Selecting Measures to Prevent Deleterious Alkali-Silica Reaction in Concrete

<u>Rationale for the AASHTO PP65 Prescriptive Approach</u>

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Notice

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16. Abstract

PP65-11 provides two approaches for selecting preventive measures: (i) a performance approach based on laboratory testing, and (ii) a prescriptive approach based on a consideration of the reactivity of the aggregate, type and size of structure, exposure conditions, and the composition of cementitious materials being used. The options for preventive measures included in the prescriptive approach of PP65-11 are to (i) control the alkali content of the concrete to a maximum allowable level, (ii) use a minimum level of supplementary cementitious material (SCM) or combination of SCMs, or (iii) use a combination of these two options .This document is intended to provide the background information that was used in the development of the prescriptive approach.

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SI* (MODERN METRIC) CONVERSION FACTORS

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)		
^{o}F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^{o}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)		
^{o}C	Celsius	1.8C+32	Fahrenheit	^{o}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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1.0 Introduction

In 2011, AASHTO published PP65-11 “Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction.” As the title suggests, these guidelines can be broken down into two steps as follows:

- Evaluating aggregate reactivity (for alkali-silica and alkali-carbonate reactive aggregates)
- Selecting preventive measures (for alkali-silica reactive aggregates only)

Aggregate reactivity is evaluated by considering one or more of the following options: (i) field performance history, (ii) petrographic assessment, (iii) chemical composition (for quarried carbonates), (iv) data from accelerated mortar bar tests (AASHTO T 303[1]), and (v) data from concrete prism tests (ASTM C 1293). A detailed description and explanation of the approach used in PP65-11 for evaluating aggregates is provided in FHWA-HIF-09-001 (Thomas et al. 2008).

PP65-11 provides two approaches for selecting preventive measures: (i) a performance approach based on laboratory testing, and (ii) a prescriptive approach based on a consideration of the reactivity of the aggregate, type and size of structure, exposure conditions, and the composition of cementitious materials being used. The two test methods used for the performance-based approach are the accelerated mortar bar test for evaluating combinations of cementitious materials and aggregates (ASTM C 1567) and the concrete prism test (ASTM C 1293), with preference given to the latter test. The rationale behind the use of these two tests and the preference for the concrete prism test is presented in FHWA-HIF-09-001 (Thomas et al. 2008) and in Thomas et al. (2006).

The options for preventive measures included in the prescriptive approach of PP65-11 are to (i) control the alkali content of the concrete to a maximum allowable level, (ii) use a minimum level of supplementary cementitious material (SCM) or combination of SCMs, or (iii) use a combination of these two options (that is controlling the alkali content of the concrete and using SCM). The precise level of alkali permitted or SCM required depends on a number of factors including: the aggregate reactivity, type and size of structure, exposure conditions, and the composition of the cement and SCM being used. Although this approach is described in FHWA-HIF-09-001 (Thomas et al. 2008) no information is given to explain the rationale behind the selection of the criteria used to establish the risk of ASR or the levels of prevention required beyond stating that the approach is similar to that developed in Canada (CSA A23.2-27A) and in Europe (RILEM TC191-ARP: AAR-7).

This document is intended to provide the background information that was used in the development of the prescriptive approach.

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[1] AASHTO T 303 is equivalent to ASTM C 1260.

2.0 Outline of the AASHTO PP65 Prescriptive Approach

The prescriptive approach of AASHTO PP65 can be summarized in the following steps:

Step 1. Determine aggregate reactivity class: The aggregate is tested in either the accelerated mortar bar test (AMBT), AASHTO T 303, or, preferably, the concrete prism test (CPT), ASTM C 1293. The criteria in Table 1 are used to classify the aggregate reactivity, which can range from "R0 – non-reactive" through to "R3 – very highly reactive."

Table 1. Classification of aggregate reactivity.

