IDAHO TRANSPORTATION DEPARTMENT

RESEARCH REPORT

RP 182C

Materials Acceptance Risk Analysis: Portland Cement Concrete

By

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The objective of this study is to evaluate the Idaho Transportation Department's (ITD) Portland Cement Concrete (PCC) specifications using the current state of knowledge and the opinions of the subject matter experts. The survey method employed in this study was the Delphi Method. Based on the current state of the knowledge and the survey, the authors recommend that the ITD use performance-based specifications and change its aggregate gradation requirements. The use of performance-based concrete-mix design would improve the performance and long-term durability of concrete projects while reducing the cost of production. The authors further recommend that the ITD adopt a performance-based approach to concrete practice by emphasizing end product characteristics. An Excel spreadsheet program was developed for use in the aggregate gradation optimization. In this study, ITD specifications for alkali-silica reactivity (ASR) mitigation were compared with practices in five other states with severe ASR problems. ASTM C1260 and ASTM C1567 tests were performed on concrete made from several Idaho aggregates and combinations of different Class F fly ashes and lithium-nitrate solution to identify the most effective ASR mitigation methods. The results of ASR tests indicate that the Class F fly ash used in S.E. Idaho shows expansion rate of less than 0.1 percent (threshold for acceptance) if replace cement in excess of 30 percent. Tests also indicate low expansion rates when 20 percent of the cement is replace with Class F fly ash and mixed with 5 percent of the recommended lithium nitrate dosage.</td></tr>
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APPROXIMATE CONVERSIONS TO SI UNITS				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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Abstract

The Portland Cement Concrete (PCC) specifications used by the Idaho Transportation Department (ITD) were evaluated using the current state of the knowledge in concrete mix design and the opinions of the subject matter experts (SME) in the field using the Delphi Method. Based on the results of this study, recommendations have been made to enhance the performance and long-term durability of PCC in ITD projects. In addition, several Idaho aggregates were tested for alkali-silica reactivity (ASR) using the American Society for Testing and Materials (ASTM) ASTM C1260 procedures and recommendations for mitigation have been made based on the results of ASTM C1567 tests.

In the present study, the ”e- Delphi“ approach was employed to assess the ITD PCC specifications which play a major role in the selection of materials, construction of structures, and assurance of quality and durability of the structures. The Delphi Method is a systematic, interactive, iterative process for consensus-building among members of a group of experts who are anonymous to each other. The Method recognizes the value of expert opinion, experience, and intuition. It also allows the use of expert opinions when limited information is available and complete scientific knowledge is lacking. In addition to the Delphi study, the ITD PCC specifications were evaluated and compared with the current state of knowledge and practices in the field.

Based on the results of this study, it is recommended that ITD adopt a performance-based approach to its concrete practice by emphasizing the end product characteristics and providing flexibility to contractors and suppliers to use innovative construction techniques and equipment. This would allow the development of better and more cost-effective products which meet ITD performance specifications. The approach involves the inclusion of the concrete specifications as part of the design process with emphasis on the desired characteristics including but not limited to strength, durability, and workability. An Excel spreadsheet program was developed for use in the optimization of the aggregate gradation in order to improve the workability of concrete.

Idaho is known to have reactive aggregates, and within Idaho, the aggregates obtained from sources in the Snake River Plain are particularly reactive. In this study, five states with severe ASR problems were identified, and their specifications were compared with ITD specifications. Several aggregates were obtained from sources in the Snake River Plain and tested for potential reactivity. The ASTM C1260 procedures were employed to identify reactive aggregates, and the ASTM C1567 test was then used to examine the effectiveness of several Class F fly ashes, lithium-nitrate solution, and combinations thereof in mitigating ASR. In this study, it was concluded that Class F fly ash with finer particles (under 45 microns) is effective in the mitigation of ASR. In addition, Jim Bridger and Navajo power plants fly ash (commonly used in southeast Idaho) proved effective, as long as it replaced more than 30 percent of the cement in concrete. These Class F fly ashes are also effective when they replace 20 percent of the cement, in combination with 5 percent of the recommended lithium nitrate dosage.

Chapter 1
Review of ITD Portland Cement Concrete Specifications and Recommendations

Delphi Study

Delphi Method[1]

The Delphi method is a systematic, interactive, iterative process for consensus-building among members of a group of experts who are anonymous to each other. It recognizes the value of expert opinion, experience, and intuition as a research tool when limited information is available and full scientific knowledge is lacking.[1]

The Delphi method was developed by Dalkey and Helmer of the Rand Corporation in the 1950's to predict the probability, frequency, and intensity of possible enemy attacks. This method was originally designed to have a very narrow scope but over the years the applications of the method have broadened to include a wide range of methodologies. In addition to the method's original intent of prediction, the Delphi method has been used to define objectives and as the basis for evaluating participant practices. The Delphi method has been employed in a variety of areas including government, medical, environmental and social studies, as well as business and industrial research.

The Delphi method is conducted anonymously to encourage a true debate among the participants independent of personality, and it does so by eliminating the force of oratory and pedagogy. The method is facilitated by a panel of researchers who refined questions and developed a series of sequential questionnaires. The opinions of participants are shared without mentioning the source, and this allows the participants to revise their opinion after being exposed to the opinions of the other subject matter experts.

The modes of communications of Delphi method have varied drastically since its inception. The conventional/classical Delphi was conducted by face-to-face interaction in a conference room or through letters. In the modern world, communication occurs by telephone, video conference, and/or e-mail. The method of using email as the main source of communication between the facilitator and participants is known as "e-Delphi".[2]

Delphi Process

In the present study, "e- Delphi" was employed to assess the ITD Portland cement concrete (PCC) specifications. These specifications play a major role in selection of materials, construction of structures, and also in maintaining quality and durability of the structures. A group subject matter experts (SME) was identified among the ITD engineers, contractors, concrete suppliers, and scientists. Of the 50 identified SMEs, 22 volunteered to participate in the study. Fifty percent of the study group consisted of ITD employees with the other half representing suppliers, contractors, and scientists.

The Delphi study participants were provided with three documents explaining the methodology of study, the strategy, and a summary of ITD PCC specifications along with a comparison of the ITD specifications for Alkali-Silica reactivity (ASR) with those of five other states with similar problems. In this process, the study group was informed that the outcome of this study would benefit all parties involved and might improve the quality and durability of concrete, while reducing the cost of concrete production. A total of two rounds were sufficient to gauge the degree of consensus among the participants and better understand the issues involving ITD PCC specifications from the ITD and non-ITD perspectives.

Characteristics of the Delphi Technique[3]

The following are the main characteristics of the Delphi method that help in focusing on the issues and distinguish it from other methodologies.

Anonymity

One of the main attributes of the Delphi process is the anonymity of participant. This anonymity reduces the effect of dominant individuals and minimizes the "band-wagon" effect. The use of electronic communication systems such as e-mail to exchange information enables participants to express their opinions freely, encourages open critique, and provides confidentiality.

Controlled Feedback

The iterative process of the Delphi method allows researchers to design the questionaires for subsequent rounds. This enables participants to revise their earlier statements at any time and minimizes the effects of data noise.

Statistical Response

The Delphi process aims at producing a quantitative expression of individual and group opinions from qualitative expressions of thought. The quantitative feedback from the participants is summarized statistically, using mean, median, and standard deviation.

Structuring of Information Flow

The moderator controls the interaction among participants by processing the information and filtering irrelevant content. This avoids the usual problems such as: influence, imposing opinions, shyness, etc. within the group dynamics.

Role of the Moderator

The Moderator plays a key role in the Delphi method, by initiating group activities, managing the feedback received, and for keeping the discussion focused. The moderator may screen responses, form new questions based on those responses, summarize contributions, remove identity revealing information, discard contributions that do not address the problem, and decide on the order and style

of each round. The moderator also can detect when discussions reach an impasse, makes necessary decisions, and provides guidance for the continuation of the discussions. [3] Steps in Delphi Study
A summary of the Delphi process used in this study along with the full description of the Delphi method, the process used to conduct this study, the complete survey results, and the recommendations made by the SMEs are provided in Appendix A.

Summary of the Delphi Study Results

The overall consensus among the participants was that the current ITD Standard Specifications for PCC are largely consistent with the practices in other state DOTs throughout the country. However, there were a few comments regarding ITD specifications and current concrete practices that require further discussion. The following is a summary of the results and recommendations made by the participants.

Based on the survey results, the participants felt that a combination of the Performance- and Prescriptive-based approach to PCC mix design would improve its performance. The majority of the participants felt that ITD could improve its aggregate gradation and reduce the amount of Portland cement in the specifications. In addition, the participants felt that ITD needs to specify a maximum water/cement ratio and use admixtures to control the workability of the concrete. Also, the participants recommended using advanced techniques to better control the quality of the concrete during production and the placement.

Concrete Mix Design, Background Information.

Performance and Prescriptive Based Mix Design

In general, there are two approaches to concrete mix design, prescriptive based and performance based mix designs. In a prescriptive approach, the client specifies the required material, proportions, and construction methods based on fundamental principles that provide satisfactory performance. In the performance-based approach, the client identifies the fundamental requirements, such as strength, durability, and air content, and relies on the concrete producers and contractors to develop concrete mixes that meet those requirements.

Mix design and proportioning are two different activities; however, normally they are grouped and referred to as “mix design”. Actual mix design establishes the required characteristics of a concrete mix to meet or exceed certain characteristics such as target strength, entrained air content, workability, etc. In the prescriptive approach, the mix is proportioned by requiring minimum cement content, specific aggregate grading, admixture dosage and all other pertinent information. In the performance approach, the target goals such as strength and entrained air are specified, and the designer is allowed to select the material and produce a mix that meets the target goals.

The current concrete specifications in many state transportation departments including Idaho are largely based on the prescriptive approach; however, there is a tendency to adopt a combination of both the Performance/Prescriptive approach. States and agencies identified as using a combination of the Performance/Prescriptive are Illinois Department of Transportation (IDOT), Iowa Department of

Transportation (Iowa DOT) and the Federal Aviation Association (FAA) (FAA P-501).[4] Their latest specifications were used in providing guidelines for ITD specification modifications. A comprehensive overhaul of the ITD PCC specifications is outside of the scope of this study; however, in this study, two major areas of concern identified by the SMEs are analyzed using the current state of the published literature. A summary of the PCC specifications used by ITD, Iowa DOT and Michigan Department of Transportation (MDOT) are provided in Appendix A.5 and A.7.

The Use of Excessive Cement in Concrete and Its Consequences

Strength of concrete is directly related to the amount of cement and the water-cement ratio. Well proportioned concrete requires a minimum amount of paste necessary to bind the aggregate and fill the voids. Increasing the quantity of cement and using minimum water-cement ratio generally provides a denser and more durable concrete with high early strength. However, it also increases the potential for uncontrolled cracking. Excessive cement increases water demand in concrete even with minimum water-cement ratio; consequently, it increases the amount of paste necessary for durable concrete and is ultimately responsible for shrinkage. The additional paste disperses the aggregate in concrete resulting in an unstable concrete as shown in Figure 1 and Figure 2.[5]

The use of excessive cement in concrete will have an adverse effect on the long term performance of concrete. In addition, the cost can be significant. Excessive cement used in concrete increases the minimum amount of paste required for concrete and produces an unstable product. Excessive cement also promotes shrinkage and cracking and increases the chance of ASR in concrete. The primary cause of potential cracking in a concrete mixture itself is due to many other environmental effects. The admixtures and the amount of cement in the mix are the main factors affecting concrete cracking. The cement content and the admixtures influence the water demand in concrete, and this directly relates to volume change and consequently the potential for uncontrolled cracking.

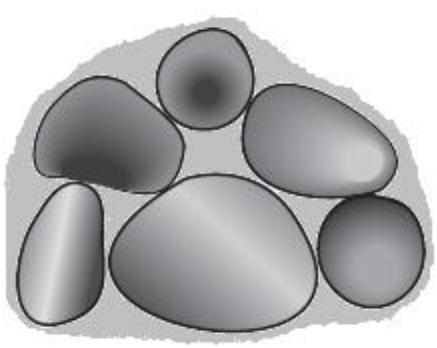


Figure 1. Cement Slurry Filling the Voids[5]

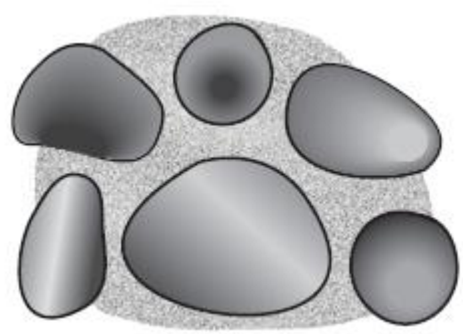


Figure 2. Dispersed Aggregate in Cement Slurry[5]

The constituents of concrete, water and cement, when mixed with each other, initiate a chemical reaction emitting heat known as the “Heat of Hydration”. This enables the binding of fine and coarse aggregates to form concrete. The basic assumption of the design is that the voids of the coarse aggregate are filled with the fine aggregate, and the voids of the fine aggregate are filled with the cement slurry formed from the combination of cement and water. A well designed mix with an appropriate water-cement ratio provides a workable concrete along with strength and durability. If the amount of paste is increased, the cement slurry produced disperses the aggregate instead of filling the gaps, thereby increasing the workability but decreasing the design strength. The excessive cement used in concrete also increases the heat of hydration. This causes expansion of the concrete in the initial stages with subsequent contraction during cooling, creating cracks on the surface due to the temperature gradient. Uneven cooling of concrete also promotes curling in large slabs used in pavement.

Mixing Water for Concrete

Any natural water which is drinkable and has no pronounced odor or taste (potable water) can be used in concrete mix. It is important that the water used in concrete mix is free from harmful materials. ASTM C403 (AASHTO T197) tests ensure that impurities in the mixing water do not adversely affect the concrete including the setting time. The acceptance criteria for water used in concrete are provided in ASTM C1602. Excessive impurities in water may affect the setting time and concrete strength, efflorescence, staining, corrosion of reinforcement, volume instability, and durability.

Water with less than 2000 ppm of total dissolved solids could be used without any harm to the product; otherwise, it should be tested for its effects on strength and setting time. Some of the impurities effecting concrete strength, setting time, and durability are summarized below. For more detailed information, see the “Design and Control of Concrete Mixtures” published by the Portland Cement Association (PCA).[5]

Alkali Carbonate and Bicarbonate

Sodium and potassium carbonates and bicarbonates affect setting time, concrete strength, and aggravated ASR, and their presence in excess of 1000 ppm must be tested.[5]

Chloride

Chloride ions may cause corrosion of reinforcement. The acid-soluble chloride ion level at which reinforcement corrosion begins is about 0.2 percent to 0.4 percent by mass of cement (0.15 percent to 0.30 percent water soluble). Chloride can be introduced in concrete by many sources including admixtures, aggregates, cementitious materials, mixing water, deicing salt, or seawater. In natural water, a high dissolved content of solids are most often sodium chloride and sodium sulfate. Both of these solids can be tolerated in concrete mixes up to 20,000 ppm with a low potential for corrosion.

The ACI 318-08 Building Code [6] in section 4.3, Table 4.3.1 provides limits for the percent water soluble chloride ion content in concrete mix by weight of cement.

- Prestressed concrete (Classes C0, C1, C2) ≤ 0.06 percent.
- Ordinary reinforced concrete exposed to chloride in service (Class C2) ≤ 0.15 percent.

- Ordinary reinforced concrete used in dry environment (Class C0) ≤ 0.10 percent.
- Other constructions (Class C1) ≤ 0.30 percent.

ACI 318-08 Building Code does not limit the amount of chloride in plain concrete.

Sulfate

High sulfate content in mix water causes expansion in concrete and deterioration due to sulfate attack. ASTM C1602 limits the amount of sulfate to 3,000 ppm.

Carbonates and Bicarbonates of Calcium and Magnesium

Carbonates of calcium and magnesium are not soluble in water and are not found with sufficient concentration in water to affect the strength of concrete. Bicarbonates of calcium and magnesium are present in some municipal water sources with concentrations up to 400 ppm. This is not considered harmful.

Magnesium Sulfate and Magnesium Chloride

Magnesium sulfate and magnesium chloride can be found in high concentrations in water and may not be harmful. However, concentration of magnesium chloride should be limited to 40,000 ppm, and magnesium sulfate should be less than 25,000 ppm.

Iron Salts

Natural groundwater typically does not have iron salt concentration of more than 30 ppm; however, in acid mine water, the concentration can be excessive. The concentration of iron salts in the water should be limited to 40,000 ppm. For more information on other types of harmful salts in water consult the PCA Manual.[5]

Characteristics of Aggregate

In concrete, aggregate is the filler, and it is held together by the cement paste. Aggregate typically accounts for 60 to 70 percent of concrete by volume and compared to the cement paste, it is more chemically stable and less susceptible to volume changes. Therefore, while maintaining desired concrete properties in concrete mixes, it is desirable to maximize the volume of aggregate and minimize the volume of cement content. In the past, it was assumed that the smallest percentage of voids produced the most suitable concrete performance. In recent years, however, it has been discovered that the production of satisfactory and economical concrete requires the aggregate of low void content, but not negligible void content. The voids in aggregate can be tested using ASTM C29 or American Association of State Highway and Transportation Officials (AASHTO) AASHTO T19.

Aggregate gradation and particle-size distribution for fine and coarse aggregates are determined by sieve analysis as outlined in ASTM C136 and AASHTO T27. The gradation distribution is generally expressed as a percentage of material passing through each sieve. The “Design and Control of Concrete

Mixtures" published by PCA provides the limits for fine aggregate and one size of coarse aggregate as shown in Figure 3.[5]

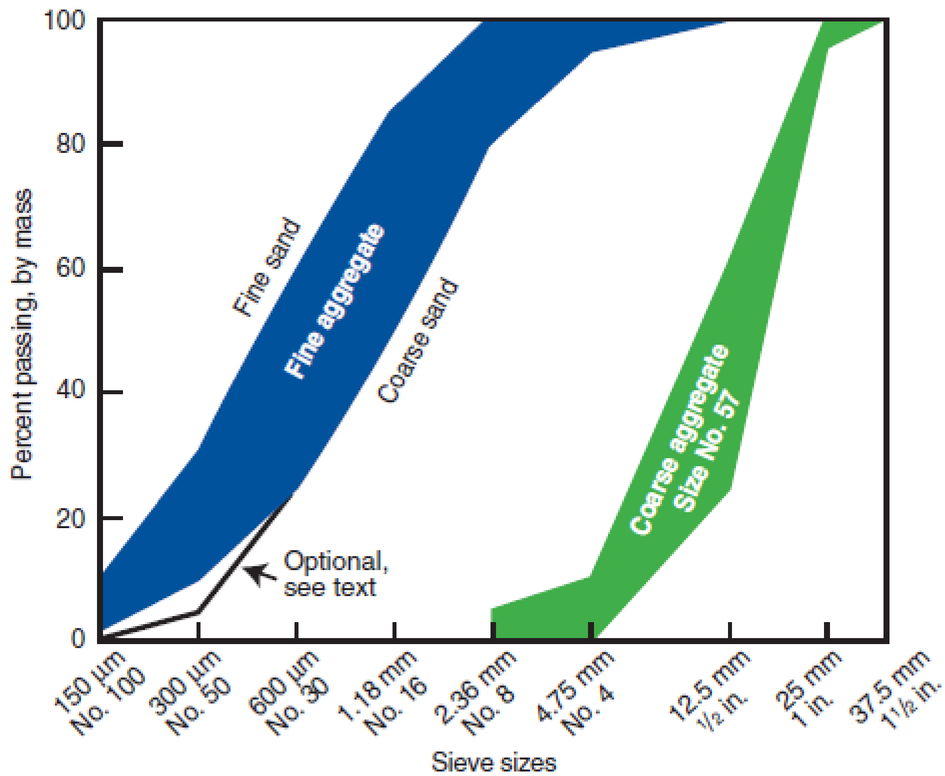


Figure 3. Curves Indicate the Limits Specified in ASTM C33 for Fine Aggregate and for One Commonly Used Size Number (Grading Size) of Coarse Aggregate[5]

The gradation and the maximum aggregate size affect relative aggregate proportions as well as the cement and water demand, workability, pumpability, economy, porosity, shrinkage and durability of concrete. Very often, the aggregate properties affect the amount of mix water. For a workable concrete, there must be enough paste to coat the aggregate particles. However, excessive water in the paste reduces long-term concrete durability by reducing strength and increasing permeability. Therefore, with a constant cement content and constant consistency, there is an optimum for every combination of aggregates that will produce the most effective water-cement ratio and highest strength.[5] In general, smaller aggregate size requires more paste due to high surface to volume ratio. That is why in many specifications, the amount of fine aggregate is limited. In an optimum mixture, the particle interface is minimized which results in better response to high frequency, high amplitude vibrators.

The uniformity of concrete is highly dependent on the variation of coarse and fine aggregate. In general, very fine aggregate requires more cement and water resulting in uncontrolled cracking and very coarse sand and coarse aggregate produces unworkable concrete. Aggregate with no significant deficiency or excess of any size provides a well distributed and smooth gradation distribution. A well graded aggregate provides a workable concrete with satisfactory performance, including reduced shrinkage and permeability.

Aggregate used in concrete must be clean, hard, strong, durable, and free from any kind of absorbed chemical, clay coating, and any other fine material. Before mixing with paste, aggregates are often washed. It would be helpful to know the service record of an aggregate, in particular, its potential for ASR. While service record is an indicator, it may not be an accurate one and in most cases, ASR tests are necessary to predict its potential for reactivity. A good habit to develop is to prepare trial batches of concrete using the project aggregate for a specific project to establish the final characteristics of the concrete and if necessary make adjustments to produce workable and durable concrete.

Fine Aggregate

Many PCC pavement specifications require the sand gradation to meet ASTM C33 or AASHTO M6/M43. These specifications permit a wide range in fine aggregate gradation. In general, the most desirable aggregate gradation depends on the type of the work, richness of the mixture, and the maximum size of the aggregate. In general, for constant water-cement ratio with a correct proportion of fine and coarse aggregate, a wide range in gradation can be used without a significant impact on the strength. By adjusting the concrete mixture to suit the gradation of the local aggregate, the best economy can be achieved.

Fine aggregate grading as specified by ASTM C33 (AASHTO M6) is generally satisfactory for most concrete. The ASTM C33 (AASHTO M6) limits for fine aggregate with respect to sieve sizes are given in Table 1.

Table 1. Fine Aggregate Grading Limits (ASTM C33/AASHTO M6)[5]

Sieve Size	Percent Passing by Mass
9.5 mm (⅜ in.)	100
4.75 mm (No. 4)	95 -- 100
2.36 mm (No. 8)	80 – 100
1.18 mm (No. 16)	50 – 85
600 µm (No. 30)	25 – 60
300 µm (No. 50)	5 – 30 (AASHTO 10 – 30)
150 µm (No. 100)	0 – 10 (AASHTO 2 – 10)

The ASTM C33 provides upper and lower limits for the percentage of material passing/retained on sieves from ⅜ in. to No. 100 (9.5 mm to 150 µm). When applied indiscriminately, ASTM C33 requirements may increase the potential for uncontrolled cracking. In general, for a concrete with high cement factor, coarse sand is required. ASTM C33 or the AASHTO specifications permit the minimum percentage (by mass) of material passing the 300 µm (No. 50) and 150 µm (No. 100) sieves to be reduced to 5 percent and 0 percent respectively, provided:[5]

1. The aggregate is used in air-entrained concrete containing more than 400 lb of cement/yd^3 and having air content more than 3 percent.

2. The aggregate is used in concrete containing more than 500 lb of cement/yd^3 when the concrete is not air entrained.

3. An approved supplementary cementitious material (SCM) is used to supply the deficiency in material passing through two sieves.

Other requirements of ASTM C33 (AASHTO M6) are:

1. The fine aggregate must not have more than 45 percent retained between any two consecutive standard sieves.

2. The fineness modulus must not be less than 2.3 nor more than 3.1, neither may it vary more than 0.2 from typical value of the aggregate source. If this value is exceeded, the fine aggregate should be rejected unless suitable adjustments are made in proportions of fine and coarse aggregate.

The amount of fine aggregate passing through No. 50 and No. 100 sieves affect the workability, surface texture, air content, and bleeding of concrete. Most specifications allow 5 percent to 30 percent to pass a No. 50 sieve. The lower limit provided by ASTM C33 may be desired for easy placing conditions or when concrete is mechanically finished. For hand-finished concrete with a smooth surface texture, fine aggregate with at least 15 percent passing through the 300 µm (No. 50) sieve and 3 percent or more passing through the 150 µm (No. 100) sieve should be used.

The upper limit of ASTM C33 is more suitable for masonry mixtures. The sub-300 µm portion of fine aggregate directly influences the water requirement and therefore increases the potential for uncontrolled cracking when used in pavement.

Figures 4 and 5 represent a sand gradation used in an actual project.[7] This gradation increases the potential for uncontrolled cracking which does not meet the requirements of ASTM C33. The extra fine sand requires more water which increases the bulk volume of the concrete, thus promoting cracking.

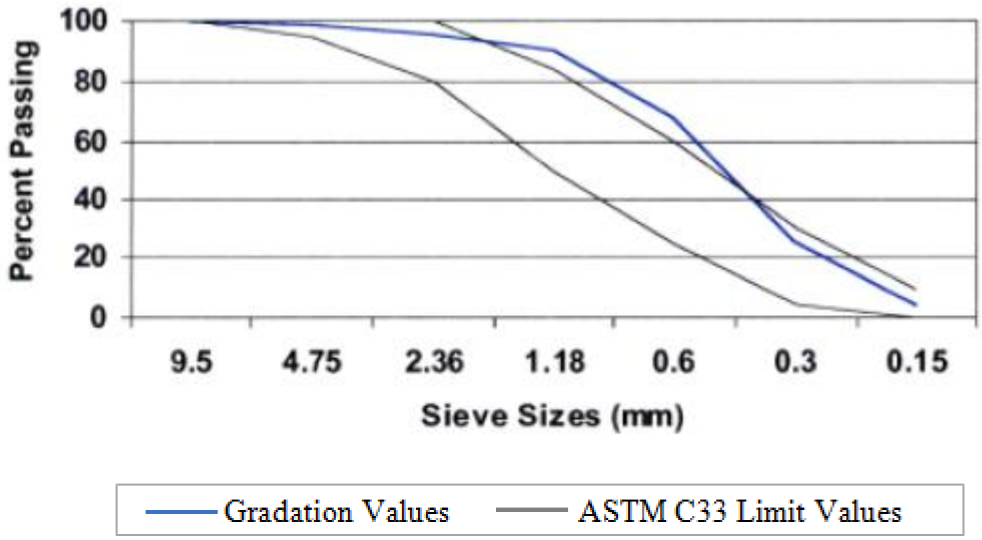


Figure 4. Grading Distribution of Sand that Does Not Meet ASTM C33 Limits and Results in a Mixture Prone to Uncontrolled Cracking(7)

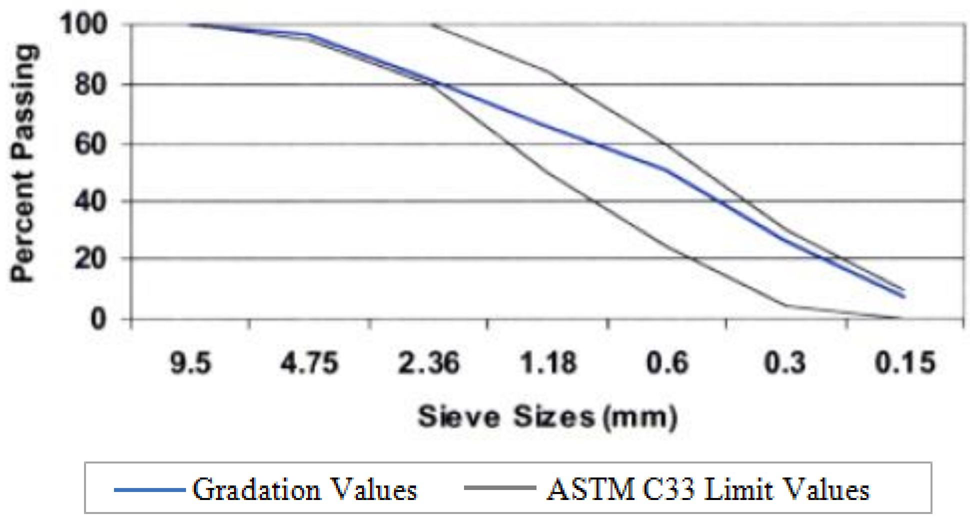


Figure 5. Grading Distribution of Sand with High Bulking Volume that Results in a Mixture Prone to Uncontrolled Cracking(7)

Bulking factor is an increase in volume compared to dry sand (Figure 6). For fine sand, it is more than twice that of coarse sand.(5) The bulking volume directly influences the bulk shrinkage and moisture requirements. The sand in Figure 5 above has a high potential for cracking although it meets the grading limits of ASTM C33. This sand has high bulking volume reflected by nearly 60 percent passing the 1.18 mm (No. 16) sieve. This sand has more than 50 percent of the sand smaller than 600 µm (No. 30) sieve.

Concrete made with this sand will likely have a high bulk volume which increases water demand and potential for uncontrolled cracking.(7)

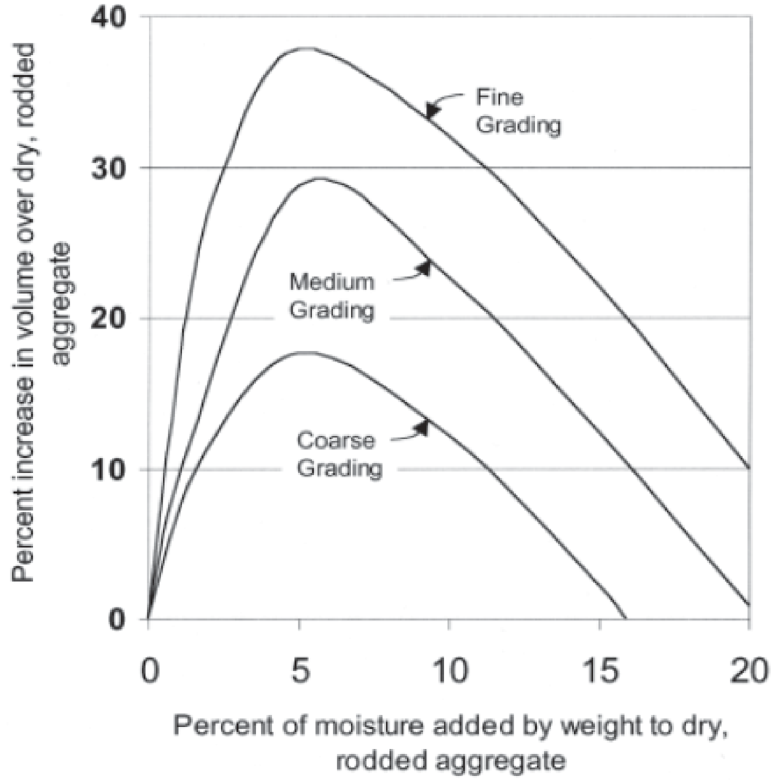


Figure 6. Bulking Volume Increase for Surface Moisture on Graded Sand(7)

The ASTM C33 limits the fineness modulus of sand to a minimum of 3.1. This value is too low for sand used in pavement concrete. Fineness modulus of up to 3.8 can provide excellent results for pavement.(7) Normally, the lower gradation limit of ASTM C33 for sand will have a fineness modulus of 3.45. However, well-graded sand with a fineness modulus of above 3.1 may not be available locally and may require the use of manufactured sands for desired characteristics. The sand gradation shown in Figure 7 is considered acceptable for use in pavement concrete without any concern for shrinkage.

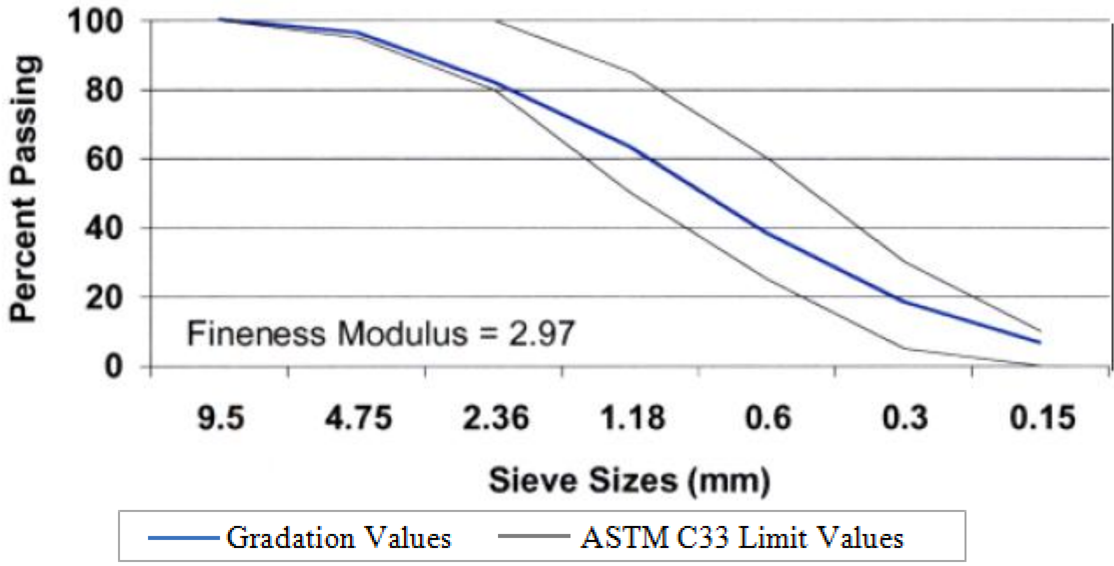


Figure 7. Grading Distribution of Well-graded Sand with Little Potential to Contribute to Uncontrolled Cracking[7]

Note that each interval represents the amount of aggregate passing the sieve size shown.

Coarse Aggregate

ASTM C33 (AASHTO M80) provides a wide range of gradations for coarse aggregate with variety of grading sizes. Table 2 shows the grading requirements for coarse aggregate.[5] As long as the proportion of fine aggregate to total aggregate produces concrete of good workability, the grading for coarse aggregate can be varied moderately without appreciably influencing the water and cement requirements. If a wide variation of coarse aggregate gradation occurs, it may be necessary to change the mix proportion. It is more economical to maintain uniformity in manufacturing and handling coarse aggregate than to reduce variations in gradation.

The maximum size coarse aggregate to be used in a project is normally limited by local availability, maximum fraction of the minimum concrete thickness or reinforcement spacing, and the ability of the equipment to handle the aggregate size. The aggregate size can influence the economy of the concrete. Coarse aggregate size requires less paste and consequently less cement due to smaller surface to volume ratio compared to smaller aggregate size. Figure 8 provides cement and water requirements for different maximum nominal aggregate sizes in concrete with and without entrained air, for slump of approximately3 inches.

Maximum aggregate size also influences the concrete strength for the same water-cement ratio. For a given water-cement ratio, concrete with smaller aggregate size may have higher compressive strength and in particular, this is true for high-strength concrete. For high-strength concrete, the optimum maximum size of coarse aggregate depends on some other factors including the relative strength of cement paste, strength of aggregate particles, and the cement-aggregate bond.

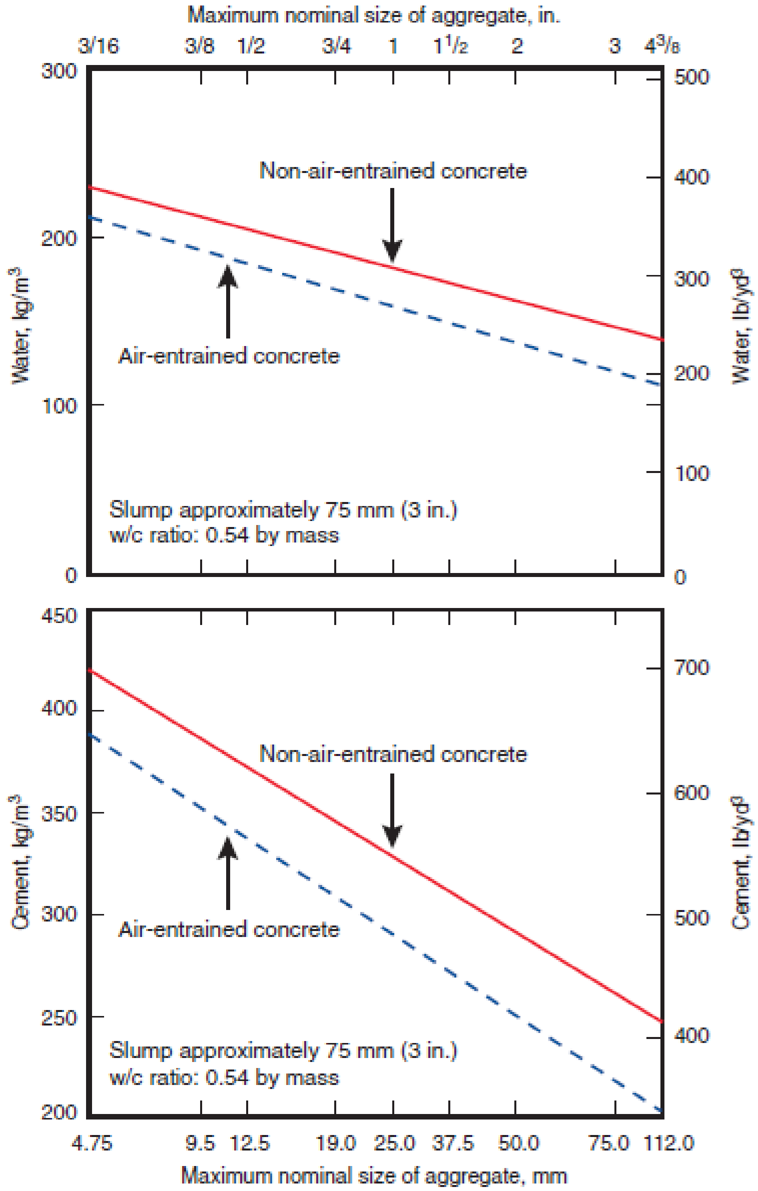


Figure 8. Cement and Water Contents in Relation to Maximum Size of Aggregate for Air-Entrained and Non-Air-Entrained Concrete[5]

Table 2. Grading Requirements for Coarse Aggregate (ASTM C33 and AASHTO M80)[5]

Size number	Nominal size, sieves with square openings	Amounts finer than each laboratory sieve, mass percent passing												
		100 mm (4 in.)	90 mm (3½ in.)	75 mm (3 in.)	63 mm (2½ in.)	50 mm (2 in.)	37.5 mm (1½ in.)	25.0 mm (1 in.)	19.0 mm (¾ in.)	12.5 mm (½ in.)	9.5 mm (⅜ in.)	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)
1	90 to 37.5 mm (3½ to 1½in.)	100	90 to 100	–	25 to 60	–	0 to 15	–	0 to 5	–	–	–	–	–
2	63 to 37.5 mm (2½to 1½in.)	–	–	100	90 to 100	35 to 70	0 to 15	–	0 to 5	–	–	–	–	–
3	50 to 25.0 mm (2 to 1 in.)	–	–	–	100	90 to 100	35 to 70	0 to 15	–	0 to 5	–	–	–	–
357	50 to 4.75 mm (2 in. to No. 4)	–	–	–	100	95 to 100	–	35 to 70	–	10 to 30	–	0 to 5	–	–
4	37.5 to 19.0 mm (1½to ¾ in.)	–	–	–	–	100	90 to 100	20 to 55	0 to 15	–	0 to 5	–	–	–
467	37.5 to 4.75 mm (1½ in. to No. 4)	–	–	–	–	100	95 to 100	–	35 to 70	–	10 to 30	–	–	–
5	25.0 to 12.5 mm (1 to ½in.)	–	–	–	–	–	100	90 to 100	20 to 55	0 to 10	0 to 5	–	–	–
56	25.0 to 9.5 mm (1 to ⅜ in.)	–	–	–	–	–	100	90 to 100	40 to 85	10 to 40	0 to 15	0 to 5	–	–
57	25.0 to 4.75 mm (1 in. to No. 4)	–	–	–	–	–	100	95 to 100	–	25 to 60	–	0 to 10	0 to 5	–
6	19.0 to 9.5 mm (¾ to ⅜ in.)	–	–	–	–	–	–	100	90 to 100	20 to 55	0 to 15	0 to 5	–	–
67	19.0 to 4.75 mm (¾ in. to No. 4)	–	–	–	–	–	–	100	90 to 100	–	25 to 55	0 to 10	0 to 5	–
7	12.5 to 4.75 mm (½ in. to No. 4)	–	–	–	–	–	–	–	100	90 to 100	40 to 70	0 to 15	0 to 5	–
8	9.5 to 2.36 mm (⅜ in. to No. 8)	–	–	–	–	–	–	–	–	100	85 to 100	10 to 30	0 to 10	0 to 5

The type of coarse aggregate influences the temperature sensitivity of concrete and its thermal expansion and contraction. Temperature sensitive concrete will developed uncontrolled cracking. For example, limestone, basalt, and granite have lower coefficient of thermal expansion compared to quartz. In selecting coarse aggregate for use in concrete, the type of aggregate should be included in design calculations and joint spacing requirements.

Another cause of uncontrolled surface cracking in concrete is the use of dirty coarse aggregate. Aggregates with about 8 percent or more passing through the 75 mm (No. 200) sieve are considered dirty. The dust, usually in the form of clay particles, provides additional fines that absorbs moisture from the mixture and decreases workability. Normally, concrete made with dirty aggregate are "gritty" or difficult to finish. In addition, the fines may rise to the surface during consolidation, creating a paste-rich surface which promotes plastic shrinkage and surface cracking. Concrete made with dirty aggregate will have a reduced strength due to the lack of good bond with the paste. A loss of 1,500 psi or more in compressive strength could occur if dusty aggregate is used in concrete.

Aggregate Gradation

In concrete mix designs, since aggregates are chemically and dimensionally more stable than cement paste, it is important to maximize the amount of aggregate. Maximization of the volume of aggregate in concrete is controlled through by optimization of the gradation.

Aggregates constitute most of the volume of the total concrete composition. The two major characteristics of the aggregate which effect the proportioning of the concrete mixtures are grading and nature of the particles as they affect the workability of the concrete. An economical mixture of concrete with the same amount of cementitious material and water can only be obtained by optimizing the gradation of the aggregate.

Aggregate grading influences the water demand, workability, and paste content which in turn impacts the risk of segregation, bleeding, and increased shrinkage of concrete paving mixes. It is always desirable to blend different aggregate sizes to obtain a smooth grading curve for the combined aggregate. There are many ways to optimize the aggregate gradation. A number of numerical and graphical methods are used to optimize gradation. The closer the aggregate gradation to the optimum, the lower is the risk of aggregate related issues as described above.

Gradation

An economical, workable, and denser concrete is obtained by optimizing the gradation. There are many ways to optimize the gradation; among them the popular is the Shilstone method. There are three main tools which help in optimizing the aggregate gradation.

1. The Coarseness/Workability Factor Chart (CFC) provides overview of the mixture.
2. The 0.45 Power Chart shows the trends.
3. The Percent of Aggregate Retained on Each Sieve (PARS) shows the detail.

Coarseness/Workability Chart

From the mathematically combined gradation, the coarseness and workability factors are calculated and then plotted on the CFC.

The coarseness factor is defined as given by Figure 9.

$$\text{Coarseness Factor} = \frac{100 \text{ - \% Passing } \tfrac{3}{8} \text{ Sieve}}{100 \text{ - \% Passing No. } 8\,(2.36\text{ mm}) \text{ Sieve}} \times 100$$

Figure 9. Coarseness Factor Equation

Workability factor is the combined percent passing No. 8 (2.36 mm) sieve as defined by Figure 10. The workability factor shall be increased by 2.5 percent for each increase of 94 lb of cement over 564 lb/yd^3.

$$\text{Workability Factor} = \text{\% Passing No. } 8\,(2.36\text{ mm}) \text{ Sieve} + 2.5\left(\frac{\text{Cement Content}}{94} - 6\right)$$

Figure 10. Workability Factor Equation

Shilstone recommends a target coarseness factor of 60 and a workability factor of 35. Shilstone also recommends that, for aggregate size of 1 to 1½ in. (25 mm to 37.5 mm), a workability factor of 34 to 38 be employed when the coarseness factor is 52. When the coarseness factor is 68, Shilstone recommends a workability factor of 32 to 36.

The CFC, as shown in Figure 11, consists of 5 zones.

- **Zone I**: Gap-graded and has high potential for segregation.
- **Zone II:** Optimum mixture for nominal maximum size aggregate from 50 mm (2 in.) through 19 mm (¾ in.).
- **Zone III:** Optimum mixture for nominal maximum size aggregate smaller than 19 mm (¾ in.).
- **Zone IV**: Excessive fines (sandy) and has high potential for segregation.
- **Zone V**: Excessive amount of coarse and intermediate (rocky) and is not plastic.

For an optimum gradation of a given aggregate, the point should plot above the control line with workability factor between 28 and 45 and inside the well graded zone with coarseness factor of between 45 and 75 as denoted by Zone II.

The following CFC is obtained from the "Integrated Materials and Construction Practices for Concrete Pavement" (IMCP) published by the Federal Highway Administration (FHWA).[8]

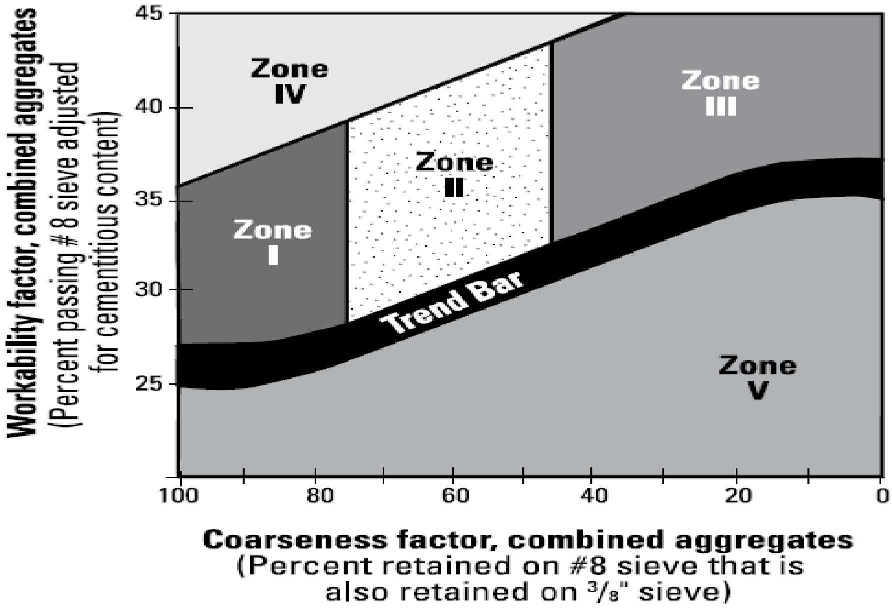


Figure 11. Coarseness/Workability Chart[8]

0.45 Power Chart

The 0.45 Power Chart provides a means to describe an ideal gradation in a mix design. In this chart, the sieve sizes (in microns) raised to 0.45 power are plotted along the x-axis, and the cumulative amount of total aggregate passing each sieve is plotted and compared to a line on the 0.45 Power chart as shown in Figure 12. A well-graded, combined aggregate-in-concrete mix normally follows a trend from the nominal maximum aggregate size to the 2.36 mm (No. 8) sieve and bends downwards. Any deviation from this line indicates a deviation from an optimum aggregate gradation.

In this chart, moving from right to left, any point that falls below the line indicates an excessive amount of aggregate retained on the sieve, and any point that falls above the line indicates the shortage of material retained on the corresponding sieve.

