %. Therefore, once a day the air content is measured in front of and behind the paver. The calculated loss of air is then added to the 6% and that is the value for the target air content +1.5/-1 %. For example, if the loss of air due to paving is 1.75%, the specifications call for air content ahead of the paver is 7 plus 1.75 plus 1.5 = 9.25 or 7 plus 1.75 minus 1 = 7.75, i.e. between 7.75 and 9.25.

In addition, Iowa is using ternary mixes with 40% replacement of cement. The 40% replacement includes 20% Class C fly ash and 20% Ground Granulated Blast Furnace Slag. The use of slag along with the Class C fly ash creates a mix that has a higher workability, lower permeability and higher resistance to ASR than mixes containing Class C fly ash alone.

During further discussion, it was mentioned that fly ashes used from a plant in Council Bluffs, during the late 1980 and early 90's were of bad quality due to the methods being used to burn the coal. They felt that some of the problems seen in pavements during those years may have been partially caused by these fly ash sources. Nebraska most likely would have experienced similar problems which could have had some responsibility in the premature deterioration being seen. However, improvements to the coal burning operations in recent years have greatly increased the quality of fly ash being produced there. It was also asked if they felt there was any advantage to inter-grinding fly ash with the cement. It was their opinion that realistically there was no real advantage to inter-grinding the fly ash other than for convenience (This was consistent with Dr. Mike Thomas' position when he was in Lincoln on April 1, 2005).

There was a short discussion concerning the effectiveness of an Air Void Analyzer. The AVA is used to evaluate the air void system of freshly placed concrete. Even though the AVA samples only the top layer of the concrete, it is their opinion that it still represents a reasonably accurate description of the air void system since the top layer of the concrete typically has the worse air content. Therefore, this test can give a reasonably accurate indication of the concrete's ability to resist freeze thaw attack.

The experts at CTRE and Iowa DOT indicated that they doubted the type of the air entraining agent used in the concrete mix would have a significant impact on the performance of the hardened concrete pavement.

Recommendations Provided

According to the petrographic analysis done on cores taken from Nebraska pavements and studied by CTRE experts, almost all the pavements have a poor air void system. As was the case in Iowa, there is a significant difference between the air content at the top and bottom of the core. Although, there was a small amount of ASR gel found in the cores this could not have been the primary cause of the severe cracking that was observed. In addition, the air voids were extensively filled with ettringite which further degraded the air void system.

Several other issues were discussed regarding Nebraska's current mix design and specifications. It was suggested that it may be advantageous to either use a higher replacement level of Class C fly ash or ternary mixes. CTRE personnel agreed that mixes with about 35% (by weight) Class C fly ash, 20% Class C and 20% slag, or 20% Class C and 20% Class F, could have promise for good results. Such mixes would have low permeability as well as high resistance to ASR. Note that studies have demonstrated that more Class C fly ash is needed (about 20-25%) to have the same ASR resistance as Class F (about 15-20%). Lower values would offer little help and in some cases can even add to the problem. The 17% Class C fly ash that has historically been used in Nebraska may not be the most effective cement replacement value.

The current specification (2005) in Nebraska for percent air content is 5.0 to 7.5 percent. However, since this value is for concrete in front of the paver, it was suggested that this may be too low. In addition, Nebraska currently uses Magnesium Chloride as its deicing chemical. According to CTRE and Iowa DOT experts, this can be detrimental to Portland cement concrete.

Summary:

The meeting at Iowa State was extremely helpful to the progress of our study. The team from Iowa was very knowledgeable and willing to help. In their opinion, the deterioration being seen in Nebraska roads is most likely due to the same type of problems that Iowa had been seeing. They suggested that Class C fly ash is probably not directly responsible for the deterioration although it may be leading to some the problems causing the deterioration. A substandard air void system could be mostly responsible for the deterioration. Several steps were suggested to ensure an adequate air void system such as increasing the specifications for target air content as well as putting limitations on vibrator frequencies. A change in the mix design would also be very helpful. In addition, other factors such as joint cutting, joint sealing and deicing chemicals need to be evaluated.