Aggregate-Reactivity Class	Description of Aggregate Reactivity	One-Year Expansion in CPT (%)	14-Day Expansion in AMBT (%)
R0	Non-reactive	≤ 0.04	≤ 0.10
R1	Moderately reactive	> 0.04, ≤ 0.12	> 0.10, ≤ 0.30
R2	Highly reactive	> 0.12, ≤ 0.24	> 0.30, ≤ 0.45
R3	Very highly reactive	> 0.24	> 0.45

Step 2. Determine level of ASR risk: Based on the aggregate-reactivity class determined in Step 1 (Table 1) and the size and exposure conditions of the concrete under construction, the level of ASR risk is determined using the criteria in Table 2. The risk may range from Level 1 (lowest or negligible risk) through to Level 6 (highest risk).

Table 2. Determining the level of ASR risk.

	Aggregate-Reactivity Class			
Size and exposure conditions	**R0**	**R1**	**R2**	**R3**
Non-massive[2] concrete in a dry[3] environment	Level 1	Level 1	Level 2	Level 3
Massive[2] elements in a dry[3] environment	Level 1	Level 2	Level 3	Level 4
All concrete exposed to humid air, buried or immersed	Level 1	Level 3	Level 4	Level 5
All concrete exposed to alkalis in service[4]	Level 1	Level 4	Level 5	Level 6

Step 3. Determine level of prevention: Based on the level of ASR risk determined in Step 2 (Table 2) and the classification of the structure[5], the level of prevention required is determined using Table 3. The level of prevention required may range from Level V (no measures necessary) to Level ZZ (extreme preventive measures necessary).

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[2] A massive element has a least dimension > 3 ft (0.9 m).

[3] A dry environment corresponds to an average ambient relative humidity lower than 60%, normally only found in buildings.

[4] Examples of structures exposed to alkalis (sodium and potassium) in service include marine structures exposed to seawater and highway structures exposed to deicing salts (e.g. NaCl) or anti-icing salts (e.g. potassium acetate, potassium formate, sodium acetate, sodium formate, etc.).

[5] Table 4 is intended to provide guidance in selecting the Class of Structure. See further discussion in Section 5.0.

Table 3. Determining the level of prevention.

	Classification of Structure (Table 4)			
Level of ASR Risk (Table 2)	**S1**	**S2**	**S3**	**S4**
Risk Level 1	V	V	V	V
Risk Level 2	V	V	W	X
Risk Level 3	V	W	X	Y
Risk Level 4	W	X	Y	Z
Risk Level 5	X	Y	Z	ZZ
Risk Level 6	Y	Z	ZZ	[††]

[††] It is not permitted to construct a Class S4 structure (see Table 4) when the risk of ASR is Level 6. Measures must be taken to reduce the level of risk in these circumstances.

The levels of prevention V, W, X, Y, Z, and ZZ are used in Tables 5 to 8.

Table 4. Structures classified on the basis of the severity of the consequences should ASR[6] occur (modified for highway structures from RILEM TC 191-ARP).

Class	Consequences of ASR	Acceptability of ASR	Examples[7]
S1	Safety, economic, or environmental consequences small or negligible	Some deterioration from ASR may be tolerated	▪ Non-load-bearing elements inside buildings ▪ Temporary structures (e.g. < 5 years)
S2	Some safety, economic, or environmental consequences if major deterioration	Moderate risk of ASR is acceptable	▪ Sidewalks, curbs, and gutters ▪ Service life < 40 years
S3	Significant safety, economic, or environmental consequences if minor damage	Minor risk of ASR acceptable	▪ Pavements ▪ Culverts ▪ Highway barriers ▪ Rural, low-volume bridges ▪ Large numbers of precast elements where economic costs of replacement are severe ▪ Service life normally 40 to 75 years
S4	Serious safety, economic, or environmental consequences if minor damage	ASR cannot be tolerated	▪ Major bridges ▪ Tunnels ▪ Critical elements that are very difficult to inspect or repair ▪ Service life normally > 75 years

Step 4. Identification of preventive measures: Based on the level of prevention required that was determined in Step 3 (Table 3), a number of prescriptive options are presented as acceptable measures for preventing ASR; including:

Option 1 – limiting the alkali content of the concrete (Table 5)

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[6] This table does not consider the consequences of damage due to ACR. This practice does not permit the use of alkali-carbonate reactive aggregates.