This chart is normally used as a guide for optimizing aggregate gradation and cannot be incorporated into aggregate specifications. An optimum gradation of aggregate normally falls within a few percent of this line.[8]

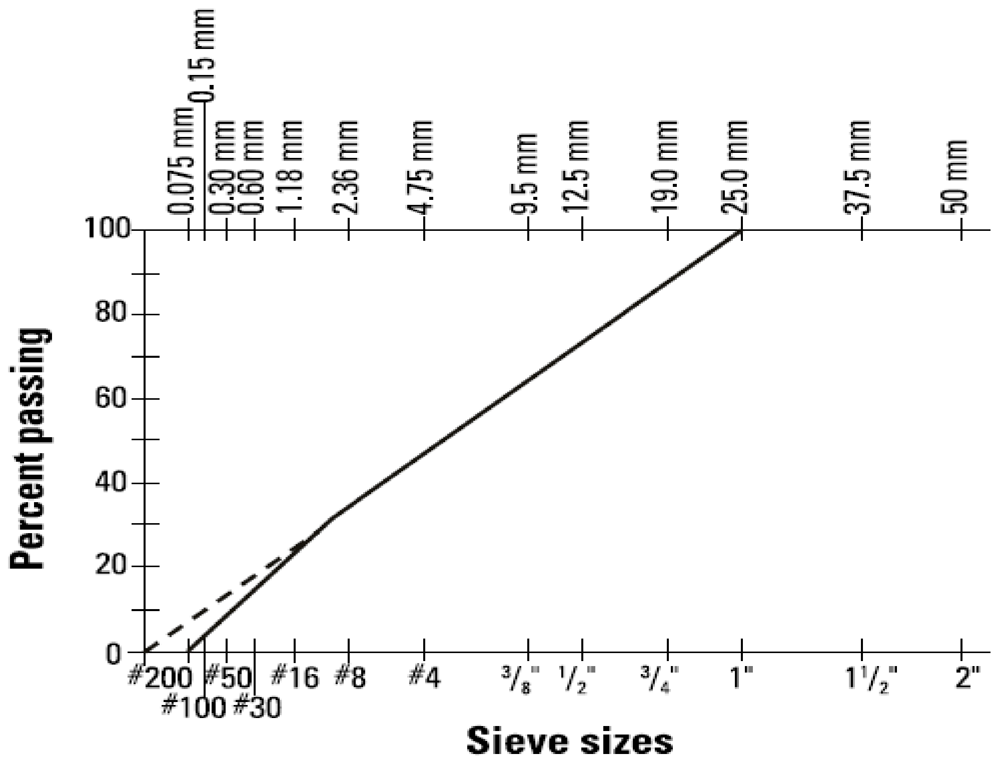


Figure 12. O.45 Power Chart for 25 mm (1 in.) Nominal Maximum Size Aggregate IMCP[8]

The Percent Aggregate Retained Chart

The percent aggregate retained chart provides detailed information on the deficiency in two adjacent sieve sizes. This method was first proposed by Shilstone in 1990.[9] In this chart, the mathematically combined percent retained aggregate particle distribution is plotted with respect to their corresponding sieves. An optimum combined aggregate particle distribution exhibits no gaps in the intermediate particles.

A well-graded aggregate combination will have no significant peaks, while a gap-graded aggregate combination will have significant peaks and dips. Shilstone recommends that the sum of percent retained on two consecutive sieves should be at least 13 percent for an optimum gradation. When there is a deficiency on two adjacent sieve sizes, they tend to balance out each other; however, problems arise when there is a deficiency in three adjacent sizes. The following chart (Figure 13) is borrowed from the ACPA Professors Seminar presented by Dr. Peter Taylor of Iowa State University.[10] In this chart, a well-graded aggregate should fall within the blue lines for all sieves. In using this chart, the goal is to not have too many peaks and valleys.

A smoother curve that is closer to the center of the limits indicates better workability and performance. A rule of thumb in using this chart is that, for a well-graded combined aggregate, the percent retained on each individual sieve falls between 8 percent and 18 percent.

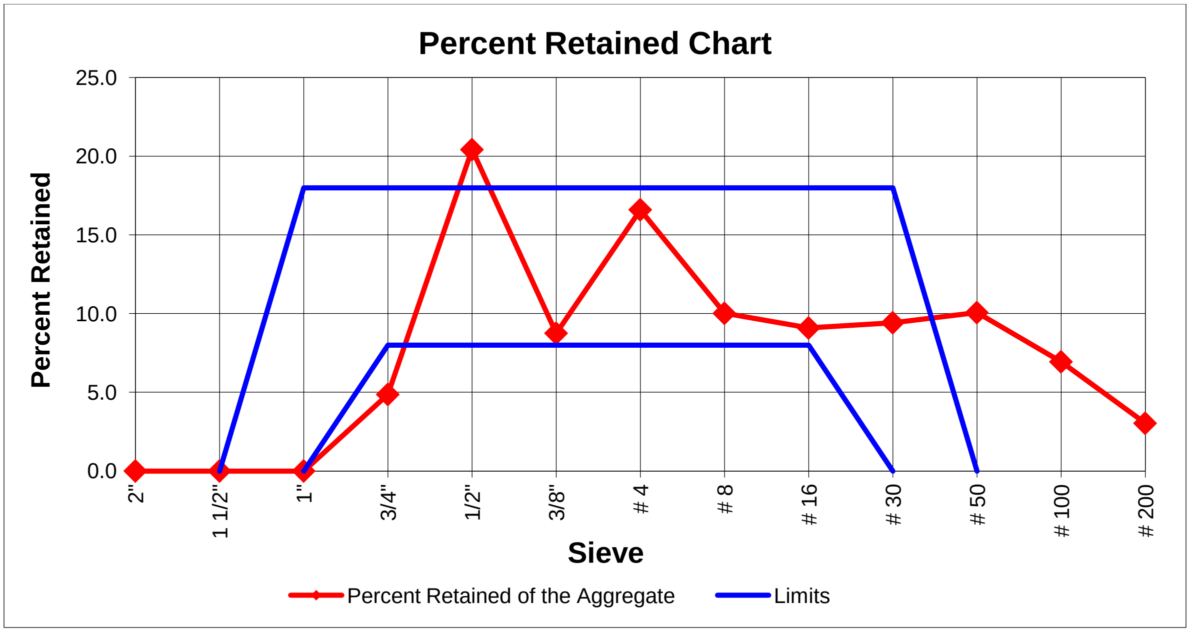


Figure 13. Percent Retained Chart, from Peter Taylor, ACPA Professor's Seminar[10]

Optimum Aggregate Blend

An optimum aggregate blend could be determined by the use of all three charts as well as practical experience. For an intended application, the CFC is utilized to determine the desirability of the aggregate combination for the mixture. The 0.45 Power Chart and the Percent Aggregate Retained Charts are used to verify the results of the CFC and identify deviations from a well-graded aggregate.

ITD Portland Cement Concrete Mix Design

The current ITD specification for PCC used in its projects is evaluated and compared with the similar concrete practices in several other state DOTs including Iowa, Illinois, Texas, Michigan, Washington, and Virginia.[11,12,13,14,15,16] A summary of the ITD PCC specifications along with specifications for several other states are provided in Appendix A.

A comparison of the ITD PCC specifications with other states reveals that the ITD's current practice is in line with those of many other state transportation departments. Based on the result of this comparison and the recommendations made by the Delphi study group, some modifications in ITD PCC specifications could improve the performance and long-term durability of concrete and may reduce the final cost of the product. The recommendations include:

- Reduce the amount of cement by improving the aggregate gradation.
- Avoid the use of gap-graded aggregate in concrete by modifying the current aggregate gradation specifications.

- For ASR mitigation, it is necessary to replace Portland cement in excess of 30 percent by Class F fly ash. More tests will be necessary to determine the effect of excessive Class F fly ash on concrete strength and rate of strength gain.

In addition, it was noted that the current ITD concrete specifications are relatively prescriptive and allows a limited flexibility to the contractor/supplier to design a concrete mix which satisfies ITD's final product specifications. This issue was also observed in the Delphi study where the majority of participants recommended a combination of prescriptive/performance-based mix design in ITD specifications.

ITD basic mix design parameters are presented in Table 3. The specification of 660 lb cement for some concrete classes is more than the cementitious material necessary to meet the performance requirements. The use of excessive cement increases the water demand as dictated by the water-cement ratio, which in turn increases the paste necessary for a dense and stable concrete product. In addition, the excessive paste promotes uncontrolled shrinkage cracking, segregation, and ASR problems.

Table 3. ITD Basic Mix Design Parameter[17]

Concrete Class in 100 psi 28-Day[a]	Minimum Cement Content lb/yd^3 [b]	Maximum Water Cement Ratio	Slump in. (maximum)	Air Content (percent)
45 and greater [c,d,e]	660	0.44	4	0 - 6
35 to less than 45[c,d,e]	560	0.44	4	0 - 6
30	560	0.49	5	6.5 ± 1.5
22	470	0.60	8	0 - 6
15	380	0.60	8	0 - 6
Seal Concrete	660	0.60	8	0 - 6

a. Numerical part of class designation is the specified compressive strength when tested in accordance with applicable test listed in Subsection 502.02 - Materials.

b. It may not always be possible to produce concrete of the specified compressive strength using the minimum cement content. No separate payment will be made for additional cement required to meet the specified compressive strength.

c. Concrete classes designated as "A" will have a slump of 1.5 to 3.5 in. (40 mm to 90 mm) and air content of 6.5 ± 1.5 percent.

d. Concrete classes designated as "B" will have a slump of 5 in. (125 mm) max. and air content of 6.5 ± 1.5 percent.

e. Concrete classes designated as "C" will have a maximum water cement ratio of 0.40, (water reducer required), a slump of 1.5 to 3.5 in. (40 mm to 90mm) and air content of 6.5 ± 1.5 percent.

ITD Specification section 703.2 provides gradation guidelines for fine aggregate to be used in concrete as in Table 4. The comparison of the ITD gradation specifications for fine aggregate with standard gradation provided by the PCA (Table 1) reveals that the ITD specification is too broad and allows a gap-gradation aggregates to pass the acceptance criteria. For example, in a mix design, fine aggregate gradation could pass the ITD approval criteria without having 2.36 mm (No. 8 sieve) aggregate in the mix. This missing aggregate plays an important role in concrete performance.

Section 703.03 of ITD specifications provides guidelines for coarse aggregate intended for use in ITD concrete projects. The comparison of the ITD coarse aggregate gradation in Table 5 with the standard gradation for coarse aggregate provided by PCA (Table 2) shows the same trend of broad-range aggregate for different sizes.

ITD Specification, Section 703.02 for Fine Aggregate in Concrete

Fine aggregate for concrete shall conform to the gradations shown in Table 4.

Table 4. ITD Specification for Fine Aggregate[17]

Sieve Size	Percent Passing
⅜ in. (9.5 mm)	100
No. 4 (4.75 mm)	95-100
No. 16 (1.18 mm)	45-80
No. 50 (0.30 mm)	10-30
No. 100 (0.15 mm)	2-10
No. 200 (0.075 mm)	0-4

For concrete wearing surfaces (pavements, approach slabs, and bridge decks), the percent passing the No. 200 (0.075 mm) sieve size shall be 0-2 except that up to 3 percent passing the No. 200 (0.075 mm) sieve will be accepted if the sand equivalent is at least 80.

ITD Specification, Section 703.03 Coarse Aggregate for Concrete: Coarse aggregate for concrete shall conform to the gradations shown in Table 5.

Table 5. ITD Specification for Coarse Aggregate[17]

SIEVE SIZE	PERCENT PASSING					
	COARSE AGGREGATE SIZE NO.					
	1	2a	2b	3	4	5
2½ in. (63 mm)						100
2 in. (50 mm)					100	95-100
1½ in. (37.5 mm)				100	95-100	
1 in. (25 mm)		100	100	95-100		35-70
¾ in. (19 mm)	100	95-100	80-100		35-70	
½ in. (12.5 mm)	90-100			25-60		10-30
⅜ in. (9.5 mm)	40-70	20-55	10-40		10-30	
No. 4 (4.75 mm)	0-15	0-10	0-4	0-10	0-5	0-5
No. 8 (2.36 mm)	0-5	0-5		0-5		

Size No. 2a or 2b may be provided when coarse aggregate Size No. 2 is required. Sizes 4 and 5 shall be a combination of 2 or more coarse aggregates.

Evaluation of ITD Aggregate Gradation

As mentioned in Sections 2.10.2 and 2.10.3, the 0.45 power Chart and the percent aggregate retained chart provide a means to describe an ideal gradation and detailed information on the deficiency in 2 adjacent sieve sizes. In the following sections, the ITD aggregate sieve analysis results for 4 ITD projects are plotted on the 0.45 power charts and percent passing charts for evaluation. In all four cases, the ITD aggregate gradation displays a deficiency in aggregate sizes and deviates from an optimum aggregate gradation. As compared to sample numbers 1 and 2, the deviation from an optimum aggregate gradation was worse for sample numbers 3 and 4.

Table 6. ITD Aggregate Sieve Analysis Result Sample 1 (Source Le-145–c)

Sieve Size	Sieve Size (in.)	Coarse Agg. % Passing	Fine Agg. % Passing
63 mm	2½ in.	100.0	100.0
50 mm	2 in.	100.0	100.0
37.5 mm	1½ in.	100.0	100.0
25 mm	1 in.	100.0	100.0
19 mm	¾ in.	83.0	100.0
12.5 mm	½ in.	35.0	100.0
9.5 mm	⅜ in.	14.0	100.0
4.75 mm	No. 4	1.0	99.0
2.36 mm	No. 8	0.9	75.0
1.18 mm	No. 16	0.0	58.0
600 µm	No. 30		45.0
300 µm	No. 50		21.0
150 µm	No. 100		5.0

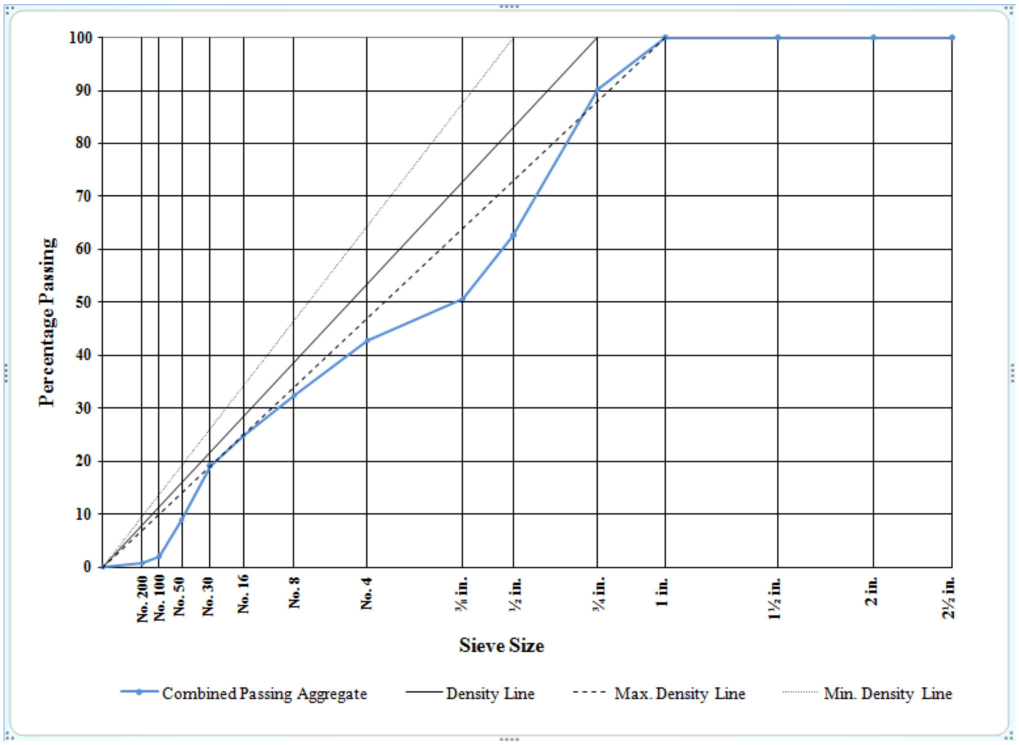


Figure 14. 0.45 Power Chart of the ITD Aggregate Sample 1 (Source Le-145–c)

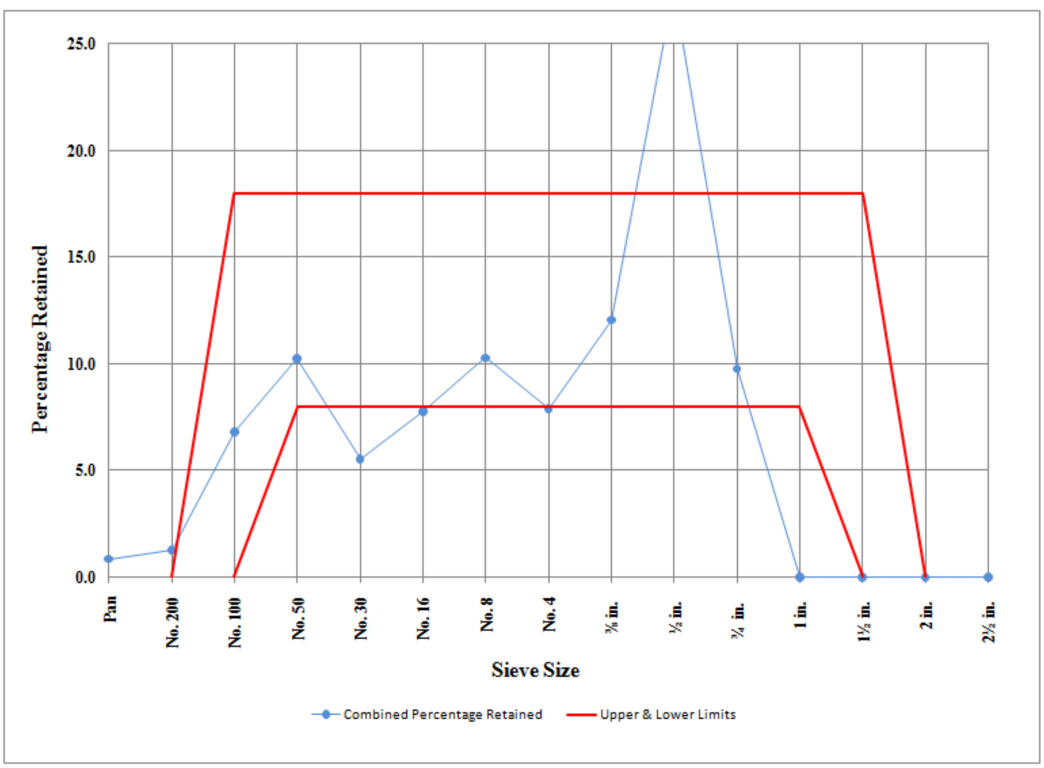


Figure 15. Percentage Retained Chart of the ITD Aggregate Sample 1 (Source Le-145–c)

Table 7. ITD Aggregate Sieve Analysis Result Sample 2 (Project: ST-1786(609))

Sieve Size	Sieve Size (in.)	Coarse Agg. % Passing	Fine Agg. % Passing
63 mm	2½ in.	100.0	100.0
50 mm	2 in.	100.0	100.0
37.5 mm	1½ in.	100.0	100.0
25 mm	1 in.	100.0	100.0
19 mm	¾ in.	83.0	100.0
12.5 mm	½ in.	94.0	100.0
9.5 mm	⅜ in.	67.0	100.0
4.75 mm	No. 4	43.0	97.9
2.36 mm	No. 8	3.7	89.0
1.18 mm	No. 16		80.0
600 µm	No. 30		69.0
300 µm	No. 50		26.0
150 µm	No. 100		4.0
75 µm	No. 200		0.6

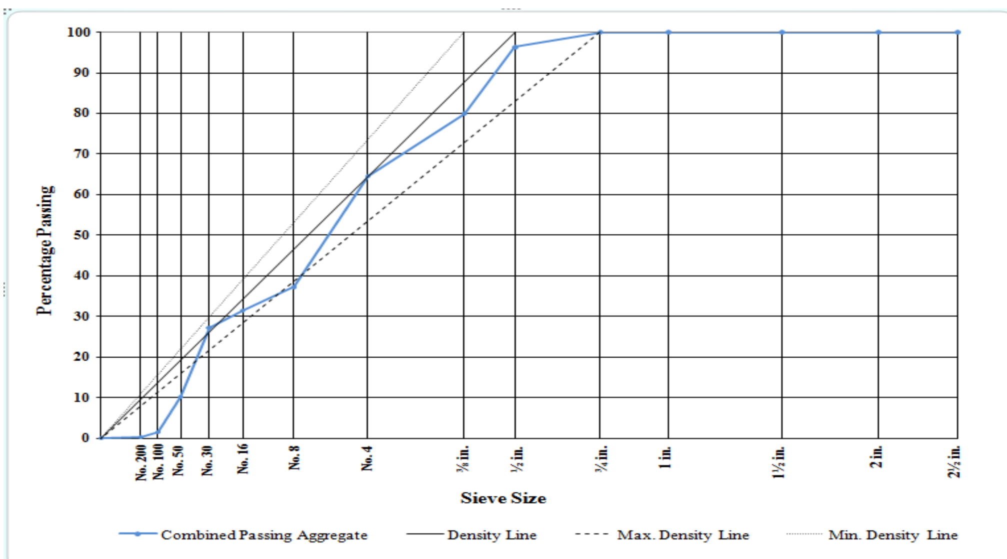


Figure 16. 0.45 Power Chart of the ITD Aggregate Sample 2 (Project: ST-1786(609))

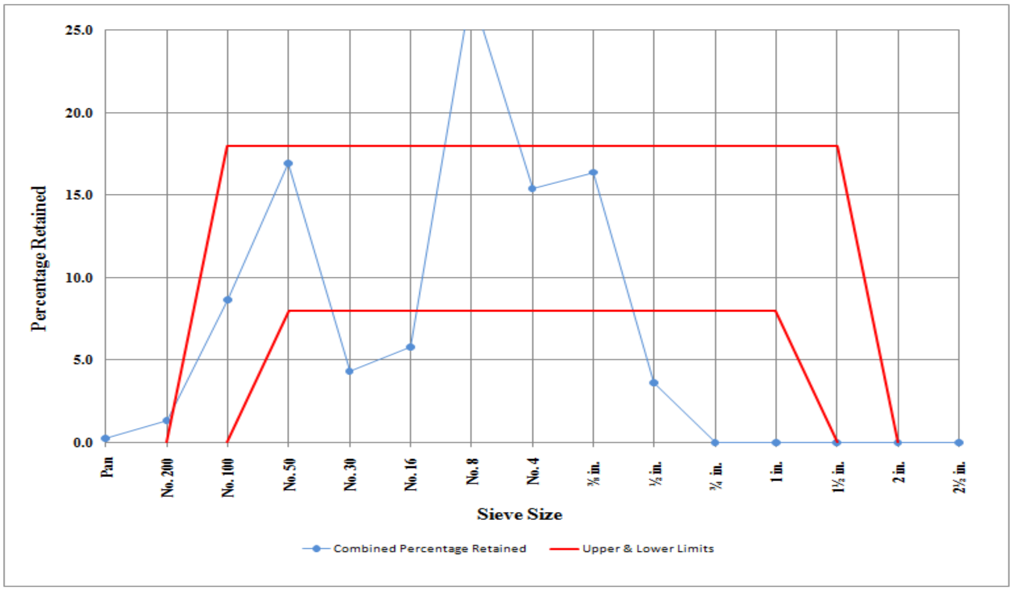


Figure 17. Percentage Retained Chart of the ITD Aggregate Sample 2 (Project: ST-1786 (609))

Table 8. ITD Aggregate Sieve Analysis Result Sample 3 (Project: NH-4110(140))

Sieve Analysis Done on 5/13/05			
Sieve Size	**Sieve Size (in.)**	**Coarse Agg. % Passing**	**Fine Agg. % Passing**
63 mm	2½ in.	100.0	100.0
50 mm	2 in.	100.0	100.0
37.5 mm	1½ in.	100.0	100.0
25 mm	1 in.	100.0	100.0
19 mm	¾ in.	90.0	100.0
12.5 mm	½ in.	47.0	100.0
9.5 mm	⅜ in.	26.0	100.0
4.75 mm	No. 4	4.0	99.0
2.36 mm	No. 8		84.0
1.18 mm	No. 16		59.0
600 µm	No. 30		47.0
300 µm	No. 50		22.0
150 µm	No. 100		4.0
75 µm	No. 200		1.0

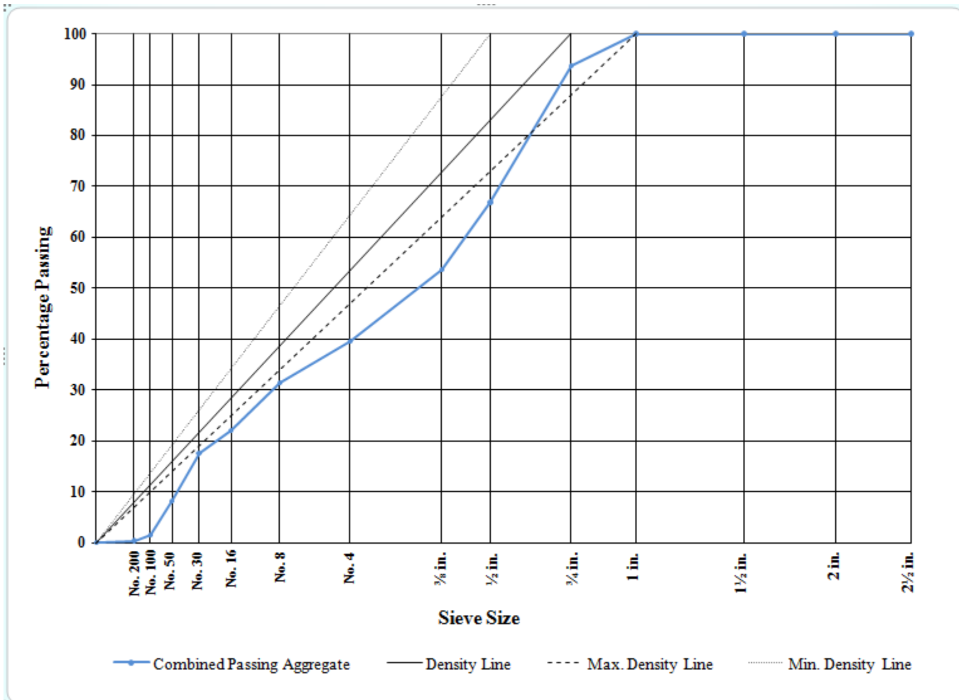


Figure 18. 0.45 Power Chart of the ITD Aggregate Sample 3 (Project: NH-4110(140))

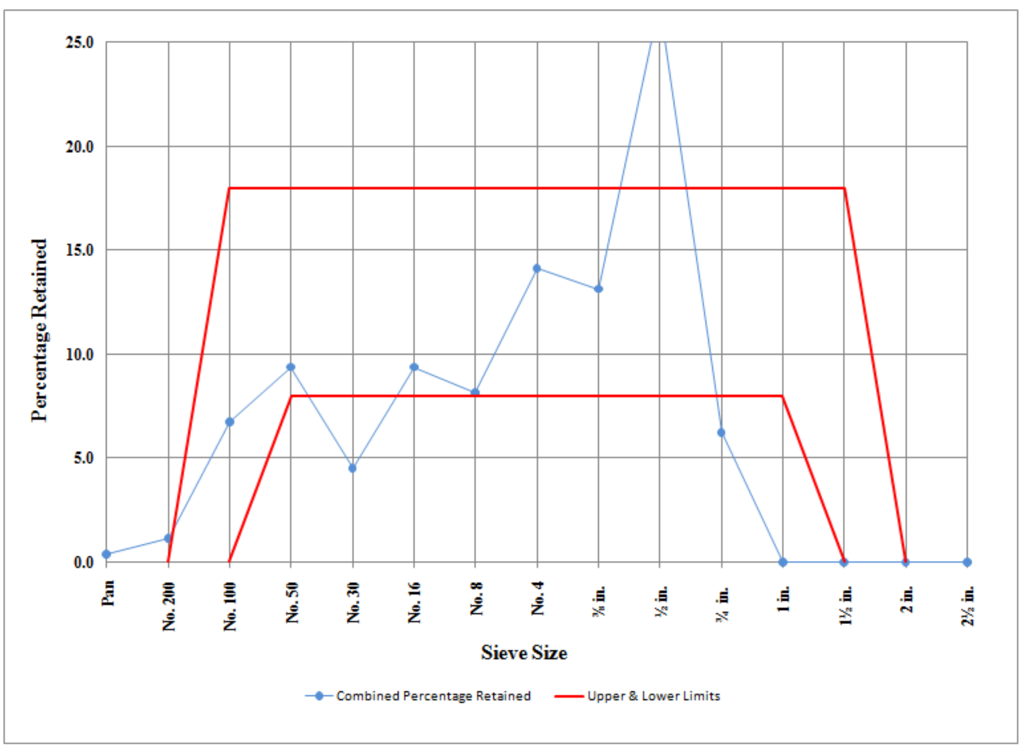


Figure 19. Percentage Retained Chart of the ITD Aggregate Sample 3 (Project: NH-4110(140))

Table 9. ITD Aggregate Sieve Analysis Result Sample 4 (Project: NH-4110(140))

Sieve Analysis Done on 7/7/05			
Sieve Size	Sieve Size (in.)	Coarse Agg. % Passing	Fine Agg. % Passing
63 mm	2½ in.	100.0	100.0
50 mm	2 in.	100.0	100.0
37.5 mm	1½ in.	100.0	100.0
25 mm	1 in.	98.4	100.0
19 mm	¾ in.	88.0	100.0
12.5 mm	½ in.	47.0	100.0
9.5 mm	⅜ in.	29.0	100.0
4.75 mm	No. 4	2.8	99.0
2.36 mm	No. 8		
1.18 mm	No. 16		57.0
600 µm	No. 30		
300 µm	No. 50		23.0
150 µm	No. 100		5.0
75 µm	No. 200		0.4

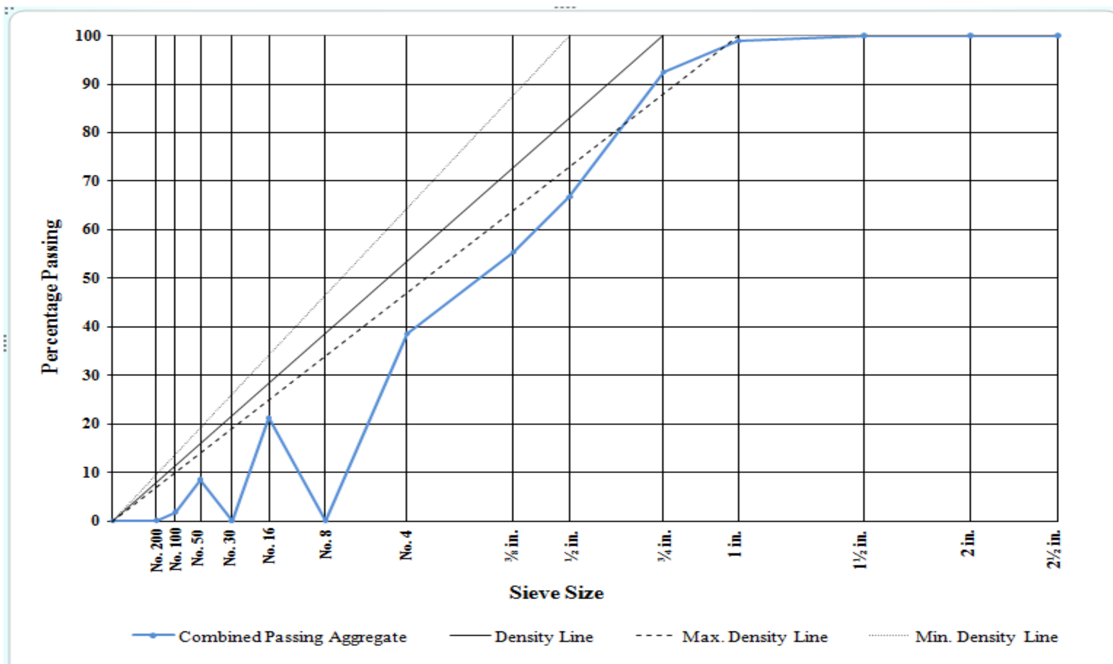


Figure 20. 0.45 Power Chart of the ITD Aggregate Sample 4 (Project: NH-4110(140))

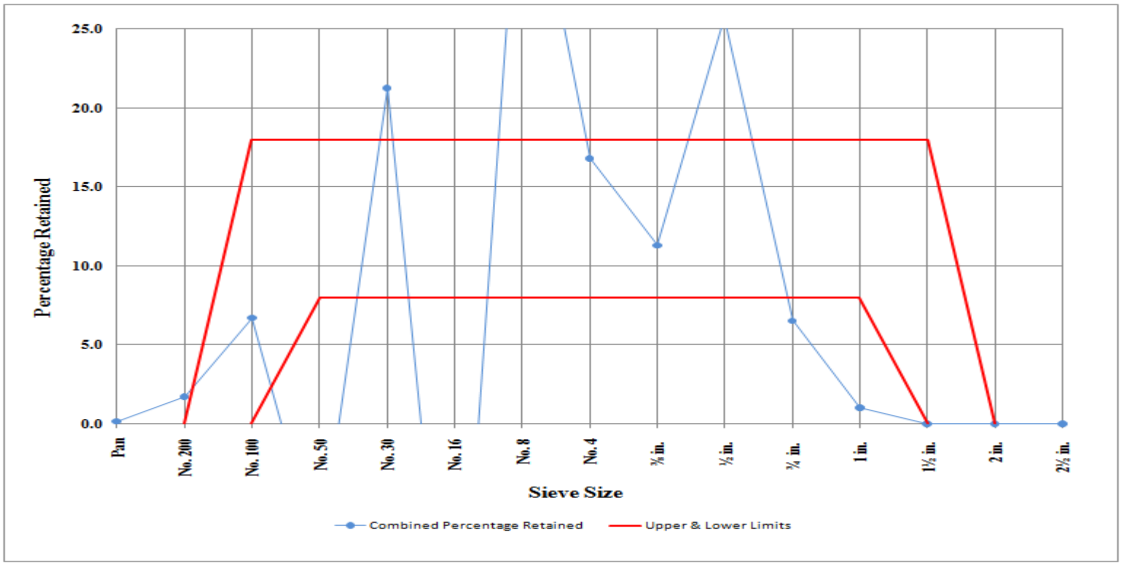


Figure 21. Percentage Retained Chart of the ITD Aggregate Sample 4 (Project: NH-4110(140))

Evaluation of ITD Concrete Mix Design

The concrete mix designs using each of the above mentioned aggregate samples were evaluated using Coarseness/Workability charts as shown below. The plot for each sample was evaluated for coarseness and workability. The plot of the first sample shows that the mix was very close to coarse-gap graded, as the point falls at the edge of region II. Normally, the point for a well graded aggregate falls within the center part of region II. The plots of the second sample indicate that the mix was well graded for minus ¾ in. aggregate, as the point falls within the region III. For samples 3 and 4, the coarseness factors were 44.54 and 165.08 respectively, indicating a sandy mix which falls in the Zone IV and beyond the limits of the chart. As a result these could not be plotted. The evaluation of these few samples from ITD mix design indicates that the ITD mix design may not be consistent. Furthermore, depending on the aggregate gradation, ITD mix design can deliver different coarseness and workability factors. This is in part due to the broad range of ITD specifications for the aggregate gradation that allows approval of aggregate without No. 8 (Gap-graded). Without No. 8 aggregate well-graded aggregate gradation cannot be achieved.

It should be noted that the sample size for the evaluation of ITD aggregate gradation and mix design was small and cannot offer conclusive evidence of deviation from the CFC. However, the evaluation of the plots for the sieve analysis confirms that ITD aggregate gradation specification allows the use of gap-graded aggregate in a mix design.

An example of a well-graded aggregate-mix design was provided by the Iowa DOT. [11] These plots show the consistency of the mix for a project with 76 different sieve analyses and mix designs.

Table 10. Concrete Mix Proportions for Sample 1 (Source: Le-145–c)

Mix Proportions (lb)	
Cement	564
Coarse Aggregate	- -
Fine Aggregate	1,705
Water	1,266
Total	3,809

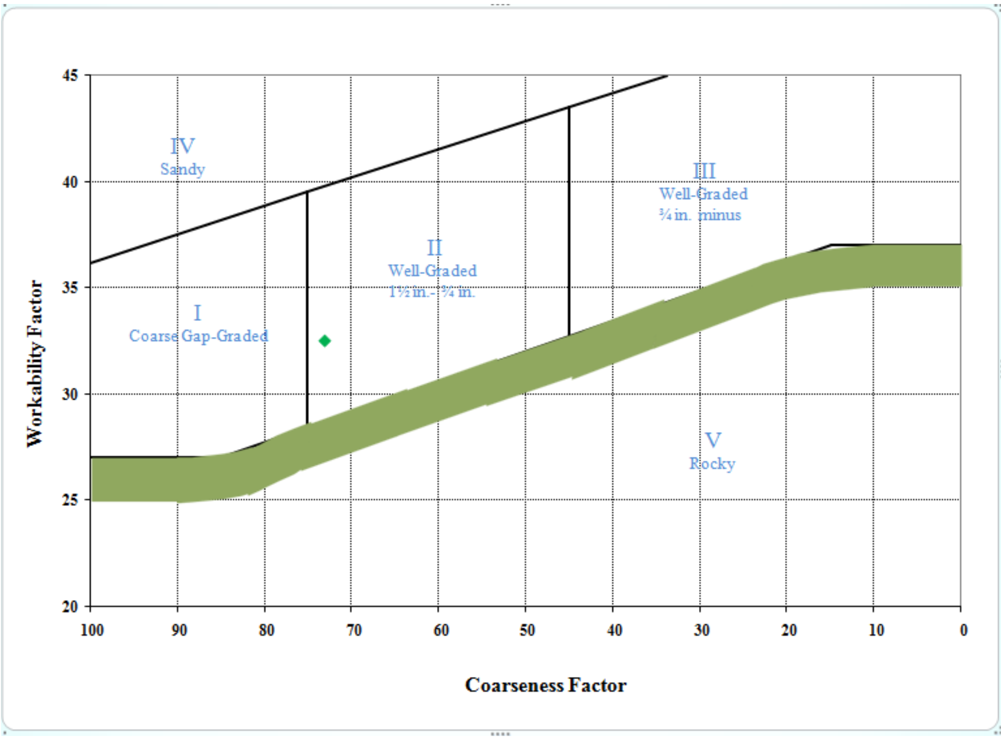


Figure 22. Coarseness Factor Chart of the Sample 1 (Project: Le-145–c)

Table 11. Concrete Mix Proportions for Sample 2 (Project: ST-1786(609), Source No: BL-84c)

Mix Proportions (lb)	
Cement	600
Cementious Material	--
Coarse Aggregate	1,748
Fine Aggregate	1,135
Water	254
Total	**3,737**

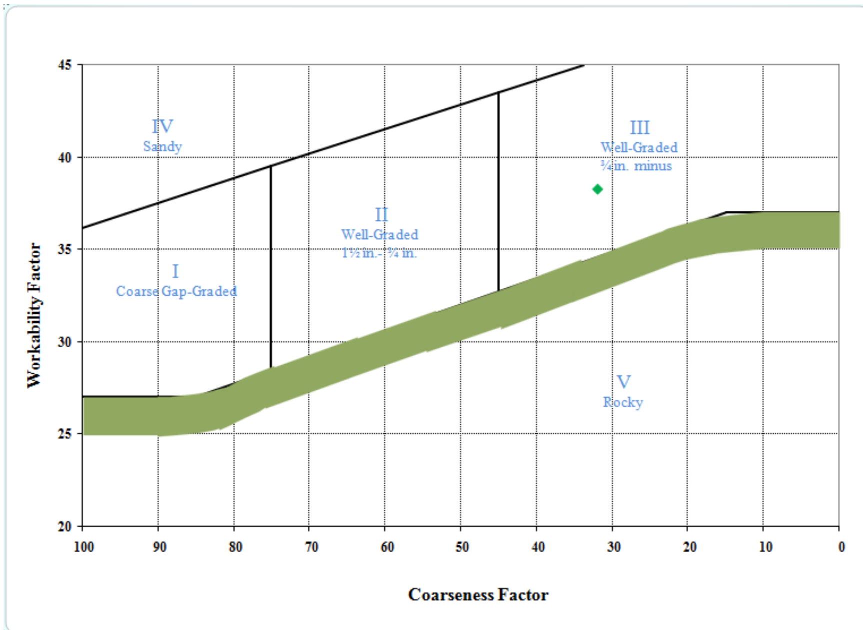


Figure 23. Coarseness Factor Chart of the Sample 2 (Project: ST-1786(609))

Table 12. Concrete Mix Proportions for Sample 3 (Project: NH-4110(140)) 6/23/05 at 0810

Mix Proportions (lb)	
Cement	5,601
Cementious Material	1,122
Coarse Aggregate	17,505
Fine Aggregate	10,500
Water	1,735
Total	**36,463**

Table 13. Concrete Mix Proportions for Sample 4 (Project: NH-4110(140)) 7/14/05 at 0615

Mix Proportions (lb)	
Cement	5,640
Cementious Material	1,131
Coarse Aggregate	17,400
Fine Aggregate	10,335
Water	1,868
Total	**36,374**

Samples from Iowa Department of Transportation and Its Forms

Iowa Department of Transportation (Iowa DOT) uses the Shilstone tools to optimize its aggregate gradation, coarseness and workability. Approval of the mix design depends on these tools. Iowa DOT provides the Excel sheets to the contractor, and the contractor is responsible to enter the results of the sieve analysis and the mix design into the Excel sheet. The plots are then sent to the Iowa DOT materials engineers for approval. In the following sections, the Excel program used by Iowa DOT, the plots for a project with 76 aggregate sieve analysis results, and the concrete mix corresponding to each sieve analysis are provided. In this section, several other charts used by Iowa DOT for the control of other aspects of the mix design are provided as well. These include a separate control chart for coarseness and workability, a size control chart for different aggregate grades, a control chart for plastic air content, a control chart for unit weight for consistency of concrete, a control chart for aggregate moisture content, and a control chart for water-cement ratio.

In these charts, a change in the limit envelops for Samples 68 through 74 were observed. It appears that for those concrete batches, the contractor needed finer aggregate in the mix, and, for approval, the limits of the charts were altered.

Component Blending for PCC Mixes

Sieve Size	% Passing Cement	% Passing Fly Ash	% Passing Mineral Admixture
#200	100.0	100.0	
#325	90.0	88.0	

		A Number	Producer & Location
% Coarse	67	A58002	River Products Columbus Jct
% Fine	33	A29502	Cessford Spring Grove
Crs. Frac.	77		
Int. Frac.	23	A58002	River Products Columbus Jct

Sieve Size	% Passing Coarse	% Passing Intermediate	% Passing Fine	% Passing Paste (dependent on MIX DESIGN)	% Passing (Combined Agg)	% Retained (Combined Agg)	% Passing (Combined Tot) (dependent on MIX DESIGN)	% Retained (Combined Tot)
1 1/2"	100.0	100.0	100.0	100.0	100.0	0.0	100.0	0.0
1"	96.0	100.0	100.0	100.0	97.9	2.1	98.6	1.4
3/4"	76.0	100.0	100.0	100.0	87.6	10.3	91.3	7.2
1/2"	43.0	99.0	100.0	100.0	70.4	17.2	79.3	12.0
3/8"	30.0	94.0	100.0	100.0	63.0	7.5	74.1	5.2
#4	12.0	48.0	95.3	100.0	45.0	17.9	61.6	12.5
#8	6.0	18.0	87.7	100.0	34.8	10.2	54.4	7.1
#16	4.9	14.6	74.5	100.0	29.4	5.4	50.6	3.8
#30	3.9	11.1	44.9	100.0	18.5	10.8	43.1	7.6
#50	2.8	7.7	6.3	100.0	5.4	13.2	33.9	9.2
#100	1.8	4.2	0.5	100.0	1.7	3.7	31.3	2.6
#200	0.7	0.8	0.2	100.0	0.6	1.2	30.5	0.8
#325	-	-	-	96.2	-	-	29.0	1.5
Liquid	-	-	-	63.8	-	-	19.2	9.8
						100.0		100.0

(dependent on MIX DESIGN)

Sieve Size	Coarse % Retained (Individual)	Coarse Weight (lbs)	Intermediate % Retained (Individual)	Intermediate Weight (lbs)	Total Weight (lbs)	Cummulative Weight (lbs)	Fine Weight (lbs)
1 1/2"	0.0	0.0	-	-	0.0	-	-
1"	4.0	9.2	-	-	9.2	-	-
3/4"	20.0	46.0	-	-	46.0	-	-
1/2"	33.0	75.8	1.0	0.7	76.5	131.7	-
3/8"	13.0	29.9	5.0	3.4	33.3	165.0	-
#4	18.0	41.4	46.0	31.6	73.0	237.9	-
#8	6.0	13.8	30.0	20.6	34.4	272.3	-
#16	1.1	2.4	3.4	2.4	4.8	-	-
#30	1.1	2.4	3.4	2.4	4.8	-	-
#50	1.1	2.4	3.4	2.4	4.8	-	-
#100	1.1	2.4	3.4	2.4	4.8	-	-
#200	1.1	2.4	3.4	2.4	4.8	-	-
Pan	0.7	1.6	0.8	0.5	2.2	26.2	-
	100.0	229.8	100.0	68.7	298.5	298.5	152.7

* Coarseness Factor =	56.8	
* Workability Factor =	34.8	
* Adjusted Workabilty =	34.5	(dependent on MIX DESIGN)
Fineness Modulus, coarse =	6.63	
Fineness Modulus, intermiediate =	5.02	Higher F.M. indicates coarser gradation
Fineness Modulus, sand =	2.89	
Fineness Modulus, combined =	5.15	

POWER 45 DATA

Sieve	Size
#200	
#100	1
#50	3
#30	6
#16	11
#8	23
#4	47
3/8	95
1/2	125
3/4	190
1	250
1 1/2	375

CW DATA

56.82	34.8
36	
100	
45	3
75	34.
45	3
75	32.
45	3
75	30.
45	3
75	29.
0	3
10	3
20	3
30	34.
75	28.
81.35	27.
90	
100	
0	3
10	3
20	3
30	32.
75	26.
81.35	25.
90	
100	
45	40.
75	36.
45	42.
75	3
72	2
72	3
75	26.
75	39.
45	43.73
45	30.87

Info | Gradation | % Retained | Power | CW | 955QMC | Mix Design | Actual Design

Figure 24. Sieve Analysis Results, Iowa DOT's Project

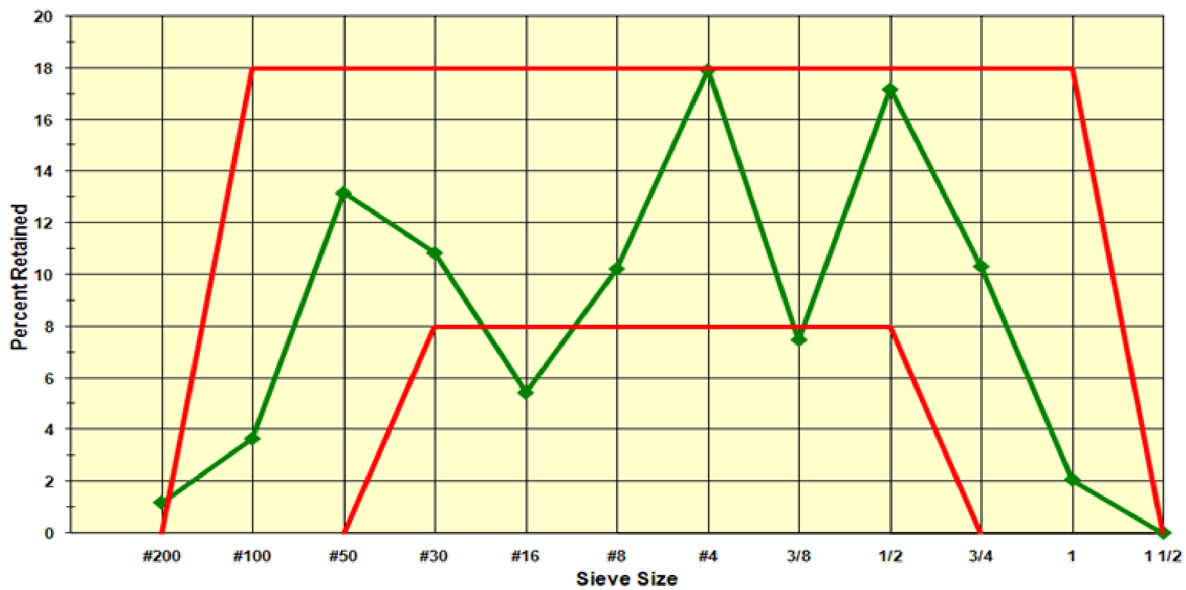


Figure 25. Combined Aggregate Gradation Percentage Retained Chart Iowa DOT's Project. The red lines indicate the upper and lower limits.