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APPENDIX B: UNIVERSITY OF NEBRASKA WORKSHOP

Minutes of Roundtable Meeting at the University of Nebraska - Omaha

Date: July 22, 2005

Location: Peter Kiewit Institute, Omaha, NE

Attending:
Dr. Ramon L. Carrasquillo, P.E., Carrasquillo Associates
Dr. Tarun R. Naik, P.E., Professor at the University of Wisconsin Milwaukee
Mr. Todd Hanson, P.E., PCC Engineer with the Iowa DOT
Mr. William E. Cook, P.E., Material Engineer with Iowa DOT
Mr. Kevin Merrymann, P.E., PCC Field Engineer with Iowa DOT
Dr. Kejin Wang, Assistant Professor at Iowa State University
Mr. Jim Grove, P.E., PCC Paving Engineer at the Center for PCCP Technology
Mr. Richard L. Donovan, P.E., Paving Materials Expert with USACOE

Mr. Moe Jamshidi, Materials and Research Engineer with NDOR
Mrs. Amy Starr, Research Engineer with Nebraska Department of Roads
Mrs. Lieska Halsey, Research Engineer with Nebraska Department of Roads
Mr. Mick Syslo, P.E., Pavement Design Engineer with NDOR

Mr. Thomas J. Schroeder, Fossil Fuels Manager with NPPD
Mr. Billy Wendland, Assistant Unit Train & Track Coordinator with NPPD
Mr. Richard M. Kotan, P.E., Manager of Rail Operations with OPPD
Mr. Ron Boro, Manager of Fossil Fuels with Omaha Public Power District

Dr. Maher K. Tadros, P.E., Charles J. Vranek Distinguished Professor at UNL
Dr. Amgad Girgis, Research Assistant Professor at UNL
Dr. James Goedert, P.E., Dept. of Construction Systems Chair at UNL
Dr. George Morcous, P.E., Assistant Professor at UNL
Dr. Yong-Rak Kim, Assistant Professor at UNL
Dr. Joseph Benak, Professor at UNL
Mr. Matt Kleymann, Research Assistant at UNL

Objective:

Determine ways for allowing the use of Class C fly ash in pavements while still meeting expected performance criteria.

Summary of Presentations

Moe Jamshidi - Nebraska Department of Roads

The Nebraska Department of Roads gave a presentation on their research which led to the decision to ban Class C fly ash. They began noticing that pavements less than 10 years old were displaying severe deterioration. In June 2003 they started to investigate such pavements as well as pavements that were performing well. The scope of the project included compiling project backgrounds, visual inspections, core extractions as well as laboratory tests such as air void analysis, petrographic examination, ASR reactivity tests, and mechanical properties tests. After phase I of the project it was concluded that all pavements exhibiting premature deterioration had in common the following variables: (1) poor air system, (2) large entrapped air voids, (3) type I/II cement batched with Class C fly ash, (3) poor drainage, and (4) no load transfer.

The same investigation was done in phase II which examined pavements that were performing well. It was concluded that the following variables were common to all good performing pavements: (1) low entrained air, (2) inter-ground cements (IPN/IPF), (3) cements batched with Class F fly ash, (4) high compressive strength, (5) poor drainage, and (6) no load transfer. After comparing the pavements from phase I and phase II, NDOR concluded that the only substantial difference between the good and bad performing pavements was that the poor performing pavements all had type I/II cement batched with 17% Class C fly ash. Therefore, Class C fly ash was banned from use in concrete pavements.

Currently NDOR is involved in ongoing research to address some of the issues being seen in Nebraska pavements. They are in the process of requiring additional air content and have made improvements to the drainage design. NDOR feels that the premature deterioration starts with ASR and is probably advanced by freezing and thawing. They have done testing with various ternary and quaternary mixes that have promising results. They have seen that ASTM C 1567 is a very reliable indicator of good and bad performing pavement. Therefore, any potential future mixes must be able to pass this test.