[7] The types of structures listed under each Class are meant to serve as examples. Some owners may decide to use their own classification system. For example, sidewalks or curbs and gutters may be placed in the Class S3.

Table 5. Maximum alkali contents in portland cement concrete to provide various levels of prevention.

Prevention Level	Maximum Alkali Content of Concrete (Na_2Oe)	
	lb/yd^3	kg/m^3
V	No limit	
W	5.0	3.0
X	4.0	2.4
Y	3.0	1.8
Z[8]	Table 8	
ZZ[8]		

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[8] SCMs must be used in prevention levels Z and ZZ.

Option 2 – using supplementary cementing materials, SCM[9] (Table 6)

Table 6. Minimum levels of SCM to provide various levels of prevention.

Type of SCM[10]	**Alkali level of SCM (% Na_2Oe)**	**Minimum Replacement Level[11] (% by mass of cementitious material)**				
		Level W	**Level X**	**Level Y**	**Level Z**	**Level ZZ**
Fly ash ($CaO \leq 18\%$)	**≤ 3.0**	15	20	25	35	Table 7
	> 3.0, ≤ 4.5	20	25	30	40	
Slag	**≤ 1.0**	25	35	50	65	
Silica Fume[12] ($SiO_2 \geq 85\%$)	**≤ 1.0**	1.2 x LBA or 2.0 x KGA	1.5 x LBA or 2.5 x KGA	1.8 x LBA or 3.0 x KGA	2.4 x LBA or 4.0 x KGA	

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[9] For Option 2, the minimum amount of SCM determined from Table 6 may be adjusted based on the alkali level of the portland cement using Table 7.

[10] The SCM may be added directly to the concrete mixer or it may be a component of a blended cement. SCMs should meet the requirements of AASHTO M 295, M 302 or M 307. Blended cements should meet the requirements of AASHTO M 240 or ASTM C 1157.

[11] The use of high levels of SCM in concrete may increase the risk of problems due to deicer salt scaling if the concrete is not properly proportioned, finished, and cured.

[12] The minimum level of silica fume (as a percentage of cementitious material) is calculated on the basis of the alkali (Na_2Oe) content of the concrete contributed by the portland cement and expressed in either units of lb/yd^3 (LBA in Table 6) or kg/m^3 (KGA in Table 6). LBA is calculated by multiplying the cement content of the concrete in lb/yd^3 by the alkali content of the cement divided by 100. For example, for a concrete containing 500 lb/yd^3 of cement with an alkali content of 0.81% Na_2Oe, the value of LBA = 500 x 0.81/100 = 4.05 lb/yd^3. For this concrete, the minimum replacement level of silica fume for Level Y is 1.8 x 4.05 = 8.1%. KGA is calculated by multiplying the cement content of the concrete in kg/m^3 by the alkali content of the cement divided by 100. For example, for a concrete containing 300 kg/m^3 of cement with an alkali content of 0.91% Na_2Oe, the value of KGA = 300 x 0.91/100 = 2.73 kg/m^3. For this concrete, the minimum replacement level of silica fume for Level X is 2.5 x 2.73 = 6.8%. Regardless of the calculated value, the minimum level of silica fume shall not be less than 7% when it is the only method of prevention.

Table 7. Adjusting the minimum level of SCM based on the alkali content in the portland cement.

Cement Alkalis (% Na_2Oe)	**Level of SCM**
≤ 0.70	Reduce the minimum amount of SCM given in Table 6 by one prevention level[13]
> 0.70, ≤ 1.00	Use the minimum levels of SCM given in Table 6
> 1.00, ≤1.25	Increase the minimum amount of SCM given in Table 6 by one prevention level
> 1.25	No guidance is given

Option 3 – limiting the alkali content of the concrete and using SCM (Table 8)

Table 8. Using SCM and limiting the alkali content of the concrete to provide exceptional levels of prevention.