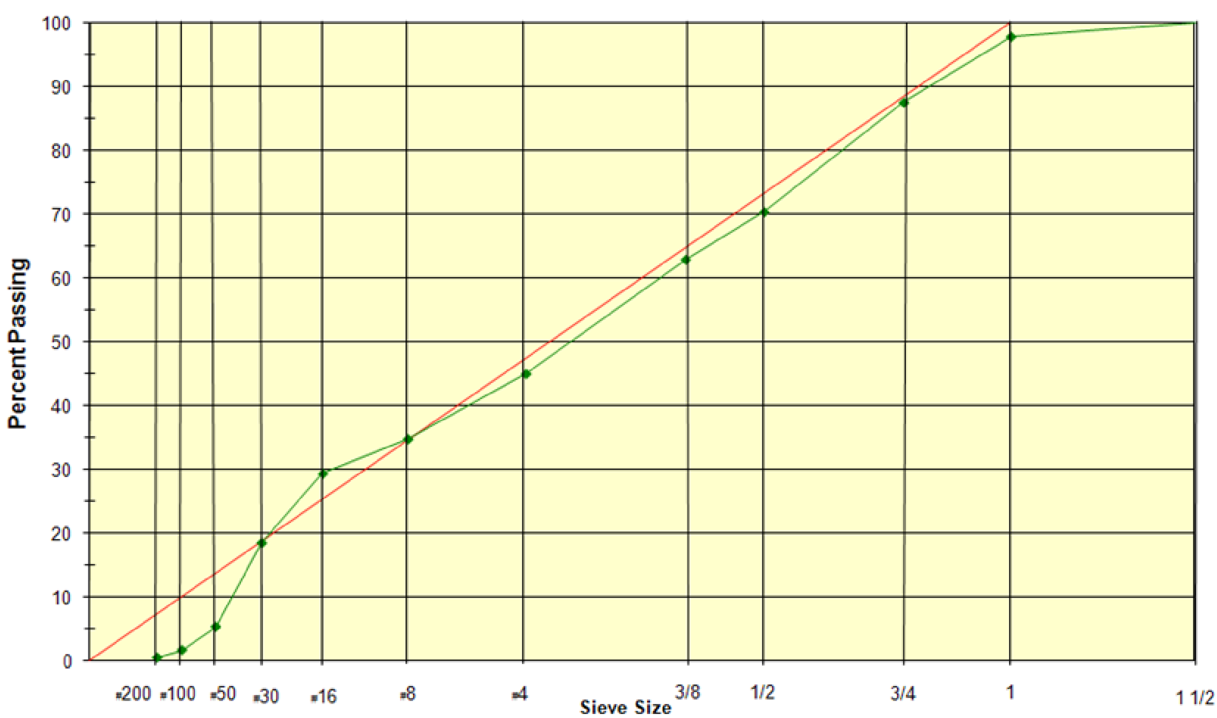


Figure 26. Combined Aggregate Gradation Power 45 Scale Chart, Iowa DOT's Project

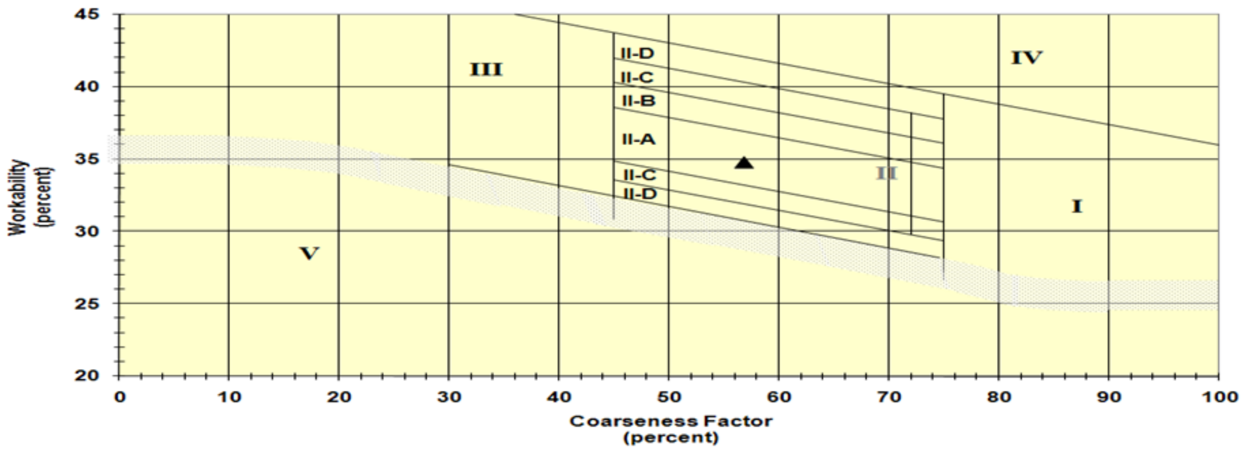
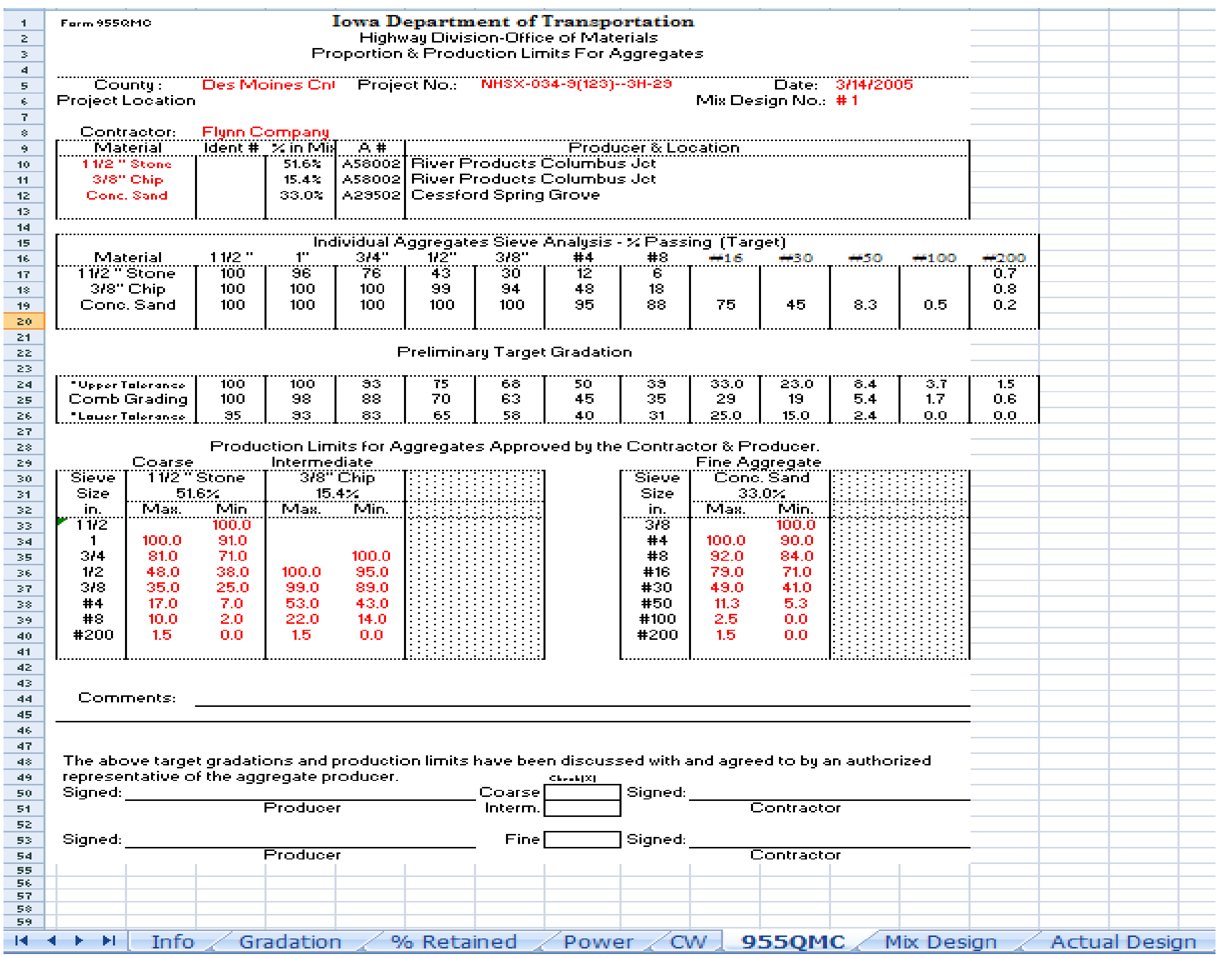


Figure 27. Coarseness Factor Chart, Iowa DOT's Project



Form 955QMC

Iowa Department of Transportation
Highway Division-Office of Materials
Proportion & Production Limits For Aggregates

County : Des Moines Cnt   Project No.: NHSX-034-9(123)--3H-29   Date: 3/14/2005
Project Location   Mix Design No.: # 1

Contractor: Flynn Company

Material	Ident #	% in Mix	A #	Producer & Location
1 1/2 " Stone		51.6%	A58002	River Products Columbus Jct
3/8" Chip		15.4%	A58002	River Products Columbus Jct
Conc. Sand		33.0%	A29502	Cessford Spring Grove

Individual Aggregates Sieve Analysis - % Passing (Target)

Material	1 1/2 "	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
1 1/2 " Stone	100	96	76	43	30	12	6					0.7
3/8" Chip	100	100	100	99	94	48	18					0.8
Conc. Sand	100	100	100	100	100	95	88	75	45	8.3	0.5	0.2

Preliminary Target Gradation

*Upper Tolerance	100	100	93	75	68	50	39	33.0	23.0	8.4	3.7	1.5
Comb Grading	100	98	88	70	63	45	35	29	19	5.4	1.7	0.6
*Lower Tolerance	95	93	83	65	58	40	31	25.0	15.0	2.4	0.0	0.0

Production Limits for Aggregates Approved by the Contractor & Producer.

Sieve Size in.	Coarse 1 1/2 " Stone 51.6% Max.	Min	Intermediate 3/8" Chip 15.4% Max.	Min.
1 1/2		100.0		
1	100.0	91.0		
3/4	81.0	71.0		100.0
1/2	48.0	38.0	100.0	95.0
3/8	35.0	25.0	99.0	89.0
#4	17.0	7.0	53.0	43.0
#8	10.0	2.0	22.0	14.0
#200	1.5	0.0	1.5	0.0

Sieve Size in.	Fine Aggregate Conc. Sand 33.0% Max.	Min.
3/8		100.0
#4	100.0	90.0
#8	92.0	84.0
#16	79.0	71.0
#30	49.0	41.0
#50	11.3	5.3
#100	2.5	0.0
#200	1.5	0.0

Comments:

The above target gradations and production limits have been discussed with and agreed to by an authorized representative of the aggregate producer.

Signed: ____________ Producer   Coarse [ ] Interm. [ ]   Signed: ____________ Contractor

Signed: ____________ Producer   Fine [ ]   Signed: ____________ Contractor

Info | Gradation | % Retained | Power | CW | 955QMC | Mix Design | Actual Design

Figure 28. Sieve Analysis Results Submitted for Approval, Iowa DOT's Project

Total Cementitious Material *(IN LBS)*

Combined Target Gradation

Sieve	Target	Toleranc	Limits (+)	Limits (-)
1 1/2"	100	5	n/a	n/a
1"	98	5	100.0	93.0
3/4"	88	5	93.0	83.0
1/2"	70	5	75.0	65.0
3/8"	63	5	68.0	58.0
#4	45	5	50.0	40.0
#8	35	4	39.0	31.0
#16	29	4	33.0	25.0
#30	19	4	23.0	15.0
#50	5.4	3	8.4	2.4
#100	1.7	2	3.7	0.0
#200	0.6	Max 1.5	2.0	0.0

Print

FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE
FALSE

Combined Percent Passing

Date	05/10/05	05/11/05	05/12/05	05/16/05	05/16/05	05/17/05	05/17/05	05/18/05	05/19/05	05/19/05	05/20/05	05/23/05	05/23/05	7/20/2005	7/22/2005	7/25/2005	7/27/2005	7/28/2005
Test #	1	2	3	4	5	6	7	8	9	10	11	12	13	73	74	75	76	77
1 1/2"	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
1"	96.4	97.1	97.3	97.5	97.9	97.9	98	96.8	97.8	97.7	98.1	97.9	96.2	98.1	97.9	98.5	97.4	97.3
3/4"	84.4	81.7	85.4	88	87.8	88.3	88.2	84.8	87	88	87.9	89	84.9	85.3	85.9	86	86.4	86.8
1/2"	64	62.5	68.4	71.3	70.4	72.2	71.4	67.7	70	71	71.7	71.9	69	69.6	70.4	70.5	71.1	71.3
3/8"	56	53.9	58.5	64	62.6	64.1	62.6	59.6	61.6	61.7	62.5	62.7	60.3	61.9	63.3	63.4	62	62.8
#4	41	39.2	41.4	45	44.9	45.6	44.7	43.6	45.1	44.7	44.4	44.3	44	44.9	46	45.3	43.9	45.4
#8	32.3	31.3	32.4	34.3	34.3	33.8	33.5	34.5	34.3	35	35.2	34.9	35.4	35.6	35.1	35.2	34	35.4
#16	27	26.2	27.1	29	28.6	27.8	28.2	29	28.5	30	29.8	29.5	30.4	30	29.8	30	30	31.3
#30	17.1	15.8	16.4	17.5	17.5	16.9	17.3	17.6	16.9	19	17.8	18.9	18.6	18.8	18.1	18.5	18.5	19
#50	6.0	5.3	6.0	4.6	4.8	4.7	4.8	4.9	4.5	5.5	5.6	6.0	4.9	6.5	6.0	6.2	5.9	7.1
#100	2.1	1.8	2.1	1.5	1.5	1.6	1.7	1.7	1.7	1.8	2.0	1.9	1.9	2.3	2.2	2.3	2.3	2.2
#200	1.6	1.4	1.5	0.9	1.0	1.1	1.1	1.2	1.2	1.2	1.4	1.4	1.4	1.6	1.7	1.6	1.7	1.7
Coarseness Factor	65.0	67.1	61.4	54.8	56.9	54.2	56.2	61.7	58.4	58.9	57.9	57.3	61.5	59.2	56.5	56.5	57.6	57.6
Workability Factor	32.3	31.3	32.4	34.3	34.3	33.8	33.5	34.5	34.3	35.0	35.2	34.9	35.4	35.6	35.1	35.2	34.0	35.4
Zone Compliance of Individual Gradation Test	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
utive Gradation Tests in Noncompliant Zones																		

Project Units (English = E, Metric = M) =	M
Total Noncompliant Volume of Concrete =	0
Gradation Testing Frequency =	1200
Gradation Tests Performed =	75
Gradation Tests Required Based on Volume of Concrete Placed =	67
Average Coarseness Factor =	57.5
Average Workability Factor =	34.8
Zone for Compliant Gradation Tests =	II-A
Air Tests Taken	291
Air Tests Required	292

Gradation / CW Average / CW Time / 1" / 3|4" / 1|2" / 3|8" / #4 / #8 / #16 / #30 / #50 / #100 / #200 / Air / Air1 / Air2 / Ai 3 / Air4 / Air5 / Unit1 / Unit2

Figure 29. Sieve Analysis Results Submitted by the Contractor for an Entire Project, Iowa DOT's project (Representative Control Chart)

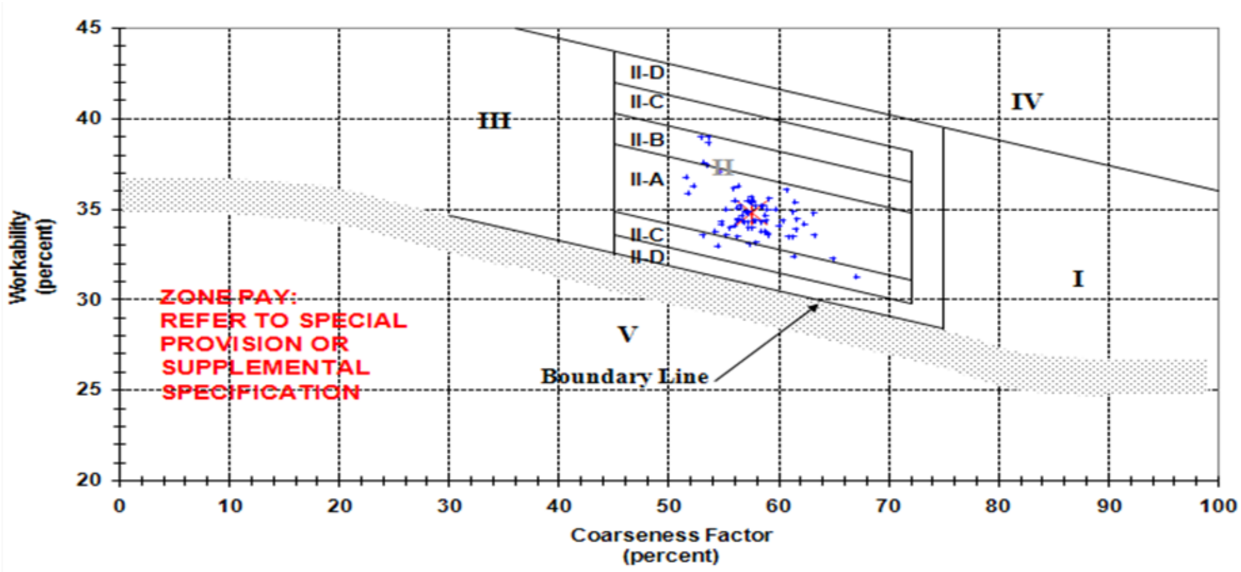


Figure 30. Average Project Workability Factor vs. Coarseness Factor, Iowa DOT's Project

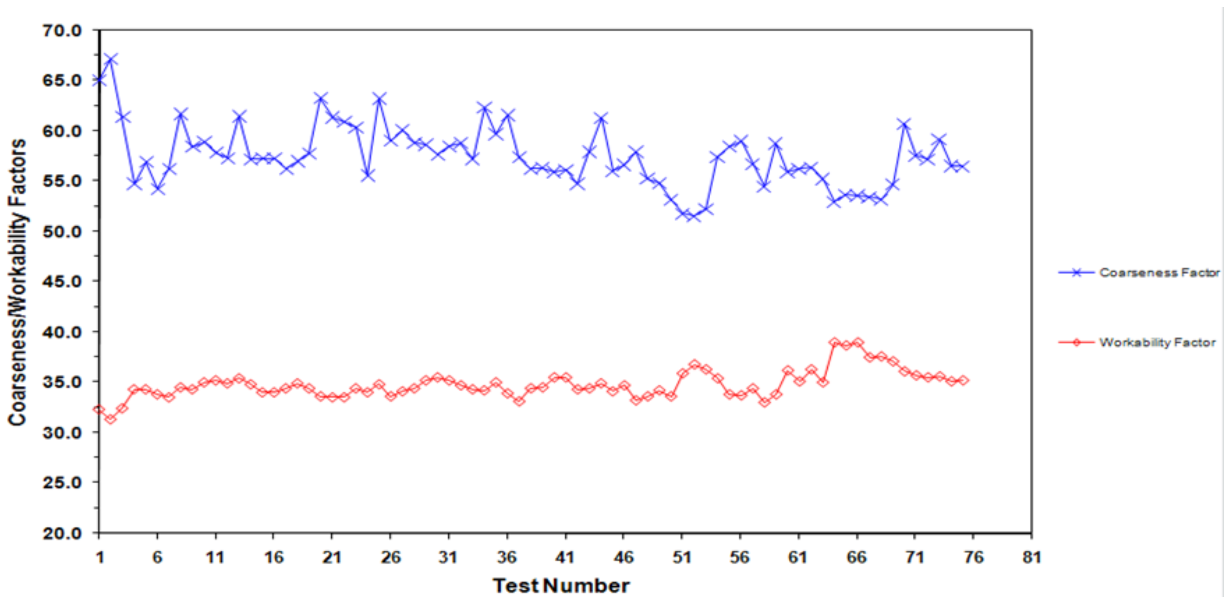


Figure 31. Control Chart for Coarseness and Workability Factors, Iowa DOT's Project

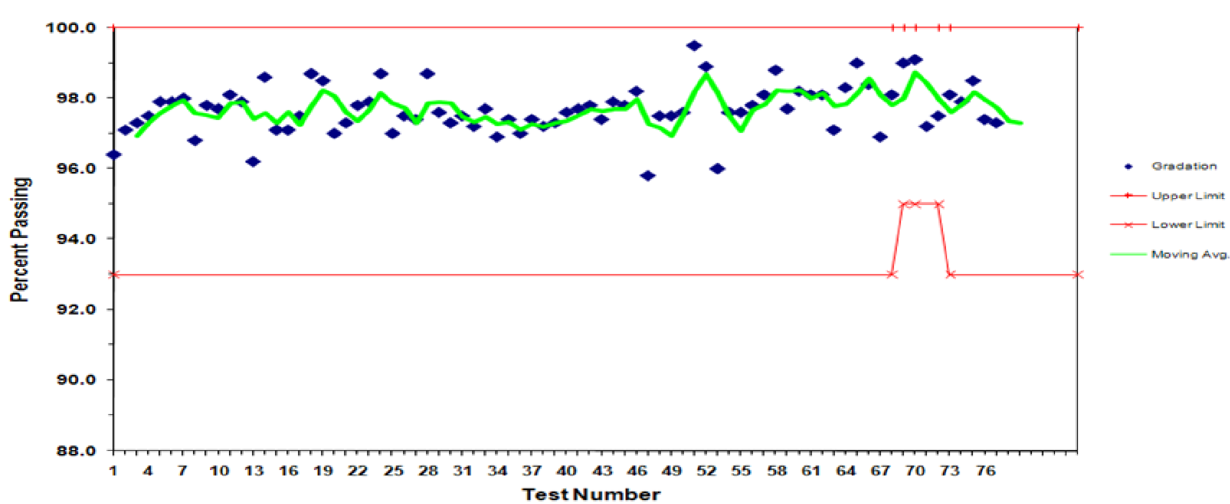


Figure 32. Control Chart for 1 in. Sieve, Iowa DOT's Project

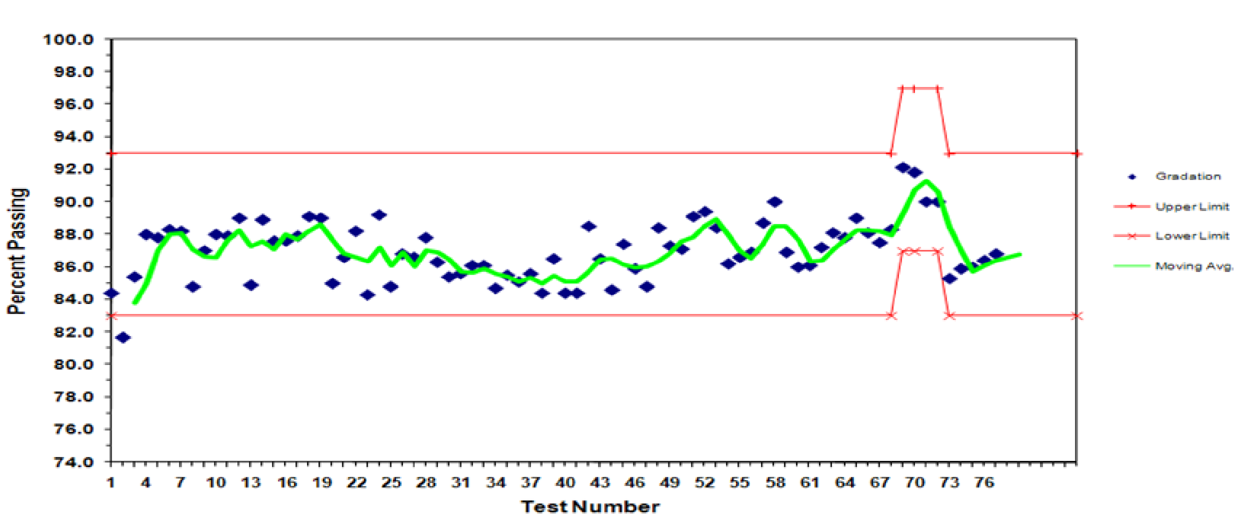


Figure 33. Control Chart for ¾ in. Sieve, Iowa DOT's Project

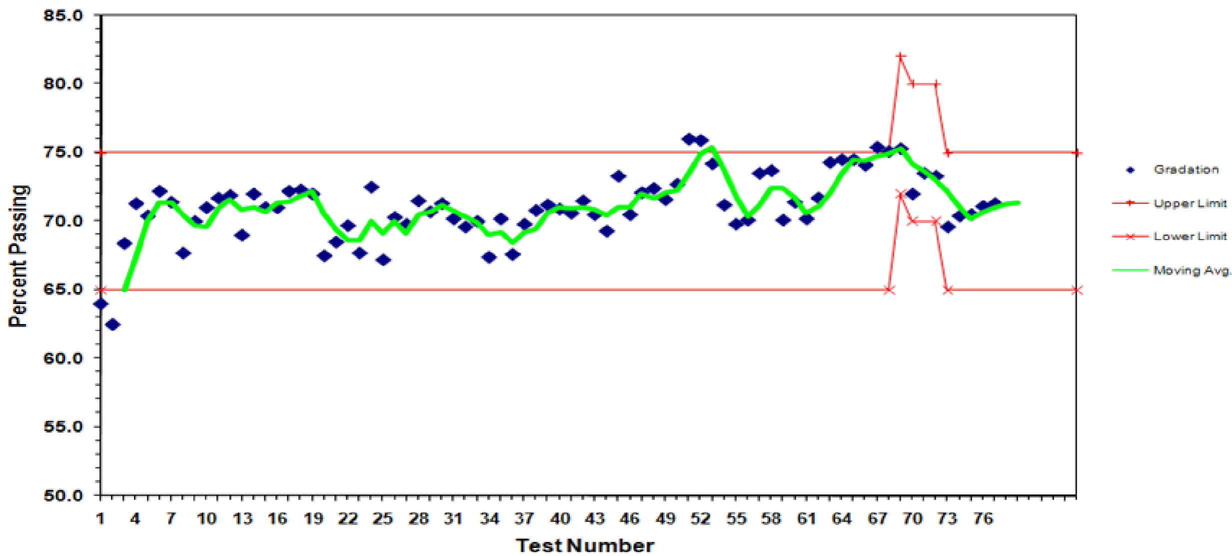


Figure 34. Control Chart for ½ in. Sieve, Iowa DOT's Project

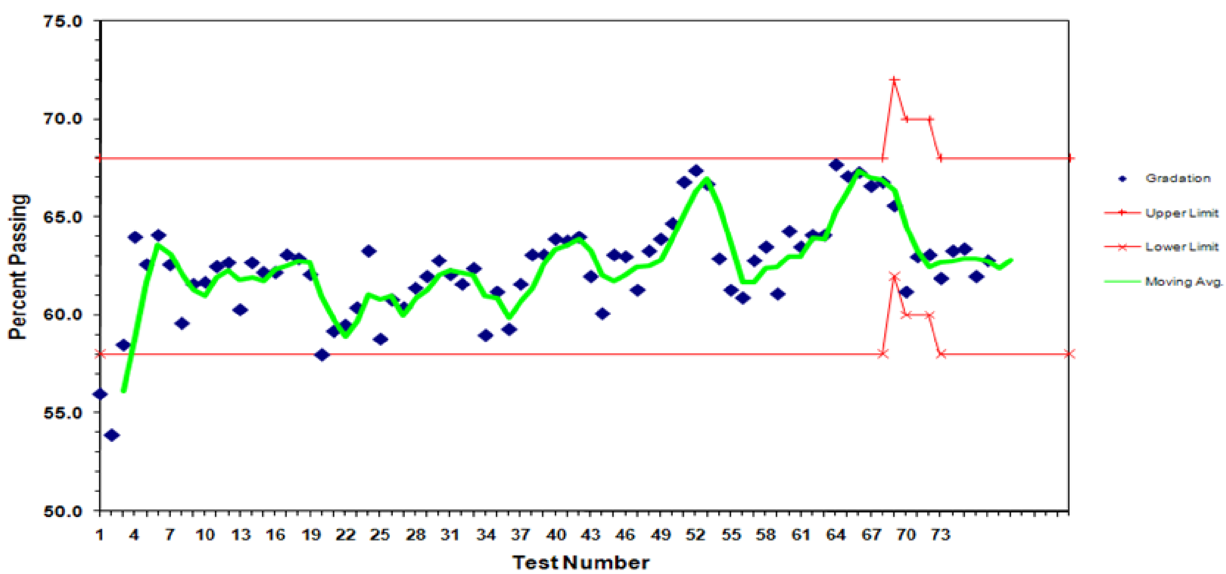


Figure 35. Control Chart for ⅜ in. Sieve, Iowa DOT's Project

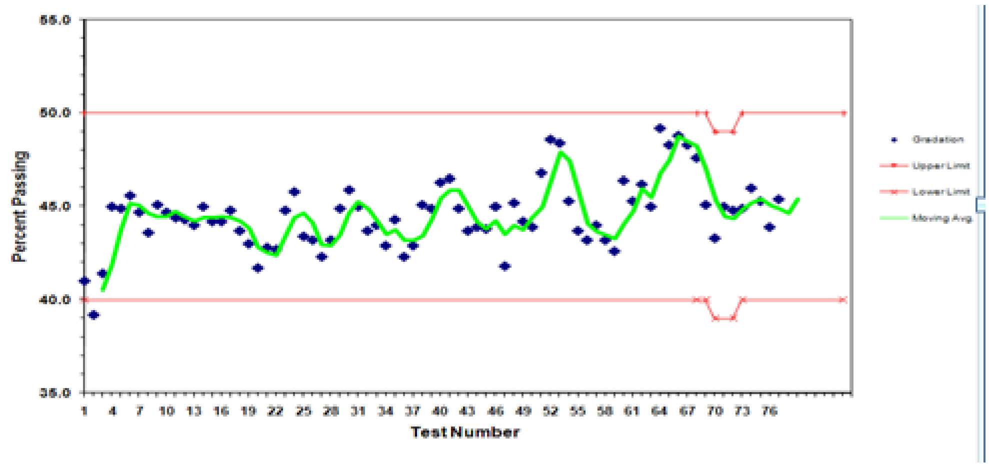


Figure 36. Control Chart for No. 4 Sieve, Iowa DOT's Project

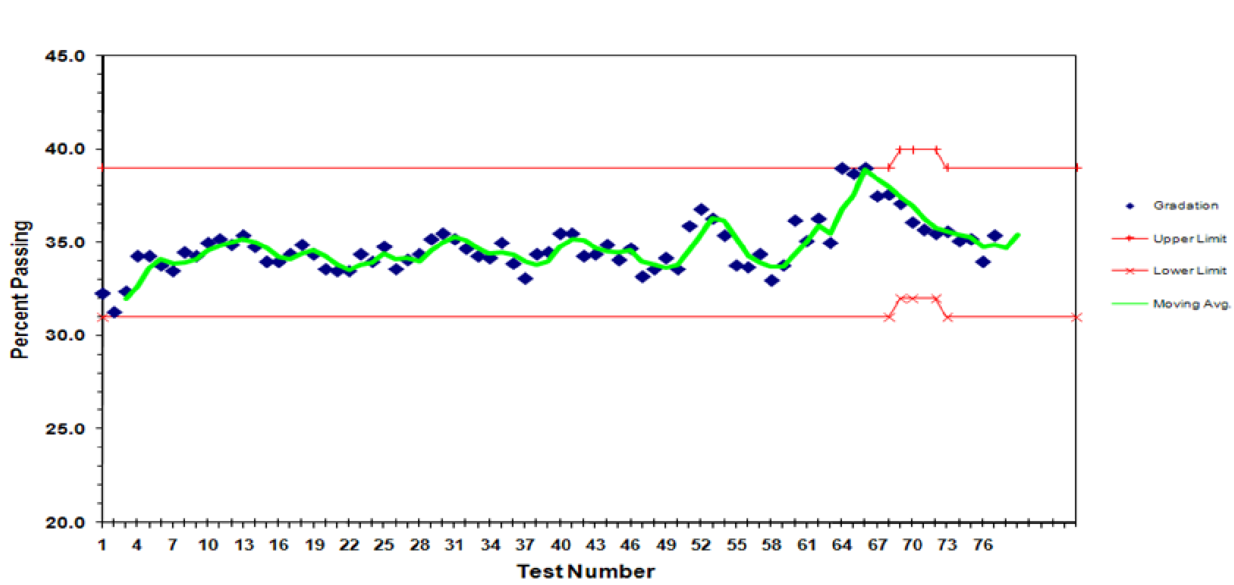


Figure 37. Control Chart for No. 8 Sieve, Iowa DOT's Project

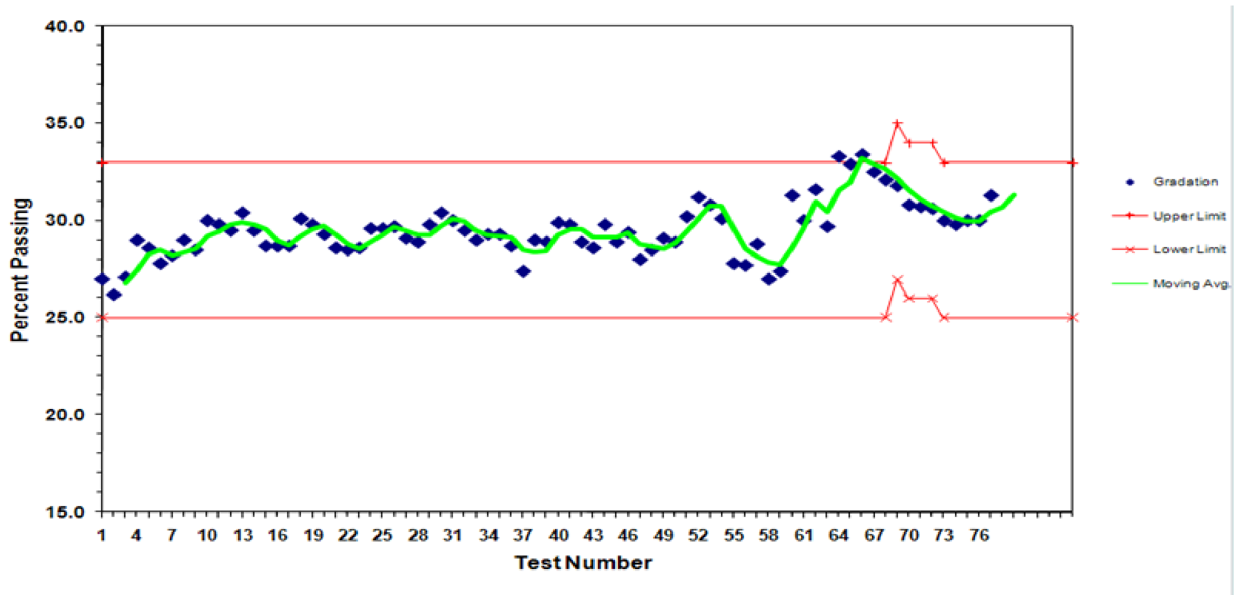


Figure 38. Control Chart for No. 16 Sieve, Iowa DOT's Project

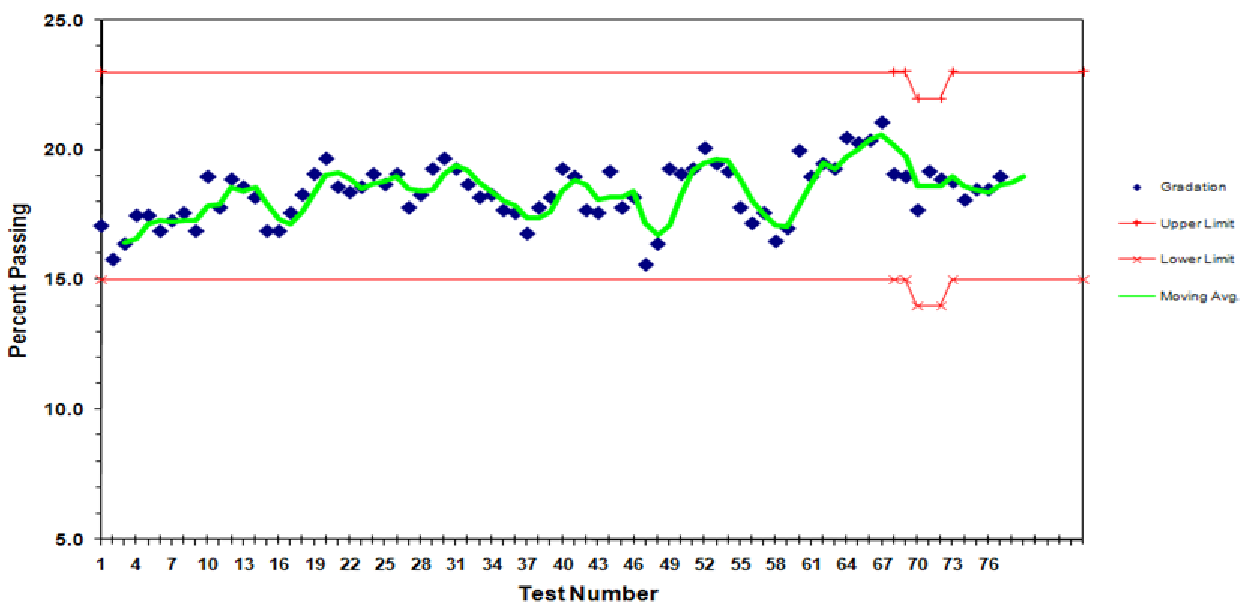


Figure 39. Control Chart for No. 30 Sieve, Iowa DOT's Project

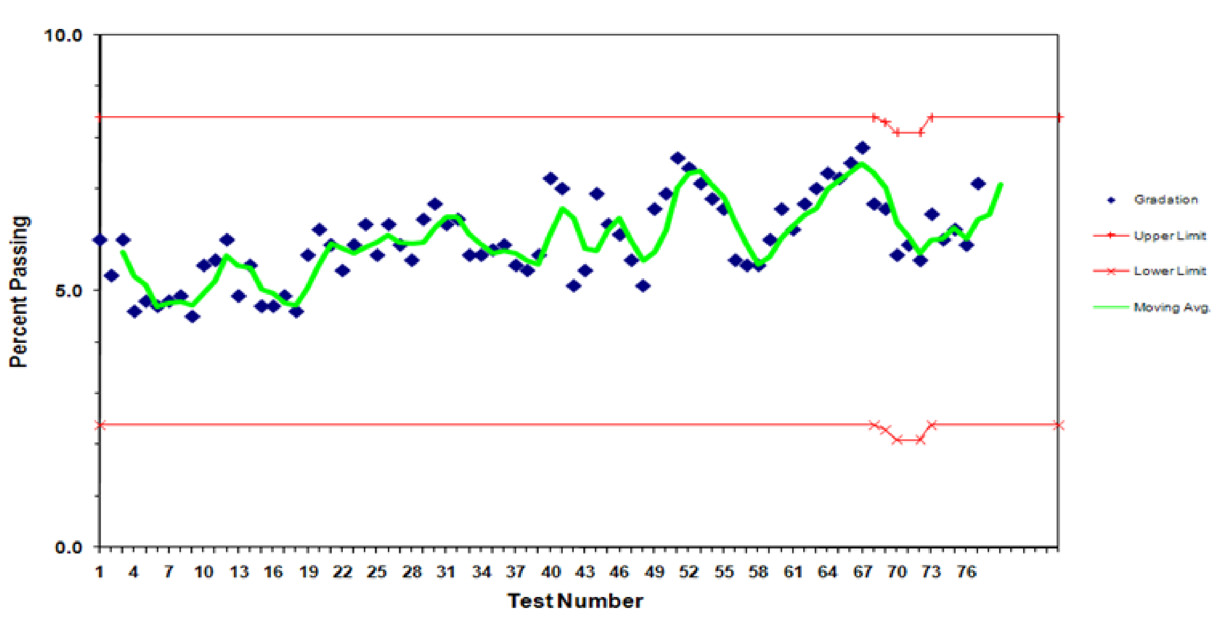


Figure 40. Control Chart for No. 50 Sieve, Iowa DOT's Project

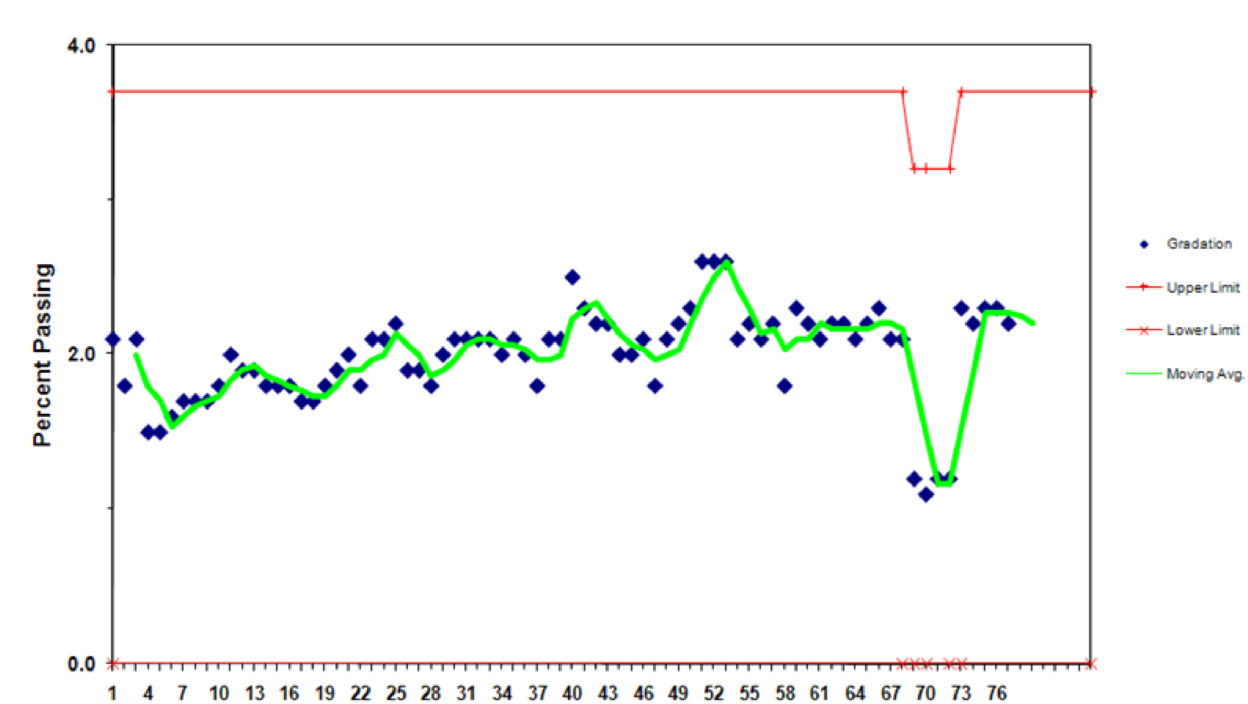


Figure 41. Control Chart for No. 100 Sieve, Iowa DOT's Project

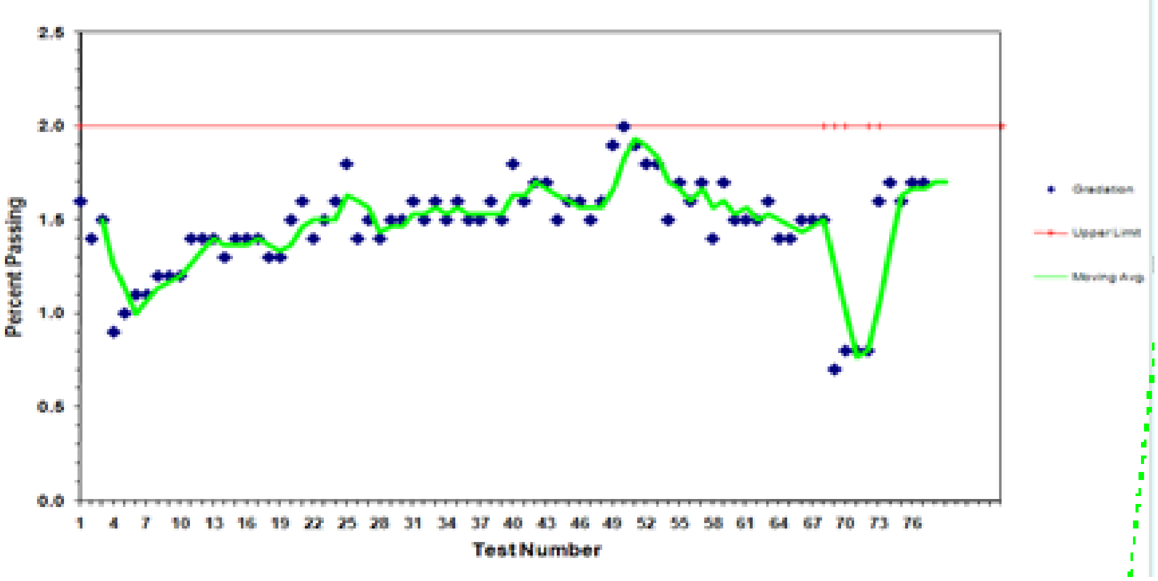


Figure 42. Control Chart for No. 200 Sieve, Iowa DOT's Project

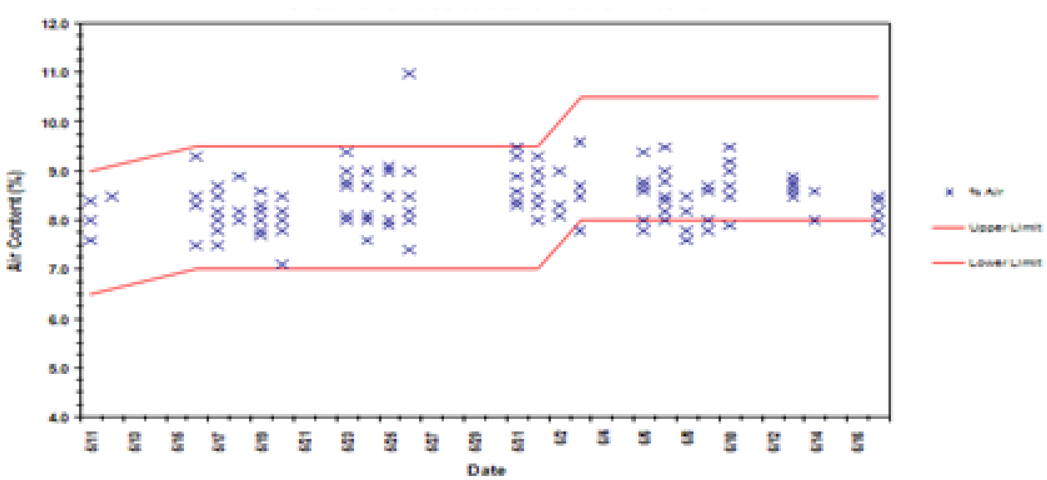


Figure 43. Control Chart for Plastic Air Content Before Paver, Iowa DOT's Project

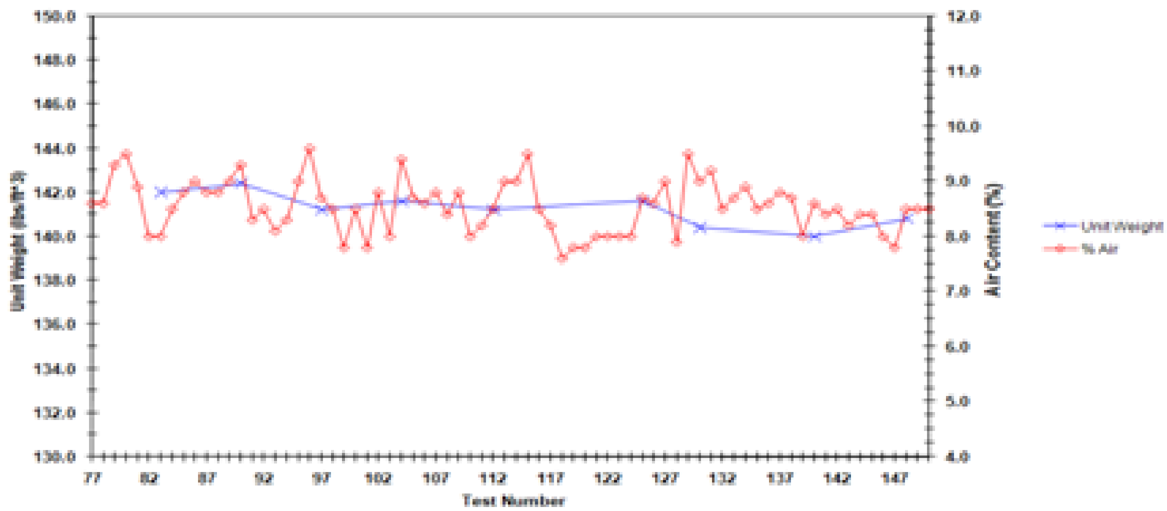


Figure 44. Control Chart for Unit Weight with Comparison of Plastic Air Content Before Paver, Iowa DOT's Project

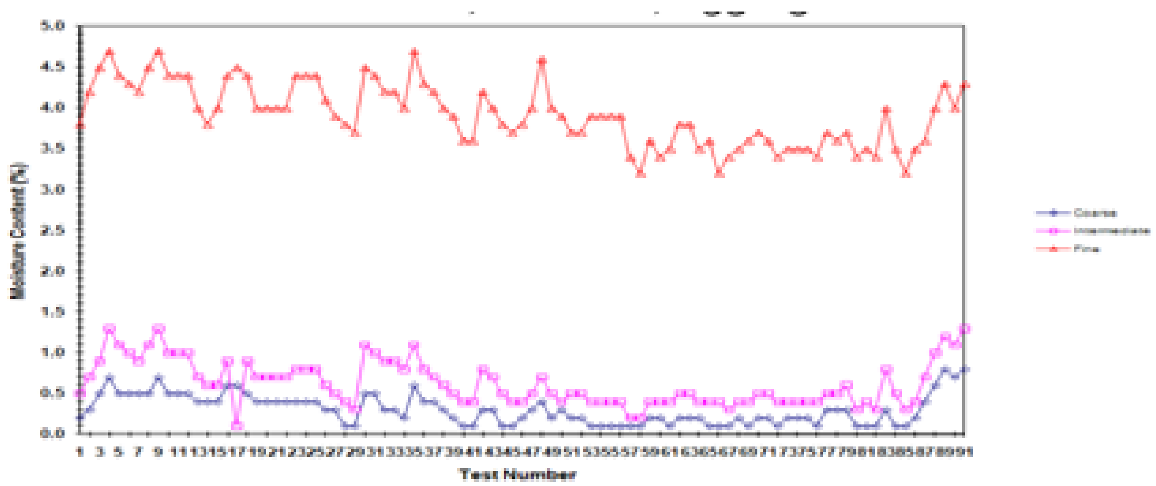


Figure 45. Control Chart for Moisture Content of Coarse, Intermediate, and Fine Aggregate, Iowa DOT's Project

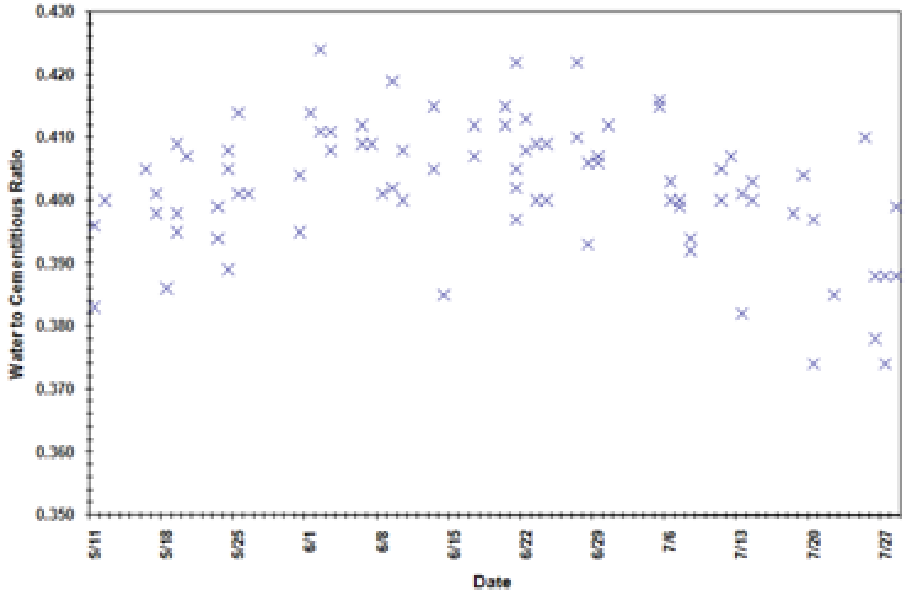


Figure 46. Control Chart for Water to Cementitious Ratio, Iowa DOT's Project

Recommendations for ITD Portland Cement Concrete Specifications

In the past several years, significant changes have taken place in concrete technology and construction methods. There are new concepts in construction methods which have not been incorporated into the concrete specification and practices of different agencies, including transportation departments. The new approach in the concrete industry puts the emphasis on the end product including performance and durability and less emphasis on the means and methods used to produce the end product. This approach allows reasonable flexibility for the contractor and supplier to use innovative construction methods and equipment. This may reduce the cost of production and could result in a better product. The new concepts are intended to develop a consistent evaluation methods resulting in an improved end product.