After NDOR's presentation a brief discussion was held. Dr. Carrasquillo believed that it was extremely important to understand the mechanism of the distress in order to find the proper steps to mitigate it. One way to do this is to try and duplicate the deterioration. He suggested taking cores from pavements showing premature deterioration, but to take the cores away from the area were the distress can be seen. Then using accelerated exposure tests, try to duplicate the deterioration.

Dr. Naik raised concerns with the potential upcoming changes to the air content specifications. By specifying such a high percent for air content in front of the paver, approximately 8-12%, it is difficult to determine what the final strength of the concrete will be. Therefore, contractors need to be aware that the strength of the concrete in front of the paver will be significantly lower than concrete behind the paver due to the change in air content. However, NDOR as well as IDOT added that strength is not really an issue of concern for them and that by specifying a minimum amount of cementitious material they are confident that they will achieve adequate strength.

Dr. Ramon Carrasquillo said that anytime Class C fly ash is used as a SCM, there is the concern of chemical reactions. He added that staining of the concrete such as what is seen in Nebraska pavement could be an indication of the presence of a gel formed from a chemical reaction.

The team from the IDOT noted that they have experience extremely similar deterioration problems during approximately the same time period. However, it is difficult to compare pavements of the two states because of the difference in aggregates.

Iowa uses a far less reactive fine sand compared to Nebraska's sand and gravel. Also, Iowa is able to use a higher percentage of limestone. The combination of these differences makes Iowa's pavements far less susceptible to ASR. It was added that in both states the deterioration, especially in its later stages, appeared to be a result of freeze thaw failure. It could be that there is something different triggering the deterioration that eventually leads to a similar type of distress.

Todd Hanson – Iowa Department of Transportation

Mr. Todd Hanson, a PCC engineer with the Iowa IDOT gave a presentation on research done with Class V gravel and the importance of an adequate air void system on concrete durability. Starting in the 1940's a research study was done between Nebraska and Kansas to investigate map cracking that was being seen. It was shown that the sand and gravel aggregates used in Nebraska and Kansas are reactive, especially the portion of the gravel that is + #4. The exact mechanism of the cracking was not determined, but it seemed to be related to surface shrinkage caused by severe drying and Alkali-Silica Reaction. Limestone "Sweetening" was effective in controlling the deterioration as well as some pozzolans although it was unclear why. It was concluded that more research was need at that time (1968).

Mr. Hanson also presented research done by Ash Grove with Type IPF cement and Class C fly ash. The four mixes that were experimented with were a straight type I/II mix, Duracem F, and Duracem F w/ 20% C ash. All of these mixes showed similar strength performance. The mix using Duracem F w/ 20% C ash showed the lowest expansion for the ASTM 1260 test. However, the study showed that such mixes may have a significantly increased set time.

Iowa has experience very similar problems in pavements constructed between 1984 and 1994. An investigation into these pavement showed that almost all contained an inadequate air void system. Therefore, in 1995 changes were made to the specifications requiring an increase in air content to 7 ± 1% as well as range for vibration frequency from 5000 to 8000 vpm's. According to the IDOT, no pavement constructed after 1995 has shown signs of premature deterioration. A study of these pavements showed that the pavements constructed after 1995 have greatly increased air content as well as much smaller spacing factors.

Currently Iowa is using ternary mixes with slag and Class C fly ash. It specification for air content is 6% (+Paver Loss) +1.5/-1%. Recent pavements have also been constructed using 55% Class V Gravel (Platte River sand and gravel) and 45% Limestone. The pavements have used type IP cement (type I/II blended with 17% calcinated clay) and 15% Class C fly ash.

According to studies down in Iowa, the primary cause of the deterioration being seen was an inadequate air void system. A study done on vibration in 1995 showed that vibrating had a large influence on air content and vibrator trails could be seen were pavements were over vibrated. This deterioration was not directly related to the use of Class C fly ash. However, often when cement with dehydrated gypsum was used with a reactive ash, the C_3A would react with the gypsum creating a flash set. The contractors would then have to over-vibrate the concrete in order to place it.