Prevention Level	**SCM as Sole Prevention**	**Limiting Concrete Alkali Content Plus SCM**	
	Minimum SCM Level	**Maximum Alkali Content, lb/yd^3 (kg/m^3)**	**Minimum SCM Level**
Z	SCM level shown for Level Z in Table 6	3.0 (1.8)	SCM level shown for Level Y in Table 6
ZZ	Not permitted	3.0 (1.8)	SCM level shown for Level Z in Table 6

The prescriptive approach does not allow the option for using lithium compounds as a preventive measure. Research has shown that the efficacy of lithium compounds in controlling expansion due to ASR is highly influenced by the nature of the reactive aggregate (Tremblay et al. 2007). Currently, it is not possible to prescribe the required lithium dose based on aggregate reactivity or mineralogy and, consequently, lithium compounds must be tested using the performance approach to determine the minimum dose required with a specific aggregate.

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[13] The replacement levels should not be below those given in Table 6 for prevention level W, regardless of the alkali content of the portland cement.

3.0 Assumptions Made in the Development of the Prescriptive Approach

A number of assumptions were made in developing the overall framework of the prescriptive approach used in AASHTO PP65 including:

- The level of ASR risk increases as the reactivity of the aggregate, the size of the structure, or the availability of moisture in service increases and if the structure is exposed to alkalis during service.
- Some concrete structures can tolerate a higher level of risk than other structures (i.e., a concrete sidewalk), either because the consequences of ASR are less severe or because the required service life is less (or both). In such cases, a lower level of prevention may be warranted.
- Deleterious ASR can be prevented solely by limiting the alkali content of the concrete.
- The maximum alkali content that can be tolerated without the occurrence of deleterious ASR will depend mainly on the reactivity of the aggregate and whether the concrete is exposed to alkali compounds in service.
- Deleterious ASR can be prevented by using supplementary cementing material (SCM), or combination of SCMs.
- The amount of SCM required to prevent deleterious ASR depends upon a number of factors including (but not limited to) the following:
    - The reactivity of the aggregate
    - The composition of the SCM (particularly CaO, $SiO_{2,}$ and Na_2Oe)
    - The alkali provided to the system by the portland cement
    - The presence of alkali compounds in service

The justification for these assumptions and evidence supporting the selection of limits for maximum alkali contents and minimum SCM replacement levels is presented in the following sections.

4.0 Risk of ASR

In the prescriptive approach, the risk of ASR is determined by the degree of aggregate reactivity, the exposure conditions of the structure, and the size of the element.

4.1 Effect of Aggregate Reactivity

Clearly, for a given set of cementitious materials, mixture proportions, and exposure conditions, the risk of deleterious reaction would be expected to increase as the reactivity of the aggregate increases. Intuitively, it is reasonable to anticipate that greater preventive measures are required to suppress expansion as the aggregate reactivity increases. Indeed, the main reason for classifying aggregate reactivity in the context of the prescriptive approach is because it has been generally observed that aggregates of higher reactivity require greater preventive measures; this is demonstrated in the sections dealing with preventive measures below. The question that remains is how to measure "reactivity."

CSA A23.1 (Appendix B) divides ASR into two categories based on the nature of reactive silica and the observed timeframe before damage is manifested; these categories are:

- Reaction with poorly crystalline or amorphous silica minerals such as opal, cristobalite, and volcanic and artificial glasses. These minerals react rapidly and may cause damaging reaction in a few years when present in amounts as little as 1%.
- Reaction with varieties of quartz such as cryptocrystalline, microcrystalline, or strained quartz. These minerals react more slowly, take longer to produce damage, and are generally required to be present in greater quantities than poorly crystalline and amorphous forms of silica.

However, it is difficult to classify aggregate reactivity based solely on mineralogical observations as aggregates may contain various types of reactive minerals in different quantities, and the extent to which reactive minerals cause damage in concrete depends on other factors, such as particle size, texture, porosity of the reactive rock types, etc.

In AASHTO PP65, the reactivity of aggregates is classified on the basis of either the 1-year expansion in the concrete prism test (ASTM C 1293) or the 14-day expansion in the accelerated mortar bar test (AASHTO T 303, ASTM C 1260) (see Table 1). The guidelines published by the Federal Highway Administration, report FHWA-HIF-09-001 (Thomas et al. 2008), which was the precursor to PP65, recommended that only the concrete prism test be used to classify the reactivity of aggregates, as it was generally considered that the accelerated test was too aggressive and insensitive to differences in aggregate reactivity. This was in agreement with the 2004 edition of CSA A23.2-27A.