The recommended approach is to include the concrete specifications as part of the design process with emphasis on the desired characteristics, including but not limited to strength, durability, and workability. Indeed, the specifications need to be part of the design process. As a rhetorical question we ask: Is it better to have a well-built, poorly designed pavement or a well-designed, poorly built pavement? Obviously, each of these alternatives is deficient. The solution is to include specifications as a part of the design process.

In the recommended approach, the contractor is entirely responsible for the end product defined by the client's job requirements. For example, for a pavement project, the end product would meet the requirements for:

- Strength.
- Durability.
- Workability.
- Concrete Permeability.
- High quality end product consistent with the availability of the local material.
- Mitigation of ASR.
- Surface smoothness.
- Air content.
- Concrete consistency.
- Flexural strength.
- Any other specifications which improve performance and quality of the end product.

The contractor will be responsible to prove that the end product meets the clients specifications using various tests, trial batches, and if necessary, test sections. If the final product doesn't conform to the client's specifications, the contractor is responsible for fixing the problem--even if he must to remove the completed work.(4)

A complete list of recommendations to overhaul the ITD PCC specifications is beyond the scope of this project. The following recommendations are some of the important steps necessary to initiate changes in ITD specifications and familiarize ITD personnel with the new concepts and national trends.

Recommendations

The complete list of recommendations including the one currently practiced by ITD is given below.

a. *Preconstruction Meeting and Agreement on Submittals:* Prior to bidding, the ITD engineers and personnel will meet with the contractor and develop an agreement on the job-specific submittals which must be provided to the client. These submittals may include but are not limited to:

    - Specification of end products and test methodology.
    - Availability of materials and supplies.
    - Project requirements and construction schedule.
    - Requirements for the contractor acceptance testing.
    - Monitoring of acceptance testing.
    - Qualification of the inspectors and laboratories that will provide testing.
    - Certification of the testing personnel, cement mill, aggregate, supplementary materials, admixtures, water quality, etc.

b. *Process Control Submittals:* The contractor is responsible for the submission of materials related to the product-quality-control testing agreed to in the preconstruction meeting. These submittals may include but are not limited to:

- Statistical quality control data on the output of the concrete mix plant.
- Aggregate quality.
- Coarseness and workability for the combined aggregate gradation. An Excel spreadsheet program has been developed to analyze aggregate for coarseness and workability. Many agencies provide the Excel spreadsheets to the contractor in the preconstruction meeting and require the contractor to enter the data and send the results to the client for approval. If this procedure is not practical for ITD, it may be possible for ITD to request the data and have a resident engineer enter the data in the program and review the results for approval or rejection. The data requested from the contractor, as defined by the client, may include: sieve analysis results, detailed mix design, a list of supplementary materials used in the mix, admixtures, moisture content of the aggregate, aggregate quality, air content, concrete temperature, etc.

c. *Acceptance Testing Agreement:* In the preconstruction meeting, ITD and the contractor will agree on the laboratories and/or persons qualified and acceptable to both parties to conduct the tests. During the production and construction, tests will be performed by the qualified laboratories or persons to assure the final product performance and long-term durability. The tests may include: concrete compressive strength, tensile strength using a split-cylinder, flexural strength using four-point bending, pavement thickness, air content, consistency, surface smoothness, etc.

d. *Cementitious Material:* The cementitious material used in the concrete must be approved by ITD in the preconstruction meeting based on the cement's quality and availability throughout the project. Cementitious materials may include: Portland cement, fly ash, silica fume, or any other material necessary to improve the quality and long-term durability of the concrete.

- Hydraulic cement: Hydraulic cement (Portland cement) used in the concrete must conform to the requirements of the ASTM C150 (AASHTO M85), ASTM C595 (AASHTO M240) or ASTM C1157 depending on the type of cement used in the concrete.
- The contractor will decide on the amount of cement needed to meet the ITD specifications. ITD will specify the maximum water-cement ratio as well as the maximum amount of cement used in concrete, which may not exceed 570 lb/yd^3.
- The workability of concrete shall be adjusted using super plasticizer. Addition of water to adjust the workability shall be avoided.
- Supplementary cementitious material (silvica fumes, fly ash): The contractor is responsible to prove that the aggregate used in the concrete is not reactive by providing test results

(ASTM C1260) conducted by a qualified laboratory, as agreed in the contract. If the aggregate proves reactive, a mitigation technique and test result using ASTM C1567 will be necessary to prove the long-term durability of the final product. The fly ash or any other supplementary cementitious material used in the concrete must conform to the ITD specifications for the composition.

- Currently, ITD requires a minimum of 1:4 and a maximum of 1:3 ratio of fly ash to cement used in concrete when the aggregate used in concrete has proven to be reactive. Based on the ASR tests conducted in this study, it would be necessary to replace cement in concrete with unprocessed fly ash in excess of 30 percent to mitigate ASR in Idaho aggregate. A combination of 20 percent fly ash with 5 percent of the recommended dosage for lithium nitrate solution is also effective in mitigating ASR. The results of these tests and the recommendations are presented in Chapter 2.

e. *Aggregate:* The source and the gradation of the aggregate will be decided by the contractor. The maximum size aggregate also shall be selected by the contractor to meet the requirements for the specific project. The aggregate shall be clean. If necessary, the aggregate must be washed before mixing with the paste. If D-cracking is a problem, the maximum aggregate size should be limited to ¾ in.

  - Aggregate gradation evaluation: Aggregate gradation will be evaluated using coarseness/workability factor, 0.45-chart, and the percent-passing chart as described in Section 2.18. Depending on the agreement, the contractor or the ITD resident engineer will provide the charts for review and acceptance. The contractor is also responsible to provide the sieve analysis results with the mix design to ITD for approval.
  - Aggregate quality: The aggregate quality must meet the requirements for the deleterious substances as outlined in Tables 1 and 2 in ASTM C33.
  - The soundness test will be performed in accordance with ASTM C88, based on the ITD requirement to use either using magnesium sulfate or sodium sulfate tests.
  - A degradation evaluation will be performed based on Idaho T-15 aggregate degradation test or the Micro-Deval test.
  - The aggregate will not contain more than 20 percent flat or elongated pieces.

f. *Water Quality:* Water used in the mix shall confirm to the ASTM C1602 requirements. Potable water may be used without testing.

g. *Chemical admixtures:* The admixtures used in concrete such as super plasticizers, retardants, accelerators, air entraining agents, etc. must be certified chemicals and be approved by ITD engineers. For all concrete admixtures containing calcium chloride will not be approved.

Excel Spreadsheet Program

An Excel sheet was developed to plot the coarsenesses/workability factor, 0.45 power chart, and percent passing chart as shown in Figure 47. As an input, the Excel program requires the sieve analysis result for both course and fine aggregates as well as the mix recipe for the concrete. The input columns are designated by blue color. If more than one aggregate source is used, the columns for the second aggregate may be filled otherwise they must remain blank. Once the data has been entered, changing the tabs would provide the corresponding graph of the three tools for the entered data. In the 0.45 chart tab, the sheet automatically plots the boundaries (i.e. minimum & maximum density lines) for the given density line and shows the gradation line of the given aggregate in a blue line. In a similar manner, in the Percentage Retained Chart tab, the boundary lines are shown in red while the gradation line is shown in blue. In the Coarseness/Workability Factor Chart tab, according to the corresponding value of coarseness and workability values, a green diamond shaped point is displayed in the regions. The point will be displayed in the chart only if the value of the workability is in between 20 to 45.

Idaho Transportation Department
Optimized Aggregate Gradation For Concrete

Sieve Size	Sieve Size (in.)	Coarse Agg. 1 % Passing	Coarse Agg. 2 % Passing	Fine Agg. 1 % Passing	Fine Agg. 2 % Passing	Combined % Passing	Combined % Retained
63 mm	2½ in.					0.0	100.0
50 mm	2 in.					0.0	0.0
37.5 mm	1½ in.					0.0	0.0
25 mm	1 in.					0.0	0.0
19 mm	¾ in.					0.0	0.0
12.5 mm	½ in.					0.0	0.0
9.5 mm	⅜ in.					0.0	0.0
4.75 mm	No. 4					0.0	0.0
2.36 mm	No. 8					0.0	0.0
1.18 mm	No. 16					0.0	0.0
600 µm	No. 30					0.0	0.0
300 µm	No. 50					0.0	0.0
150 µm	No. 100					0.0	0.0
75 µm	No. 200					0.0	0.0
	Pan					0.0	0.0
Percentage of Agg.		0.00%	0.00%	100.00%	0.00%	100.00%	

Mix Proportions	
Cement	1
Cementious Material	
Coarse Aggregate # 1	
Coarse Aggregate # 2	
Fine Aggregate # 1	1
Fine Aggregate # 2	
Water	0
Total	2

Sieve Analysis | 0.45 Power Chart | Percentage Retained Chart | Coarseness Factor Chart

Figure 47. Excel Program for ITD

Risks Involved in the Use of Concrete and Supplementary Cementitious Material

One of the objectives of this study was to evaluate the risks involved in the use of cementitious supplementary material in different ITD projects including: pavement, sidewalks, curbs and gutters. After evaluation of the ITD concrete specifications, it appears that the main risk involved is the concrete itself. In risk analysis for concrete performance and long-term durability, it is important to assure that the concrete used in the projects is durable which includes the reactivity of aggregate for potential ASR attack. If a concrete mix is well-designed with a good aggregate gradation, it may take 10 to 20 years for ASR to develop depending on the type of the structure. ASR may start earlier (8-10 years) to develop in the areas such as wheel tracks on the roadway where the stresses and moisture are high due to traffic load. It may take longer (15-20 years) for ASR to develop in the sidewalks, curbs and gutters where the stresses and moisture relatively low. A poor aggregate gradation and/or poor mix design may also develop uncontrolled cracks in concrete; therefore, it is necessary to evaluate the specifications for the concrete mix design prior to conducting risk analysis for the use of fly ash in concrete for the ASR mitigation. However, as a general rule it is highly recommended that, if the aggregate is known to be reactive, fly ash for mitigation purposes should be used. The fly ash replacement not only eliminates or reduces the chance of premature deterioration of concrete; it may also be more cost effective. Chapter 3 of this report provides recommendations for mitigation of ASR in some of the Idaho aggregates from the sources in the Snake River basin.

Chapter 2
Alkali-Silica Reactivity

Introduction

Alkali Silica Reactivity in concrete is a worldwide problem which results in premature deterioration of PCC used in structures and pavements. ASR was first discovered by Stanton in 1940, when he observed the premature failure of certain concrete in California.[18] Since then, ASR has received extensive review, but it continues to constitute the major cause of premature deterioration of concrete in structures and pavements.

It is widely known that there are three components necessary for ASR to take place: reactive silica, sufficient alkali, and moisture. The absence of any one of these components will prevent ASR and the associated deterioration. Reactive Silica refers to the aggregates that tend to breakdown while exposed to the high alkaline solution in concrete. This reacts with the alkali-hydroxides to form alkali silica gel, which absorbs water and expands resulting in the disruption of the surrounding concrete. There are many types of siliceous aggregates from different rock sources that are susceptible to ASR, but not all siliceous aggregates are prone to ASR.

Alkalis such as sodium and potassium come from Portland cement and other sources like aggregates, chemical admixtures, supplementary cementing materials, and from external sources like seawater and deicing salts. The presence of moisture is important when considering the potential for ASR damage in structures. Concrete mixtures with highly reactive aggregates and high alkali cements have shown little or no expansion in dry environments and the structures which are exposed to constant moisture have exhibited ASR induced damage.

Concrete Cracks

Concrete is a brittle material with low tensile strength that cracks when the tensile force induced on concrete exceed its strength. Tensile stresses induced on concrete arise from different sources including: externally applied forces and induced stresses from internal chemical reactions. Some of stresses sources in concrete includes: static and dynamic loads from traffic, thermal cycles, freeze and thaw, wetting and drying, internal chemical reactions including hydration process and alkali-silica reaction. The stresses imposed on concrete are not independent, and they combine to increase the overall level of stress in concrete. This concept is illustrated in Figure 48.[19]

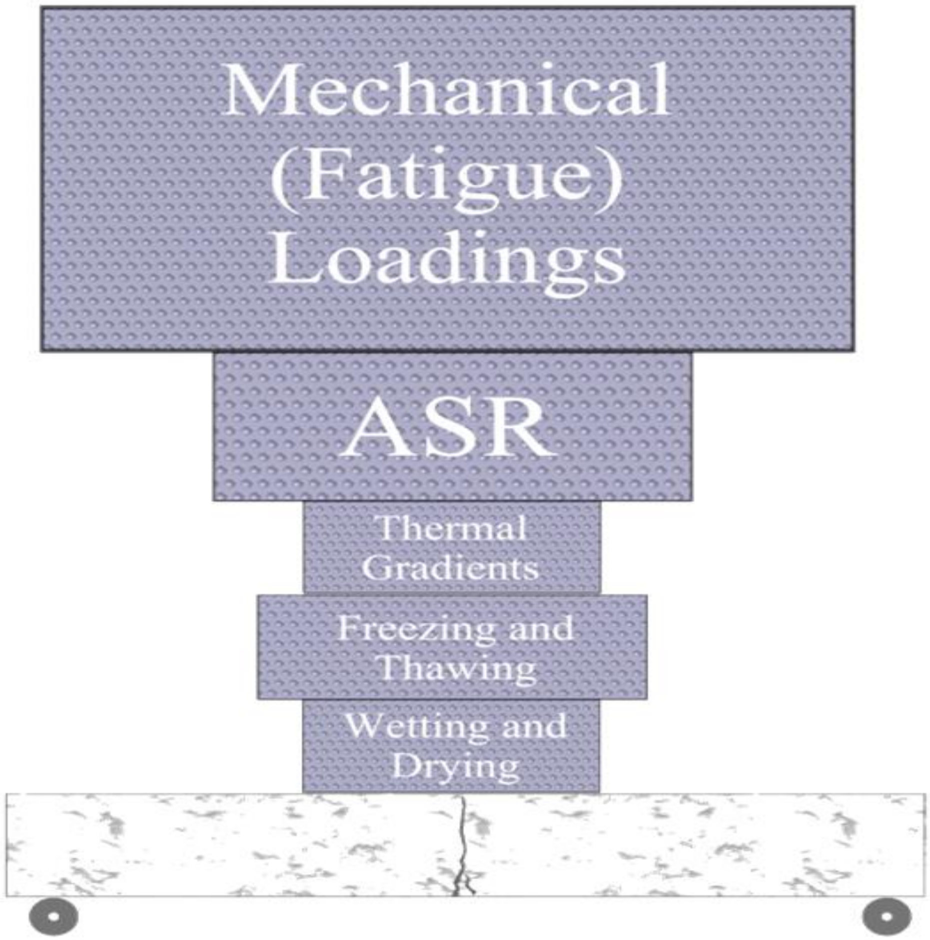


Figure 48. Some of the Stress Sources in Concrete[19]

In general, stresses induced on concrete are additive. It is difficult to assign the cause of cracking to one individual source, unless that source alone induces stresses larger than the tensile strength of concrete. Figure 49 illustrates this concept by comparing pavement on the driving lane and shoulder which were poured at the same time. The signs of ASR are quite obvious in the driving lane while the shoulder shows no sign of distress. Of course, this doesn't mean that the shoulder will not display ASR cracking in the future. The other sources of stress normally accelerate ASR cracking, and it takes more time for shoulders to develop the sign of ASR.

Figure 49. Comparison of ASR Cracks in a Driving Lane and Shoulder[19]

Mitigating ASR

In the past 68 years, many attempts have been made to prevent ASR deterioration due to the potential for extending the life cycle of concrete almost indefinitely. Some of the mitigation techniques used in preventing ASR includes: the use of Class F fly ash, silica fume, slag, lithium nitrate solutions, and other admixtures. Traditionally, Class F fly ash has been used as a partial replacement for Portland cement in concrete to improve the resistance of concrete to chemical attacks such as ASR and to reduce the heat of hydration. In general, the effectiveness of fly ash in mitigating ASR depends on its fineness and chemical composition. The chemical composition of fly ash, in turn, depends on the variety of coal used in the power plant and the combustion process.

A finer fly ash reduces water demand in concrete by increasing the concrete density. This produces more workable and stronger concrete.[20] A finer fly ash also provides greater surface area promoting a faster reaction with alkali in cement and so preventing alkali reaction with reactive silica. The mineralogy of fly ash also dictates its efficacy when used in concrete.[21] There are 2 types of chemical compound in fly ash: those which increase expansion such as: Calcium Oxide (CaO), Magnesium Oxide (MgO), Sulfur Trioxide (SO_3), and alkali and those which reduce expansion such as: Silicon Dioxide (SiO_2), Aluminum Oxide (Al_2O_3) and Ferric Oxide (Fe_2O_3).

Literature Review

In 1987, the Congress established the "Strategic Highway Research Program" (SHRP) to improve the performance and durability of the nation's roads and makes them safer for motorists and workers. This project with a $150 million budget was a 5-year research program targeting those products with a high pay-off. State transportation agencies played a major role in guiding this research under the National Academy of Science. In 1993, SHRP concluded the project, which had identified 130 products in support of its mission. Five of those products addressed by SHRP were associated with detecting, identifying, mitigating, and avoiding ASR in concrete structures and roadways.(18) This study resulted in three major publications:

1. ASR: An overview of research, SHRP-C-342, SHRP Product 2011.(22)
2. Eliminating or Minimizing ASR, SHRP-C-343, SHRP Product 2011.(23)
3. Handbook for the Identification of ASR in Highway Structures, SHRP-C-315, SHRP Product 2010.(24)

As a result of this study, 2 rapid tests were also developed to detect ASR and screen aggregate for potential reactivity.

1. Rapid Identification of ASR Products in Concrete, AASHTO T299, SHRP Product 2013.(24)
2. Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to ASR, AASHTO T303, and SHRP Product 2009.(24)

In 1996, the AASHTO Task Force Implementation established the Lead State Program and invited over 30 states to join the team in seven technology groups; ASR being one of them. The mission of ASR group was "to provide a clearinghouse to share and deliver information and technical assistance in identification, prevention, and rehabilitation of alkali silica reactivity to the public, private, and academic sector transportation".(18)

One of the key elements of the Lead State Team was to conduct a national survey to assess the extent of ASR in the nation. A total of 38 states participated in the survey.(25) Many participants indicated that they have experienced unexpected cracking problems and about 50 percent of them attributed it to ASR. The remaining states weren't sure if it was related to ASR.

The survey result indicated that ASR is a nationwide problem, and more states are experiencing the problem due to increase in the use of Portland cement concrete and diminishing of nonreactive-aggregate sources. Furthermore, as the Environmental Protection Agency (EPA) imposes stricter regulations on coal power plants the quality of fly ash used to mitigate ASR deteriorates. A complete list of survey questions, a list of participants, their answers to the questions, the methods of testing for ASR and the mitigation techniques used in each state are presented in the Appendix B.

Alkali Silica reactivity in New Mexico State is a well known problem to highway engineers. Normally, a bridge which would last 80 to 100 years without ASR problems will require major rehabilitation in 25 years and replacement in 50 years when affected by ASR.(22) Prior to 1970, the use of low-alkali cement was believed to adequately mitigate ASR in concrete; however, significant deterioration in some structures with low-alkali cement within seven to ten years proved otherwise. The solution recommended since then was to partially replace low-alkali cement with fly ash, and this approach has proved to further reduce ASR-induced deterioration in concrete structures.

In 1977, the New Mexico State Highway and Transportation Department (NMSHTD) initiated a research project to identify the type and level of additive to be used to mitigate ASR in its concrete projects. In this research project, fly ash and lithium nitrate were investigated as additives for use in reducing ASR. The testing criterion used for acceptability was 0.1 percent expansion rate at 14 days evaluated using Standard Method of Test for Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction (AASHTO T 303-96).(24)

This study showed that for most aggregates in New Mexico, 25 to 27 percent Class F fly ash by weight replacement of total cementitious material was sufficient to reduce ASR related distress in concrete. When lithium nitrate was introduced in the mix, there was a reduction in the amount of fly ash needed to mitigate ASR. This experimental study concluded that a combination of lithium nitrate and Class F fly ash is a viable combination for mitigating ASR. This study also concluded that the blend of Class F and Class C fly ash do not reduce ASR adequately and suggested that the no more than 10 percent CaO in fly ash should be used in any concrete.

The Accelerated Mortar Bar Test (AMBT), ASTM C1260 (AASHTO 303), originally developed in 1986 by Oberholster and Davies, is a severe screening test used to determine the potential ASR in Portland cement concrete.19) However, this test has many advantages over other tests such as the Concrete Prism Test (CPT), ASTM C1293, since the AMBT takes only 16 days to complete compared to 1 or 2 years for other tests. However, the advantage of getting a quick answer could be offset by the severity of the test which could over estimate the reactivity.

The majority of the focus today is on the AMBT (ASTM C1260 and modifications) and the CPT (ASTM C1293) tests. The CPT test seems more realistic since it does not require a soak solution of Sodium Hydroxide (NaOH); however, this test requires the specimen to be exposed to temperature of 100°F for 1 to 2 years in a high humidity environment. This is unrealistic when compared to the real environment. Also, is it unpractical for a contractor to wait two years to determine aggregate reactivity? In two years, a lot can change; the composition of cement may not be the same, the aggregate may change, and the supplementary cementitious material (SCM) may not be the same.

The results of multi-laboratory tests for ASTM C1293 show a large range of variation as shown in Figure 50.(26) The range of acceptable values that might be obtained from two different labs repeating a

CPT test is approximately 145 percent of a given initial value; however, the range of values from having another lab repeat an AMBT test is approximately 90 percent of a given initial value.(26) There are very limited realistic field data supporting 0.04 percent expansion limit with the 2 years CPT test. The controlled outdoor exposure specimens show that the CPT test is not reliable in predicting performance.

Although not adopted by the FHWA, recent research data indicates that AMBT (ASTM C1260) with a threshold of 0.08 percent at 28-days is a better predictor than CPT test (ASTM C1293) with a threshold of 0.04 percent in 2 years.(26, 27) Table 14 compares ASTM C1260 (14- and 28-day tests) and ASTM C1293 test with field data.(28) In this table the false negative and false positive are defined as follows:

- False Negative (False -)
  - ASTM threshold predicts no failure (negative) but the field specimens show failure. This will result in premature structural loss.
- False Positive (False +)
  - ASTM threshold predicts failure (positive) but the field specimens show no failure. This will result in increased construction cost to bring non-reactive aggregate for use in concrete.

In general, the objective is to minimize both false negative and false positive predictions. A risk analysis was concluded by Malvar & Lenke on the data collected by Stokes et al.(26)

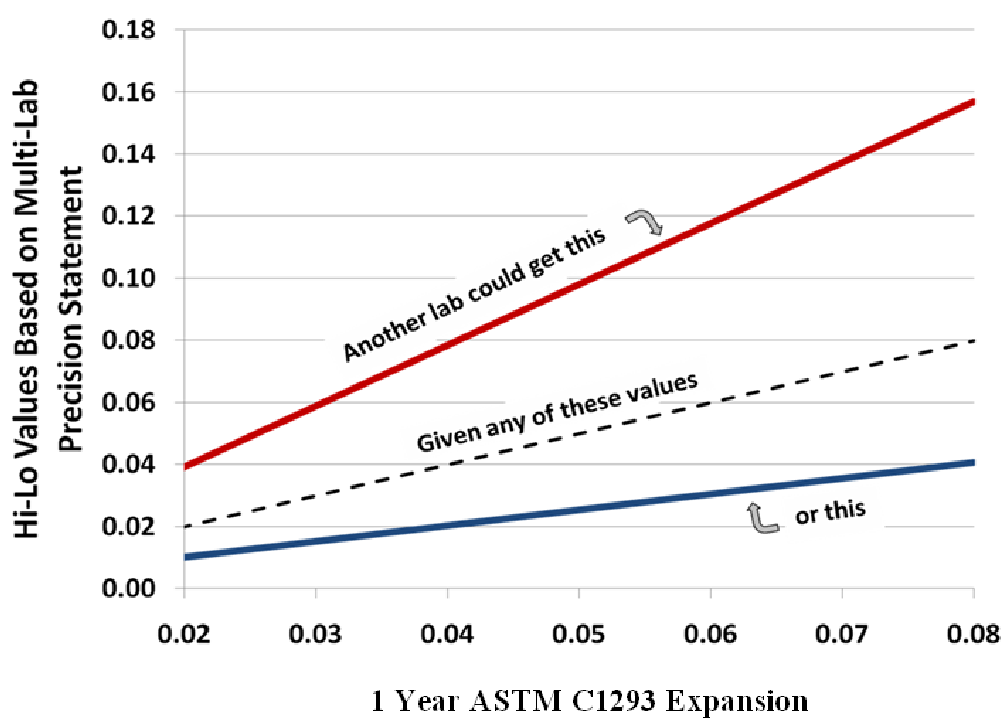


Figure 50. Multi-Laboratory Test Results for CPT Test (ASTM C1293)(26)

Table 14. Comparison of ASTM C1260 (14- and 28-Day Tests) and ASTM C1293 Test with Field Data[28]

Data from Stokes et al. and Fournier et al.

Mixture	14-day AMBT			28-day AMBT		2-year CPT		Outdoor Slab		14-day	14-day	28-day	2-year	14-day	14-day	28-day	2-year
	0.10%	0.08%	expansion	0.08%	expansion	0.04%	expansion	0.04%	expansion	0.10%	0.08%	0.08%	0.04%	0.10%	0.08%	0.08%	0.04%
Aggregate	(P/F)	(P/F)	%	(P/F)	%	(P/F)	%	(P/F)	%	False +	False +	False +	False +	False -	False -	False -	False -
Aggregate A	F	F	0.391	F	0.617	F	0.207	F	0.255	0	0	0	0	0	0	0	0
A w/ 10 % silica fume	F	F	0.142	F	0.402	P	0.038	F	0.086	0	0	0	0	0	0	0	1
B	F	F	0.278	F	0.464	F	0.157	F	0.11	0	0	0	0	0	0	0	0
B w/ 20 % class F fly ash	P	P	0.048	F	0.125	P	0.008	F	0.044	0	0	0	0	1	1	0	1
B w/ 30 % class F fly ash	P	P	0.021	P	0.04	P	-0.007	P	0.03	0	0	0	0	0	0	0	0
B w/ 7.5 % silica fume	F	F	0.112	F	0.282	P	0.03	F	0.042	0	0	0	0	0	0	0	1
B w/ 10 % silica fume	P	P	0.078	F	0.225	P	0.023	P	0.018	0	0	1	0	0	0	0	0
C	P	F	0.093	F	0.193	F	0.226	F	0.176	0	0	0	0	1	0	0	0
D	F	F	0.173	F	0.212	F	0.087	F	0.171	0	0	0	0	0	0	0	0
D w/ 7.5 % silica fume	P	F	0.089	F	0.169	P	0.036	F	0.103	0	0	0	0	1	0	0	1
D w/ 10 % silica fume	P	F	0.083	F	0.178	P	0.03	F	0.075	0	0	0	0	1	0	0	1
D w/ 12.5 % silica fume	P	P	0.066	F	0.141	P	0.024	F	0.085	0	0	0	0	1	1	0	1
E	F	F	0.463	F	0.7	F	0.269	F	0.395	0	0	0	0	0	0	0	0
E w/ 20 % class F fly ash	P	P	0.065	F	0.137	F	0.05	F	0.145	0	0	0	0	1	1	0	0
E w/ 30 % class F fly ash	P	P	0.034	P	0.036	P	0.021	F	0.087	0	0	0	0	1	1	1	1
E w/ 7.5 % silica fume	F	F	0.13	F	0.276	F	0.079	F	0.152	0	0	0	0	0	0	0	0
E w/ 10 % silica fume	F	F	0.12	F	0.284	P	0.039	F	0.081	0	0	0	0	0	0	0	1
F	F	F	0.36	F	0.587	F	0.092	F	0.141	0	0	0	0	0	0	0	0
F w/ 20 % class F fly ash	P	P	0.037	P	0.067	P	0.016	F	0.048	0	0	0	0	1	1	1	1
F w/ 30 % class F fly ash	P	P	0.026	P	0.041	P	0.011	P	0.019	0	0	0	0	0	0	0	0
F w/ 7.5 % silica fume	P	F	0.09	F	0.222	P	0.028	F	0.062	0	0	0	0	1	0	0	1
F w/ 10 % silica fume	P	P	0.068	F	0.177	P	0.028	P	0.029	0	0	1	0	0	0	0	0
G	F	F	0.419	F	0.603	F	0.221	F	0.254	0	0	0	0	0	0	0	0
G w/ 20 % class F fly ash	F	F	0.175	F	0.271	F	0.04	F	0.101	0	0	0	0	0	0	0	0
G w/ 20 % class F fly ash	P	P	0.062	F	0.091	P	0.02	P	0.038	0	0	1	0	0	0	0	0
H	F	F	0.854	F	1.04	F	0.231	F	0.4	0	0	0	0	0	0	0	0
H w/ 20 % class F fly ash	F	F	0.395	F	0.461	F	0.085	F	0.247	0	0	0	0	0	0	0	0
H w/ 30 % class F fly ash	P	F	0.088	F	0.141	F	0.05	F	0.128	0	0	0	0	1	0	0	0
										0	0	11	0	36	18	7	36
									Stoke's et al. paper	0	0	4	0	43	29	7	32

- 0.08% at 28 days minimizes False –, but has higher False +
- Next best is DOD's 0.08% at 14 days
- 0.10% at 14 days and 0.04% at 2 years are equally bad

Note: these outdoor slabs had added alkali to match CPT Na_2O of 1.25% by mass of cement

The percentages of False Negatives and False Positives are shown in Table 15.[28] This table shows that the added risk in using ASTM C1260 (14- and 28-days) with threshold of 0.08 percent is only 18 percent, while the added risk for ASTM C1260 (14-day) and ASTM C1293 (2 years) tests with threshold of 0.10 percent and 0.04 percent, respectively is 36 percent.

Table 15. Comparison of Number of False Negatives and False Positives for ASTM C1260 and ASTM C1293 Tests and Added Risks[28]

Percentages of False + and –

Use both sets of slab data: with and without added alkali

- **Generally field data does not have added alkali**

Risk	Additional Alkali in Slab	14-day	14-day	28-day	2-Year
		0.10%	0.08%	0.08%	0.04%
False Negative	No	25	11	4	18
	Yes	36	18	7	36
False Positive	No	11	14	29	4
	Yes	0	0	11	0
Added Risk	No	36	25	32	21
	Yes	36	18	18	36

- **0.08% at 28 days may be too strict (29% False +)**
- **Optimum between 0.08% at 14-day and 0.08% at 28-day**

In 1999, Thomas and Innis conducted experiments to evaluate different admixtures for controlling expansion due to ASR.[29] They showed that in 73 percent of the cases, CPT and AMBT were in agreement. In 2000, Grosbois and Fontaine compared the results of CPT and AMBT for several different aggregate types.[30] They concluded that for ASTM C1260 test, carbonate aggregate was conservative and a 0.08 percent or 0.06 percent threshold would have been more appropriate. For sandstone, both methods predicted similar reactivity. For igneous and metamorphic rock, in 2 cases ASTM C1260 seemed to be more conservative. In 2000, Strange conducted ASR studies on the existing buildings in Canada and concluded that in 46 percent of the cases AMBT indicated more conservative results compared to CPT.[31] This study was not conclusive since 70 percent of the structures which showed reactivity were constructed between 1930 and 1950. Although some of the previous work has indicated that CPT provides more realistic results, in 2000, Jensen and Fournier showed that some CPT tests on aggregates indicated no reactivity while actual field tests demonstrated reactivity.[32]

The question is: is it worse to wait for 2 years for the results of an ASTM C1293 tests? The answer of course is: no. Furthermore, it is necessary to find a more reliable and faster test to verify the reactivity of the aggregate for ASR. Therefore, for all practical purpose, the ASTM C1260 test (14- or 28-day) would provide relatively reliable results in the absence of a more accurate test.

Thresholds for Acceptance

Malvar and Lenke[28] have compared the thresholds for Acceptance of CPT and AMBT tests adopted by the American Society for Testing and Materials (ASTM), The Department of Defense (DOD), and the proposed thresholds by Stokes, et al.[26] as shown below.

- ASTM C1260 (Accelerated Mortar Bar Test, AMBT)
  - ASTM 0.10 percent at 14-day (of exposure)
  - DOD 0.08 percent at 14-day
  - Stokes, et al. 0.08 percent at 28-day
- ASTM C1293 Concrete Prism Test, (CPT)
  - ASTM 0.04 percent at 1 year (without SCM)
  - ASTM 0.04 percent at 2 years (with SCM)
- Field Data (exposed slabs) (typically worse than blocks)
  - Assumed 0.04 percent at 6 to 10 years

SCM = Supplementary Cementitious Material,
e.g. fly ash, ground granulated blast furnace slag, silica fume

The plot of data for ASTM C1260 tests for 14- and 28-day percent expansion is shown in Table 16.[28] From this data set, it appears that the majority of test results are below a percent expansion limit of 0.06 percent for 14-day and below 0.13 percent for 28-day. Based on this data, Melvar and Lenke[28] recommended the expansion limits for the ASTM C1260 test as shown in Table 16.

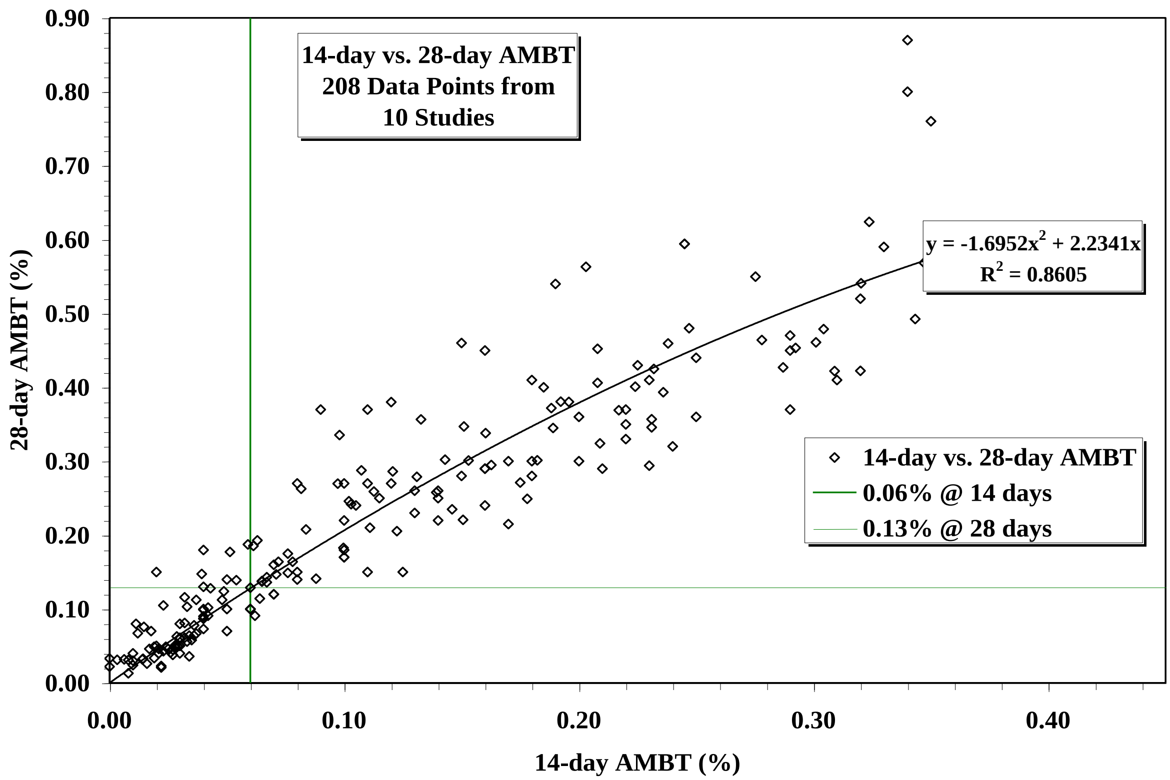


Figure 51. Visualization of 14-day versus 28-day Thresholds[28]

Table 16. Recommended Thresholds for AMBT (ASTM C1260)[28]

Risk	Additional Alkali in Slab	14-day 0.08%	28-day 0.08%	14-day 0.06%	28-day 0.13%
False Negative	No	11	4	4	4
	Yes	18	7	11	11
False Positive	No	14	29	25	21
	Yes	0	11	11	7
Added Risk	No	25	32	29	25
	Yes	18	18	21	18

- Recommended use 0.13% at 28-day based on data
- Next best choice is 0.06% at 14-day

Aggregate from Snake River Basin, Potential for ASR

Concrete pavement on I-84 around the Mountain Home area (MP 90-114) was constructed between 1992 and 1997. Starting in 2000, the pavement showed some level of distress, cracking, and concrete pop-ups. The cracking could be attributed to both chemical and physical processes; however, there was no field examination or aggregate analysis to identify the exact source of deterioration. After field review and examination of the aggregate and the concrete pavement cores, it was suspected that ASR was the cause of cracking. In 2002, ITD changed its standard specifications for concrete and required testing and mitigation for ASR in new concrete.(33)

The initial study of the pavement on I-84 started in 2002, when ITD hired consultants to test the pavement cores using petrographic analysis; examine the aggregate using ultra-violet waves, and test cores for ASR using Uranyl Acetate solution. The mix design, the aggregate sources utilized in the project, and the contractors' records were examined, and FHWA agreed to include this stretch of I-84 in their research program. The five-year research program focused on Lithium Nitrate treatment and crack monitoring using the French technique.(33) ITD hired a consultant to perform crack monitoring, investigate treatment options, study the aggregate used in the pavement, and make recommendations for treatment.(33)

The field and laboratory examination of the pavement and the aggregate sources showed that the cracking could be attributed by both physical and chemical processes. The identified chemical causes that have been identified include: ASR and internal/external sulfate attack. The aggregate pits were examined to characterize the nature of the aggregate including deposition and composition. Aggregate samples were examined to determine the relative-percent rock type, angularity and to identify amorphous and crystalline silica or other rock types associated with ASR related minerals.(34)

Aggregate Source Description

The aggregate sources used in this project are located on the Western Snake River Plain, commonly called a rift zone, bounded on the north and south by fault zones. This area is generally covered by basalts, silicic volcanic rock, granite, and meta sediments of the Snake River Plain. The aggregate examined in this area are fluvial deposits of the Snake River and/or remnants of the Bonneville Flood deposits. Four aggregate sources were used in the I-84 project: EL-37C, EL-120, EL-116C, and OW-110. The first three sources are located on the north side of the Snake River and the last source (OW-110) is located on the south side. All 4 sources contain aggregate rock type that can be potentially reactive in concrete including sandstones, quartzite, granite, and silicic volcanic rock. The findings of this study indicated that the source EL-116C was most likely to be reactive followed by OW-110, EL-120, and EL-37C. EL-37C appears to be the least likely to be reactive. The concrete slab replacement from the EL-37C aggregate currently does not exhibit signs of surface cracking indicating ASR.(34)

ITD ASR Mitigation Practice and Comparison with Other States

The ITD 2004 Concrete Specifications in Sections 703.02 and 703.03 for fine and coarse aggregate, respectively, requires the contractor to conduct AASHTO T303 (ASTM C1260) to determine the potential reactivity of the aggregate. Aggregate found to be potentially reactive per AASHTO T303 requires mitigation measures. Expansion greater than 0.10 percent as determined by AASHTO T303 will be considered potentially reactive and will require mitigation. Mitigation measures may include the use of fly ash, lithium nitrate admixtures, or other materials as approved by ITD engineer. ITD requires that Class F fly ash be used in concrete for mitigation purposes and specifies a ratio of minimum 1:4, maximum 1:3 of fly ash to Portland cement used in concrete.

Idaho is one of the states with reactive aggregate, and, within Idaho, the aggregate obtained from the sources in the Snake River Plain are particularly reactive. The majority of states have some sort of reactive aggregate and they have provisions for ASR mitigation in their concrete specifications. In this study five states with severe ASR problems were identified and their specifications were compared to ITD's specifications. A summary of the standard specifications for identifying reactive aggregate and provisions for mitigation for the states of Idaho, California, Virginia, Washington, New Mexico, and Nevada are compared in Section 2.9. The comparisons of the ASR specifications for 6 states with sever ASR problems including Idaho is provided in Section 2.9. The comparison of the specifications for ASR show that that there are some variations in the specifications among different states in source approvals, mitigation methods, testing techniques, and threshold levels for acceptance. Some states like Nevada are stricter and do not allow any reactive aggregate to be used in their concrete. Washington does not require mitigation for ASR if the expansion rate is under 0.20 percent using AASHTO T303 (ASTM C1260) test. The Idaho ASR specifications are generally in line with the practices of many DOTs and the current state of the research on ASR.

Comparison of ASR Specifications and Mitigation Techniques for Six Different States

- **For Aggregate Sources that are Reactive According to AASHTO T303**

  Idaho: ASTM C1293 or ASTM C295 and modified AASHTO T303.

  California: ASTM C1293 or ASTM C1260.

  Virginia: ASTM C227, ASTM C441 and ASTM C1260.

  Washington: ASTM C1293 or ASTM C1260.

  New Mexico: AASHTO T303, ASTM C1293.

  Nevada: ASTM C289.

- **For Aggregate Sources Identified as Reactive for ASR**

  Idaho:

  1. Expansion of mortar bars shall not exceed 0.10 percent at 14-day with the addition of Class F fly ash, lithium nitrate, or other ASR mitigation additives.
  2. The aggregate blend percentages used in the testing are reported and are within 2 percent of the blend percentages proposed in the mix design and to be used on the project. Aggregates may also be tested separately.
  3. The materials used in the expansion testing are the same materials (aggregate sources, cement, fly ash, mitigation additive) and at the same proportions reported in the proposed mix design and to be used on the project.
  4. When Class F fly ash is used, ensure the Calcium Oxide (CaO_2) content of the fly ash used on the project meets the 1.5 percent tolerance as established by the specifications.
  5. When lithium nitrate is used, ensure the lithium nitrate dosage is reported as a volume & as a percentage of the standard or full dose.