Richard Donovan – US Army Corps of Engineers

In Mr. Donovan's presentation, a brief overview of the evolution of using fly ash by the USACOE was given. In 1983, in response to EPA RCRA guidance, Class F or C fly ash was allowed with selected optional requirements and replacement at 20% of cementitious volume. In 1987, both Class C and Class F ash were required to have a maximum LOI of 4% and 15-35% replacement of cementitious mass was used. In 1997, the requirement for LOI was reduced to 3% for freeze thaw environments.

In 2002, in response to Public Law 106-398, ASTM C 1260 was used to identify reactive aggregates. For non-reactive aggregates, the requirements are those set in 1997. For concrete using reactive aggregates, Class F fly ash was required with a CaO limit of 8%. The percent replacement was determined based on the modified ASTM C 1260 or C 1567.

In 2004 new specifications were implemented. For non-ASR reactive aggregates Class F or slag was to be used at 15-35% replacement by mass. The use of Class C fly ash is not permitted. For ASR reactive aggregates, Class F fly ash with a CaO limit of 8% is required with the replacement percentage determined by ASTM C 1567.

Based on their study, fly ashes with higher CaO require much higher replacement levels to effectively mitigate ASR. The replacement percentage needed to control ASR varies with the source of aggregate. For use with some aggregates, replacement levels of 35% still won’t effectively control ASR. Using high amounts of Class C fly ash also raises a host of other concerns such as constructability, shrinkage potential, set time and loss of aggregate interlock. Ternary mixes are not used by the USACOE because of the limited number of silos available during the construction of airfields.

It was also suggested that the use of deicers, particularly Potassium Acetate, when used with a high lime fly ash, will greatly increase ASR expansion. It was found that in some case where materials were shown to be innocuous by ASTM C 1567, the presence of Potassium Acetate made these materials reactive.

During the presentation by Mr. Donovan it was noted by the team from Iowa that ASTM C 1260 (or 1567) is very conservative. Some aggregates and mixes in Iowa that have been proven to be successful, fail the test. Almost, everything that Iowa uses will fail the test, but perform well in the field. They have also found that the test is very dependent on the cement that is used and also seemed to correlate with the amount of magnesium present. However, NDOR's experience is that for Nebraska aggregates, ASTM C 1567 is consistent regardless of cement source.

Tarun Naik – University of Wisconsin Milwaukee

Dr. Naik's presentation was focused on using coal ash to manage ASR. He explained that for Alkali-Silica Reaction to occur there must be three conditions present. There must be sufficient alkali in the concrete, presence of reactive aggregates and sufficient moisture. Therefore there are different ways to mitigate ASR. A reduction in the total available alkali will minimize ASR, which can be accomplished by using low alkali cement. The use of a mineral additive can be effective by consuming alkalis during the pozzolanic reaction and effectively reduce the total available alkalis. ASR can also be minimized by controlling the moisture in the concrete. This can be accomplished several ways such as improving the impermeability of the concrete, reducing the gel pore size or reducing the ingress of water into the concrete. Also, using a water reducer to lower the necessary water demand will help.

The effectiveness of using Clean-Coal Ash or CCA was discussed. Test data was presented using several mixtures incorporating combinations Class C, Class F and Clean-Coal Ash. These mixes were non-air entrained. According to Dr. Naik it is easier to manage non-air entrained concrete in the lab because it eliminates the variable of mixes having different air content. Based on ASTM 1260, the mixes that were most effective in controlling ASR expansion were the ones using large amounts of CCA. Particularly, 20% C ash with 60 % CCA and 0% Na_2SO_4 or 20% F ash with 60% CCA and Na^2SO^4 of 4%. Generally, Clean-Coal Ash was most effective in controlling ASR, followed by Class F ash then Class C ash. Using coal ash creates pozzolanic reactions in the concrete that consumes the alkali, reduces the gel pore size and mobility of the alkalis, controlling ASR expansion.