For the concrete prism test, an expansion value of 0.04% at one year was selected to separate non-reactive from reactive aggregates. This is consistent with other specifications such as CSA A23.2-27A (which uses 0.040% as the limit) and with the recommendations in Appendix X1 of ASTM C 33 "Standard Specification for Concrete Aggregates." It is believed that the expansion limit of 0.04% has become the "standard" criterion used, as cracking is often observed to initiate at this level of expansion. In the PP65 prescriptive approach, the risk of ASR is considered to be negligible if the aggregate is classified as non-reactive and no preventive measures are required.

An expansion value of 0.12% was selected to delineate between moderately-reactive and highly-reactive aggregate to be consistent with CSA A23.2-27A. This value was chosen by CSA as it separates the expansion behavior of two aggregates, the Spratt limestone and the Sudbury gravel (Figure 1)[14], that have been widely used by researchers worldwide, and results from tests on these aggregates form much of the database that was used to develop the CSA standard practice.

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[14] The results of a round-robin study involving 27 laboratories from Canada, the USA and France showed average 1-year expansion values of 0.093% (standard deviation of 0.037) for Sudbury and 0.176 % (standard deviation of 0.045) for Spratt aggregate.

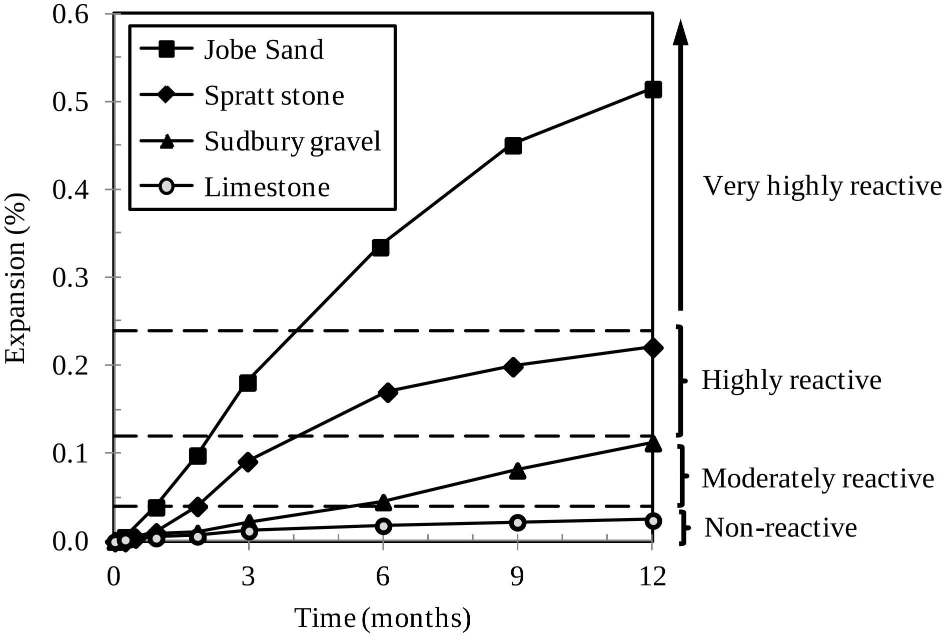

Figure 1. Expansion of various aggregates tested in accordance with CPT (ASTM C 1293).

At the time of writing FHWA-HIF-09-001 (Thomas et al. 2008), the Canadian Standard only had three classes of aggregate reactivity, with “highly reactive” being the highest. However, a number of studies in North America (Folliard et al. 2006; Touma et al. 2001) have indicated that there are many aggregates that produce significantly greater expansion than the Spratt aggregate and thus present a greater risk of ASR (and require a greater level of prevention). Consequently, a fourth category of “very highly reactive” aggregate was included in FHWA-HIF-09-001 and AASHTO PP65 to represent aggregates that produce 1-year expansions in excess of 0.24%. CSA modified A23.2-27A in 2009 to include a fourth category, “extremely reactive” aggregate, but an expansion value in excess of 0.230% was used to define these aggregates.