  Virginia: Aggregate may contain materials deleteriously reactive with alkalis in the cement, if cement contains less than 0.60 percent alkalis (percent Na_2O plus 0.658 percent K_2O) and contains a minimum of 7 percent silica fume or 15 percent Class F fly ash as specified to be effective in preventing harmful expansion due to alkali-aggregate reaction by ASTM C441.

  Washington: Aggregates tested in accordance with AASHTO T303 or ASTM C1260 with expansion greater than 0.20 percent are ASR and will require mitigating measures. Aggregates tested in accordance with ASTM C1293 with expansion greater than 0.04 percent are ASR and will require mitigating measures.

  New Mexico: All aggregates shall be evaluated for reactivity by AASHTO T303-96 or by ASTM C1293. This test will be performed utilizing standard Rio Grande Type I-II low alkali cement from the Tijeras Plant. This cement shall have alkali content between 0.5 percent and 0.6 percent. Aggregates that exhibit mean mortar bar expansions at 14-day greater than 0.10 percent shall be considered potentially reactive. If ASTM C1293 is used, the aggregate shall be considered to be innocuous if the average expansion measured at the end of 1 year is less than 0.04 percent.

  Nevada: Aggregates from any source having a history of ASR in concrete will not be approved for use.

- **Tests to be Conducted by Contractor**

  Idaho: AASHTO T303, ASTM C1293 or ASTM C295 testing to determine the potential ASR of the aggregates.

California: Aggregate producer submit its certified test results from qualified lab to Material Engineering and Testing Service (METS) for approval.

Virginia: Where there is potential for ASR, provide results of tests conducted in accordance with ASTM C227 or ASTM C1260.

Washington: The Contractor may submit an alternative mitigating measure through the Project Engineer at the State Materials Laboratory for approval along with evidence in the form of test results from ASTM C1567 that demonstrate the mitigation when used with the proposed aggregate controls expansion to 0.20percent or less. The agency may test the proposed ASR mitigation measure to verify its effectiveness. Passing petro graphic analysis (ASTM C295) accepted by WSDOT prior to August 1, 2005, is acceptable as proof of mitigation until the aggregate source is reevaluated.

New Mexico: All aggregates are evaluated by AASHTO T303 or by ASTM C1293, to determine ASR.

Nevada: Samples of aggregates to be tested by ASTM C289 at least 30 working days before anticipated use.

- **Reactive Aggregate**

  Idaho: Expansion greater than 0.10 percent as determined by AASHTO T303, or greater than 0.04 percent as determined by ASTM C1293. If ASTM C295 shows an aggregate composition containing greater than the indicated percentage Mineral Limit Optically strained, Micro fractured, or microcrystalline quartz 5.0 percent (max) Chert or Chalcedony 3.0 percent (max.), Tridymite or Cristobalite 1.0 percent (max.), Opal 0.5 percent (max.), Natural Volcanic glass 3.0 percent (max.).

  California: ASTM C1260 is a test that is commonly used to test the reactivity of an aggregate.

  New Mexico: The test procedure using the actual cement, fly ash and, if desired, any of the ASR inhibiting admixtures is conducted. The minimum amount of Class F fly ash, and the minimum amount of ASR inhibiting admixture required to provide a maximum expansion at 14-day is less than 0.10 percent. Report the Fly Ash required as a percentage of the cement weight.

- **Accelerated Detection of Potentially Deleterious**

  Idaho: AASHTO T303 meet AASHTO M307 with a maximum available alkali of 1.5 percent, and have not more than 10 percent retained when wet-sieved on the No. 325 screen.

  New Mexico: Acceptability was judged by comparing expansion measured at 14-days of age in AASHTO T303-96, criteria of 0.1 percent.

- **Fly Ash**

  Idaho: Natural Pozzolans and fly ash shall conform to AASHTO M 295 except that loss on ignition (LOI) shall not exceed 1.5 percent for all Classes Class-F fly ash shall be used, and available alkalis in the fly ash (as Na_2O) shall not exceed 1.5 percent. In addition, Calcium Oxide (CaO) content shall not exceed 11 percent. The Contractor shall submit the manufacturer's certification of material class and conformance to material specifications.

  California: Low fly ash proportion (<20 percent), moderate fly ash proportion (20-30 percent), high fly ash proportion (> 30 percent).

  Virginia: Cement with Minimum 15 percent Class F fly ash (maximum cement alkali 0.60 percent) Cement with Minimum 20 percent Class F fly ash (maximum cement alkali 0.68 percent) Cement with Minimum 25 percent Class F fly ash (max cement alkali 0.75 percent) Cement with Minimum 30 percent Class F fly ash (maximum cement alkali 0.83 percent).

  Washington: Low Alkali Cement shall be used, the percentage of alkalis in the cement shall not exceed 0.60 percent by weight calculated as Na_20 plus 0.658 K_20 or by using 25 percent Class F fly ash by total weight of the cementitious materials.

  New Mexico: As required to mitigate ASR expansion, but not less than 20 percent by weight of cement only for binary blends; as long as the total Pozzolan dosage is at least 20 percent.

  Nevada: Approved Type F or Type N Pozzolan, or with cement designated Type IP use 1:4 Pozzolan: Cement by mass. The limitation on replacement of cement with Pozzolans is a maximum of 17 percent.

- **Lithium Nitrate Admixtures.**

  Idaho: 30 percent Lithium Nitrate by mass in aqueous solution.

  California: 30 percent Lithium Nitrate by mass in aqueous solution.

  Virginia: Lithium Nitrate (30 percent by mass in aqueous solution).

  New Mexico: 0.55 gal/yd^3 of solution for each pound of cement sodium equivalent (30 percent by mass in aqueous solution).

  Nevada: 30 percent aqueous Lithium Nitrate solution.

Mitigation of Alkali Silica Reactivity with Fly Ash

In this study, several aggregate samples were obtained from sources along the Snake River Plain and tested for potential reactivity. If the aggregate was identified as reactive, different mitigation experiments were conducted to identify the most effective and economical method. The admixtures used in this experiment were six samples Class F fly ash from different sources, lithium nitrate solution, and combination of both with different percentages and dosage. ASTM C1260 test (14- and 28-day) was employed to identify the reactivity of the aggregate and ASTM C1567 test (14- and 28-day) were used to examine the effectiveness of the admixtures in reducing ASR. The testing methods selected in this study were primarily based on the current state-of-the-art research and the time constraints of the project.

For the purpose of this study, 8 aggregates from the sources within ITD districts 3, 5, and 6 were obtained. Table 17 shows the aggregate source used in this study.

Table 17. Aggregate Sources Tested in This Study

Aggregate Source	ITD District
EL116C EL37C OR-8-C OR-16-C	District 3
PW84 BG 112	District 5
BN-136-C BN-140-C	District 6

The six Class F fly ash samples tested in this study came from six different power plants located in the western United States. These are identified in Table 18. The mineral compositions of these fly ash samples meet the specification limits set forth by ITD.

Table 18. Class F Fly Ash Sources and Locations

No.	Class F Fly Ash Source	Location
1	Jim Bridger Fly Ash	Green River, Wyoming
2	Navajo Fly Ash	Albuquerque, New Mexico
3	Four Corners Fly Ash	Fruitland, New Mexico
4	Gallup Fly Ash	Gallup, New Mexico
5	San Juan Fly Ash, San Juan Generating Station	Water Flow, New Mexico
6	Cholla Fly Ash	Joseph City, Arizona

The mineral composition of the fly ash samples listed above is provided in Table 19 (collected from the source company). The Lithium Nitrate used in this experiment was 30 percent concentration provided by FMC Corporation, in Charlotte, North Carolina, USA**.** The list of experiments conducted and those in progress are given in Section 2.11 below.

Initially, the ASTM C1260 tests were conducted on three aggregate sources, BG112, BN 136-c, and PW84 using fine aggregate, coarse aggregate, and the combination (40/60) to identify the level of their reactivity. Table 23 lists 14- and 28-day expansion rates for each of these sources, and Figure 53 compares the results graphically.

Table 19. Chemical Analysis of Class F Fly Ash

Source	Silicon Dioxide SiO_2 (percent)	Aluminum Oxide Al_2O_3 (percent)	Ferric Oxide Fe_2O_3 (percent)	Calcium Oxide CaO (percent)	Magnesium Oxide MgO (percent)	Sulfur Trioxide SO_3 (percent)	Moisture Content (percent)	Loss of Ignition (percent)	Sodium Oxide Na_2O (percent)	Potassium Oxide K_2O (percent)	R Factor (percent)
Gallup Fly Ash	61.78	26.05	3.94	3.05	1.38	0.28	0.03	0.28	2.07	1.15	-0.49
San Juan Fly Ash	59.86	28.94	3.44	3.98	1.24	0.31	0.04	0.39	2.10	1.16	-0.30
Four Corners Fly Ash	62.06	26.01	3.96	3.20	1.37	0.26	0.03	0.25	2.04	1.11	-0.45
Cholla Fly Ash	62.42	23.19	3.88	3.96	1.62	0.32	0.04	0.27	2.37	1.43	-0.27
Navajo Fly Ash	58.50	22.10	4.50	7.40	-	0.50	0.00	0.20	0.70	0.32	-
Jim Bridger Fly Ash	64.30	16.60	4.00	6.20	-	0.90	0.10	0.30	1.17	0.43	-

***R Factor is R = CaO-5/Fe_2O_3 deals with the sulfate resistance of fly ash.**

List of Experiments Conducted

Aggregate Source: PW84

1. ASTM C1260 with Coarse Aggregate.
2. ASTM C1260 with Fine Aggregate.
3. ASTM C1260 with Coarse Aggregate (60 percent) & Fine Aggregate (40 percent).

Aggregate Source BN136-c

4. ASTM C1260 with Coarse Aggregate.
5. ASTM C1260 with Fine Aggregate.
6. ASTM C1260 with Coarse Aggregate (60 percent) & Fine Aggregate (40 percent).

Aggregate Source BG112

7. ASTM C1260 with Coarse Aggregate.
8. ASTM C1260 with Fine Aggregate.
9. ASTM C1260 with Coarse Aggregate (60 percent) & Fine Aggregate (40 percent).

Aggregate Source: PW84

10. ASTM C1567 Jim Bridger’s fly ash 25 percent replacement with cement.
11. ASTM C1567 Navajo fly ash 5 percent replacement with cement.
12. ASTM C1567 Four Corners fly ash 25 percent replacement with cement.
13. ASTM C1567 Cholla’s fly ash 25 percent replacement with cement.
14. ASTM C1567 Gallup fly ash 25 percent replacement with cement.
15. ASTM C1567 San Juan fly ash 25 percent replacement with cement.
16. ASTM C1567 Four Corners fly ash 5 percent replacement with cement.
17. ASTM C1567 Four Corners fly ash 10 percent replacement with cement.
18. ASTM C1567 Four Corners fly ash 15 percent replacement with cement.
19. ASTM C1567 Four Corners fly ash 20 percent replacement with cement.
20. ASTM C1567 Jim Bridger’s fly ash 30 percent replacement with cement.
21. ASTM C1567 Jim Bridger’s fly ash 35 percent replacement with cement.

Lithium Nitrate with High Alkali Cement

22. ASTM C1567 with 100 percent lithium nitrate and 0 percent fly ash.
23. ASTM C1567 with 80 percent lithium nitrate and 5 percent f Jim Bridger’s fly ash.
24. ASTM C1567 with 60 percent lithium nitrate and 10 percent Jim Bridger’s fly ash.
25. ASTM C1567 with 40 percent lithium nitrate and 15 percent Jim Bridger’s fly ash.
26. ASTM C1567 with 20 percent lithium nitrate and 20 percent Jim Bridger’s fly ash.

Lithium Nitrate with Low Alkali Cement

27. ASTM C1567 with 100 percent lithium nitrate
28. ASTM C1567 with 80 percent lithium nitrate and 5 percent Jim Bridger's fly ash.
29. ASTM C1567 with 60 percent lithium nitrate and 10 percent Jim Bridger's fly ash.
30. ASTM C1567 with 40 percent lithium nitrate and 15 percent Jim Bridger's fly ash.
31. ASTM C1567 with 20 percent lithium nitrate and 20 percent Jim Bridger's fly ash.

Aggregate Source: OR-8c

32. ASTM C1567 Jim Bridger's fly ash 30 percent replacement with cement.
33. ASTM C1567 Jim Bridger's fly ash 35 percent replacement with cement.
34. ASTM C1567 with 100 percent lithium nitrate and 0 percent fly ash.
35. ASTM C1567 with 80 percent lithium nitrate and 5 percent Jim Bridger's fly ash.
36. ASTM C1567 with 20 percent lithium nitrate and 20 percent Jim Bridger's fly ash.

Aggregate Source: EL-37

37. ASTM C1567 Jim Bridger's fly ash 30 percent replacement with cement.
38. ASTM C1567 Jim Bridger's fly ash 35 percent replacement with cement.
39. ASTM C1567 with 100 percent lithium nitrate and 0 percent fly ash.
40. ASTM C1567 with 80 percent lithium nitrate and 5 percent Jim Bridger's fly ash.
41. ASTM C1567 with 20 percent lithium nitrate and 20 percent Jim Bridger's fly ash.

Aggregate Source: BN-140c

42. ASTM C1567 Jim Bridger's fly ash 30 percent replacement with cement.
43. ASTM C1567 Jim Bridger's fly ash 35 percent replacement with cement.
44. ASTM C1567 with 100 percent lithium nitrate and 0 percent fly ash.
45. ASTM C1567 with 80 percent lithium nitrate and 5 percent Jim Bridger's fly ash.
46. ASTM C1567 with 20 percent lithium nitrate and 20 percent Jim Bridger's fly ash.

Aggregate Source: OR-16c

47. ASTM C1567 Jim Bridger's fly ash 30 percent replacement with cement.
48. ASTM C1567 Jim Bridger's fly ash 35 percent replacement with cement.
49. ASTM C1567 with 100 percent lithium nitrate and 0 percent fly ash.
50. ASTM C1567 with 80 percent lithium nitrate and 5 percent Jim Bridger's fly ash.
51. ASTM C1567 with 20 percent lithium nitrate and 20 percent Jim Bridger's fly ash.

Aggregate Source: EL-116

52. ASTM C1567 Jim Bridger's fly ash 30 percent replacement with cement.
53. ASTM C1567 Jim Bridger's fly ash 35 percent replacement with cement.
54. ASTM C1567 with 100 percent lithium nitrate and 0 percent fly ash.
55. ASTM C1567 with 80 percent lithium nitrate and 5 percent Jim Bridger's fly ash.
56. ASTM C1567 with 20 percent lithium nitrate and 20 percent Jim Bridger's fly ash.

Aggregate Source: BN-136c

57. ASTM C1567 Jim Bridger's fly ash 30 percent replacement with cement.
58. ASTM C1567 Jim Bridger's fly ash 35 percent replacement with cement.
59. ASTM C1567 with 100 percent lithium nitrate and 0 percent fly ash.
60. ASTM C1567 with 80 percent lithium nitrate and 5 percent Jim Bridger's fly ash.
61. ASTM C1567 with 20 percent lithium nitrate and 20 percent Jim Bridger's fly ash.

Aggregate Source: BG-112

62. ASTM C1567 Jim Bridger's fly ash 30 percent replacement with cement.
63. ASTM C1567 Jim Bridger's fly ash 35 percent replacement with cement.
64. ASTM C1567 with 100 percent lithium nitrate and 0 percent fly ash.
65. ASTM C1567 with 80 percent lithium nitrate and 5 percent Jim Bridger's fly ash.
66. ASTM C1567 with 20 percent lithium nitrate and 20 percent Jim Bridger's fly ash.

Table 20. ASTM C1260 for the Aggregate Source PW-84

	14-day Expansion	28-day Expansion
CA	0.1536	0.2665
FA	0.1775	0.2869
CA & FA	0.1655	0.2782

CA: Coarse Aggregate, FA: Fine Aggregate

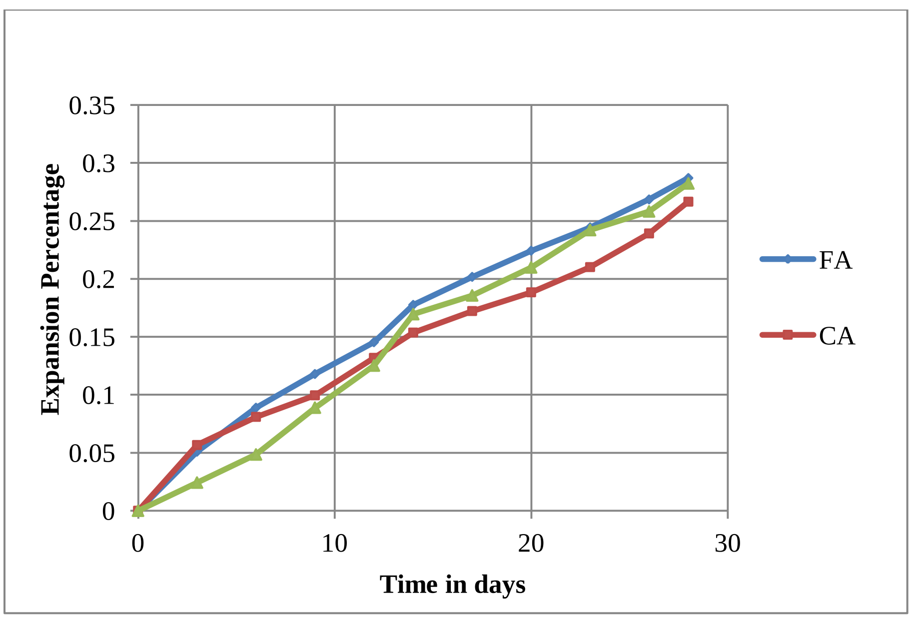


Figure 52. Comparison of ASTM C1260 (CA, FA, CA & FA) for the Aggregate Source PW-84

Table 21. ASTM C1260 for the Aggregate Source BN-136-C

	14-day Expansion	28-dayExpansion
CA	0.1234	0.2426
FA	0.2575	0.5040
CA & FA	0.2654	0.4585

CA: Coarse Aggregate, FA: Fine Aggregate

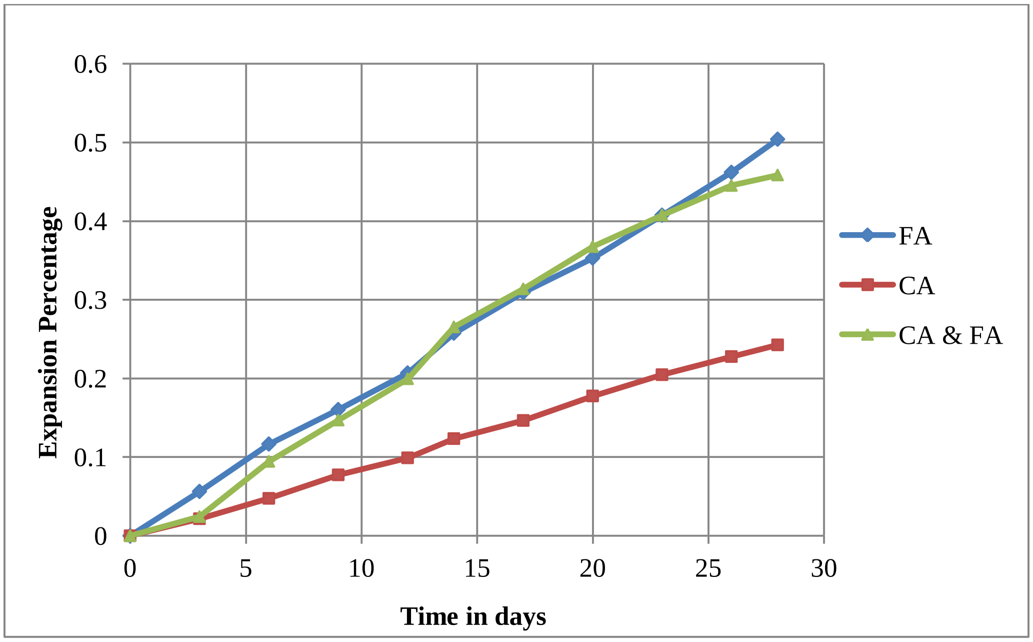


Figure 53. Comparison of ASTM C1260 (CA, FA, CA & FA) for the Aggregate Source BN-136-C

Table 22. ASTM C1260 for the Aggregate Source BG-112

	14-day Expansion	28-day Expansion
CA	0.1302	0.2537
FA	0.2077	0.4376
CA & FA	0.1701	0.3410

CA: Coarse Aggregate, FA: Fine Aggregate

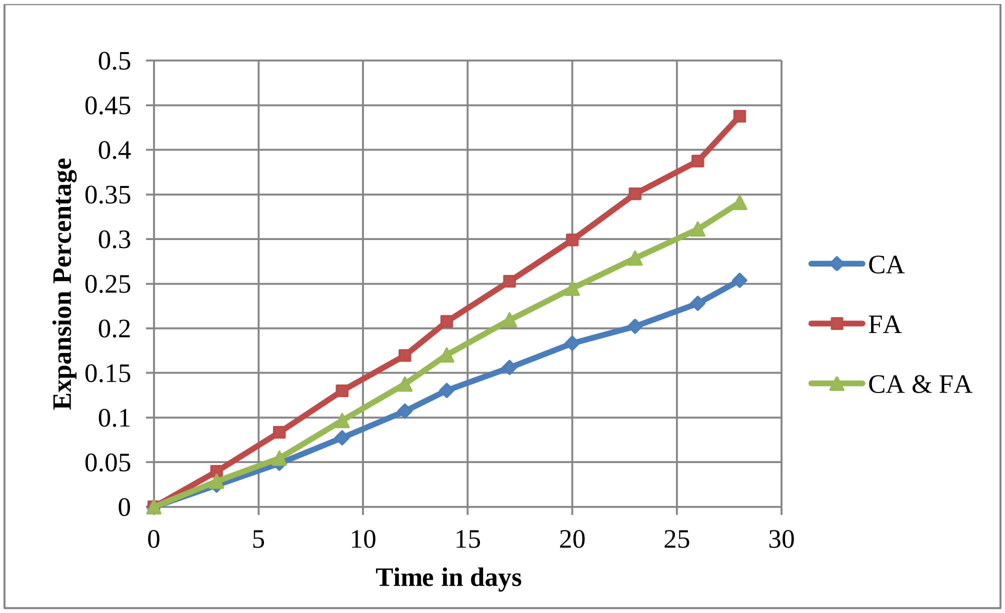


Figure 54. Comparison of ASTM C1260 (CA, FA, CA & FA) for the Aggregate Source BG-112

Table 23. Comparison of All Aggregate Sources PW84, BG112 and BN-136c (ASTM C1260)

	14-day Expansion	28-day Expansion
PW-84	0.1695	0.2823
BN-136c	0.2654	0.4585
BG 112	0.1701	0.3410

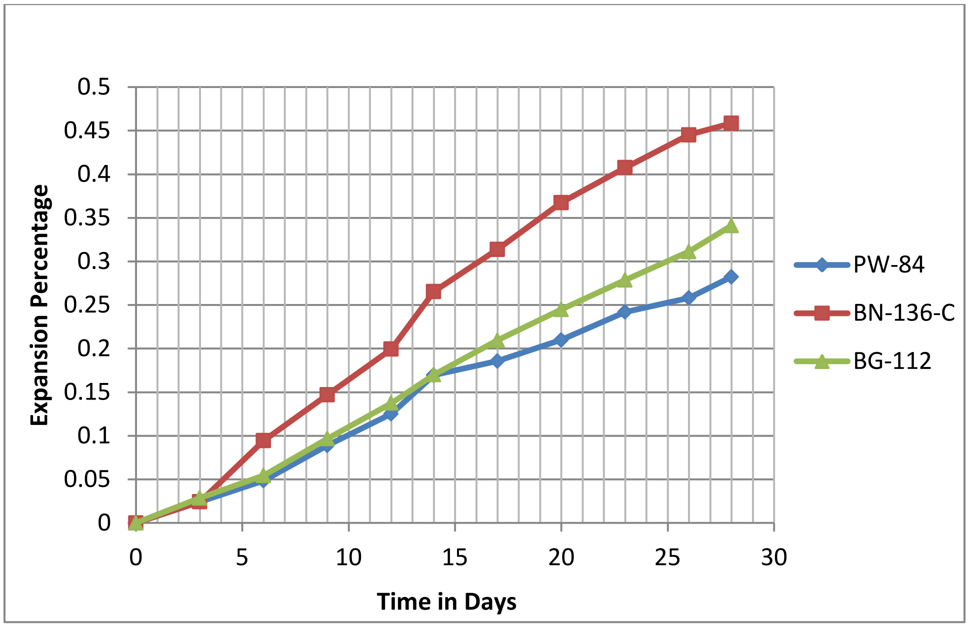


Figure 55. Comparison of the ASTM C1260 Test for 3 Selected Aggregate Sources

Standard ASTM C1567 tests (16-day mortar bar tests) as well as extended ASTM C1567 tests (28-day mortar bar) were conducted on all 6 fly ash samples using 25 percent cement replacement. The expansion rate for each was measured, and these results are displayed in Table 24 and presented graphically in Figure 56. The results indicated that the Jim Bridger and the Navajo fly ash samples failed the test by exceeding the 0.1 percent limit for both 14- and 28-day tests. For the remaining 4 fly ash samples, the expansion rates were within the limit as shown in Table 24.

Table 24. Average Percent Expansion for PW-84 Using 25 Percent Fly Ash

Fly Ash	14-day Expansion	28-day Expansion
Jim Bridger	0.1219	0.1492
Navajo	0.1230	0.1468
Four Corners	0.0308	0.0608
Cholla	0.0250	0.0331
Gallup	0.0417	0.0598
San Juan	0.0375	0.0449

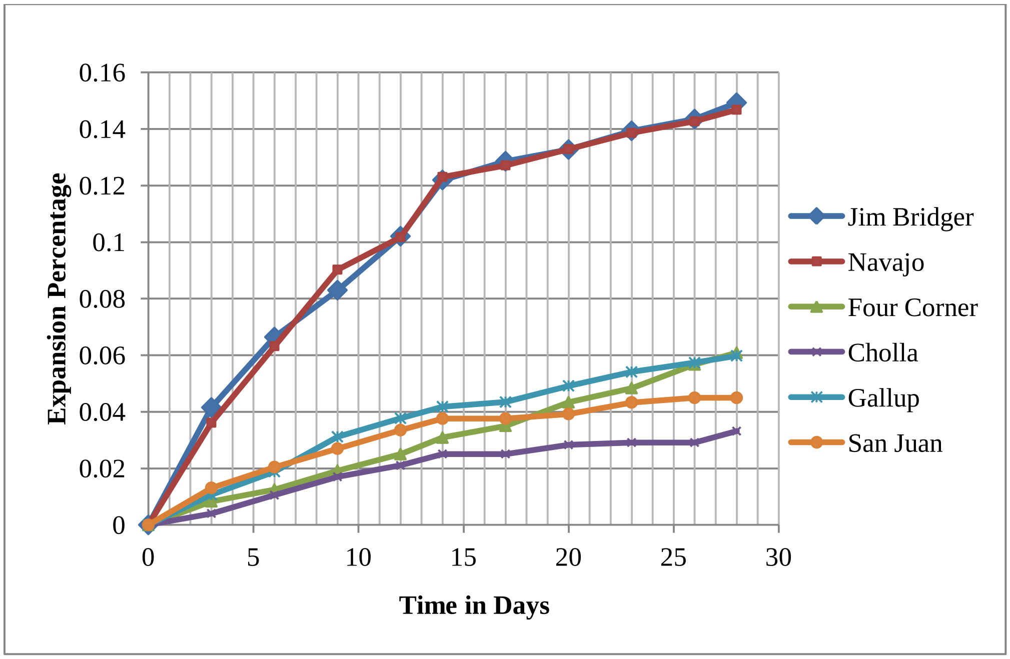


Figure 56. Results of ASTM C1567 Tests for All Fly Ashes with 25 percent Replacement Used on PW-84

The Jim Bridger and Navajo fly ashes, were supplied directly from the power plants without any further processing. Whereas the remaining 4 fly ashes, with expansion rate of below 0.1 percent, were processed and the particle sizes for these 4 fly ashes was less than 45 micron. These fly ash samples were processed using centrifugal methods at the Salt River Material Group (SRMG) located in Scottsdale, Arizona. The processing reduced the particle size to less than 45 microns, that is, the particles passing through ASTM sieve size No. 325.

Out of 4 fly ashes which proved effective with 25 percent replacement, Four Corners fly ash was selected and tested with different percentages (5 percent, 10 percent, 15 percent, 20 percent, and 25 percent). From the results obtained, it was determined that Four Corners fly ash was also effective with 20 percent replacement as shown in Table 25 and Figure 57. The Jim Bridger fly ash was chosen from the 2 fly ashes which were not effective with 25 percent cement replacement. It was tested by increasing the fly ash percentages to 30 percent and 35 percent. Both these tests were found to be effective compared to the 25 percent replacement as shown in Table 26 and Figure 58. Replacing fly ash in excess of 30 percent may delay the 28-day strength gain in concrete.

Table 25. Average Percent Expansion for PW-84 Using Four Corners Fly Ash

Fly Ash (percent)	14-day Expansion	28-day Expansion
5 percent	0.0652	0.1628
10 percent	0.0857	0.1492
15 percent	0.0399	0.0701
20 percent	0.0375	0.0723
25 percent	0.0308	0.0608

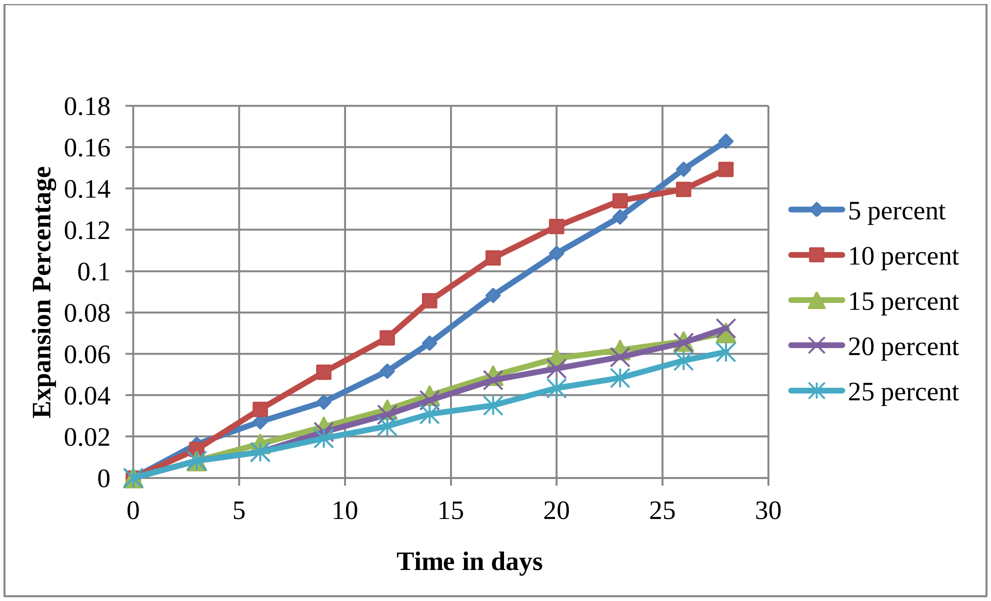


Figure 57. Results of ASTM C1567 Tests for the Four Corner Fly Ash (PW-84) with (5 percent, 10 percent, 15 percent, 20 percent, and 25 percent)

Table 26. Average Percent Expansion for PW-84 Using Jim Bridger Fly Ash (25 percent, 30 percent, and 35 percent)

Fly Ash (Percent)	14-day Expansion	28-day Expansion
25 percent	0.1219	0.1492
30 percent	0.0358	0.0676
35 percent	0.0234	0.0497

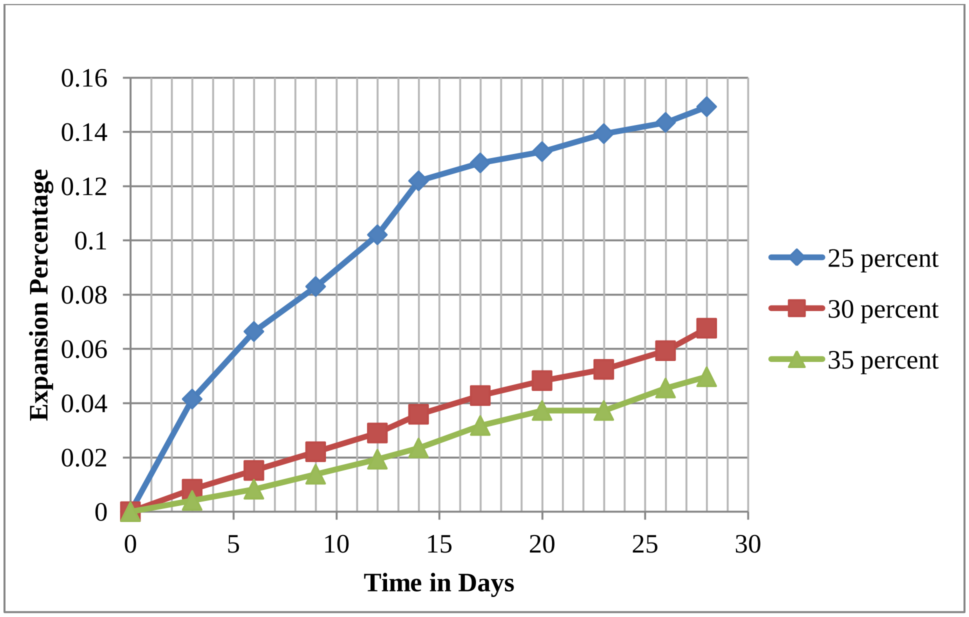


Figure 58. ASTM C1567 Test Using Jim Bridger Fly Ash for PW-84 (25 percent, 30 percent, and 35 percent)

Experiments with Lithium Nitrate and Combination with Fly Ash

Lithium Nitrate solution with 30 percent concentration is known to be effective in the mitigation ASR in concrete. The recommended dosage for effectiveness is 0.55 gallon of lithium nitrate for every pound of alkali in the cement. Several experiments were conducted on PW84 aggregate using different dosages of lithium nitrate solution with high and low alkali cements and different percentages of fly ash from the Jim Bridger plant. High alkali cement was obtained by adding kiln dust. The percentage of alkali in the cement was increased from 0.54 percent to 0.9 percent. Table 27 and Figure 59 show the results of the ASTM C1567 tests (14- and 28-day) for different combination of lithium nitrate and Jim Bridger fly ash with low alkali cement. Table 28 and Figure 60 show the results with high alkali cement.

Note: The dosage of lithium nitrate is the percent of recommended dosage that provides 0.55 gallon per pound of alkali in the cement.

Table 27. ASTM C1567 Test (PW-84) with Different Combination of Lithium Nitrate and Jim Bridger Fly Ash with Low Alkali Cement

Percent Combination	14-day Expansion	28-day Expansion
100 Percent ($LiNO_3$) & 0 Percent Fly Ash	0.0545	0.0803
80 Percent ($LiNO_3$) & 5 Percent Fly Ash	0.0332	0.0636
60 Percent ($LiNO_3$) & 10 Percent Fly Ash	0.0437	0.0957
40 Percent ($LiNO_3$) & 15 Percent Fly Ash	0.0603	0.1014
20 Percent ($LiNO_3$) & 20 Percent Fly Ash	0.0408	0.0599

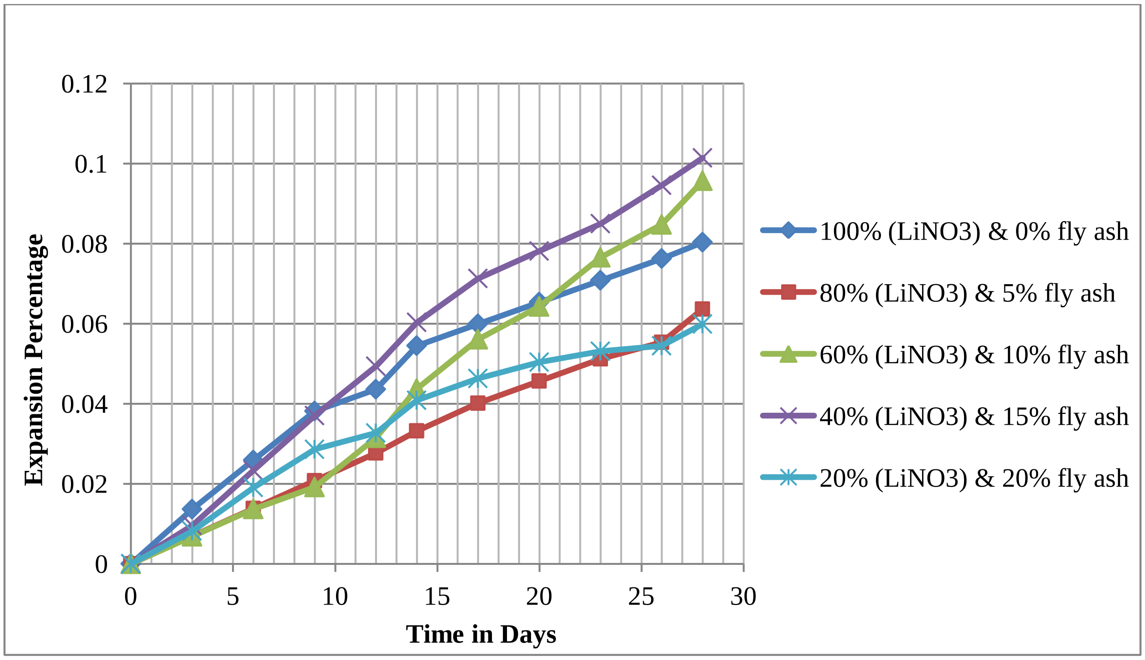


Figure 59. ASTM C1567 Combination of Lithium Nitrate ($LiNO_3$) & Fly Ash (Jim Bridger) & Low Alkali Cement (Without Dust)

Table 28. ASTM C1567 Test with Different Combination of Lithium Nitrate and Jim Bridger Fly Ash with High Alkali Cement (with Dust)

Percent Combination	14-day Expansion	28-day Expansion
100 Percent ($LiNO_3$) & 0 Percent Fly Ash	0.0384	0.0782
80 Percent ($LiNO_3$) & 5 percent Fly Ash	0.0438	0.0862
60 percent ($LiNO_3$) & 10 Percent Fly Ash	0.0872	0.1240
40 percent ($LiNO_3$) & 15 Percent Fly Ash	0.0847	0.1366
20 percent ($LiNO_3$) & 20 Percent Fly Ash	0.0584	0.0923

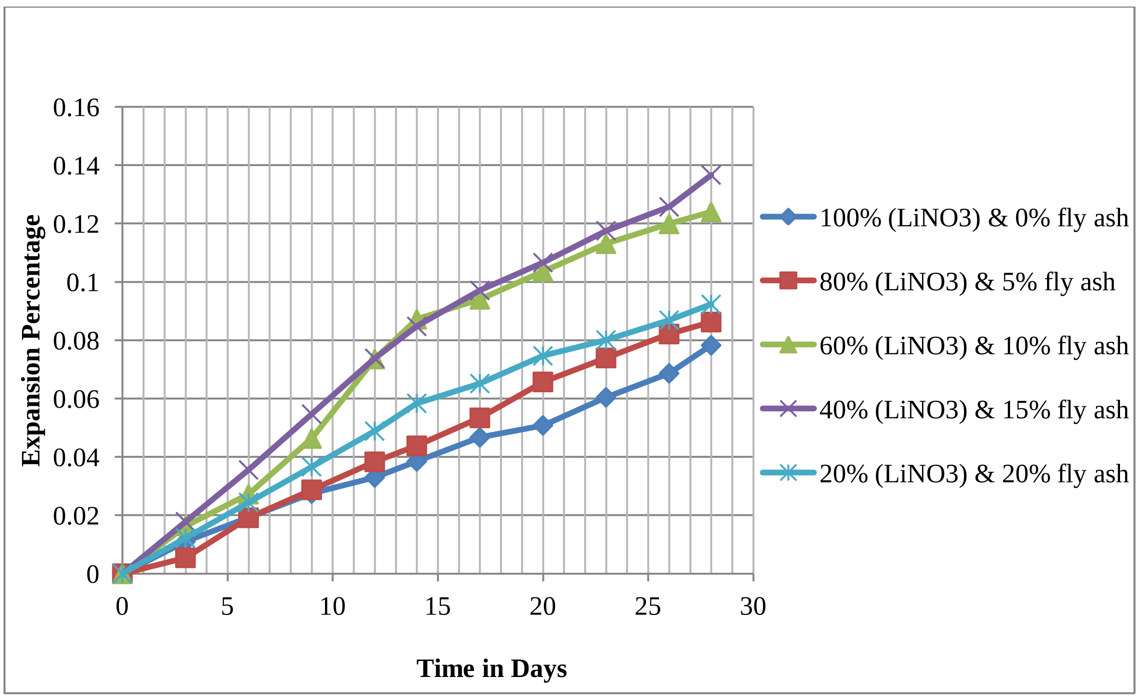


Figure 60. ASTM C1567 Combination of Lithium Nitrate ($LiNO_3$) and Fly Ash (Jim Bridger) with High Alkali Cement (With Dust)

Table 29 and Figure 61 summarize the results of the combination of lithium nitrate solution and fly ash for both low and high alkali cement.

Table 29. Summary of Test Results for the Combination of Lithium Nitrate and Fly Ash with Low and High Alkali Cement

	14-day Expansion With Dust	14-day Expansion Without Dust	28-day Expansion With Dust	28-day Expansion Without Dust
100 Percent ($LiNO_3$) & 0 Percent Fly Ash	0.0384	0.0545	0.0782	0.0803
80 Percent ($LiNO_3$) & 5 Percent Fly Ash	0.0438	0.0332	0.0862	0.0636
60 Percent ($LiNO_3$) & 10 Percent Fly Ash	0.0872	0.0437	0.1240	0.0957
40 Percent ($LiNO_3$) & 15 Percent Fly Ash	0.0847	0.0603	0.1366	0.1014
20 Percent ($LiNO_3$) & 20 Percent Fly Ash	0.0584	0.0408	0.0923	0.0599

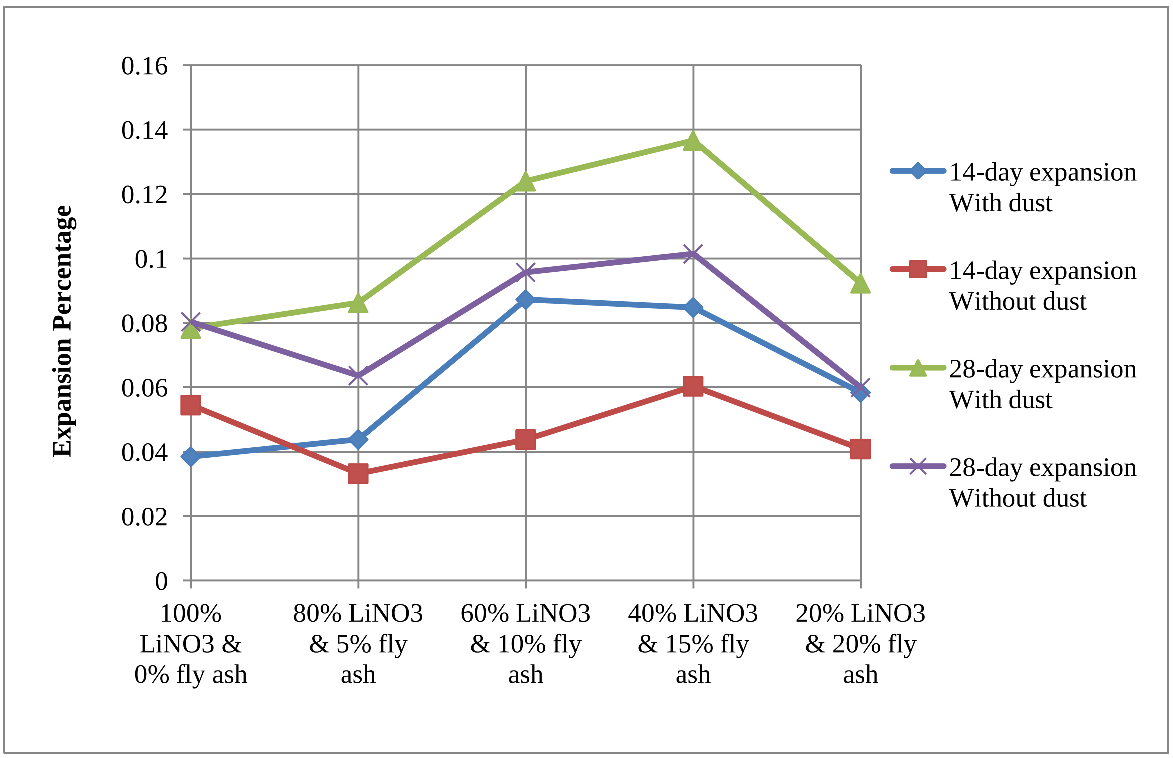


Figure 61. ASTM C1567 Test Comparing High Alkali and Low Alkali Cements

Experiments with the Remaining Aggregate Sources

The above experiments were conducted on PW84 to identify the most effective and economical mixture for ASR mitigation. The results indicated that 100 percent of the recommended dosage for lithium nitrate solution was effective in mitigation ASR on PW84 aggregate with both high and low alkali cement when using the ASTM C1567 (14- and 28-day) test. The use of 100 percent lithium nitrate was tested on the remaining 7 aggregates to examine its effectiveness. Table 30 and Figure 62 summarize the result of these experiments. The results indicate that, indeed, 100 percent of recommended dosage for lithium nitrate solution is effective in all of the samples known to be reactive. These experiments were conducted using both low and high alkali cement in the concrete. From the results of the aggregate source PW84 the most effective combinations of lithium nitrate and fly ash were chosen and experiments were conducted with the remaining aggregate sources. The combinations include 80 percent lithium nitrate and 5 percent fly ash & 20 percent lithium nitrate and 20 percent fly ash. The results are shown in Tables 31, 32 and the plots are presented in Figures 63 and 64. In addition, the Jim Bridger fly ash with 30 percent and 35 percent replacement was tested on the remaining aggregates and the results are shown in Tables 33 and 34 and the plots are depicted in Figures 65 and 66.