The state of Wisconsin has relatively non-reactive aggregates and allows 35% of either Class C or Class F fly ash. However, Class F fly ash is not readily available in Wisconsin so most pavement projects use Class C fly ash.

Kejin Wang – Iowa State University

Dr. Wang presented common causes of deterioration and the effect of fly ash on these types of deterioration. Some of the common causes of deterioration in pavements are shrinkage, freezing-thawing, cement aggregate reaction (CAR), and other various problems such as abnormal setting or poor workability.

The use of fly ash generally reduces free shrinkage in concrete. Free shrinkage is the shrinkage of mortar samples that are not restrained from shrinking by aggregates or other means. Typically, at a given consistency, the higher the percent fly ash the lower the free drying shrinkage. Since the samples are kept at a given consistency, for a higher dose of fly ash a lower amount of water is need to achieve the desired consistency. Therefore, this confirms that a reduction in water will reduce shrinkage.

Since freeze-thaw resistance is direct function of a concrete’s air void system, the use of fly ash has no major effect on freezing-thawing durability if the strength and air content are kept constant. However, since some fly ashes have slow strength gains, the concrete may not have sufficient strength at a given time. In addition, some fly ashes will increase the dosage of AEA needed, depending on the fly ash's LOI and fineness. It was her recommendation to check the air content in front of and behind the paver to ensure that proper air content is being achieved in the placed concrete. It is also important to have an adequate spacing factor (< 0.2mm) which generally decreases as the air content increases. It was pointed out by Dr. Carrasquillo that for a given air void system, concrete with fly ash will generally have higher resistance to freezing and thawing because the decrease in permeability will reduce the ingress of moisture.

Another common cause of deterioration is cement-aggregate reaction such as ASR. The effectiveness of fly ash in reducing the expansion to such reactions is very dependent on the fly ash being used therefore it is important to analyze the fly ashes chemistry. It has been shown that the amount of CaO, MgO, SO_3 can have an effect on how well a given fly ash can mitigate expansion due to chemical reactions. The amount of soluble alkali can also have an effect on the dosage of fly ash required to control expansion. It is important to realize what type of gel in being formed during the cement-aggregate reaction, as different gels with different compositions can cause different degrees of expansion.

The use of fly ash in concrete pavements can cause other problems such as abnormal setting and reduced workability. According to ACI Comm. 226 all Class F and most Class C fly ashes will increase the time of setting. However, some C ashes with high amounts of free lime and calcium content can have reduced set time and workability. This can lead to other problems such as over or under consolidation.

Dr. Wang concluded that many factors such as the chemistry and reactivity of a given fly ash can have a significant effect on concrete durability. Controlling things such

CaO, MgO, soluble alkali and sulfate content may be important steps in producing a durable concrete.

Maher Tadros - University of Nebraska

A brief presentation was given by the University of Nebraska describing the issues that they will investigate as well as potential mixes using Class C fly ash. The issues included ensuring an adequate air void system, the effect of deicing salts, chemical reactions such as ASR or sulfate attack, and cutting and sealing joints. The University Nebraska presented three classes of mixes that looked promising. The mixes were: (1) 40% replacement of cement with Class C fly ash, (2) 40% replacement of cement with a combination of Class C and Class F fly ash, and (3) 40% replacement of cement with Class C fly ash and Ground Granulated Blast Furnace Slag.

General Discussion

The general consensus of the group was that Class C fly ash could be used in Nebraska pavements although extensive research was needed to determine the processes causing the deterioration and to develop a mix that would mitigate such problems.

Dr. Tarun Naik commented that the mix using 20% Class C and 20% Class F fly ash would be a very good mix with respect to ASR resistance. NDOR and IDOT both agree with this comment and said that their testing has shown this mix to be very effective in reducing expansion as well as having extremely low permeability. In addition, Dr. Carrasquillo suggested that with the chemical composition of Nebraska's fly ash, 40% Class C fly could be a very good mix. A concern was raised by Dr. Naik that higher replacements such as 40% may raise other durability issues such as salt scaling because of the lower cement value. His suggestion was to replace 20% of the cement with Class

C fly ash and then add an additional 20% Class C fly ash as a replacement of the finest particles of the fine aggregate. The total 40% Class C fly ash would greatly reduce expansion from ASR, while the mix would still maintain a cement factor similar to that of current NDOR mixes.