Table 30. ASTM C1567 Comparison of All Sources (100 Percent of the Recommended dosage for Lithium Nitrate)

Time in Days	PW-84	OR-8-C	OR-16-C	EL-116-C	BN-140-C	EL-37-C	BN-136-C	BG-112
14	0.0545	0.0512	0.0432	0.0416	0.0481	0.0467	0.0444	0.0400
28	0.0803	0.0863	0.0972	0.0873	0.0935	0.0986	0.0861	0.0828

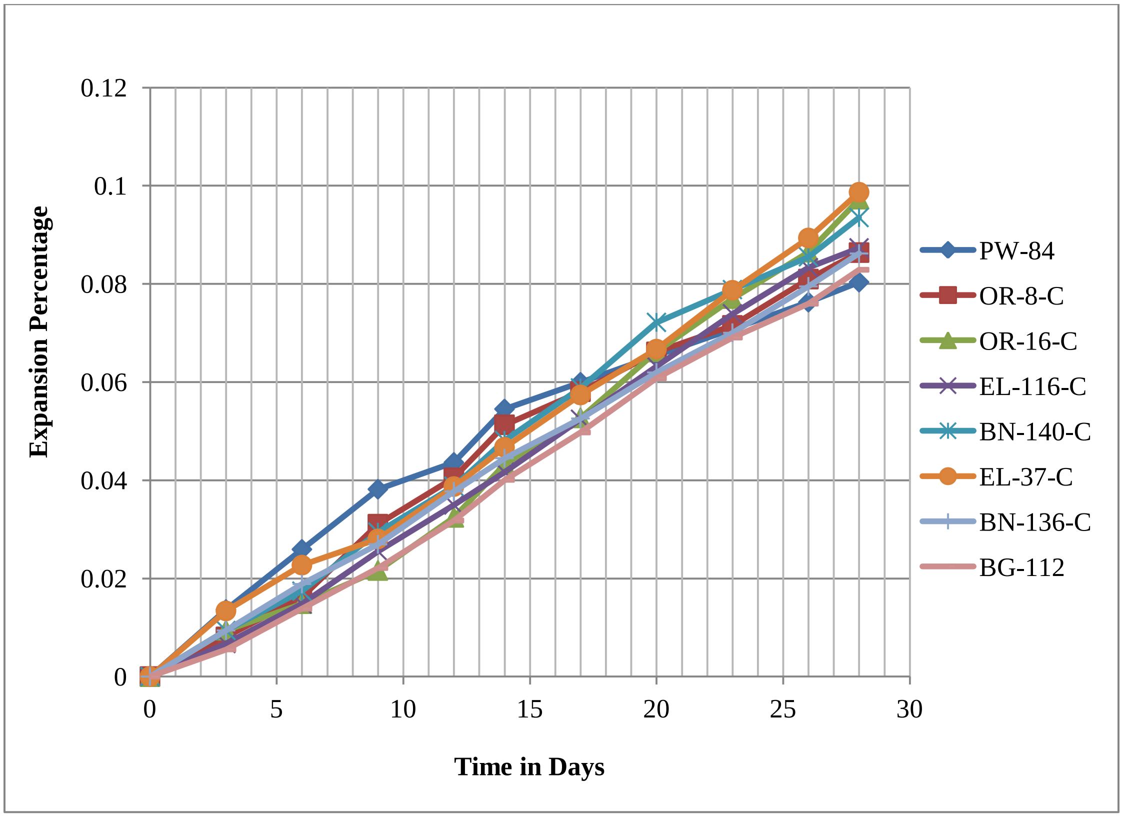


Figure 62. ASTM C1567 Comparison of All Sources 100 percent Lithium Nitrate and Low Alkali Cement

Table 31. ASTM C1567 Comparison of All Sources (20 percent Lithium Nitrate & 20 percent Jim Bridger Fly Ash)

Time in Days	PW-84	OR-8-C	EL-37	BN-140-C	OR-16-C	EL-116	BN-136-C	BG-112
14	0.0408	0.0266	0.0272	0.0280	0.0257	0.0269	0.0215	0.0364
28	0.0599	0.0466	0.0531	0.0480	0.0447	0.0457	0.0470	0.0661

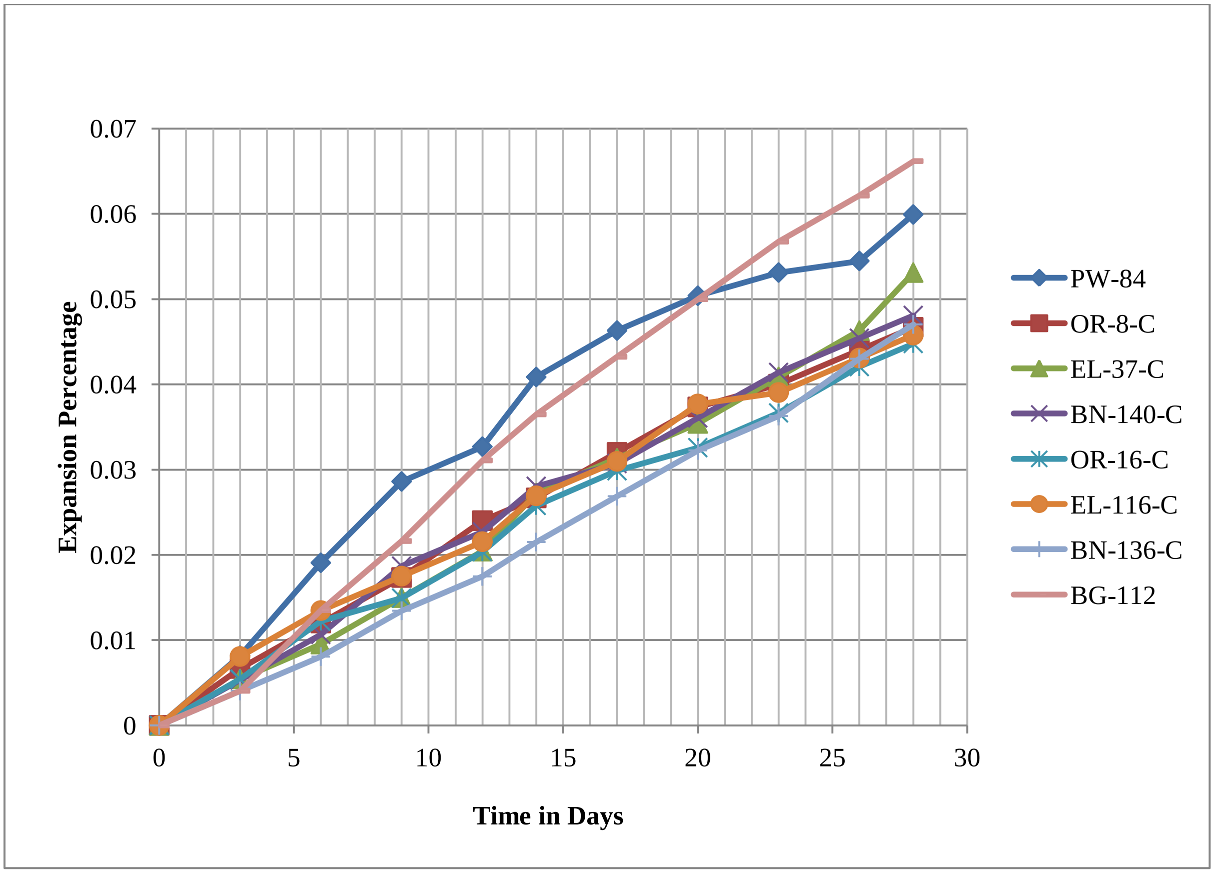


Figure 63. ASTM C1567 Comparison of All Sources 20 percent Lithium Nitrate & 20 Percent Fly Ash and Low Alkali Cement

Table 32. ASTM C1567 Comparison of All Sources (80 percent Lithium Nitrate & 5 Percent Jim Bridger Fly Ash)

Time in Days	PW-84	OR-8-C	EL-37	BN-140-C	OR-16-C	EL-116	BN-136-C	BG-112
14	0.0332	0.0266	0.0373	0.0306	0.0258	0.0296	0.0250	0.0374
28	0.0636	0.0506	0.0692	0.0533	0.0448	0.0485	0.0527	0.0681

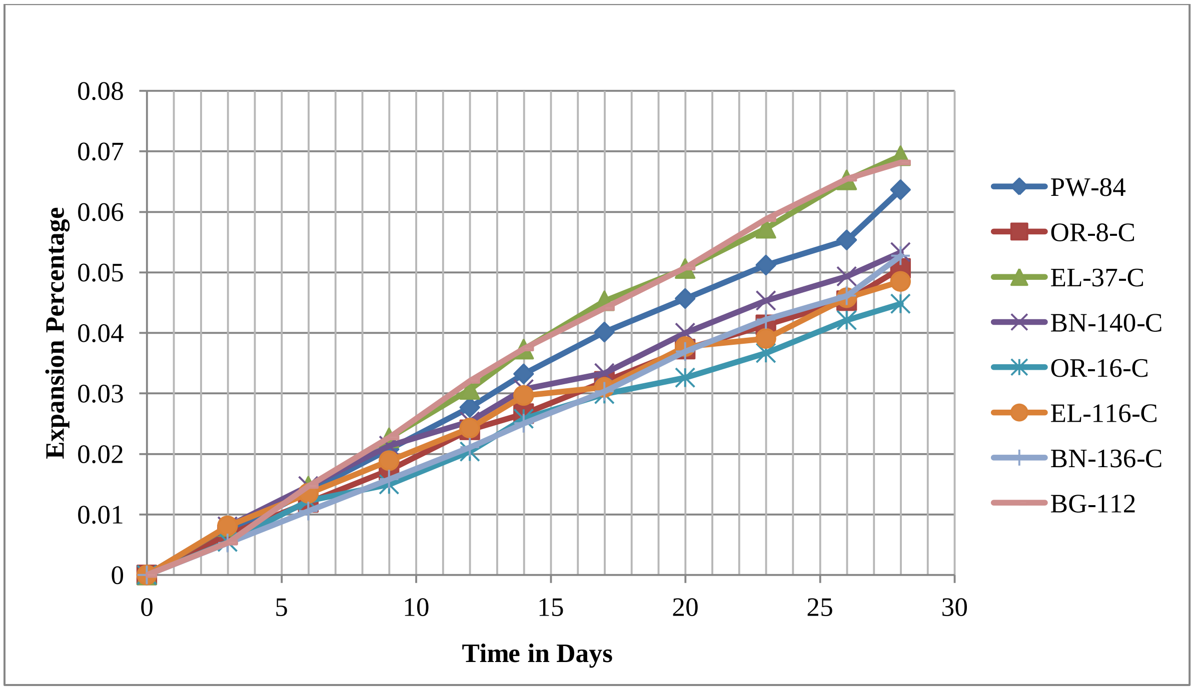


Figure 64. ASTM C1567 Comparison of All Sources 80 Percent Lithium Nitrate & 5 Percent Fly Ash and Low Alkali Cement

Table 33. ASTM C1567 Comparison of All Sources (30 Percent Jim Bridger Fly Ash)

Time in Days	PW-84	OR-8-C	EL-37	BN-140-C	OR-16-C	EL-116	BN-136-C	BG-112
14	0.0358	0.0338	0.0482	0.0334	0.0496	0.0376	0.0348	0.0398
28	0.0676	0.0757	0.0857	0.0734	0.0885	0.0752	0.0764	0.0823

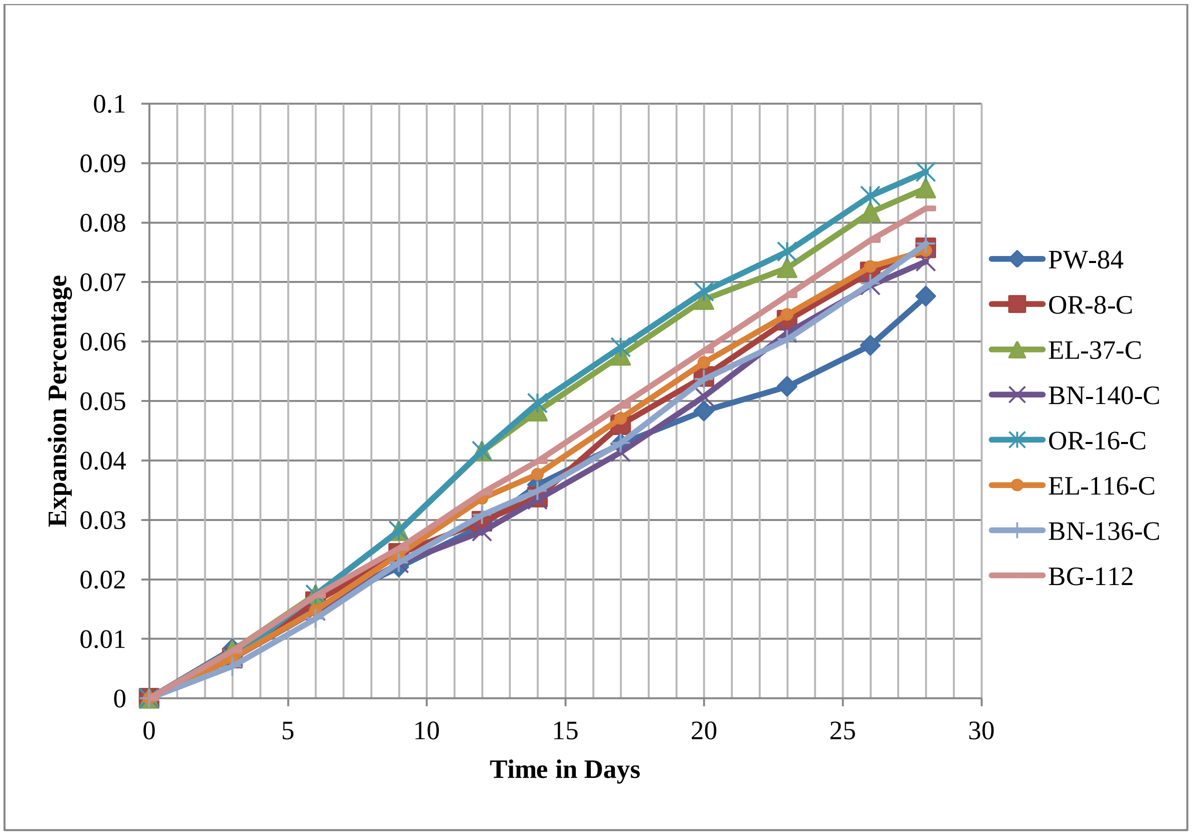


Figure 65. ASTM C1567 Comparison of All Sources with 30 Percent Jim Bridger Fly Ash and Low Alkali Cement

Table 34. ASTM C1567 Comparison of All Sources (35 Percent Jim Bridger Fly Ash)

Time in Days	PW-84	OR-8-C	EL-37	BN-140-C	OR-16-C	EL-116	BN-136-C	BG-112
14	0.0234	0.0235	0.0469	0.0340	0.0390	0.0335	0.0413	0.0421
28	0.0497	0.0497	0.0831	0.0599	0.0727	0.0670	0.0746	0.0771

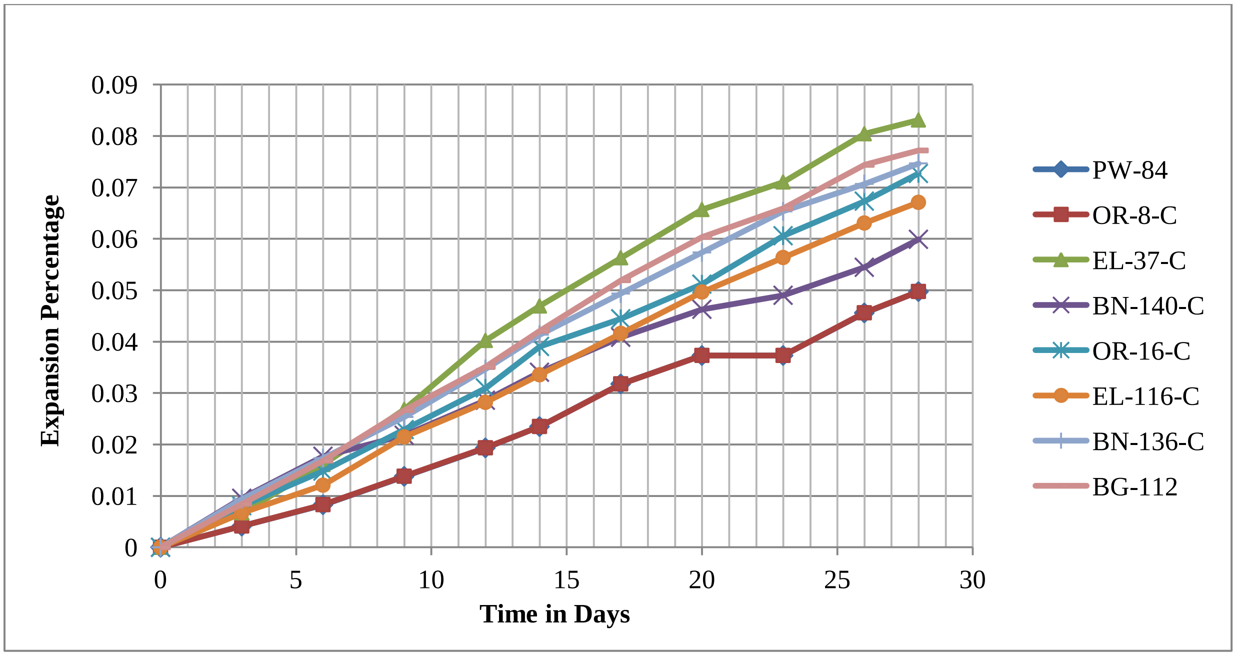


Figure 66. ASTM C1567 Comparison of All Sources with 35 Percent Jim Bridger Fly Ash and Low Alkali Cement

Conclusions

The conclusions drawn from this study are summarized below.

- Lithium nitrate solution is effective in mitigating ASR when 100 percent of the recommended dosage is used in the solution for all aggregate sources used in this experiment.
- Unprocessed Class F fly ash obtained directly from the Jim Bridger and Navajo power plants, tested on all aggregate sources, is effective in mitigating ASR when 30 percent and 35 percent fly ash replaces cement in the concrete. The 25 percent cement replacement did not meet the expansion limit of 0.1 percent. Replacing cement with fly ash in excess of 30 percent may delay the 28-day strength gain in concrete.
- The processed Class F fly ashes with particle size under 45 microns were effective in mitigating ASR when tested on PW84 aggregate. It didn't seem relevant to test these fly ashes on the remaining aggregate sources since it would be prohibitively expensive to use manufactured fly ash from Arizona.
- The combination of lithium nitrate and Class F fly ash from the Jim Bridger plant was effective when 20 percent and 15 percent fly ash was used in combination with 20 percent and 40 percent of the recommended dosage for lithium nitrate, respectively.

Recommendations

The following recommendations are drawn from the above experiments.

1. The most cost effective ASR mitigation method is to use 20 percent Class F fly ash replacement in combination with lithium nitrate (30 percent concentrate) in the amount of 20 percent of the recommended dosage for every pound of alkali in concrete.
2. Perhaps, the most cost effective method of mitigating ASR is to replace cement with Class F fly ash in excess of 30 percent. Excessive fly ash in concrete will delay the 28-day strength gain, and its effects on concrete strength and long-term performance require further study due to change in the composition of the fly ash obtained from different plants.

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Appendix A

Flow Chart Representation of Delphi Process

The following Flow Chart summarizes the Delphi Process and the sequence of actions necessary to complete the study

Delphi Planning

- Transposing the framework into a set of questions
- Formation of the criteria for participants selection
- Preparing the questionnaires and supporting letters

↓

Delphi Round 1

- Objective: To test the desirability and feasibility of components of the framework
- Contents of the survey: framework components for each issue, desirability and feasibility rating on a scale of 1 to10 and space for comments.
- Survey requirements: Ask participants to share their experiences and opinions about the components of the framework.

↓

Delphi Round 2

- Objective: To inform the participants of the results from Round1 and provide opportunity to review their ratings in the light of the average results
- Contents of the survey: average result, participant score from the Round1, amendments, comments
- Monitoring the return rate. Reminder sent via e-mail ten days after the second questionnaire was sent.

↓

Decision on Delphi Progress

Progress to Delphi Round 3 or Termination of the Delphi Survey

- Analysis of the results from Round 2 with regards to changes of opinions.
- Design more refined questionnaire in the areas of disagreement among the participants.
- Proceed to Round 3.

Delphi Round 3

- Objective: To inform the participants of the results from Round2 and give them an opportunity to review their scorings in the light of the average results
- Contents of the survey: The result of statistical analysis, participant's rating from the Round 2, amendments, comments

Progress to Delphi Round 4 or Termination of the Delphi Survey

- Analysis of the results from Round 3 with regards to changes of opinion and size of the sample.
- Decision to terminate the study.

Analysis of the Delphi Study Results

- Analysis of the results in categories of desirability and feasibility
- Analysis of the significance of the new components.
- Consideration and analysis of the qualitative feedback, i.e. the comments of the participants
- Analysis of the impact of the results of the survey on the proposed framework

Delphi Study Survey Questions Along with Summary of the Responses

Round One

In round one of this study, four questions were posed to generate initial discussions among the participants. The participants were provided with the ITD PCC specifications and were asked to respond to these questions in relation to the following two queries:

1. What is the risk of the material failing to meet ITD specifications?
2. What is the consequence of that material failing to meet ITD specifications?

The four questions were as follows:

1. Based on your knowledge, experience, and/or observations, and in reference to the summary of the ITD specifications for Portland cement concrete (PCC), is there any ITD specification that does not closely relate to field performance and needs modification(s)? Please list the specification, provide a brief comment about it, and make suggestions for improvement, if you have any.

2. In some of the ITD PCC specifications, there are limitations set forth for material acceptance including aggregate, cement, supplementary cementations material (SCM), and concrete as a final product. Some examples of the limitations include maximum alkali in cement 0.6 percent, loss of less than 12 percent in sodium sulfate soundness test, maximum CaO in fly ash 11 percent, maximum expansion of 0.1 percent in an ASTM C1260 test, and many other limitations as shown in the ITD specification summary. What does each of these limitations mean to you? In your opinion, is there any correlation between these specifications and the field performance of concrete?

3. In some areas of Idaho (mainly along the Snake River basin), the aggregate is known to be reactive and to promote Alkali-Silica reaction (ASR), resulting in premature deterioration of concrete. Please make comments on the ITD Specifications (attached) in dealing with ASR problems, and in your opinion, what works the best in mitigating ASR in concrete, considering Class F fly ash, silica fume, lithium nitrate solution, and combination and/or other Supplementary Cementations Material (SCM) used for mitigation. Are there any other methods to mitigate ASR in concrete? In your opinion, are there any other factor(s) contributing to the premature deterioration of PCC and what is the remedy for prevention?

4. In the ITD specifications, there are two tables depicting ITD specifications for the concrete mix recipe with and without fly ash. Please make any comments you may have regarding ITD mix recipes.

Round Two

All responses received from the participants, in the first round, were categorized, summarized, and shared with all the participants without mentioning the source of responses. A more detailed survey questions were developed to quantify the degree of consensus among the participants.

These responses received are presented below. For each category one or more multiple-choice questions are posed to gauge the degree of consensus among the participants.

The overall consensus among the participants was that the current ITD PCC specifications are largely consistent with the practice in other state DOTs throughout the country. However, there were a few comments regarding ITD concrete specifications and current concrete practices that required further discussion. In the following pages, a summary of the related comments and questions are presented along with the results of the survey of each question.

Voting Scale used in this study:

Confidence (In Validity of Argument or Promise)

- Certain
  - low risk of being wrong
  - decision based upon this will not be wrong because of this "fact"
  - most inferences drawn from this will be true
- Reliable
  - some risk of being wrong
  - willing to make a decision based on this but recognizing some chance of error
  - some incorrect inferences can be drawn
- Risky
  - substantial risk of being wrong
  - not willing to make a decision based on this alone
  - many incorrect inferences can be drawn
- Unreliable
  - great risk of being wrong
  - of no use as a decision maker

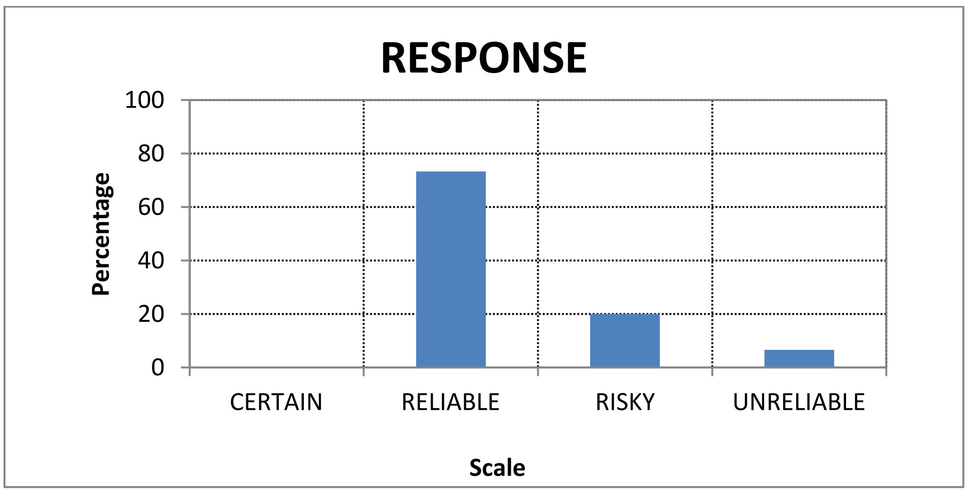
NOTE: The comments presented below are the summary of the responses from the round one survey questions.

Comment 1: There were several recommendations to replace the ITD's prescriptive-based approach to concrete mix design with a performance-based approach or a combination of perspective and performance-based approaches when durability is a prime concern. Changing ITD concrete practices to performance or to a combination of performance/prescriptive based mix design could eliminate some of the tests required for approval, which may result in cost reduction without compromising the quality and durability of concrete.

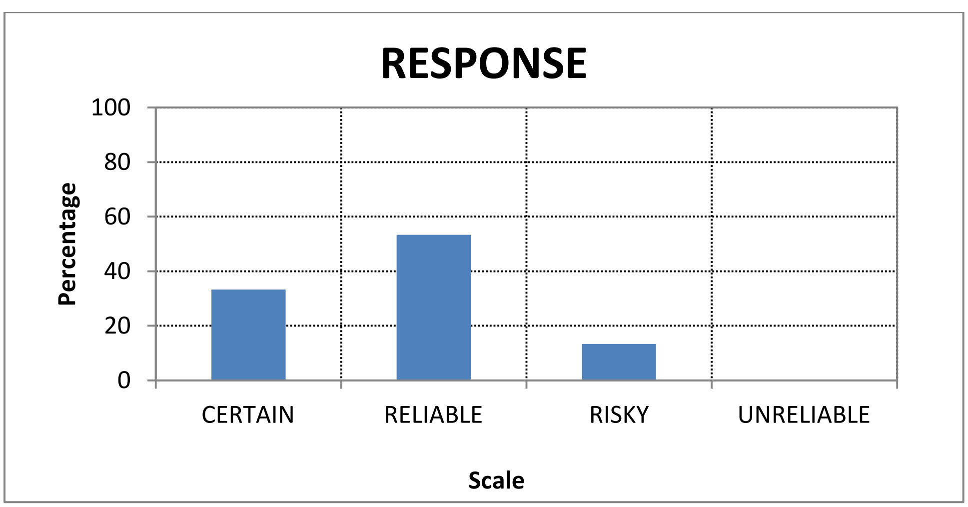
Background: In general, there are two approaches to concrete mix design, prescriptive based and performance based mix designs. In a prescriptive approach, the client specifies the required material, proportions, and construction methods based on fundamental principles that provide satisfactory performance. In performance based approach, the client identifies the fundamental requirements, such as strength, durability, and volume changes, and relies on the concrete producers and contractors to develop concrete mixes that meet those requirements. In general, the supplier/contractor does perform some tests during the mix design preparation.

Question 1.a: Please mark your view of how a performance based or a combination of performance/prescriptive based approach to concrete mix design reduces the risk of failure to provide satisfactory performance.

1.a.1 Performance-based mix design reduces the risk of failure to provide satisfactory performance.

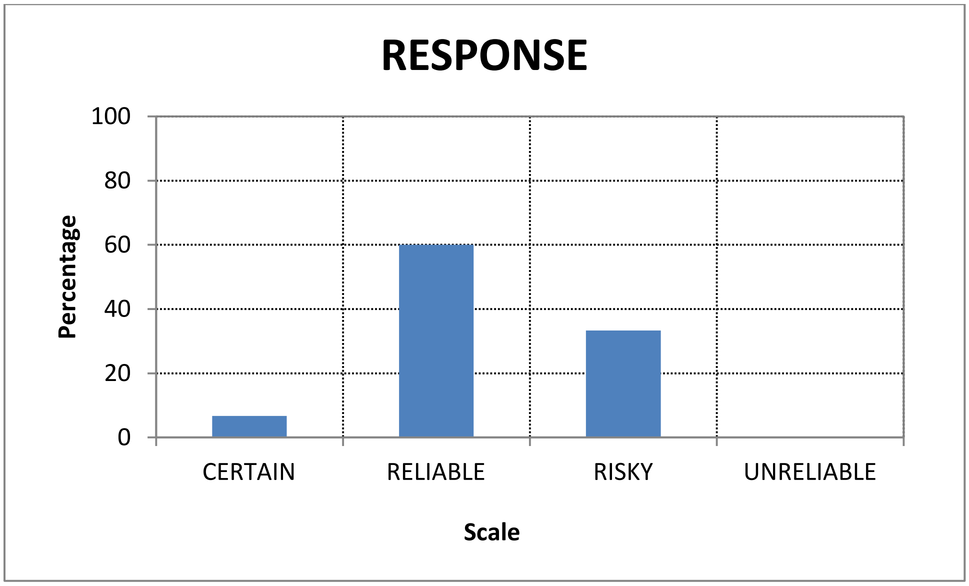


1.a.2 A combination of performance/prescriptive-based approaches reduces the risk of failure to provide satisfactory performance.

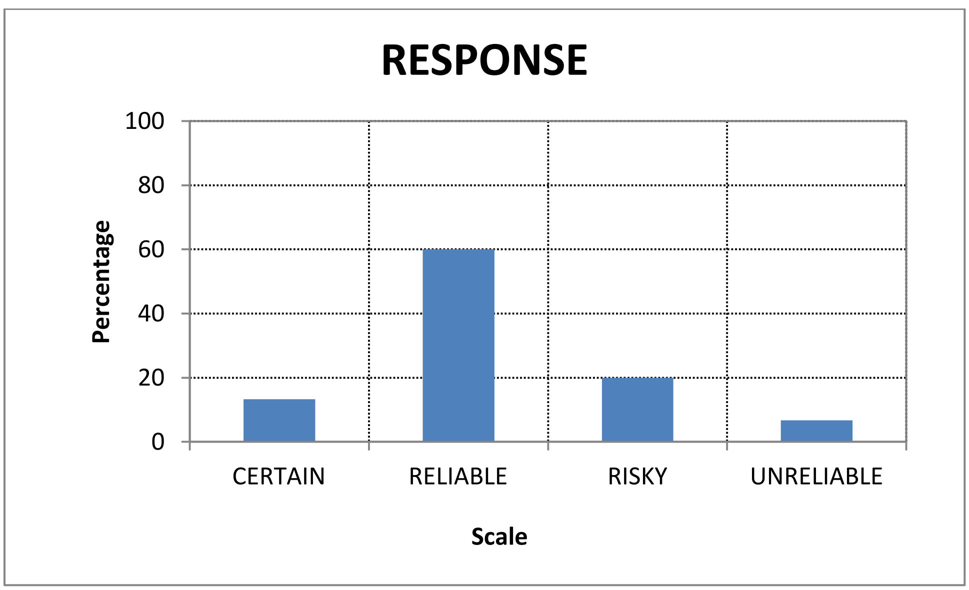


Question 1.b: Please mark your view of how a performance based or a combination of performance/prescriptive based approach to concrete mix design reduces the cost of concrete production.

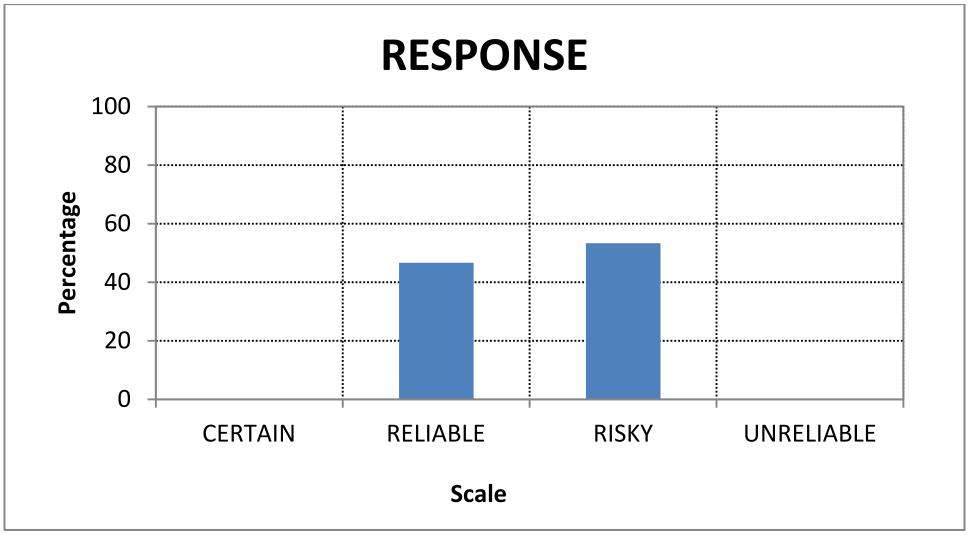
1.b.1 Performance-based mix design reduces the cost of concrete production.



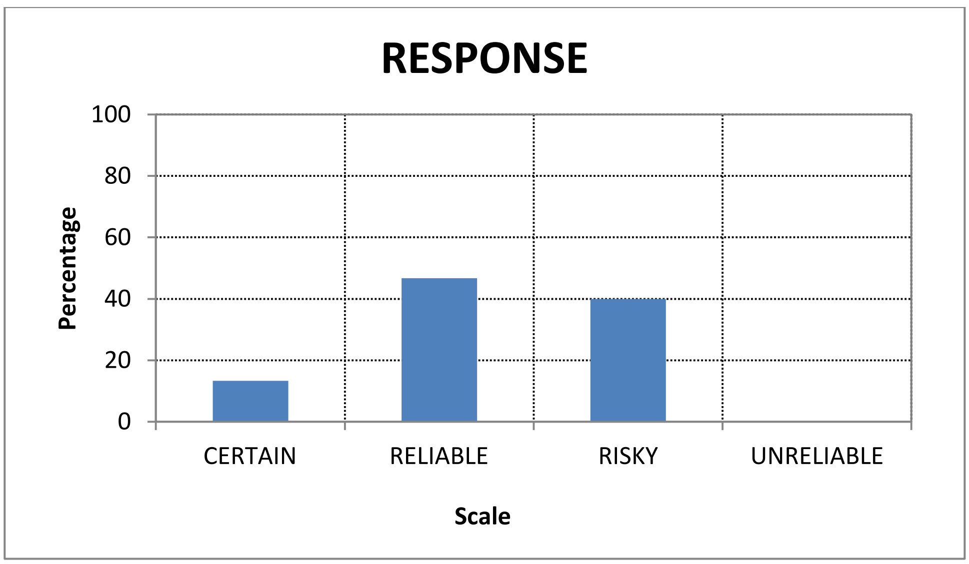
1.b.2 A combination of performance/prescriptive-based approach reduces the cost of concrete production.



Question 1.c: What is the chance of failure for concrete to meet the specified performance and durability criteria if ITD adopts a performance based approach to concrete mix design and eliminates some of the required material acceptance tests, thereby allowing the suppliers/contractors to draw on their own knowledge and experience in designing a mix recipe to meet the specified performance and durability criteria.

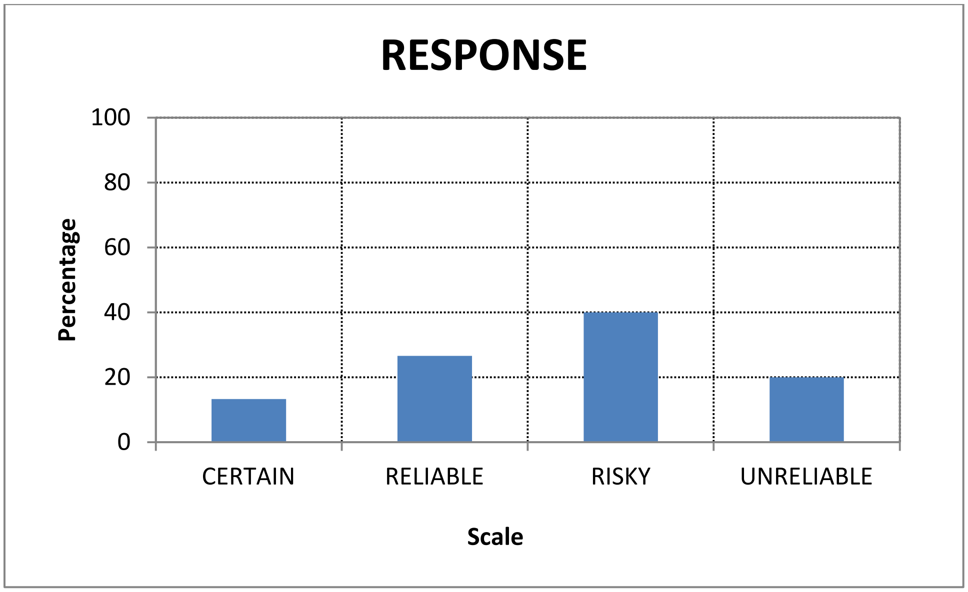


Question 1.d: What is the chance of failure for concrete to meet the specified performance and durability criteria if ITD adopts a combination of performance/prescriptive based approach to concrete mix design and eliminates some of the required material acceptance tests, thereby allowing the suppliers/contractors to draw on their knowledge and experience in designing a mix recipe to meet the specified performance and durability criteria.



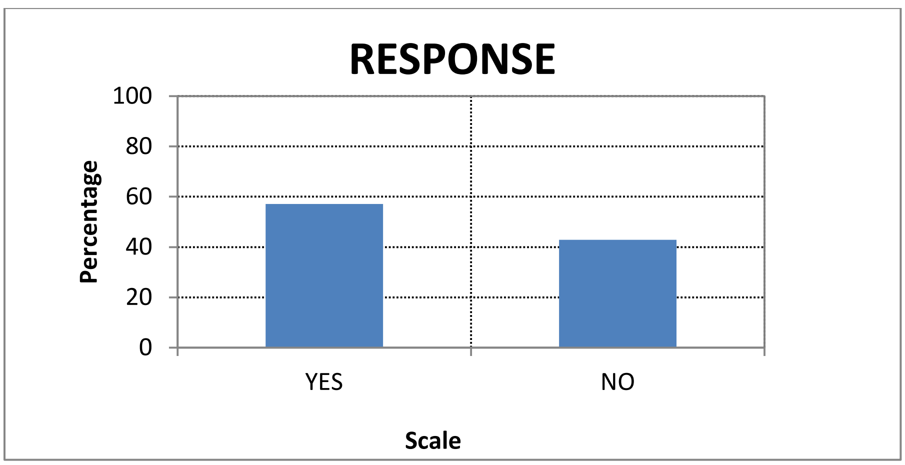
Comment 2: Slump test does not directly relate to the field performance. There is a need to measure consistency of concrete. There were suggestions to measure unit weight of concrete as a measure of consistency when performing an air content test (AASHTO T152/ASTM C231) by weighing the material in the air pot and calculating the unit weight.

Question 2: Using unit weight of freshly mixed concrete as a measure of concrete consistency is more reliable than using slump for the same purpose.

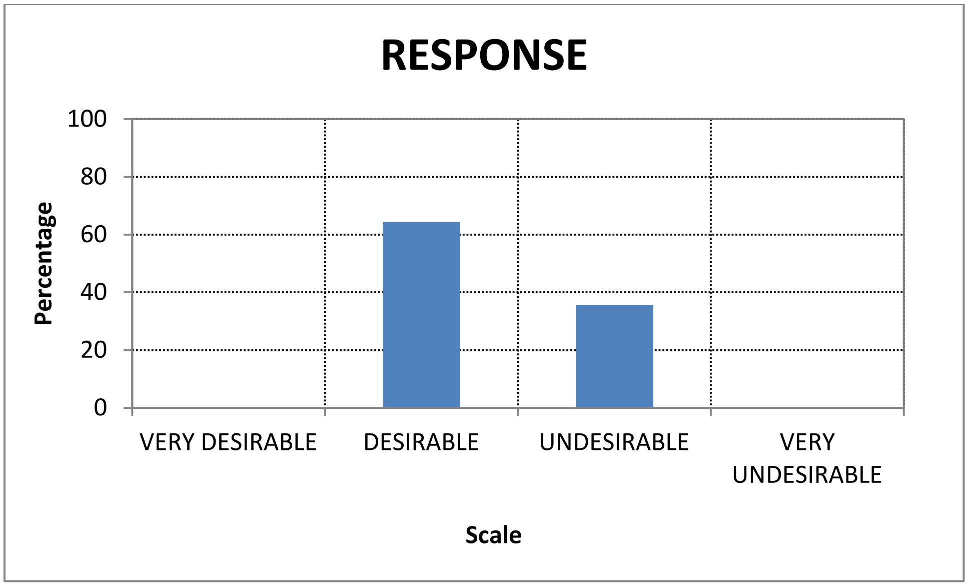


Comment 3: Several participants indicated that the current ITD aggregate gradation specifications need to be reviewed and modified. There were calls for designing a denser concrete.

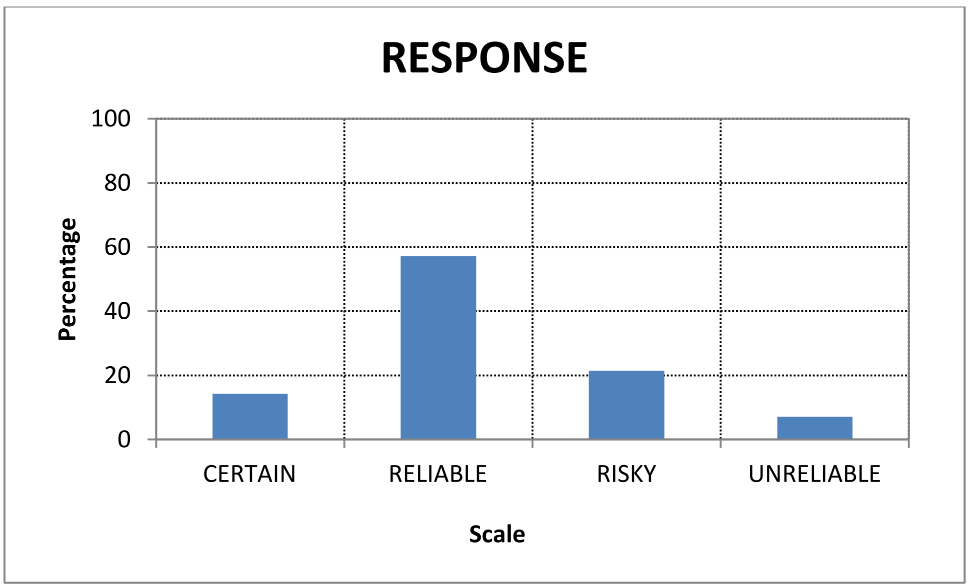
Question 3.a: In your opinion, does ITD need to improve its aggregate gradation specifications?



Question 3.b: How would you rate the current ITD Aggregate gradation specifications?

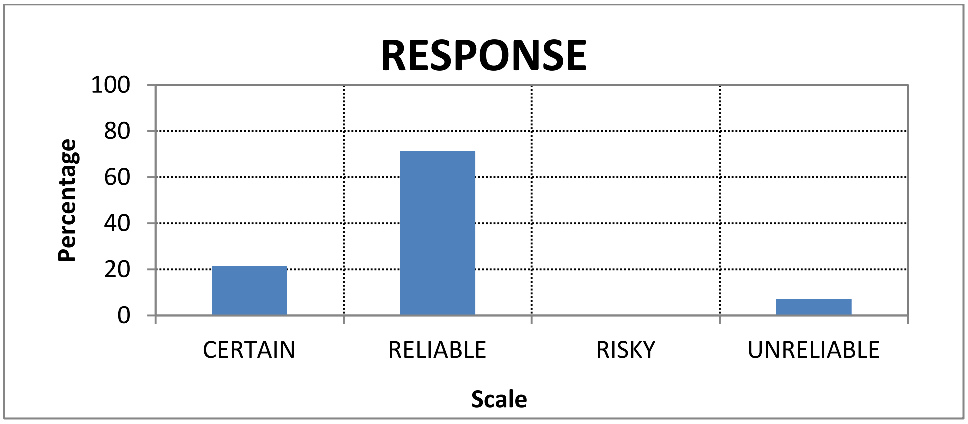


Question 3.c: An improved aggregate gradation improves the workability and stability of concrete.



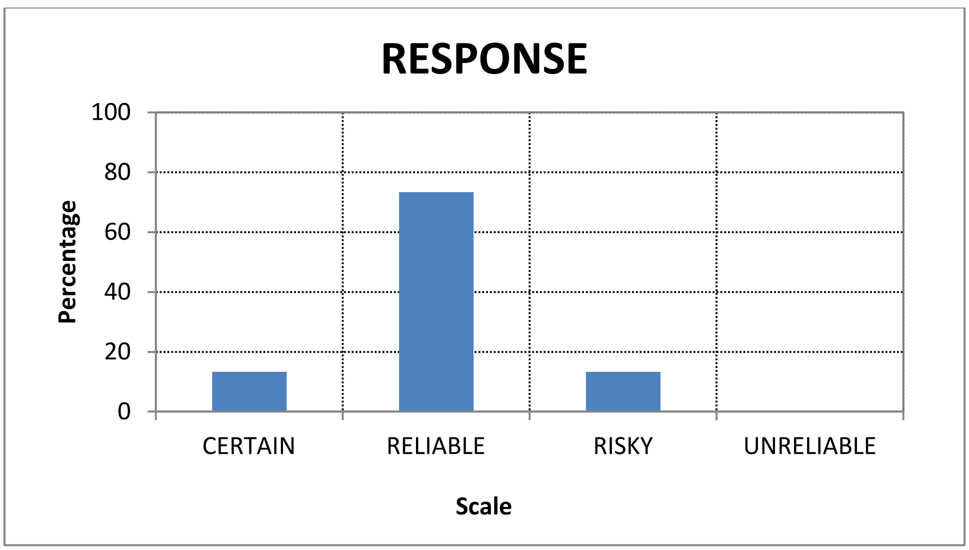
Comment 4: There were comments about the use of advanced methods to better control the quality of concrete during production and placement. These advanced techniques include a) a maturity test using a calorimeter, which records time and temperature for freshly mixed concrete and b) an air-void analyzer.

Question 4: Adopting the advanced techniques listed in Comment 4 to measure early strength, heat of hydration, consistency, and air content of concrete provides more reliable information about the quality of concrete.



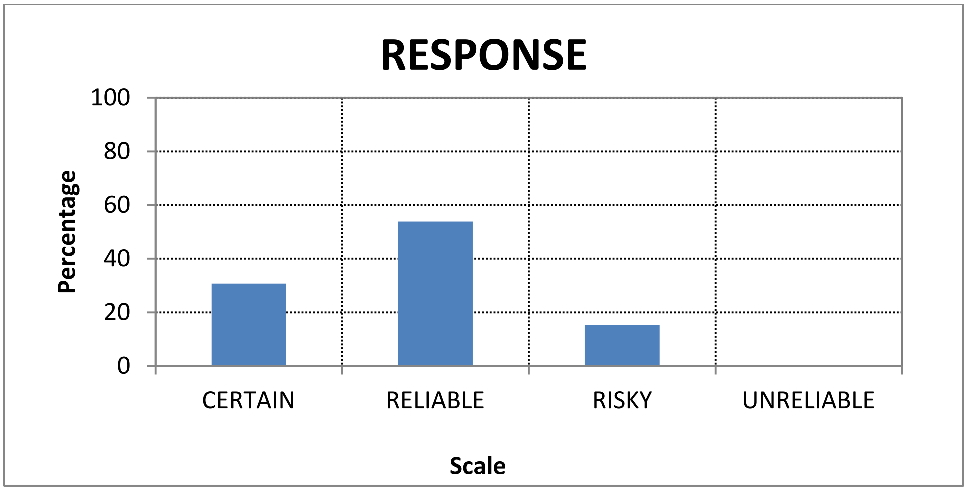
Comment 5: Currently, ITD specifies maximum slump for concrete used in different projects. If the slump does not meet the specifications, generally suppliers add water to control workability, which could have an adverse effect on its performance. ITD need to use additives to control concrete workability.

Question 5: The use of super plasticizers and additives to control workability of concrete would improve the quality of its performance.