NDOR added that is would be unwise to conclude that what works in Iowa will be guaranteed to work in Nebraska. Nebraska has different sources and availability of materials, mainly aggregates, that makes it more difficult to produce a durable pavement. Unlike Iowa, Nebraska is forced to use reactive sand & gravel and is limited to using only 30% limestone because of availability. NDOR did add that Iowa is doing good things, especially with air content and drainage which are all issues that are being investigated. In addition, Todd Hanson mentioned that although Iowa is now using ternary mixes, for several years after the new specifications for air content, they successfully used type I/II cement batched with Class C fly ash.

Although there were some conflicting opinions on how reliable of an indicator ASTM C 1260 is of a concrete resistance to ASR expansion, everyone agreed that it was at least on the conservative side. According to NDOR, their research shows that all mixes that fail ASTM C 1260 fail in the field and that all mixes that pass ASTM C 1260 perform well in the field. Therefore, they feel very adamant about using the test as an indicator of a concrete’s performance in the field. The three main tests used by NDOR to determine a concrete’s durability are ASTM C 666 freeze-thaw test, NDOR’s wet-dry test, and modified ASTM C 1260 (1567) ASR expansion test. Dr. Carrasquillo added that it is important to establish an appropriate baseline for analyzing the performance of mixes during these tests. Although mixes using IPF cement have performed very well, it may not be a fair baseline since lesser performing mixes may still be adequate in the field.

There was also a discussion on using blended or inter-ground cements. It was agreed upon that having fly ash blended or inter-ground by the cement manufacturer produces a far better product then having it batched at the ready-mix plant. This is contrary to what other experts have said, but it was agreed upon by the panel members that cement manufacturers are better able to control the chemical composition of the fly ash and cement and are able to produce a much better product. It was suggested that a meeting be held with representatives from various cement manufacturers to discuss the possibility of marketing a product that pre-blends cement with either C ash, F ash or both. It was discussed that such a product could be very attractive for not only Nebraska, but Kansas and Iowa as well.

There was also a short discussion on phase II of the project. It was suggested by NDOR that UNL focus on developing a mix that has high chemical durability, especially against Alkali-Silica Reaction. They felt that other issues such as type of deicing salt, air content/vibration, as well as methods of cutting and sealing joints were common concerns for all concrete pavements regardless of whether or not fly ash was used and should be investigated in separate projects. These issues have been and are continually being investigated by NDOR and need not be the focus of this project. It was also suggested that only one source of aggregate be used in the mixes. The aggregate will be an eastern Platte river sand & gravel and should provide a good representation of Nebraska's aggregates. Dr. Carrasquillo advised to keep as many things constant as possible and focus on a single issue such as resistance to ASR.

Summary of Recommendations

- It is extremely important to understanding the mechanism of the distress so that proper steps can be taken to mitigate it.
- Although something else might be initiating the deterioration, an adequate air void system is vital in achieving a durable concrete.
- There is no “one size fits all” for replacement levels needed to control ASR and the percent replacement with fly ash should be determined from testing, particularly ASTM 1567.
- There are several ways to control ASR in concrete such as reducing the total available alkali, consuming alkalis with pozzolanic reactions and controlling the moisture in the concrete.
- Many factors affect the ability of a given fly ash to limit expansion due to ASR. Controlling things such as CaO, MgO, soluble alkali and sulfate content may be important steps in producing a durable concrete.
- It was expressed by the group that all 3 proposed mixes could be successful and should be further investigated with lab testing. A particular favor was shown towards a ternary mix with both Class C and Class F fly ash.

- Dr. Tarun Naik suggested an additional mix using 20% replacement of cement with Class C fly ash and then add an additional 20% C ash as a replacement of the finest particles of the aggregate.
- It was suggested that for this project UNL focus on developing a mix that has a high resistance to chemical attacks, particularly ASR. Other issues such as air content and

deicing chemicals are common concerns of all pavements regardless of the mix design and should be addressed in a separate study.

- NDOR felt that for a mix to be able to produce a durable pavement it must pass ASTM 1567, ASTM 666 and NDOR's Wet-Dry test. However, it was suggested that is very important to establish a baseline in order to determine what "passes" these tests.
- Additional tests may be required since mixes with high replacement levels can have other problems such as salt scaling and time of set.
- It was agreed upon that having SCMs pre-blended or inter-ground produces a far better product then having it batched at the ready-mix plant.
- One cannot conclude that what works in Iowa will work in Nebraska. Nebraska has more reactive aggregates and is restricted by the availability of limestone.

APPENDIX C: PICTURES OF PREMATURE DETIORATION

Signs of deterioration at 84th and Holdrege, Lincoln, Nebraska:

Staining of Transverse and Longitudinal Joints

Staining of Transverse Joints

Cracking Running Adjacent to Free Edges and Joints

Cracking Running Adjacent to Free Edges and Joints

Cracking Parallel to the Joint

Signs of Deterioration and Hwy 77 & Van Dorn, Lincoln, Nebraska:

Staining of Joints

Map Cracking at Corner Joint

APPENDIX D: SURVEY OF STATE DOT'S

Name	State	Cementitious Material Allowed	Has your state experienced similar deterioration?	If so, what steps have been taken to minimize or preent this deterioration	Additional Comments
Jim Pappas	Delaware	Type I/II cement, fly ash, GGBFS, Silica Fume	Yes, ours was ASR though	We've found the pavement that show this were before mitigation steps were taken (slag, fly ash, etc	-
Lon Ingram	Kansas	Type I/II cement	No	-	We have experienced staining in the transverse joints but have not noticed deterioriation of the pccp that cannot be attributed to D-cracking. Some of the stained pavements are 20+ years old and performing well. We have not allowed the use of class C Ash.
Tom Pyle	California	Type I/II batched with Class F fly ash	No		If ASR and D cracking are not a possibility, I would have a petrographic study done on cores. I would also look into the absorbtion of the aggregates.
Lloyd Welker	Ohio	Type I/II cement, Type I/II cement batched with Class C fly ash, Type I/II cement batched with Class F fly ash	No	-	-
Mike O'Brien	Mississippi	Type I/II cement, Type I/II cement batched with Class C fly ash, Type I/II cement batched with Class F fly ash	No	-	-
Gerobin Carnate	Hawaii	Type I/II cement	No	-	-
Keith Johnston	Oregon	Type I/II cement, Type I/II cement batched with Class C fly ash, IPF cement, Type I/II cement batched with Class F fly ash	No	-	-
Michael Redmond	Maine	Type I/II cement, Type I/II cement batched with Class F fly ash, Silica Fume, GGBFS	No	-	Maine does not design concrete roadways. Our only concrete section of interstate I-95 has serious ASR and is being rehabilitated prior to an asphalt overlay.
Tim Aschenbrener	Colorado	IPF cement, Type I/II cement batched with Class F fly ash	Yes, It is somewhat similar but not exactly the same.	No action has been taken to date as we are not certain of its cause.	The distress we have observed includes corner breaks and map cracking in the middle of the slab. It appears to be a distress related to expansion, but it is not ASR.
Jim Wild	Virginia	Type I/II cement, Type I/II cement batched with Class F fly ash, GGBFS, Type II preblended with GGBFS, silica fume	No	-	Vermont has very few concrete pavement roads. The ones that are left have been paved over or torn up. We do have an intersection that was done about 8 years ago that still looks good and there is a street in one of our smaller cities named Winooski that i
Doug Dirks	Ilinois	Type I/II cement, Type I/II cement batched with Class C fly ash, IPF cement, Type I/II cement batched with Class F fly ash, GGBFS	No	-	Is it a combination of things? Could the problem be a combination of ASR, D-cracking, and air content? No problem has to be significant by itself, but when you add them together you have a problem.
Mike Lynch	Montana	Type I/II cement, Type I/II cement batched with Class C fly ash, Type I/II cement batched with Class F fly ash	No	-	-
Mike Brinkman	New York	Type I/II cement, Type I/II cement batched with Class F fly ash	Yes	Make sure our joints our properly sealed has seemed to help. We feel it isd a moisture related problem since we have ruled out ASR and D-cracking. We hope to obtain some cores for analysis purposes this spring.	Run a petrographic analysis (i.e. C-856).
Darin Hodges	South Dakota	Type I/II Cement batched with Class F Fly Ash	No	-	-
Hannah Schell	Ontario	Type I/II cement, Type I/II cement batched with Class C fly ash, Type I/II cement batched with Class F fly ash, Type I cement batched with slag	No	-	Although our specifications allow use of up to 25% fly ash, it is not generally used; use of slag (at 25% replacement level) is common.