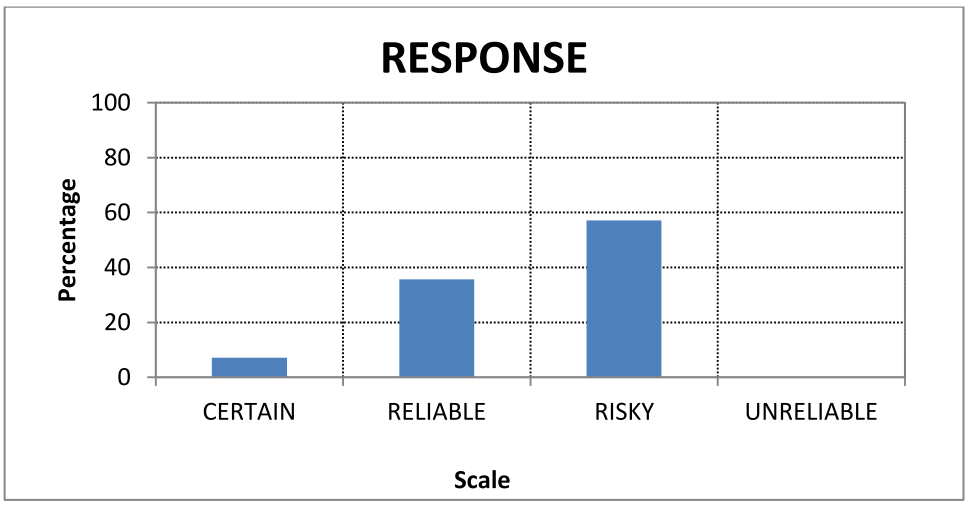


Comment 6: The use of excessive cement in concrete should be avoided. Excessive cement generates more heat of hydration promoting shrinkage and cracking. It impacts the stability of concrete by providing more paste and it promotes ASR by increasing the alkali content of the concrete.

Question 6.a: Reducing cement content of concrete, requiring maximum w/c, and controlling workability by improving aggregate gradation and the use of super plasticizers and admixtures would improve the quality and performance of concrete.

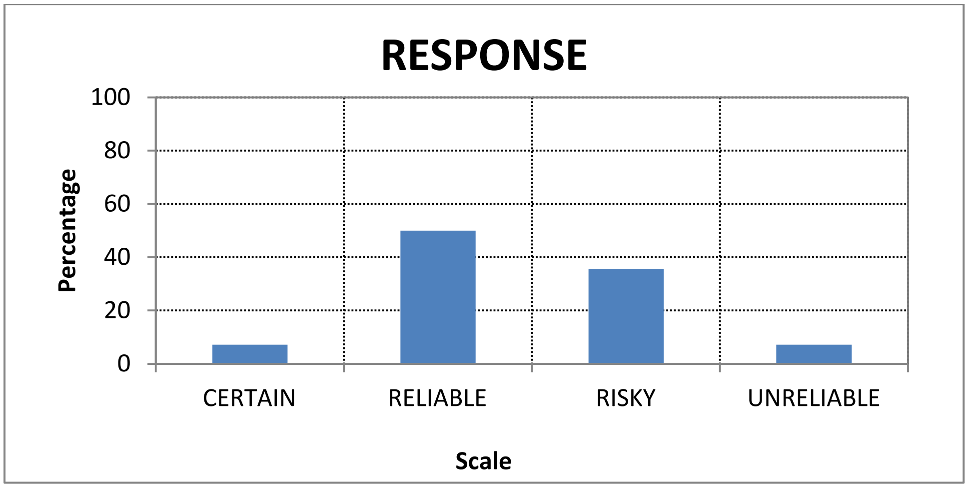


Question 6.b: Reducing cement content of concrete, requiring maximum w/c, and controlling workability by improving aggregate gradation and the use of super plasticizers and admixtures reduces the cost of concrete production.



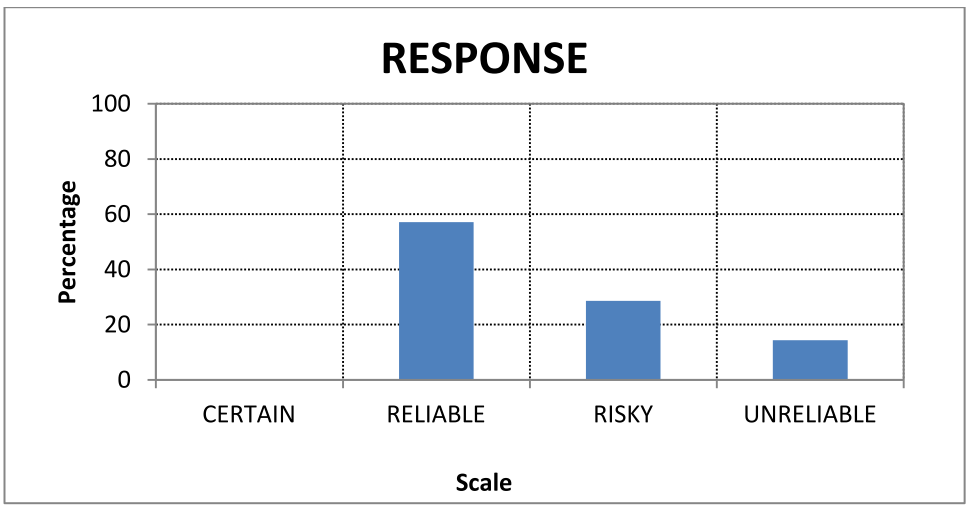
Comment 7: AASHTO M295 and ASTM C618 limit the maximum percentage of Loss On Ignition (LOI) in fly ash to 5 percent and 6 percent, respectively. The ITD current specifications limit LOI in fly ash to a maximum of 1.5 percent. Increasing the percentage LOI in fly ash may not significantly impact the performance while it qualifies fly ash produced by many power plants.

Question 7: Increasing the LOI limit to a maximum of 5 percent would not significantly impact the quality of concrete performance.

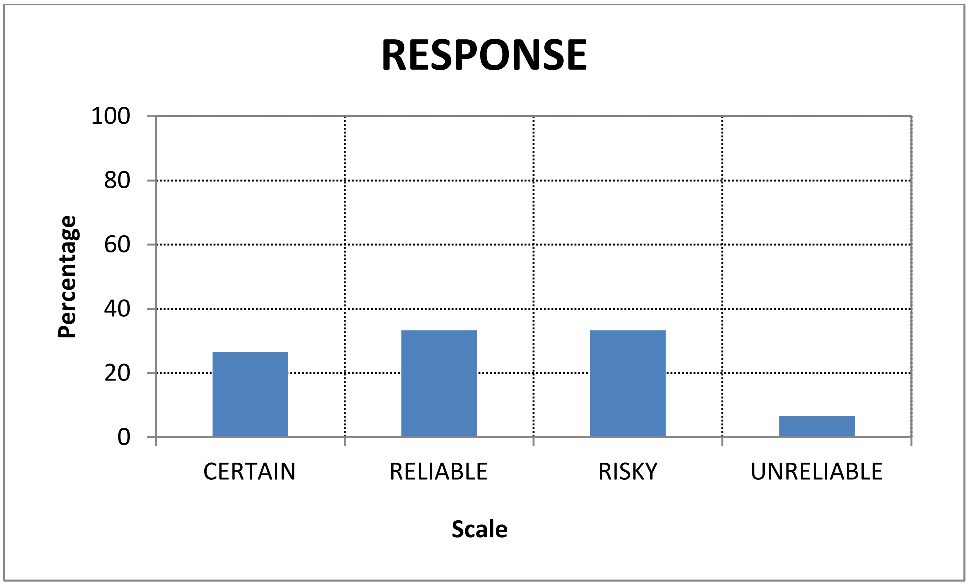


Comment 8: Currently, ITD requires an AASHTO T303 (ASTM C1260) test for identifying potentially deleterious aggregate.

Question 8.a: What is the risk of having ASR problems for an aggregate passing the AASHTO T303 (ASTM C1260) test?



Question 8.b: Requiring an ASTM C1293 test in addition to AASHTO T303 (ASTM C1260) to identify reactive aggregates would reduce the risk of having ASR problems.



Comment 9: A question was raised that having a minimum 4 percent limit for air content could be inadequate to provide freeze/thaw durability for some mixes. For mixes with large nominal maximum aggregate size (1.5 in. or larger), 4 percent minimum air content might be adequate, but for mixes with 1 in. or smaller nominal maximum size, aggregate would require more paste and 4 percent minimum air content might be inadequate. ACI 301 in Table 4.2.2.4 recommends the use of 6 percent ± 1.5 percent for severe freeze/thaw environments such as exist in Idaho.

Table 35. The Results of Round 2 Questions

Question Number	Certain	Reliable	Risky	Unreliable
1.a.1	0.00	73.33	20.00	6.67
1.a.2	33.33	53.33	13.33	0.00
1.b.1	6.67	60.00	33.33	0.00
1.b.2	13.33	60.00	20.00	6.67
1.c	0.00	46.67	53.33	0.00
1.d	13.33	46.67	40.00	0.00
2	13.33	26.67	40.00	20.00
3.a	Yes 57.14		No 42.86	
3.b	0.00	64.29	35.71	0.00
3.c	14.29	57.14	21.43	7.14
4	21.43	71.43	0.00	7.14
5	13.33	73.33	13.33	0.00
6.a	30.77	53.85	15.38	0.00
6.b	7.14	35.71	57.14	0.00
7	7.14	50.00	35.71	7.14
8.a	0.00	57.14	28.57	14.29
8.b	26.67	33.33	33.33	6.67

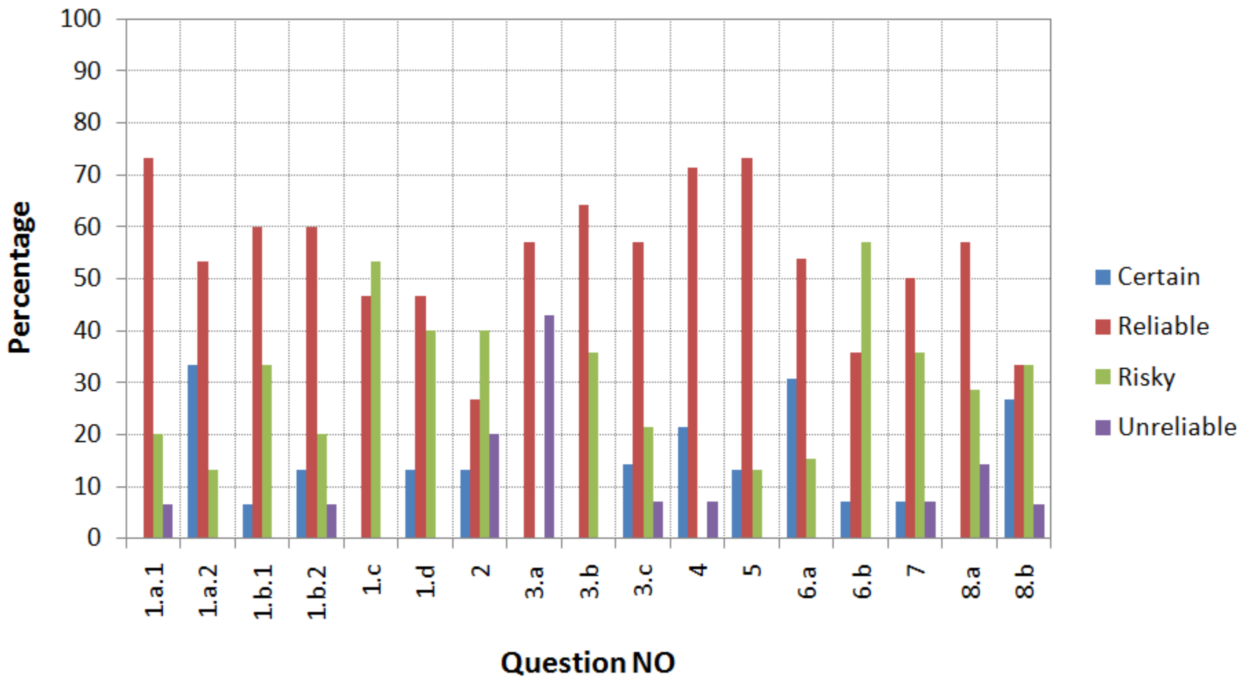


Figure 67. Summary of the Results for Round 2

Summary of the Results

Based on the survey results, the participants favor a combination of the performance/prescriptive-based approach to PCC mix design. The majority of the participants felt that ITD needs to modify its aggregate gradation, and that there is a need to reduce the amount of PCC in ITD's concrete specifications. In addition, the participants felt that ITD needs to specify a maximum water/cement and use admixtures to control the workability of the concrete. The participants also recommended the use of advanced techniques such as air void analyzer, maturity meter to better control the quality of the concrete during production and the placement.

ITD Concrete Pavement Specifications[17]

Material/ Product	TEST FOR		SPECIFICATION	REMARKS	ACCEPTANCE REQUIREMENT
Portland Cement	Portland Cement		AASHTO M85 Type I, II, or III	>0.6% of total alkali is not accepted.	Manufacturer's certification as provided in subsections 106.04 and 106.05
	Blended Hydraulic Cement		AASHTO M240 Type IP,P, or I (PM)		
Aggregates	Fine Aggregates	Fine Concrete & Plant Mix Aggregate	AASHTO M6, AASHTO M29, Idaho T-13		Source Qualifications Basalt Sources Idaho T-116 All Other Sources Idaho T-15, T-142
		Sodium Sulfate Soundness Test	AASHTO T104		
		Organic Impurities	AASHTO M6, AASHTO T21, AASHTO T71		
		Alkali Silica Reactivity	AASHTO T303		
		Sieve Analysis	AASHTO T27, AASHTO T11 Method A or B		
		Clay lumps, Friable Particles	AASHTO T112,		
	Coarse Aggregate	Coarse Concrete Aggregate	AASHTO M80		
		Sodium Sulfate Soundness Test	AASHTO T104	Loss ≤ 12 percent at 5 cycles	
		Alkali Silica Reactivity	AASHTO T303		
		Sieve Analysis	AASHTO T27, AASHTO T11 Method A or B		
		Percentage of Fracture	AASHTO TP-61		
		Elongated Particles	ASTM D4791		
		Degradation	AASHTO T96		

ITD Concrete Pavement Specifications[17] (cont.)

Material/ Product	TEST FOR		SPECIFICATION	REMARKS	ACCEPTANCE REQUIREMENT
Fly Ash		Natural Pozzolans and Fly Ash	AASHTO M295	LOI ≤ 1.5% for all classes, Class-F Fly ash is used and available alkalies ≤ 2%, CaO ≤ 9%	Manufacturer's certification as provided in subsections 106.04 and 106.05. Note: Idaho now accepts available alkalies ≤ 1.5%, CaO ≤ 11%
Concrete		Aggregate Correction Factor	AASHTO T152		
		Compressive Strength	AASHTO T22		
		Making and Curing in Field and Laboratory	AASHTO T23, AASHTO T126		
		Slump	AASHTO T119		
		Air Content	AASHTO T152		
		Pavement Straightedge Procedure	Idaho T-87		
		Temperature	AASHTO T309		
		Sampling	WAQTC TM-2		
		Rate of Evaporation	Idaho T-133		

PORTLAND CEMENT CONCRETE PAVEMENT

Basic Mix Design Parameters				
Concrete Class in 100 psi (Mpa) 28-Day	**Minimum Cement Content lb/yd^3 (kg/m^3)**	**Maximum Water Cement Ratio**	**Slump in. (mm)**	**Air Content Percent**
45 (31.0)	660 (392)	0.44	2 in. 50 mm) Maximum	4 - 7

Basic Mix Design Parameters When Fly Ash is Required					
Concrete Class in 100psi (Mpa) 28-Day	**Minimum Cement Content lb/yd^3 (kg/m^3)**	**Miminum Fly Ash Content lb/yd^3 (kb/m^3)**	**Maximum Water Cement (Plus Fly Ash) Ratio**	**Slump in. (mm)**	**Air Content Percent**
45 (31.0)	550 (326)	138 (82)	0.42	2 in. (50 mm Maximum	4 - 7

Curb and Gutter

There shall be four types of curb, gutter and traffic separators as follows:
Type A sections shall be cast-in-place Portland cement concrete.
Type B sections shall be pre-cast Portland cement concrete.
Type C sections shall be extruded Portland cement concrete.
Type D sections shall be extruded asphalt concrete.

- Specifications for Concrete for Type A, B and C Sections, Portland cement, Aggregate & fly ash are as mentioned in the table above.

- In Type B sections when bonding to hardened PCC an Epoxy resin system meeting the requirements of AASHTO M235 should be used

Responses of Delphi Study

Note: Each Bullet represents response from each participant. These responses were not edited.

QUESTION 1

Based on your knowledge, experience, and/or observations, and in reference to the summary of the ITD specifications for Portland cement concrete, is there any ITD specification that does not closely relate to the field performance and needs modification(s)? Please list the specification, provide a brief comment about it, and make suggestions for improvement, if you have any.

ITD PARTICIPANTS

- Historically, acceptance of concrete has been by strength obtained from cylinders broken in compression. Breaking beams in flexure would give a better estimation of the true field strength. This is because concrete in use does not fail in compression, but in flexure. The compressive cylinder test also has problems because the cylinder ends are not allowed to expand as the normal stress is applied for true compressive failure. The best argument for continuing to use the "cylinder break" test method is because of the immense historical data bank of cylinder break information and it is what our current standards are.
- The slump test also does not directly relate to field performance, yet any modification needed is to somehow convey to ITD staff and the contractor what the test is a measure of: consistency between batches. There are many variables that affect consistency and flowability of the concrete and too many people believe it is just a matter of how much water is in the mix.
- Mix designs may be approved for use by either of two methods. Many times contractors submit mix designs that show class strengths but cannot meet the additional 1200 psi required or the consecutive passing tests. Either the class limits are set too low for what the State needs or the testing required for mix design approval is excessive. The statistical requirements in proving mix designs should be revisited.
- In addition, I would like to see the State have a standard mix design that may be used in recipe fashion by batch plants that do not have the experience and historical data to meet the current mix design approval specifications. The mix will have enough cement, water, aggregate and approved admixtures that it will be effective and our assurance will be based on experience.
- TP-61 percent of fracture - This is typically not an issue with concrete. Although this is listed in the 502 section of the spec book, I am unsure what the specification limit is.
- D4791 Elongated particles – this relates primarily to workability and not necessarily to performance. This is typically not an issue with concrete as rounded rock is generally used.
- T-152 Agg correction factor – this number is typically very small for Idaho aggregates. Most ITD mixes use ¾" nominal max aggregate. The aggregate correction factor generally applies to 1 ½" or larger aggregate.
- I don't believe there is any specification that does not relate to field performance.

- The specification itself does not require any fracture specification for course or fine aggregate. This affects workability of the plastic concrete. The only exception that I can think of would be to require rounded (i.e., "natural) course and fine aggregate for the casting of drilled shafts.
- The current ITD gradation specs for coarse and fine aggregate for concrete need to be reviewed and updated. The High Performance Concrete spec and self consolidating concrete specs being used by other states utilize more densely graded aggregate gradations for increased workability at lower w/c ratios rather than gap graded aggregate gradations (which are prone to segregation). ITD should move to a process of specifying a maximum aggregate size and require the contractor to submit their gradation as part of the mix design rather than specifying specific gradations for CA and FA. The contractors/producers would then be able to better optimize their designs.
- The current specs and field acceptance testing for slump and air and 28 day lab cured cylinders tells more about the mix design and material delivered to the job than the material finally incorporated into the structure. Many things may happen to the concrete after testing (water may be added, concrete is pumped, vibration/consolidation) that change these characteristics in the structure. More relevant testing is needed to determine acceptability. New methods such as the air void analyzer, maturity meter and AASHTO 318 that better represent the material in the structure are/have been developed. These methods should be evaluated and incorporated into the acceptance process for ITD to the extent they are practical in the field and the equipment is affordable and maintainable by the Department.
- Slump should be eliminated as an acceptance parameter. Slump is not a good indication of w/c ratio, only of consistency or uniformity of the material delivered. Slump changes as a function of aggregate size and shape, time and admixtures.
- It is generally accepted that failure to meet these specifications will result in an unacceptable product, usually in terms of concrete durability, but occasionally in strength.

NON –ITD PARTICIPANTS

- One issue that I would like to see addressed related to field performance involves onsite testing of the concrete. The specification calls for the standard battery of field tests: slump, air, temperature, sampling, and compressive strength. I believe that unit weight (AASHTO T121/ASTM C138) should be included in the standard battery of tests. Since everyone already is performing an air test (AASHTO T 152 / ASTM C 231), it is easy to weigh the material in the air pot and calculate the unit weight. The unit weight is a better indicator of consistency from load to load than slump in my opinion. You can also perform a gravimetric air calculation as a check on the accuracy of the air meter. Having data on the unit weight from load to load can provide information to help troubleshoot if there are sudden changes in the mix. A yield calculation can be performed as well to make sure that the delivered material will be adequate to complete the job.
- I am curious why in the spec, the aggregate correction factor in AASHTO T 152 is included as a separate test? Could it not be implied that since the aggregate correction factor is included in

AASHTO T 152 for calculating the correct air content it is necessary to perform? I am curious if other people would agree with me that having the information twice is redundant? Maybe I am missing the reason to have it included separately?

- As you know from an empirical perspective, the behavior of PCC is well understood. However, on a chemical basis, it is a complex material whose mechanism and interaction are not fully understood. Therefore, as a subject matter expert, your opinion is very important in identifying the degree of consensus among a group of experts.
- The slump test AASHTO 119 has less relevance today with water reducing admixtures than it did in the past. Testing of water cement ratios in the field would be a more accurate test of concrete strength and performance.

QUESTION 2

In some of the ITD PCC specifications, there are limitations set forth for material acceptance including aggregate, cement, supplementary cementations material (SCM), and concrete as a final product. Some examples of the limitations include maximum alkali in cement 0.6 percent, loss of less than 12 percent in sodium sulfate soundness test, maximum CaO in fly ash 11 percent, maximum expansion of 0.1 percent in an ASTM C1260 test, and many other limitations as shown in the ITD specification summary. **What does each of these limitations mean to you?** In your opinion, **is there any correlation between these specifications and field performance of concrete?**

ITD PARTICIPANTS

- The limitations set for of 0.6 percent alkali in cement, loss of less than 12 percent in sodium sulfate soundness, maximum CaO fly ash and 0.1 percent expansion do not mean very much to me at all. These limits seem similar to that used by other states; however; I have seen little statistical basis for these bounds. There are charts showing 0.6 percent reactive silica relating to 0.1 percent expansion that peak at 6 percent reactive silica then taper off. I have seen no information on stochiometric requirements on reactive silica and alkali found in cement mixtures and the resulting chemical potential for expansion. I have heard many engineers and contractors complain about C1260 tests giving false positives and that it does not in any way predict expansion. I do not have data to back this up, but this seems to happen too often.
- I am not aware if Alkali Carbonate Reaction has been discussed or if it is potentially a problem in Idaho. I do know this reaction is sometimes confused with ASR and other States have mistakenly mitigated for ASR.
- Most of the limitations have been adapted from national trends and laboratory data.
- All of the limitations listed have an effect on long term performance, but less effect in areas with less precipitation or less freeze/thaw potential:
- Max 0.6 percent alkali – this is a nationally recognized limit at which ASR has a limited effect on the concrete (effects become significantly more pronounced above this level). Some states vary, but it is typically believed that values of 0.6 percent and below are not detrimental to the long term durability.

- Sodium Sulfate – this simulates freeze/thaw conditions. The test has a high variability and can give “false” results with certain aggregate types. I am unsure of why 12 percent loss was chosen, but it seems to be fairly common number, probably based on economics (ability to use local aggregates, haul costs, etc).
- Max CaO in fly Ash – the 12 percent seems to be a cutoff between Class C and class F fly ash, which also indicates whether a harder or softer coal was burned. Class F has lower CaO content and is primarily a filler, but also has less potential for ASR problems and lower heat of hydration. Although the 2 types of coal could probably overlap somewhat, limiting CaO to 12 percent effectively eliminates the softer coal use.
- Max expansion of 0.1 percent in ASTM C1260 – Another industry standard in which 0.1 percent was determined not detrimental to the long term durability of concrete.
- 2A.Portland Cement – less than 0.6 percent alkali to minimize the available alkalis and thus the ASR potential
- Sodium Sulfate Soundness – checks the expansive potential of the aggregates due to freeze/thaw action
- Clay lumps, friable particles – minimize deleterious material
- Degradation – aggregate durability
- Elongated Particles – minimize non-cubicle particles that could impact strength
- Flyash – minimizing the available alkalis available in the mix
- Concrete – these are quality control tests performed in the field to minimize a chance of failure after the fact, except the compressive strength which is global quality control
- 2B. There is definite correlation between these specifications and field performance of concrete (i.e., long term concrete performance. Non-conformance with one requirement may or may not be noticeable; non-conformance with a couple could be catastrophic
- All these tests relate to early strength and longevity of the product. In my opinion no one single test takes precedence over another. They are all interrelated. Major structures and concrete pavement are designed to last 40+ years yet must meet strength requirements relatively early.
- Low alkali cement is specified to help mitigate ASR and it is warranted based on ASR research. The max 11 percent CaO limit is also related to ASR mitigation. Higher lime contents such as found in type C fly ashes contribute to ASR. Flyash used for ASR mitigation should be Type F with lower CaO content per ASR research. The 0.1 percent max expansion per ASTMC1260 relates to determining how reactive a given aggregate is for ASR. The test has many limitations. Other test methods are being developed and further improved to better determine/predict ASR. These specs need to be reviewed frequently along with the most current research and test method development in an attempt to keep the specs up to date with what is available. However, changing the specs too frequently results in problems for the producers and increased costs for ITD. There has to be a balance between the latest and greatest testing and research and what the industry can implement. Some testing for ASR requires extensive testing time and producers must be given advanced notice before the requirements change.

- If allowed to remain in service premature failure may result. Sidewalks, curbs and gutters usually become the responsibility of local municipalities to maintain and they would bear the costs of maintenance.
- Premature failure of bridges and pavements pose more difficult problems for ITD as bridge and pavement outages impact the traveling public, are highly visible, repairs are difficult to schedule and the cost of repairs is higher.
- These limitations infer to me that these are important requirements and problems have and could result from failure to meet them.
- Yes, though in durability, not in workability of fresh concrete.

NON –ITD PARTICIPANTS

- In my opinion it is important to place some restrictions on material for concrete. The 0.6 percent equivalent alkali content for cement is important to address because of the ASR situation in Idaho. For ASR to take place there needs to be all of the following present: moisture, reactive siliceous component of aggregate and available alkali. By limiting the alkali content of the cement we are reducing the probability for the reaction.
- Sulfate soundness testing on aggregates is a good way to evaluate durability aspects of those aggregates. In my opinion durability testing is critical in a harsh freeze and thaw environment such as Idaho.
- The LOI limit in the spec is 1.5 percent for all classes of fly ash. AASHTO M 295 / ASTM C 618 have a max limitation of 5 percent. If I am right in my assumption, the limit of 1.5 percent LOI is to reduce the carbon fluctuation in the ash and therefore reduce the fluctuation in air content of the concrete. The ready mix supplier can compensate for higher LOI by increasing the dosage of air entraining admixture. Information from the fly ash supplier as well as tests like the foam index can help predict when LOI can vary. From the ready mix standpoint it is a major headache to deal with fluctuating air contents due to carbon content of fly ash. Theoretically if you have a load of fly ash that is 2 percent LOI and the ready mix supplier compensates their air entraining admixture adequately to correct and maintain the correct air content, then you would have concrete that could meet all of the performance criteria, but still be rejected on the basis of LOI in fly ash. I am curious to see if the group would agree that having a restriction of 1.5 percent LOI on the fly ash seems to be a little low? I think it is important to keep carbon content to a minimum, but if AASHTO/ASTM has a limit of 5 percent is ITD a little low?
- I think that ASR testing is critical to ensure long term durability of concrete in the state of Idaho. There are not many, if any, aggregates in the state that will pass the AASHTO T 303/ ASTM C 1260 test. This test is a harsh “worst case scenario” test. The 1260 is really the only reliable test that gives an indication of ASR reactivity of the aggregates with the available cements in a short period of time. The ASTM C 1293 test is also a test that can be performed, but takes a year to complete. I am curious if an aggregate fails the 1260 and passes the 1293, would the 1293 overrule the 1260? In my opinion I think that this should be discussed. I think that if an

aggregate passes the 1293 it should be considered innocuous even if it is considered potentially deleterious or deleterious by the 1260. To me the 1293 is more representative of actual conditions of the concrete in service.

- I think that there is definitely a correlation between some of these limitations and field performance. Aggregate gradation and durability limits correlate very well with the performance of the mix. Having clean, well graded aggregates affects what the ready mix producer and contractors have to do to that concrete to achieve a workable mix. Poor graded, dirty aggregates generally require more paste (usually compensated for by adding water) to achieve workability and finishibility. Concrete is only as strong and durable as its aggregates. I think that with fly ash it is important to realize that consistency in the end concrete product is the most important factor. If the LOI fluctuates above 1.5 percent and the ready mix producer can adequately maintain the air content of concrete that should still be satisfactory. Air is one of, if not the most, influential aspects on durability of concrete in Idaho. Another important aspect of durability is density and permeability of concrete paste. I believe that more of an emphasis should be placed on testing density / porosity of concrete mixes. Moisture in concrete is the root of most of the durability problems we see (ASR, freeze / thaw damage, sulfate attack, ingress of de-icers). If the paste is dense, the moisture has a harder time infiltrating the concrete and causing some of these common problems. High cement factors are often prescribed to help achieve high strengths, but that does not necessarily mean that there will be low porosity.
- All of the above referenced limitations increase the cost of producing PCC and must be added to the bid price of PCC. These restrictions/additions to the PCC mixes have come about over years of experience by ITD in various locations throughout Idaho and other areas. Are they necessary in all areas? Probably not. For example: If a concrete supplier has an ultimate strength failure on a cylinder test he will assume the mix was tested correctly and probably increase the cement content of the mix. This increases the likely-hood of ASR, which then becomes an issue that requires more expense to ITD. The real culprit in this case was probably an ITD employee who learned to test concrete during the winter when things were slow, has little prior experience, and is now asked to properly test and cure concrete in the field. The penalties are so ridiculous now that few companies want to trust their concrete to the hands of a tester that has nothing to lose by doing the testing incorrectly and ITD has everything to gain by a poor test. The current requirements of ITD have grown from a series of mistakes, made over the years that have now forced concrete producers to provide a very expensive product that is no better than the everyday concrete used locally. Cement and fly ash producers are no different. They have paid penalties in the past because their products did not meet specifications when tested by ITD and yet when tested by independent labs they were fine. There is a question of trust involved when the testing agency stands to gain financially from a faulty test and uses testing personnel with minimum PCC knowledge. The limitations listed above are probably valid in many cases but are grossly over scrutinized. Every minute detail must be addressed by the producer in duplicate and yet the attitude of “anything goes” applies to ITD’s own performance. When you factor in penalties that are sometimes higher than the cost of the concrete itself it is no wonder most producers in the State refuse to do business with ITD. The monster is one they have created themselves

- Each limitation has a cost associated with it, from testing for source approval to testing for ASR. The correlation between the specs and the field performance is not a short term relationship. Most of the specs are aimed at the long term performance and durability. It is difficult to see any correlation for this reason. The critical question is whether the additional cost to meet these specs, produces a long term difference in the performance of the concrete.

QUESTION 3

In some areas of Idaho (mainly along the Snake River basin), the aggregate is known to be reactive and to promote Alkali-Silica reaction (ASR), resulting in premature deterioration of concrete. Please make comments on the ITD Specifications (attached) in dealing with ASR problems, and in your opinion, what works the best in mitigating ASR in concrete, considering fly ash, Silica fume, and lithium nitrate solution, combination and/or other Supplementary Cementations Material (SCM) used for mitigation. Are there any other methods to mitigate ASR in concrete? In your opinion, are there any other factor(s) contributing to the premature deterioration of PCC and what is the remedy for prevention?

ITD PARTICIPANTS

- In my district, the Snake River siliceous aggregates are not found; this area does not seem to have a problem with alkali silica deterioration. In the past, local suppliers have been required to mitigate for a misunderstood scare found elsewhere. Lithium nitrate, fly ash, and silica fume have been added in a prescribed amount without reactive calculations or testing. The lithium nitrate and fly ash were used to react with the assumed alkali expansion. Fly ash and silica fume are now used to reduce permeability and reduce sulfate attack. One method has been tried after deterioration; it was to spray a lithium nitrate compound on the concrete surface and seal the existing cracks with High Molecular Weight Methacrylate. If expansion is expected, the concrete may also be sealed with silane to retard water penetration thus reducing expansion.
- ITD specifications address cement and fly ash, but do not address total alkali in the concrete, some of which may come from aggregate itself. This would require much more preliminary testing.
- In my opinion, lithium nitrate based products work the best to mitigate ASR as they change or stop the chemical reaction. Fly ash, silica fume and other SCM's may do this to a lesser effect, but primarily act as fillers to decrease permeability and slow the exchange of ions.
- Other factors contributing to ASR are the use of deicing solutions and moisture intrusion. Sealants may be effective, but I am not aware of broad use of them. Deicing is highly political, for both traveling and environmental reasons, and it is unlikely that their use will be discontinued. Based on environmental concerns, potassium acetate (?) is becoming popular, but seems to have the most damaging effects on concrete.
- Strained quartz seems to be the main factor in Idaho for ASR. More highly metamorphosed rock types may be more of a problem. Rock types (which haven't undergone petrographic analysis)

would have to be evaluated geologically. This would be subjective and difficult to apply with consistency.

- In district three we require twenty percent fly ash for all class 40 concrete from either the Boise River or the Payette River drainages. A maximum lithium nitrate dosage would be acceptable in lieu of the fly ash. Snake River aggregates need the T303 testing performed and they will not pass this test. It would be up to the supplier to prove the mitigation method meets the specification for expansion. I don’t know about silica fume as a mitigating measure and I am not familiar with other mitigating measures. I am not aware of other factors contributing to premature deterioration of PCC.
- The addition of Lithium nitrate is preferred because it is easier to measure in the field. There are fewer variables to consider and have to document.
- The current ITD specs related to ASR in combination with a shortage of fly ash meeting the ITD specs resulted in most suppliers withdrawing fly ash from their mixes and mitigating ASR with lithium nitrate. Lithium nitrate is shown to effectively mitigate ASR in laboratory tests and research supports the use of lithium nitrate to mitigate ASR. However, removing fly ash results in the loss of other benefits provided by fly ash such as reduced permeability of the mix and increased workability at lower w/c ratios. There have been problems with the use of lithium nitrate in the field such as shortened set times and loss of workability. The specs should encourage the use of fly ash (or even require the use of fly ash) in most mixes. Fly ash and other SCMs should be used to mitigate for ASR whenever possible because of the many other benefits they bring to concrete. Lithium nitrate should be used in combination with fly ash or other SCMs when mitigation cannot be obtained with SCMs alone. One of the most critical factors in concrete life is low permeability of the paste. SCMs help with reducing permeability.
- Other than shipping non-reactive aggregates I have no comment/opinion.

NON –ITD PARTICIPANTS

- ASR has a huge affect on long term concrete durability. As I mentioned above I believe that if the ASTM C 1293 test data is available it can provide a better “real world” scenario as to the potential for ASR in the concrete than the AASHTO T 303 / ASTM C 1260 can. Porosity in concrete can have a great influence on ASR due to the fact that water is one of the driving factors behind the reaction. If moisture can easily get into the concrete to create freeze / thaw issues that can create a “snowball” effect that will exacerbate ASR later in the concrete’s life. I believe that a good reactive class F fly ash is the most cost effective and practical way to mitigate ASR. Fly ash due to the physical nature of its particles often allows for water reduction and/or a workability benefit in concrete. It has a negative effect on early strengths, but a positive effect on long term strengths. Having workable concrete at a job site is critical because if it is not workable, most likely water will be added to achieve the desired workability and adding water compromises the durability of the concrete. Silica fume tends to create problems with workability due to its extremely high surface area and water demand. Admixtures are often needed to maintain workability when silica fume is being used. Curing is very important when using silica fume because the surface of the concrete tends to quickly lose moisture needed for hydration. Lithium nitrate based admixtures are very effective at mitigating ASR, but have to be used in large quantities and are expensive. Ternary cementitious blends can be effective, for example, a cement, class F fly ash, slag blend. The slag allows some ASR mitigation and compensates for some of the delay in strength gain from the class F fly ash. The problem is finding a good source of slag that would be in close proximity to Idaho to make it economical. There are other materials that can be effective at mitigating ASR. There are various natural pozzolans around the state of Idaho that could be effective, such as zeolites, calcined clays, volcanic ashes, etc. The problem is mining and processing the materials to be effective in concrete. Fly ash is naturally at a particle size distribution that makes it fairly reactive in concrete. Slags and some of the natural pozzolans need additional levels of processing to make them effective in concrete.

- In my opinion the greatest contributing factor contributing to the premature deterioration of PCC is porosity in concrete. I don't see the issues in paving and commercial work as much as in residential concrete. As I said before, a more porous concrete allows moisture in more easily which, in a harsh freeze / thaw environment like Idaho, can be extremely detrimental to the life of that concrete. Moisture is a vehicle for de-icers to get into the interior of the concrete and exacerbate the effects of water freezing and thawing. Once this freeze thaw starts to take place microfractures are created within the concrete paste. These microfractures allow more moisture to penetrate the concrete. Over time this process creates a "snowball effect" that allow the concrete to become saturated with moisture very easily. This high level of saturation can create conditions for ASR to flourish. Usually in residential concrete the effects of freeze / thaw damage are seen fairly early on due to the fact that the water to cement ratios are high and the concrete is very porous. Finishing practices can also be a major contributor to early degradation of the surface of the concrete. These problems usually stem from adding large amounts of water to the truck on the job to achieve more workable concrete. Curing procedures often are not performed or not performed adequately enough to allow a complete hydration of the concrete before it is put into service. Reducing the water added to the mix both at the plant (use of plasticizers, etc.) and at the jobsite (water added to the truck) can greatly help the durability of the concrete. Having a well graded aggregate with the largest practical nominal max coarse aggregate can reduce the amount of paste need, therefore the amount of water needed, to achieve a workable mix. Clean aggregates reduce the water demand because excessive fines create a higher water demand. The more workable we can make the mix at the batch plant by using well graded aggregates, admixtures, fly ash, etc., the less likely people are to add water to get workability at the jobsite. The best way to prevent premature deterioration of PCC is to educate people on the causes and effects of ASR, Freeze / Thaw, de-icer use, adding water to concrete, etc. Also, the positive effects of curing cannot be underestimated. There are plenty of effective ways to mitigate ASR and all of these other problems. For the most part these common problems are well understood. There are known ways to prevent these problems from happening, yet we still see them on a regular basis. Specs can be written, but until people really understand how these problems come about and how we can prevent them, there will still be issues.

- As mentioned there are many methods to mitigate ASR. I have had experience with only three. Silica Fume is the most expensive because of the relatively high cost of the silica fume itself and the dosage required to mitigate ASR. It is also expensive to handle and store. Lithium nitrate is probably the best mitigator, providing close to zero expansion, but is also fairly expensive and requires a considerable amount of expertise to control. Fly ash is the most inexpensive solution but due to poor testing by ITD in the past, no local suppliers will back their products when tested by ITD. With no local supplier of fly ash the transportation costs become exorbitant.
- My testing for ASR and mitigating it is limited to the use of fly ash. Fly ash if available was more cost effective and less detrimental to the properties of the concrete. Meeting ASR specs for all concrete sold to the State should not be required. Some applications may not need, benefit, or be cost effective by meeting ASR specs.

QUESTION 4

In the ITD specifications, there are two tables depicting ITD specifications for the concrete mix recipe with and without fly ash. Please make any comments you may have regarding ITD mix recipes.

ITD PARTICIPANTS

- Extra lines should be used for the modifiers A and B with all their parameters. All the notes that go along with the table are quite convoluted and confusing. This information should be incorporated into the table. I have no other comments about this table.
- Fly ash mixes in Idaho (using class F fly ash) tend to gain strength slower, but have higher long term strengths. Contractors prefer the workability of fly ash mixes, but mixes may need to up the cement content (above ITD minimums) to get acceptable 28 day strengths. With more stringent air quality requirements of coal burning plants, concrete producers are having difficulty getting fly ash meeting specifications. Due to less supply and increased cost, many concrete suppliers have opted to not use fly ash unless required.
- I have no significant comments. Utilizes low slump (two inch maximum) to minimize edge slump in slip form paving
- The minimum cement content (especially in Table A) should be reduced. Concrete strength is better obtained by optimizing aggregate grading rather than high paste content. We should allow the producers to optimize their mixes and reduce paste if they can get strength by optimizing aggregates. This will improve the overall quality of concrete. Mixes are developed and tested in the lab and for PCC pavements are verified in the ITD HQ lab. The minimum cement contents need to be reviewed against the industry and other state's specs.
- These mixes have a fixed, minimum amount of cement and a high design strength. Cement is, by far, the highest cost ingredient of concrete. Yet these specifications offer no incentive to minimize the most costly ingredient.
- I suggest that, at least for larger projects, standards that promote minimizing cement content, while maintaining all necessary strength and durability, be specified. Also, for larger projects, tight uniformity of strengths should be specified.

- Use of larger maximum size (course) aggregate, tighter gradation control (especially in the sand), would result in lower requirements for cement paste with similar workability and not loss in strength or durability.
- Use of approved super-plasticizers should also be promoted as workability and durability is usually improved along with higher early strengths often needed for bridge pier and abutment work. Shrinkage and related stress are minimized. Strength-based form stripping and loading of concrete structures should be allowed.

NON –ITD PARTICIPANTS

The only major issues I see with the table that was sent is the high cement content for 4500psi concrete and the low end of the air requirement. With higher cement contents come higher water contents. Even at a low w/c, more water in the mix can mean more shrinkage and a higher probability for cracking. 660lbs of cement for the straight bag mix and 688 total cementitious seems a little excessive to achieve 4500 psi at 28 days. Also, having the low limit on the air at 4 percent could be inadequate to provide freeze / thaw durability for some mixes. For mixes with a larger nominal max (1.5" or higher) this could be adequate, but for mixes at 1" nominal max or less, which would require more paste, 4 percent might not be enough. ACI 306, the standard for cold weather concreting, refers to ACI 301 Table 4.2.2.1 for recommended air contents. For severe a freeze / thaw environment, which in my opinion includes the entire state of Idaho, 6 percent +/- 1.5 percent is recommended for aggs with a nominal max. of 1" (as well as ¾"). From there as the nominal max size decreases the target air content increases. This would allow for a minimum air content of 4.5 percent or higher as nominal max size decreases.

ROUND 2

QUESTION 1

- **Comment 1:** There were multiple recommendations to replace the ITD's prescriptive based approach to concrete mix design with a performance based approach or a combination of perspective and performance based approach when durability is a prime concern. Currently, ITD uses a recipe type approach in which one recipe fits for all aggregate sources and concrete used in different locations. Realizing that the suppliers/contractors know more about their aggregate and they use their years of experience in designing a mix with a specified performance and durability, allowing the suppliers/contractors to design the mix would improve the quality of concrete while reducing the cost of production. Changing ITD concrete practice to performance or to a combination of performance/prescriptive based mix design eliminates some of the tests required for approval, which results in cost reduction without compromising the quality and durability of concrete.

Question 1.a: Please mark your view of how a performance based or a combination of performance/prescriptive based approach to concrete mix design reduces the risk of failure to provide satisfactory performance.

Question 1.b: Please mark your view of how a performance based or a combination of performance/prescriptive based approach to concrete mix design reduces the cost of concrete production.

Question 1.c: What is the chance of failure for concrete to meet the specified performance and durability criteria if ITD adopts a performance based approach to concrete mix design and eliminates some of the required material acceptance tests, thereby allowing the suppliers/contractors to draw on their knowledge and experience in designing a mix recipe to meet the specified performance and durability criteria.

Question 1.d: What is the chance of failure for concrete to meet the specified performance and durability criteria if ITD adopts a combination of performance/prescriptive based approach to concrete mix design and eliminates some of the required material acceptance tests, thereby allowing the suppliers/contractors to draw on their knowledge and

ITD PARTICIPANTS

- Responses are based on the assumption that any performance or performance/prescriptive concrete specs would include adequate acceptance criteria and testing requirements (strength, durability, etc.) that measure long term performance.
- ITD already uses a performance/prescriptive approach to concrete mix designs.
- I don't believe suppliers in my area have sufficient expertise in mix designs. Most have followed "what has been done before" rather than adopt new designs to save money or materials or produce a better mix. Many believe strength to be the only criteria, which may jeopardize longevity.
- Having the material's acceptance testing done, helps to insure that there will be a quality product. Things can change fast in the stockpiles of material being used in mix designs.
- I would not prescribe to pure performance based…some producers do not understand the volumetric of a mix design
- Performance based specifications favors the large produces, thus making concrete products difficult to obtain in remote areas. Idaho is primarily a remote area with the exception three locations. Not only is it important to allow supplies to utilize their knowledge when they produce large quantities of specification material, but the remote bridge project in Leadore, or Elk River need to have reasonably priced products.
- There have been changes in materials that would assume no impact when there have been significant impact to the durability of the concrete.
- **1.a:** This question is poorly worded. The question should be 'whether' rather than "how". As the provided multiple choice answers provide no opportunity to explain "how".
- I believe a carefully selected choice of performance and prescriptive requirements would result in a lower risk of premature failure than the existing or Performance based approaches. It has **not** been my experience that suppliers know more about their materials, what they can do, or their limitations than the consensus of specialists of Government organizations. Suppliers, after all are in the business of selling concrete while, for example, owners have the job of living with

it. Further, suppliers and manufactures rarely perform tests such as freeze-thaw durability and refute problems with it, blaming it upon poor field practices performed months or years before and typically poorly documented.

- **b:** Again this question is poorly worded. The question should be 'whether' rather than "how". As the provided multiple choice answers provide no opportunity to explain "how".
- I believe that a combination of performance/prescriptive based approach to concrete mix design will allow a supplier the freedom, in some facets of the process, to minimize costs. This would include minimizing the costs of the most expensive ingredient, cement, by decreasing it, where possible while meeting statistical strength requirements, etc.
- However unique or tighter gradation requirements, which would reduce total surface of the aggregate (and therefore cement paste requirements), improved aggregate handling techniques, etc. typically require prescriptive requirements when they are above industry standards. For example there is no drive to develop tighter, denser gradation requirements when everyone else orders their concrete as a "seven, six, or five bag mix".
- Increasing gradation requirements and 'dense' are intended to mean both larger maximum size aggregate and gradations where proportions of each smaller aggregate are selected to more accurately fill the voids in the larger aggregate. The results in both cases is less voids and surface area requiring less cement paste to fill.

NON-ITD PARTICIPANTS

- The combination of both performance and prescriptive would be the best. If ITD and the contractor/supplier could work together to the best results would be achieved.
- In a purely performance based system, the QA/QC becomes more of a responsibility of the Ready Mix producer because they have more responsibility for providing what they say they are providing. In a prescription based spec. there is less responsibility because they are told what mix to provide. Some of the cost benefit for a performance mix could be offset by the testing and QA/QC that the ready mix company will have to do to assure the quality of their product.
- The validity of these answers is very dependent on a whole bunch of issues tied to the performance tests selected.
- 1c: the chance is the same as it is now – no difference. It is currently performance based. The chance of failure comes from poor testing and field curing, not from the concrete. The chance of failure in both 1c and 1d is the same if the same personnel do the testing.

QUESTION 2

- **Comments 2:** Slump test does not directly relate to the field performance. There is a need to measure consistency of concrete. There were suggestions to measure unit weight of concrete as a measure of consistency when performing an air content test (AASHTO 152/ASTM C231) by weighing the material in the air pot and calculating the unit weight.
- **Question 2:** Using unit weight of freshly mixed concrete as a measure of concrete consistency is more reliable than using slump for the same purpose

ITD PARTICIPANTS

- Unit weights are not very sensitive to small increments of water. On many projects, slump tests are not consistent from truck to truck. This does not reflect on the unit weights.
- Slump is important in keeping a placement/pour consistent and is not necessarily related to unit weight. Unit weight can be used as a consistency measure to indirectly give a feel for air content, w/c ratio or cement factor.
- Using the slump cone is a quick way of checking you concrete consistency. Then you can do the unit weight, but slump is a very important part of the testing.
- Unit weight is a back door measure of the air content (Just as we do for the volumetric for AC pavement) Unit weight would be a waste of time as a measure of consistency.
- Slump is easy, quick and relates to primarily workability but indirectly to durability. Air content is not quick or easy and requires more skill. That being said unit weight should be recorded on all air tests, as it is accurate for predicting if there are botches in batching Concrete ingredients, or if ITD is being shorted on Yield.

NON-ITD PARTICIPANTS

- The slump test of concrete does not have any direct correlation to performance and durability concrete. Slump testing of concrete should be replaced by some kind of water/cement ratio test. Question 2 is not relevant because unit weight and the slump test have no correlation. Unit weight and the air test are closely related and do correlate.
- Do both tests – and weigh cylinders before they are broke.
- Slump does directly relate to field performance. It is a fact that higher water /cement ratios yield lower strength concrete. Again it comes to a matter of testing. If slump is not relating directly to field performance then the sample is not handled correctly and the test is invalid.

QUESTION 3

Comment 3: Several participants indicated that the current ITD aggregate gradation specifications need to be reviewed and modified. There were calls for designing a denser concrete.

Question 3.a: In your opinion, does ITD need to improve its aggregate gradation specifications?

Question 3.b: How would you rate the current ITD Aggregate gradation specifications?

Question 3.c: An improved aggregate gradation improves the workability and stability of concrete.

ITD PARTICIPANTS

- Workability of concrete depends on aggregate roundness and sphericity.
- ITD has used fineness modulus in the past. Gradation can definitely effect workability, but a necessary fraction of coarse material is desirable in high wear situations. Gradations may be modified depending on use.

- The aggregate gradation specs seem to be ok. Some areas of the construction would not be able to use a denser concrete. They need specifications for the type of work they will be doing.
- Control of the gradation will control of the mixture. The current coarse and fine aggregate gradation ranges specified work reasonably well for general guidelines. What ITD needs to consider in the mixture is the relationship of percentage of sand, coarse & fine gradation and the fineness modules. A concrete mixture all comes back to the volumetric of the materials and the grain size distribution. A change in the 1-2 percentage of sand can change slump by one inch or more. Finishers like higher percentages of sand and suppliers like higher percentages of sand as sand is the least cost product. High sand content reduces strength, requires more water to be workable, and has reduced flow characteristics. Control of the sand portion and gradation of the sand are critical factors in mix strength and workability
- ITD aggregate specifications are often difficult to meet. I'm not sure they provide better overall product.
- **3.b:** The current ITD agg. gradation spec is 'Desirable' but could be improved. This should especially be considered for large projects.

  3.c: A dense mix is generally not as workable but has better performance in terms of minimizing heat, better durability and cost. This should be considered for large or critical projects such as bridge and large concrete pavement projects

NON-ITD PARTICIPANTS

- No all ready mix producers may have the bin capacity or means to control a more uniformly graded aggregate spec. Maybe having an alternate spec were a producer can choose a more prescriptive (as exists now in the spec) or a performance spec with a denser more well graded aggregate system would give some options and not exclude anyone who didn't have the bin capacity to provide a denser graded aggregate
- Current specs are OK but may be improved with combined grading approach – which in an of itself is still not ideal.
- 3c. There are large discrepancies as to the term workability and stability that lead to varying opinions as to what they are.

QUESTION 4

Comment 4: There were comments about the use of advanced methods to better control the quality of concrete during production and placement. These advanced techniques include a) a maturity test using a calorimeter, which records time and temperature for freshly mixed concrete and b) an air void analyzer.

Question 4: Adopting the advanced techniques named in Comment 4 to measure early strength, heat of hydration, consistency, and air content of concrete provides more reliable information about the quality of concrete.

ITD PARTICIPANTS

- A maturity meter may more accurately represent actual field conditions than standard lab tests. It is my understanding that AASHTO may be adopting this test in the near future. I am not familiar with an air void meter.
- The physical tests will provide a person with most of this information. Plus its hands on so maybe more reliable?
- These test techniques have been used for quality in the construction industry for many years (15 or more) for major projects such as skyscrapers. The advantage I see in the use of these techniques is they measure the actual material in place as it is curing. Compressive strength of cylinders currently used by ITD only measure the potential strength cured under tightly controled laboratory conditions and are not a measure of actual field cure conditions.
- Advanced testing equipment tends to be unreliable in the field. ITD probably just needs to use better control over field specimens (ie, curing conditions, handling, etc)
- From what I just learned the Air Void Analyzer should be part of the mix design process then checked occasionally during production.
- I believe the maturity testing process may be of use on some structures where early loading is required or in concrete pavements placed near the beginning or end of the season where early loading is desired. However the cost of this program is high in terms of labor and should be cost justified in advance. Maintaining cure for at least 7 days should be required where costs are not justified

NON-ITD PARTICIPANTS

- Be careful with terminology - maturity is not measured with a calorimeter
- Any information is useful if it is valid.

QUESTION 5

Comment 5: Currently, ITD specifies maximum slump for concrete used in different projects. If the slump does not meet the specifications, generally suppliers add water to control workability, which could have an adverse effect on its performance. ITD need to use additives to control concrete workability.

Question 5: The use of super plasticizers and additives to control workability of concrete would improve the quality of its performance.

ITD PARTICIPANTS

- In many cases, high slump is a problem rather than low slump. Super plasticizers are completely acceptable when used correctly. Segregation can be a problem with too high slump and must be avoided.

- Yes, plus it would not change the water/cement ratio.
- Use of plasticizers can work well to control workability for certain placements where high flowability is needed and cannot be obtained by other methods. One danger is that super plasticizers will permit reducing the W/C ratio below the amount needed to hydrate the cement yet will still be flowable. Finishing can be difficult once the chemical dissipates. Workability can be better controlled by gradation and proportioning.
- Need to control the air content. Freeze – Thaw is significant.
- There are really two different issues. Water exceeding the allowable W/C ratio should not be allowed as durability and early strength are decreased.
- Maximum allowable W/C should be dependant upon a number or factors including exposure (to weather and water), importance of the structure, etc.
- Using a high rate water reducing additive will significantly improve the workability of a **dense gradation mix** or a low W/C mix. They are also useful in difficult, complex placements (lots o' rebar), and have some use where high early strengths are desired. By decreasing excess water they increase durability and can help with bleeding problems.

NON-ITD PARTICIPANTS

- For this same reason the slump test is not reliable any longer.
- As with adding anything on the job, adequate mixing becomes important. Some water reducers can tend to have adverse affects on set, especially in extreme (cold or hot) temperatures.
- ITD has trouble testing plain concrete, they certainly do not have the expertise to test or evaluate rheoplastic mixes.

QUESTION 6

Comment 6: Currently ITD concrete specifications require 660 lbs of cement per yard of concrete for 4,500 psi concrete compressive strength. This is excessive. Excessive cement generates more heat of hydration promoting shrinkage and cracking. It impacts the stability of concrete by providing more paste and it also promotes ASR by increasing the alkali content in concrete. In addition, ITD requires a maximum water-cement ratio of 0.44 and maximum slump of 2 in. for concrete used in pavement projects.

Question 6.a: Reducing cement content of concrete, requiring maximum w/c, and controlling workability by improving aggregate gradation and the use of super plasticizers and admixtures would improve the quality and performance of concrete.

Question 6.b: Reducing cement content of concrete, requiring maximum w/c, and controlling workability by improving aggregate gradation and the use of super plasticizers and admixtures reduces the cost of concrete production.

ITD PARTICIPANTS

- For paving concrete, Reducing cement content of concrete, requiring maximum w/c, and controlling workability by improving aggregate gradation and the use of super plasticizers and admixtures reduces needs to be consistent with aggregate fractured face specifications.
- The use of admixtures generally adds to the cost (probably more so than the offset of reducing the cement). Probably the reason for high cement contents is with the high coefficient of variation of current plants, it is required to exceed minimum strengths at all times. Until plants upgrade to produce less variation in batch weights, water addition, etc, the overdesign is necessary.
- With 660 lbs. of cement your almost always going to meet strength specifications. If the specs were to change a contractor wanting to save money would use 400 lbs. of cement and lots of add mixtures. What type of product would the state be stuck with then?
- This should decrease the cost of production, but there are many other factor that affect the cost of concrete to say this would reduce the cost to ITD. Rejection of product due to not meeting specifications, price of admixtures, failing strength tests, etc have cost effects.
- **6.a:** It is not certain what is intended by “requiring maximum w/c”. The above answer is predicated on this meaning a very low W/C.
- **6.b:** There are conflicting items here. A more dense aggregate mix will lesson workability as will lowering the cement content as will “requiring maximum w/c” if this mains a very low W/C. This will result in less ‘paste’, making finishing more difficult. Careful selection and use of super plasticizers will increase workability. Costs will be affected similarly. Non-commercial (dense) gradations will cost more as will additional admixtures. Decreasing cement content will decrease cost. W/C ratio doesn’t affect cost. Dense gradations should be considered for large projects where supplier costs associated with non-standard gradations can be recuperated.

NON-ITD PARTICIPANTS

- Cement is typically the most expensive component of the mix, but changing admixtures and controlling a tighter aggregate spec could offset some of the cost savings. Quality and performance of concrete “as delivered” can be totally different from concrete “as placed.” Education of contractors placing the concrete as to the proper placing and finishing procedures has a great affect on the end product quality and durability.
- The current mixes are over design substantially to overcome poor testing practices by ITD. If the tests by ITD would reflect the actual strength of the mix reducing the cement content would have already been addressed. As it is now the cement requirement is a minimum that no one dares go below because added cement is “insurance” against a poor field technician.

QUESTION 7

Comment 7: AASHTO M 295 and ASTM C 618 limit the maximum percentage of Loss on Ignition (LOI) in fly ash to 5 percent and 6 percent respectively. The ITD current specifications limit LOI in fly ash to a maximum of 1.5 percent.

Question 7: Increasing the LOI limit to a maximum of 5 percent would not significantly impact the quality of concrete performance.

ITD PARTICIPANTS

- Having a control on the amount of fly ash added is good. Too much would not produce a good product.
- I don't know why ITD is more conservative than AASHTO and ASTM
- Based upon current ITD specifications allowing for up to 7 percent deleterious material in the fine aggregate and 4.5 percent in the course aggregate one could surmise that roughly 10 percent of the aggregate can be undesirable without a risk to the product. As long as the total amount of deleterious material in the total of the mix does not exceed this roughly 10 percent value that historically has be acceptable there should be no adverse effect.
- The LOI results in the amount of burned off carbon. The increase in carbon will impact the air content of the concrete. Resulting in an increase in scaling and the concrete becoming less durable. Even more significant in the used of silica fumes.
- I don't know much about fly ash. However as we are having problems in the ASR area I'd be cautious about loosening standards. If there are material gains by allowing a higher LOI, such as wider selection of sources or better functioning fly ashes it should be considered, however even 5 percent sounds high.

NON-ITD PARTICIPANTS

- It is the ability of the Fly Ash producer to control the variation or inform the Ready Mix producer of changes in Fly Ash LOI before they receive the material that is important. If the LOI is fairly consistent, the Ready Mix producer can compensate by their dosages of AEA. Whether the LOI is at 1 percent or 3 percent is not the issue as long as the producer knows. There are tests such as the Foam Index test that can easily be performed by the Ready Mix producer to test for uniformity in the Fly Ash from load to load. It can tell them if they need to compensate with higher or lower dosages of AEA.
- As long as the producer is knowledgeable in the use of fly ash with higher LOI then quality concrete with this LOI is not a problem
- A typical case of performance or prescriptive. Measure the air entertainer needed to get the air void system desired rather than one of the parameters that may affect it.

QUESTION 8

Comment 8: Currently, ITD requires an AASHTO T303 (ASTM C1260) test for identifying potentially deleterious aggregate.

Question 8.a: What is the risk of having ASR problems for an aggregate passing the AASHTO T 303 (ASTM C1260) test?

Question 8.b: Requiring an ASTM C 1293 test in addition to AASHTO T 303 (ASTM C1260) to identify reactive aggregates would reduce the risk of having ASR problems.

ITD PARTICIPANTS

- The Contractor has this option now.
- AASHTO T303 is a very aggressive test and probably represents worst case. C1293 is probably more applicable, but may require a source to be tested well before that area is actually used (long duration test). C295 can determine if an aggregate has components which are not reactive, but are effected in the previous 2 tests.
- Yes
- From my experience neither of these tests provide reliable results.
- The C1260 should be run on a 28 day term. The C1260 provides the best results but takes a year to run. Need to qualify sources well in advance of a project. Perhaps during the Design period of the project.
- I am not certain. I was not able to read the ASTM C1293 procedure.

NON-ITD PARTICIPANTS

- 8a: Reactivity of aggregates and any correlation to T303 could be studied more to see if a relationship exists between the two.
- The only problem with the 1293 is it is a yearlong test. Some projects can't wait that long. Having 1293 data on the different aggregate sources in the state would give a more complete picture of the ASR risks and compliment the T303 test.
- If an aggregate passes the 1260 it most likely will not exhibit ASR in the field because the test environment is more harsh that the natural environment. Aggregates that pass the 1260 will most likely pass the 1293 because it is a less harsh lab test. The 1293 more accurately represents field conditions. Where the 1293 becomes valuable is in proving that an aggregate that fails the 1260 but passes the 1293 should not exhibit ASR in the field. If for example an aggregate failed the 1260/T303 and passed the 1293, an engineer could specify lithium nitrate to mitigate ASR, when the aggregate may never exhibit those symptoms in the field. Lithium nitrate is a costly admixture which raises the cost of the concrete. In this case, the 1293 provides evidence to show that the lithium nitrate may not be necessary providing cost savings in the mix.

- All aggregate tests are subjective and represent a very small percent of the total aggregate base. Until a large number of tests are performed over a long period of time the actual reactivity is speculative.

QUESTION 9

Comment 9: A question was raised that having a minimum 4 percent limit for air content could be inadequate to provide freeze/thaw durability for some mixes. For mixes with large nominal maximum aggregate size (1.5 in. or larger), 4 percent minimum air content might be adequate, but for mixes with 1 in. or smaller nominal maximum size, aggregate would require more paste and 4 percent minimum air content might be inadequate. ACI 301 in table 4.2.2.4 recommends the use of 6 percent +/- 1.5 percent for severe freeze/thaw environments such as exist in Idaho.

ITD PARTICIPANTS

- It is only a minimum.
- Idaho currently uses the 6.5 ± 1.5 percent for structural mixes (classes 30 and 40) other than concrete pavement (class 45 – larger agg). Although the specification states 0 – 6 percent, the footnotes require classes A, B, and C to have 6.5 ± 1.5 percent.
- That would be ok, the extra air would help with freeze thaw.
- ITD's specification of 5 to 8 percent is greater than 4 percent minimum or even 4.5 percent to 7.5 percent per ACI 303. We should be adequate in providing freeze/thaw durability.
- Air content is critical for thin placements of exposed concrete and should remain at minimum 4.5 percent as this will allow for some entrained air above the entrapped air of 3-4 percent.
- Idaho specifications do not differentiate between placement locations for air content for placements below the frost line or area not exposed to freeze thaw. If a change is wanted for minimum air content it should be based upon the location of the placement, not a blanket change.
- The smaller the coarse aggregate the more mortar is required. It is logical that more air is required as this is contained in the mortar.
  A dense ¾" MSA mix should have: 7 percent ± 1 percent,
  A dense 1½" MSA mix should have 5.5 percent ± 1 percent,
  A dense 3" MSA mix should have 4.5 percent ± 1 percent.
- Entrained air in concrete with no exposure to deicing salts can be decreased by 1 percent.
- Entrained air in concrete that is not exposed to harsh weather and/or water conditions can also be decreased.
- The ITD air content specs for concrete need to be reviewed and revised. The current "one size fits all" is likely adequate for obtaining optimum concrete durability. Along with the spec, the method of testing should be reviewed. Testing prior to placement in a structure and prior to consolidation does not represent the final air content in the concrete.

NON-ITD PARTICIPANTS

- More testing needs to be done to determine if 4 percent air is less durable. Some studies have suggested that 4 percent air would make the concrete less permeable to water and have better compressive strength thus making it more durable.
- I agree with table 4.2.2.4. It is better to error on the high side with air for durability purposes in Idaho.
- Parameter required is spacing factor. Total air required to achieve this for a given mix can then be calibrated in. Depends on paste content, SCM dosage and chemistry, and admixture type, haul times, handling procedures…..
- I agree with the findings in ACI 301, however, I have yet to see conclusive evidence that 4 percent air entrainment is inadequate in this area.

ITD Mix Design and Comparison with Other States(11,14,17)

MATERIAL/ PRODUCT	TEST FOR		IDAHO SPECIFICATION	IOWA SPECIFICATION	MICHIGAN SPECIFICATION
Cement	Portland Cement	Type I, IA, III, and IIIA Cements	AASHTO M85 Type I, II, or III (>0.6% of total alkali is not accepted)	ASTM C150. Type I, II, III	Conform to ASTM C150. The requirements for Gillmore or Vicat setting time and compressive strength for 7- and 28-days apply.
		Type IS, I (SM), IS-A, and I (SM)-A Blast-Furnace Slag Cements			ASTM C595
		Type IP, I (PM), IP-A, and I (PM)-A Pozzolan Cements.			ASTM C595
	Blended Hydraulic Cement	Pozzolan constituent of Type IP cement ≤ 25 weight (mass) percent of the Portland-pozzolan cement	AASHTO M240 Type IP, P, or I (PM)	ASTM C595 Type IP and IS	
		Slag constituent Type IS cement ≤ 35 weight (mass) percent of the Portland blast-furnace slag cement.			
		Type IP cement shall not contain Class-C fly ash			
	White Cement				ASTM C150, Type I except not more than 0.55% of ferric oxide (Fe_2O_3) by weight is permitted
	Masonry Cement				ASTM C91, Type N, S, or M.
	Hydrated Lime				ASTM C207, Type S or SA.
	Ground Granualated Blast Furnance Slag				ASTM C989, Grade 100, minimum. Use only as a blending material with Type I or Type IA Portland cement

ITD Mix Design and Comparison with Other States[11,14,17] (cont.)

MATERIAL/ PRODUCT	TEST FOR		IDAHO SPECIFICATION	IOWA SPECIFICATION	MICHIGAN SPECIFICATION
Aggregate	Fine Aggregate	Fine Concrete & Plant Mix Aggregate	AASHTO M6, AASHTO M29, Idaho T-13		
		Sodium Sulfate Soundness Test	AASHTO T104		
		Sand Equivalent Test			ASTM D2419
		Organic Impurities	AASHTO M6, AASHTO T21, AASHTO T71		AASHTO T21
		Alkali Silica Reactivity	AASHTO T 303		
		Sieve Analysis	AASHTO T27, Finer than No. 200 sieve, AASHTO T11 Method A or B		ASTM C136
		Clay lumps, Friable Particles, Lightweight Particles	AASHTO T112		
		Lightweight particles	AASHTO T113		
		Specific Gravity and Absorption			ASTM C128
		Angularity Index			Michigan Test Method (MTM) 118
		Mortar Strength		Iowa DOT Materials Laboratory Test Method 212 (1.5% minimum)	
		Fineness Modulus		Materials I.M. 302 (2.75% minimum	
		Shale and Coal		Materials I.M. 344 (2.0% maximum)	

ITD Mix Design and Comparison with Other States[11,14,17] (cont.)

MATERIAL/ PRODUCT	TEST FOR		IDAHO SPECIFICATION	IOWA SPECIFICATION	MICHIGAN SPECIFICATION
Aggregate	Coarse Aggregate	Coarse Concrete Aggregate	AASHTO M80		
		Sodium Sulfate Soundness Test	AASHTO T104 (Loss ≤ 12% at 5 cycles)		
		Alkali Silica Reactivity	AASHTO T303		
		Sieve Analysis	AASHTO T27, Finer than No. 200 sieve, AASHTO T11 Method A or B		
		Percentage of Fracture	AASHTO TP61		
		Degradation	AASHTO T96		
		Elongated particles	ASTM D4791		
		Selection and Preparation of Coarse Aggregate Samples for Freeze-Thaw Testing			
		Making Concrete Specimens for Freeze-Thaw Testing on Concrete Coarse Aggregate			MTM 114
		Freeze-Thaw Testing of Coarse Aggregate			MTM 115
		Specific Gravity and Absorption			ASTM C 127
		Alumina		Iowa DOT Materials Laboratory Test Method 222 (<0.5%)	
		A Freeze		Iowa DOT Materials Laboratory Test Method 211, Method A (≤6%)	
		Clay Lumps and Friable Particles		Materials I.M. 368 (≤0.5%)	
		Abrasion	AASHTO T96	AASHTO T96 (Cr. Stone ≤ 50; Gravel ≤ 35 (may be increased by 0.1% for each 1% of particles with at least one fractured face))	

ITD Mix Design and Comparison with Other States[11,14,17] (cont.)

Aggregate	Coarse Aggregate	Sieve Analysis	AASHTO T27, Finer than No. 200 sieve, AASHTO T11 Method A or B		
		Percentage of Fracture	AASHTO TP61		
		Degradation	AASHTO T96		
		Elongated particles	ASTM D4791		
		Selection and Preparation of Coarse Aggregate Samples for Freeze-Thaw Testing			
		Making Concrete Specimens for Freeze-Thaw Testing on Concrete Coarse Aggregate			MTM 114
		Freeze-Thaw Testing of Coarse Aggregate			MTM 115
		Specific Gravity and Absorption			ASTM C 127
		Alumina		Iowa DOT Materials Laboratory Test Method 222 (<0.5%)	
		A Freeze		Iowa DOT Materials Laboratory Test Method 211, Method A (≤6%)	
		Clay Lumps and Friable Particles		Materials I.M. 368 (≤0.5%)	
		Abrasion	AASHTO T96	AASHTO T96 (Cr. Stone ≤ 50; Gravel ≤ 35 (may be increased by 0.1% for each 1% of particles with at least one fractured face))	
MATERIAL/ PRODUCT	**TEST FOR**		**IDAHO SPECIFICATION**	**IOWA SPECIFICATION**	**MICHIGAN SPECIFICATION**
		Sieve Analysis of Mineral Filler			AASHTO T37
		Mortar Strength			AASHTO T71
		Particle Size Analysis			AASHTO T88
		L. A. Abrasion Resistance of Aggregate			MTM 102

ITD Mix Design and Comparison with Other States [11,14,17] (cont.)

MATERIAL/ PRODUCT	TEST FOR		IDAHO SPECIFICATION	IOWA SPECIFICATION	MICHIGAN SPECIFICATION
Aggregate	Coarse Aggregate	Coarse Concrete Aggregate	AASHTO M80		
		Sodium Sulfate Soundness Test	AASHTO T104 (Loss ≤ 12% at 5 cycles)		
		Alkali Silica Reactivity	AASHTO T303		
		Sieve Analysis	AASHTO T27, Finer than No. 200 sieve, AASHTO T11 Method A or B		
		Percentage of Fracture	AASHTO TP61		
		Degradation	AASHTO T96		
		Elongated particles	ASTM D4791		
		Selection and Preparation of Coarse Aggregate Samples for Freeze-Thaw Testing			
		Making Concrete Specimens for Freeze-Thaw Testing on Concrete Coarse Aggregate			MTM 114
		Freeze-Thaw Testing of Coarse Aggregate			MTM 115
		Specific Gravity and Absorption			ASTM C 127
		Alumina		Iowa DOT Materials Laboratory Test Method 222 (<0.5%)	
		A Freeze		Iowa DOT Materials Laboratory Test Method 211, Method A (≤6%)	
		Clay Lumps and Friable Particles		Materials I.M. 368 (≤0.5%)	
		Abrasion	AASHTO T96	AASHTO T96 (Cr. Stone ≤ 50; Gravel ≤ 35 (may be increased by 0.1% for each 1% of particles with at least one fractured face))	

Virginia DOT Concrete Pavement Specifications[16]

MATERIAL/ PRODUCT	TEST FOR				SPECIFICATION	REMARKS	ACCEPTANCE REQUIREMENT
Hydraulic Cement	Portland Cement	Type I and Type II			AASHTO M85, ASTM C150	< 1.0% alkalies	SiO_2 content shall be at least 20%
		Type II				≤ 65% C_3S	
		Type III				≤ 8% C_3S	
	Blended Hydraulic Cement				AASHTO M240 Type IP(P), or Type I(S)		
Aggregate	Fine Aggregate	Soundness Test			AASHTO T103 or T104	0% by weight	
		Organic Impurities			AASHTO M6, AASHTO T21		
		Alkali Silica Reactivity			AASHTO T303		
		Sieve Analysis			AASHTO T27	0.25% by weight	
		Deleterious Material	Clay lumps		AASHTO T112	1% by weight	
			Shale, mica, coated grains, soft or flaky particles		AASHTO T113		
			Total material passing No. 200 sieve by washing		AASHTO T11 and T27	3% by weight	
			For use in concrete subject to abrasion			5% by weight	
			For other concrete				
		Void Content			VTM-5		

Virginia DOT Concrete Pavement Specifications[16] (cont.)

MATERIAL/ PRODUCT	TEST FOR			SPECIFICATION	REMARKS	ACCEPTANCE REQUIREMENT
Aggregate	Coarse Aggregate	Coarse Concrete Aggregate				
		Soundness Test			Loss ≤ 12% at 5 cycles	
		Organic Impurities				
		Alkali Silica Reactivity				
		Sieve Analysis				
		Flat and Elongated Particles		ASTM D4791	>30% by mass of aggregate particles retained on and above the ⅜-inch sieve having a maximum to minimum dimensional ratio greater than 5	
		Deleterious Material	Coal and lignite	AASHTO T113	0.25% by weight	
			Clay lumps	AASHTO T112	0.25% by weight	
			Material passing No. 200 sieve by washing	AASHTO T11	1% by weight	
		Abrasion		AASHTO T96		
Granulated iron Blast-Furnace Slag				ASTM C989	< 50% for class F in concrete	
Silica fume				AASHTO M307	< 10% for class-F in concrete	
Fly Ash		Hydraulic Cement Concrete		ASTM C618, Class-F or Class-C	< 30% for class-F in concrete	
		Lime Stabilization		ASTM C593.		
Water				AASHTO T26, ASTM C1602,	PH between 4.5 and 8.5.	
Concrete	Compressive Strength			ASTM C39, ASTM C31, or	Min. 75% standard	
	Slump			ASTM C42	Greater than Standard	
	Air and Consistency			AASHTO T119	Max 9%	

Washington DOT Concrete Pavement Specifications[15]

MATERIAL/ PRODUCT	TEST FOR			SPECIFICATION	REMARKS	ACCEPTANCE REQUIREMENT
Portland Cement	Portland Cement			AASHTO M85 Type I, II, or III or ASTM C150	> 0.75% of total alkali is not accepted.	Manufacturer's Mill Test Report number indicating full conformance to the Specifications
	Low Alkali Cement			AASHTO M 85 Type I, II, or III or ASTM C150	> 0.6% of total alkali is not accepted.	
	Blended Hydraulic Cement			Type I (SM) (MS) or Type I (PM) (MS) cement conforming to AASHTO M240, P, or I(PM)	> 0.75% of total alkali is not accepted.	
Aggregate	Fine Aggregate (Sand or other inert Materials, or Combinations Thereof)		Particles of Specific Gravity 1.95		< 1% by weight	
			Organic Impurities			
			Alkali Silica Reactivity		Expansion > 0.2% Requires mitigation measures	
					Expansion > 0.04% Requires mitigation measures	
			Clay, Loam, Alkali, Organic Matter, or Other Deleterious Matter.	Washed thoroughly		
	Coarse Aggregate (Gravel, Crushed Stone, or Other Inert Material or Combinations thereof Having Hard, Strong, Durable Pieces Free from Adherent Coatings)	Coarse Concrete Aggregate		AASHTO M 80		
		Sodium Sulfate Soundness Test		AASHTO T104	Loss ≤ 12% at 5 cycles	
		Organic Impurities		AASHTO M6		
		Deleterious Substances	Amount Finer than U.S. No. 200		1% by weight	
			Pieces of specific gravity less than 1.95		2% by weight	
			Clay Lumps		0.50% by weight	
			Shale		2% by weight	
			Wood waste		0.50% by weight	
		Deleterious Substances for Portland Cement Concrete	Amount finer than U.S. No. 200		2.0 percent by weight	
			Pieces of specific gravity less than 1.95		2.0% by weight	
			Clay lumps		0.3% by weight	
			Shale		1.0% by weight	
			Wood waste		0.03% by weight	
		Alkali Silica Reactivity		AASHTO T303 or ASTM C1260	Expansion > 0.2% Requires mitigation measures	
				ASTM C1293	Expansion > 0.04% Requires mitigation measures	
		Percentage of Fracture		AASHTO TP-61		
		Degradation		Los Angeles Abrasion Test, AASHTO T96	Retained on the U.S. No. 4 sieve shall not have > 35% after 500 revolutions.	

Washington DOT Concrete Pavement Specifications [15] (cont.)

MATERIAL/ PRODUCT	TEST FOR		SPECIFICATION	REMARKS	ACCEPTANCE REQUIREMENT
		Sampling	WSDOT FOP for AASHTO T2		
		Organic Impurities	AASHTO T21		
		Percent of Fracture in Aggregates	WSDOT FOP for AASHTO TP61		
		Sieve Analysis of Fine and Coarse Aggregates and Aggregates in HMA	WAQTC FOP for T27/11		
		Determination of Degradation Value	WSDOT T113		
		Determination of Fineness Modulus	AASHTO T27		
		Particle Size Analysis of Soils	AASHTO T88		
		Stabilometer R Value, Untreated Materials	WSDOT T611		
		Swell Pressure and Permeability	WSDOT T611		
		Stabilometer S Value, Treated Materials	WSDOT T703		
		Determining Stripping of HMA	WSDOT T718		
		Sand Equivalent Test for Surfacing Materials	WSDOT FOP for AASHTO T176		
		Clay Lumps in Aggregates	AASHTO T112		
		Material Finer than U.S. No. 200 Sieve in Aggregates	AASHTO T11		
		Compressive Strength of Concrete	WSDOT FOP for AASHTO T22		
		Flexural Strength of Concrete	WSDOT T802		
	Fly Ash	Natural Pozzolans and Fly ash	AASHTO M295 Class-C & F	LOI≤ 1.5 percent	
	Ground Granulated Blast Furnace Slag		AASHTO M302, Grade 100 or Grade 120	Manufacturing facility shall be certified on the cement mill test certificate.	
	Micro silica Fume		AASHTO M307		
	Concrete	Aggregate correction factor	WAQTC FOP (Western Alliance for Quality Transportation Construction) for AASHTO T152		
		Water	ASTM C94M		
		Compressive Strength	AASHTO T22		
		Making and Curing in Field and Laboratory	AASHTO T23, AASHTO T126, WSDOT T 808		
		Slump	AASHTO T119		
		Air Content	WAQTC FOP for AASHTO T152.	Lower limit 3% & upper limit 7%	
		Temperature	AASHTO T309		
		Sampling	WAQTC TM-2		

APPENDIX B

ASR Survey

AASHTO LEAD STATES TEAM
SURVEY ON ALKALI-SILICA REACTIVITY (ASR)

1 - Agency Completing Survey
2 - Address
3 - Person Completing Survey
4 - Title of Person Completing Survey
5 - Phone and. Fax Number

GENERAL

6 - Has your State/Province experienced unexpected concrete cracking/deterioration problems?

7 - Has there been an assessment of the causes of the deterioration?

(a) Any that are attributed to alkali-silica reactivity (ASR)? Yes No Not sure

(b) If yes, what type of assessment was performed?

(c) What type of testing was performed?

8 - If you've determined that the cracking/deterioration is attributed to ASR, is the reaction widespread or localized?

(a) If localized, what areas of your State/Province are affected? and, who in the strict, region, or area is the contact person?

(b) Can you provide the Team with any surveys, reports, or studies on the ASR problem you've encountered?

(c) How long after construction did the cracking/deterioration occur?

(d) What detection techniques are being used?

9 - What steps or techniques are used to remediate ASR caused cracking/deterioration in new and old construction? Are you testing and/or treating the aggregate? if so, how? Are you using low-alkali cement, blended fly ash/cement, fly ash, silica fume, or ground granulated blast-furnace slag for mitigation?

10 - How long have the techniques been implemented?

(a) Are your current ASR mitigation techniques affective? Why or why not?

11- What if any, specifications address ASR in your State/Province? Can you provide a copy?

12 - Does your agency offer programs or training on detection, prevention. and remediation of ASR to Personnel within your agency?

13 - Would you be interested in additional information on ASR? What types of information would you be interested in?

Guide specifications
Testing specifications
Detection of ASR
Case histories
Remediation of ASR
SHRP reports

Other

14 - If the AASHTO Lead States Team on ASR were to offer a showcase on ASR, would your State/Province be interested in hosting such an event?

15 - If the ASR Lead States Team were to offer other services, which of the following would you be interested in?

Field Surveys
Field detection using UV testing kit
Uranyl acetate testing in the laboratory
Laboratory testing training
Assistance in writing specifications
Field inspection training
Assistance with mix design
Field demonstration assistance
Laboratory study set-up
Other

STATE	Question Cracking Problems Yes/No	Question 7 Assessment Problems Yes/No	Question Attrib. To ASR Yes/No	Question 7b Type of Assessment (See Tab 7b)	Question 7c Types of Testing (See Tab 7c)	Question 8 Is ASR widespread or Localized	Question 8a If Localized, Name of Contact Person	Question 8b Survey Results	Question 8c How Long After Construction did Problems Occur	Quest 8d Detection Techniques Used (See Tab 8d)
AL	Yes	Yes	No	g,h		Localized	Sergio Rodriguez	See Attachments	See Attachments	a,b,c
AK	No		No			N/A				
AZ	Not Sure	No	Not Sure			Not Sure				none
CO	Yes	Yes	Yes	a, b	a,b,c	Localized	See Tab 8a		5-15 years	a
CT	No	No	No			N/A				
FL	Yes	Yes	No	N/A	N/A					
GA	No	No	No		N/A	N/A				
HI	No	No	No			N/A				
ID	Yes	Yes	Yes	c	i	Localized	A.F. Stanley	See Attachments	< 10 Years	b,c
IN	Yes	Yes	Yes	b	h	Widespread		See Attachments	< 30 Years	a,b
KS	Yes	Yes	No							
KY	No	Yes	No			N/A				
ME	Yes	Yes	Yes	b,f,c		Widespread	Michael Redmond		+10 years	b,c
MD	Yes	Yes	Yes	f	d,h	Localized		No	4-7 years	c,f
MI	Yes	Yes	Not Sure	b	h	Widespread		none	< 5 years	b
MS	Yes	Yes	Not Sure	b	none	Localized	NE Area	No	10 years	a
MT	Yes	Yes	No			N/A				
NE	Yes	Yes	Yes	b,j	h	Widespread		In-Progress	5-10 years	a,b
NH	Yes	Yes	Yes	c,f	e	Widespread	Richard M. Lane	In-Progress	Unknown	b
NJ	Yes	Yes	Yes	c	h	Localized			approx 13 years	a,b
NM	Yes	Yes	Yes	a,b,c,f	d,h	Widespread	Bryce Simons	Done	5-20 Years	a,b,c
NY	Yes	Yes	No	b	h					
NC	Yes	Yes	Yes	b	b,h,j,k,l,m,n,o,p	Localized	Mrinmay Biswas	Yes (none attached)	< 8 years	a,b
NS	Yes	Yes	Yes	b,h	d,h	Localized	Gary Pyke	See Attachments	15-20 years	e
OH	No	N/A	No		N/A	N/A				
ON	Yes	Yes	Yes	Various	c,d,e	Widespread			4-10 years	a
OR	Yes	Yes	Yes	c	b	Localized	Keith Johnston	No	1-30 years	a
PA	Yes	Yes	Yes	c	d	Unknown		See Attached	4-10 years	b,c
RI	Not Sure	N/A	Not Sure	N/A	N/A	N/A	N/A	N/A	N/A	N/A
SK	Yes	Yes	Yes	b	d	Localized	Herve Bachelu	none	approx 15 years	a
SC	No	N/A	No			N/A				
SD	Yes	Yes	Yes	c,f,g	e,f,g	Widespread		Done	5-25 years	a,d
TX	Yes	Yes	Yes	b	b	Localized	El Paso			a
UT	Yes	Yes	Yes		d	Localized	Central/South Utah		approx 30 years	a
VT	Yes	Yes	Yes	Not Sure		Localized		No	approx 10 years	b
DC	No	No	No							
DC	Yes	Yes	Not Sure		d	N/A				
WT	Yes	Yes	No	c,h,i		N/A				

STATE	Question 8b Survey Results	Question 8c How Long After Construction did Problems Occur	Question 8d Detection Techniques Used (See Tab 8d)	STATE	Question 9 Remediation Techniques (See Tab 9)	Question 10 How Long Been Used	Question 10a Are Techniques Effective	Question 11 What Specs Address ASR	Question Do You Offer Training	Question 13: Are you Interested i Guide Specs	Testing Specs	Detection of ASR
AL	See Attachments	See Attachments	a,b,c	AL	a,e	Unknown	Yes	none	No		Yes	
AK				AK	g				No			
AZ			none	AZ	g,h	15 years		Attached	No	Yes	Yes	
CO		5-15 years	a	CO	a,b,c	10 Years	Yes	See Quest 9	No	Yes		
CT				CT	N/A	N/A		N/A	No	Yes	Yes	Yes
FL				FL	N/A	N/A		none	No			
GA				GA	a,e,f	> 15 years		N/A	No			Yes
HI				HI	N/A	N/A		none	No	Yes	Yes	Yes
ID	See Attachments	< 10 Years	b,c	ID	a,h	< 5 years	not sure	See Attached	No	Yes	Yes	
IN	See Attachments	< 30 Years	a,b	IN	a,e	New	not sure	See Quest 9	No	Yes		
KS				KS	a,j,l,k	>50 years	Yes	See Quest 9	No			
KY				KY	N/A	N/A		none	No			
ME		+10 years	b,c	ME	a,e,f	2 years		Attached	No			
MD	No	4-7 years	c,f	MD	e	6 years	Yes		Yes			
MI	none	< 5 years	b	MI	e	N/A		none	No		Yes	Yes
MS	No	10 years	a	MS	a	+25 years	not sure	See Quest 9	No		Yes	Yes
MT				MT								
NE	In-Progress	5-10 years	a,b	NE	l	3-4 years	Yes	See Attached	No			
NH	In-Progress	Unknown	b	NH	none	Unknown		none	No	Yes	Yes	Yes
NJ		approx 13 years	a,b	NJ	e	New		In-Progress	No			
NM	Done	5-20 Years	a,b,c	NM	a,f,g,l,n,q,t,p	Recent	Unknown	New-See Attached	Yes	Yes	Yes	Yes
NY				NY	a,q,r,s	>30 years	Yes	See Attached	No			
NC	Yes (none attached)	< 8 years	a,b	NC	a,e	8 years	Yes	See Quest 9	No	Yes		Yes
NS	See Attachments	15-20 years	e	NS	a,e	3 years	not sure	CIP Specs Attached	No	Yes	Yes	Yes
OH				OH	N/A	N/A		none	No	Yes	Yes	Yes
ON		4-10 years	a	ON	d	15 Years	Yes	Attached	No			
OR	No	1-30 years	a	OR	a,d,e	15 years	not sure	none	No			
PA	See Attached	4-10 years	b,c	PA	a,e,l	6 years	partially	See Attached	No			Yes
RI	N/A	N/A	N/A	RI	a,e	approx 15 yrs	not sure	none	No	Yes	Yes	Yes
SK	none	approx 15 years	a	SK	a,p	4 years	Unknown	See Quest 9	No	Yes	Yes	Yes
SC				SC	a	N/A			N/A	Yes	Yes	Yes
SD	Done	5-25 years	a,d	SD	a,h,m,No	approx 6 years	Yes	Yes	No	Yes	Yes	Yes
TX			a	TX	a,e,f,l,o	Recent	Unknown	See Comment	No	Yes	Yes	Yes
UT		approx 30 years	a	UT	a,h		Yes	UDOT PCCP Spec	No		Yes	
VT	No	approx 10 years	b	VT	none	N/A		none	No		Yes	Yes
DC				DC	i				No	Yes	Yes	Yes

STATE	Question 14	Question 15: Which Services are of Interest									
	Will You Host Seminar?	Field Surveys	Field Detection Using UV Test Kit	Uranyl Acetate in Laboratory	Laboratory Testing Training	Assist. In Writing Specifications	Field Inspection Training	Assist. W Mix Design	Field Demo Assist.	Laboratory Study Set-up	Other
AL	Yes										Petrograph
AK	No										
AZ	Yes		Yes	Yes	Yes					Yes	
CO	Maybe	Yes	Yes		Yes	Yes		Yes	Yes	Yes	
CT	No	Yes	Yes	Yes			Yes		Yes		
FL	No										
GA	No	Yes	Yes								
HI	No										
ID	No					Yes					
IN	No					Yes			Yes		
KS	No										
KY	No										
ME	No										
MD	No						Yes				
MI	No			Yes	Yes					Yes	
MS	Yes			Yes	Yes				Yes	Yes	
MT											
NE	No										
NH	Yes	Yes		Yes		Yes	Yes	Yes	Yes		
NJ	No										
NM	No										
NY	No										
NC	No										
NS	?						Yes		Yes		
OH	No										
ON	No										
OR	No										
PA	Yes	Yes					Yes				
RI	No	Yes			Yes	Yes	Yes	Yes	Yes		
SK	No	Yes	Yes				Yes				
SC	Yes	Yes	Yes		Yes	Yes	Yes	Yes	Yes		
SD		Yes		Yes	Yes	Yes	Yes	Yes	Yes	Yes	
TX	No										
UT	?							Yes			
VT				Yes	Yes	Yes	Yes				
DC	No										
DC	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	
WT	No		Yes				Yes				

	Test Assessment Method	Respondent
a:	Performance Review of Concrete with specific aggregates	CDOT, New Mexico
b:	Visual Inspection	CDOT, Indiana. Michigan, Nova Scotia, Nebraska, Texas, Sasketchewan, North Carolina, New Jersey,New Mexico, Mississippi
c:	Petrographic Examination C-856	Ontario, Oregon, South Dakota, Idaho, New Hampshire, Pennsylvania, New York, New Mexico
d:	Live Load Testing w two 100 ton vehicles	Ontario
e:	Strength Testing	Ontario
f:	UV Light Kit	Maine, South Dakota, Maryland, New Hampshire, New Mexico
g:	Mapping of Cracks	Alabama
h:	Obtaining Cores from Affected area	Alabama, South Dakota, Nova Scotia
i:	Refractive Index	Wisconson
j:	Chemical	Nebraska

		Test Designation	Respondent Using Method
a:	Mortar Bar Test	C-227	CDOT
b:	Petrographic Examination of Aggregates	C-295	CDOT
c:	Chemical Method	C-289	CDOT, Oregon, Ontario
d:	Petrographic Examination of Concrete	C-856	Oregon, Utah, Nova Scotia, Maryland, Washington DC, Pennsylvania, Texas, Sasketchewan, North Carolina, New Mexico
e:	Aggregates Tested	C-1260/T303	South Dakota, New Hampshire
f:	Aggregates Tested	C-227	South Dakota
g:	Aggregates Tested	C-289	South Dakota
h:	UV Test Method		Indiana, Michigan, Nova Scotia, Maryland, Nebraska, North Carolina, New Jersey, New York, New Mexico
i:	Expansion Testing		Idaho
j:	Fluorescence Image Analysis		North Carolina
k:	Chemical Analysis		North Carolina
l:	Residual Expansion		North Carolina
m:	Dynamic Modulus		North Carolina
n:	Compressive Strength		North Carolina
o:	Splitting Tensile Strength		North Carolina
p:	X-Ray Defraction		North Carolina

Contact Names for Localized ASR Distress			
Name	State or Province	Region or District	Phone Number
Gerald Peterson	CDOT	1	303/757-9134
Kenneth L. Wood	CDOT	4	970/350-2131
Michael Redmond	Maine	Statewide	207/287-2262
Keith Johnston	Oregon	Central Lab	503/986-3053
Sergio Rodriguez	Alabama	Central Lab	334/206-2410
Gary Pyke	Nova Scotia	Central Lab	902/860-2999
A.F. Stanley	Idaho	Central Lab	208/334-8443
Richard M. Lane	New Hampshire	Central Lab	603/271-3151
Herve Bachelu	Sasketchewan	Central Lab	306/787-4830
Mrinmay "Moy" Biswas	North Carolina	Central Lab	919/715-2465
Bryce Simons	New Mexico	Central Lab	505/827-5191

	Detection Technique Used	Respondent Answering
a:	Visual Observation and Inspection	CDOT, Ontario, Mississippi, Oregon, Alabama, Utah, Indiana, Nebraska, Texas, Saskatchewan, North Carolina, New Jersey, New Mexico
b:	UV Light	Maine, Alabama, Vermont, South Dakota Indiana, Michigan, Idaho, New Hampshire, Pennsylvania, Nebraska, North Carolina, New Jersey, New Mexico
c:	Petrographic C-856	Maine, Alabama, Idaho, Pennsylvania, New Mexico
d:	Copper Sulfate	South Dakota
e:	CSA Test Procedures	Nova Scotia
f:	Education of Personnel	Maryland

	Remediation Techniques Used	Limits	Respondent
a:	Low Alkalie Cement	<0.6 Alkalies	CDOT, Mississippi, Oregon, Alabama, Kansas. South Dakota, Indiana, South Carolina, Pennsylvania, Texas, Georgia, , New York (< 0.7%), Rhode Island, Maine,Nova Scotia, Washington DC,Sasketchewan, North Carolina
b:	Mandatory 20% Fly Ash	Class F	CDOT
c:	AC Overlay	Repair	CDOT
d:	Use of Non-Reactive Aggregate		Ontario, Oregon
e:	General use of fly ash, silica fume, ggbfs, slag cement		Rhode Island, Oregon, Alabama, Indiana, Michigan, Nova Scotia, Pennsylvania, Texas, Georgia, North Carolina, New Jersey, Rhode Island, Maine, Maryland
f:	Aggregate Evaluation by C-1260/T 303		Maine, Texas, Georgia
g:	Type II Cement		AZ, Alaska
h:	Allows use of Class F Ash up to 20%		AZ, Utah, South Dakota, Idaho
i:	T 303		Washington
j:	Wetting Drying Test (Attached)		Kansas
k:	Use of 30%Limestone, dolomite or approved gravel or 25% chat to mitigate ASR		Kansas
l:	Type IP Cement		Kansas, Pennsylvania, Nebraska, Texas
m:	Type V Cement		South Dakota
n:	Lithium Treatment		South Dakota
o:	Aggregate Evaluation by C-1293		Texas
p:	Silica Fume		Sasketchewan
q:	Aggregate Evaluation by C-295	% Chert	New York
r:	Aggregate Evaluation by C-227		New York
s:	flyash modified high alkali cements		New York
t:	Required Use of at least 20% Class F Ash		New Mexico

ASTM/AASHTO STANDARDS

ASTM C1260/AASHTO T303 – Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method).

ASTM C227 – Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method).

ASTM C1293 – Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction.

ASTM C1567 – Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method).

ASTM C231/AASHTO T152 – Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method.

ASTM C618/AASHTO M295 – Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete.

ASTM C403/AASHTO T197 – Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance.

ASTM C 1602 – Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete.

ASTM C 29/AASHTO T19 – Standard Test Method for Bulk Density ("Unit Weight") and Voids in Aggregate.

ASTM C136/AASHTO T27 – Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates.

ASTM C33/AASHTO M6/M43/M80 – Standard Specification for Concrete Aggregates.

ASTM C150 – Standard Specification for Portland Cement.

ASTM C595/ASTM C1157 – Standard Performance Specification for Hydraulic Cement.

ASTM C88 – Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate.

ASTM C441 – Standard Test Method for Effectiveness of Pozzolans or Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction.

ASTM C289 – Standard Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method).

ASTM C295 – Standard Guide for Petro graphic Examination of Aggregates for Concrete.

AASHTO M307 – Standard Specification for Silica Fume Used in Cementitious Mixtures.

AASHTO T299 – Standard Method of Test for Rapid Identification of Alkali-Silica Reaction Products in Concrete.

ACI 301 – Specifications for Structural Concrete for Buildings.

ACI 318 – Building Code Requirements for Structural Concrete and Commentary.