Name	State	Cementitious Material Allowed	Has your state experienced similar deterioration?	If so, what steps have been taken to minimize or preent this deterioration	Additional Comments
Jim Delton	Arizona	Type I/II cement, Type I/II cement batched with Class F fly ash	No	-	-
David H. Andrewski	Indiana	Type I/II cement, Type I/II cement batched with Class C fly ash, IPF cement, Type I/II cement batched with Class F fly ash, Type ISA,IA or IIIA,IP-A,IS,III, & IP	Yes	We are looking into the cause of the distress	If you have an idea of what is causing the distress please let me know, what kind of anti-icing chemicals are you using?
Sergio Rodriguez	Alabama		No	-	-
Paul Finnerty	Maryland	Type I/II cement, Type I/II cement batched with Class F fly ash, Slag and Silica Fume are allowed	No. There may be 1 or 2 isolated cases	-	-
Mark E. Felag	Rhode Island	-	-	-	We do not typically specify concrete pavements.
Douglas J. Schwartz	Minnesota	Type I/II cement, Type I/II cement batched with Class C fly ash, IPF cement, Type I/II cement batched with Class F fly ash	Yes, Discolorization does not always mean early failure	If it is not ASR or D-Cracking, it is probably the early stages Freeze-thaw damage due to a poor entrained-air system	Production air test results may indicate adequate air with the pressure meter but the spacing factor or bubble size may not be satisfactory to provide a system to prevent freeze-thaw. Also, the concrete may have been over-vibrated, thereby, destroying the
Andrew Freeman	Wyoming	Type I/II cement, Type I/II cement batched with Class C fly ash, IPF cement, Type I/II cement batched with Class F fly ash	Yes	We conducted a study and identified ACR as the cause. We are now screening limestone sources for ACR susceptibility.	Please let me know an e-mail address where I can send a photo, as we had a very similar appearing pavement where the ACR occurred.
Mike Mance	West Virginia	Type I/II cement, Type I/II cement batched with Class C fly ash, IPF cement, Type I/II cement batched with Class F fly ash, Type I with GGBFS	No	-	Although we allow all of the above cement/pozzolan combinations, we almost always see either Type 1 cement with Class F Fly Ash or a straight Type 1 cement mix (we rarely see Class C Fly Ash or IP Cement used).
robert skalla	New Jersey	Type I/II, Class C or F fly ash and slag	No	-	-
Thomas Baker	Washington	Type I/II cement, IPF, Type I/II cement batched with Class F fly ash, Ground Granulated Blast Furnace Slag and Microsilica			
Keith Lane	Connecticut	Type I/II cement, Type I/II cement batched with Class C fly ash, Type I/II cement batched with Class F fly ash, GGBFS is used extensively	No	-	We have very little exposed PC concrete pavements
Myron K. Banks	Georgia	Type I/II cement, Type I/II cement batched with Class C fly ash, IPF cement, Type I/II cement batched with Class F fly ash, GGBFS	No	-	-