Figure 1 shows example expansion curves for the different aggregate reactivity classes and Figure 2 shows the expansion values for approximately 80 aggregate samples.

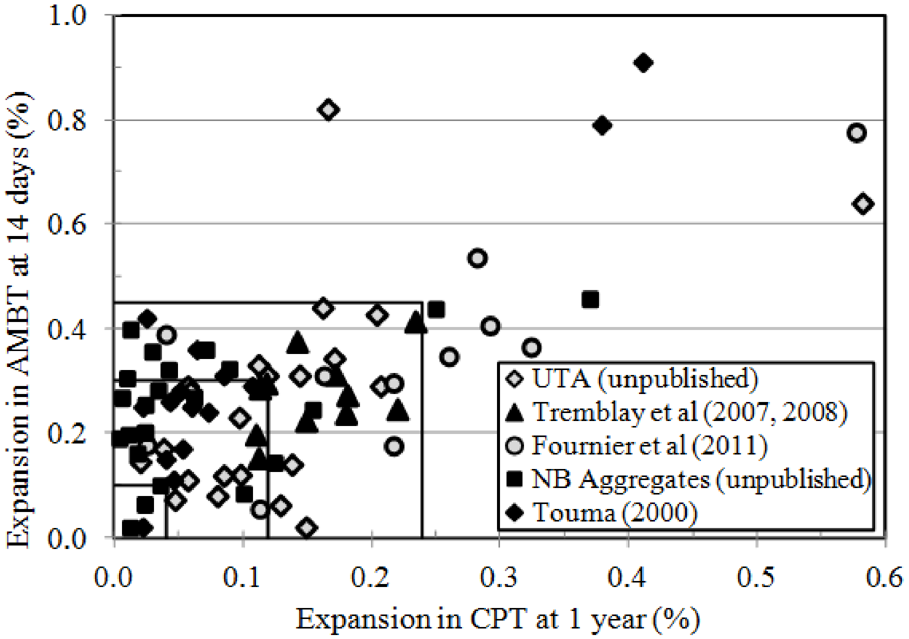

Figure 2. Relationship between 14-day expansion in AMBT (AASHTO T 303) and 1-year expansion in CPT (ASTM C 1293) for a wide range of aggregates.

Although FHWA-HIF-09-001 (Thomas et al. 2008) recommends that aggregate reactivity only be classified using the concrete prism test, AASHTO PP65 also includes criteria based on the accelerated test (see Table 1) because it was considered that in many cases in practice, only data from the accelerated test may be available. The criteria used to delineate between aggregate classes, based on the accelerated test, were based on a comparison of data from this test with data from the concrete prism test for the same aggregate (see Figure 2), although it is recognized that there is a poor correlation between the two tests.

It should be noted that the magnitude of expansion is not the only means for classifying aggregate reactivity. For example, in the U.K., guidelines for preventing ASR (Building Research Establishment 1997) infer aggregate reactivity based on the threshold alkali content in concrete that is required to initiate deleterious expansion with that particular aggregate. For the purpose of selecting preventive measures, this concept has more technical merit than using the magnitude of expansion but, given the wide range of aggregate types available in North America and the complexity of establishing the alkali threshold (not to mention the lack of an established test for determining the threshold), it is impractical to adopt this approach at this time.

4.2 Effect of Exposure Conditions and Size of Element

Alkali-silica reaction can only proceed if there is a sufficient level of moisture present in the concrete. Internal moisture is required to transport the alkali ions to the location of the reactive silica, to enable the dissolution of silica and formation of alkali-silica gel, and to sustain the swelling of the gel (which occurs due to the imbibition of water). Laboratory testing (carried out on concrete prisms) has shown (see Figure 3) that the threshold relative humidity for ASR expansion can vary from one aggregate to another, and that ASR expansion ceases or is largely reduced if the relative humidity surrounding the concrete specimens falls below 70 to 80% (Pedneault 1996).

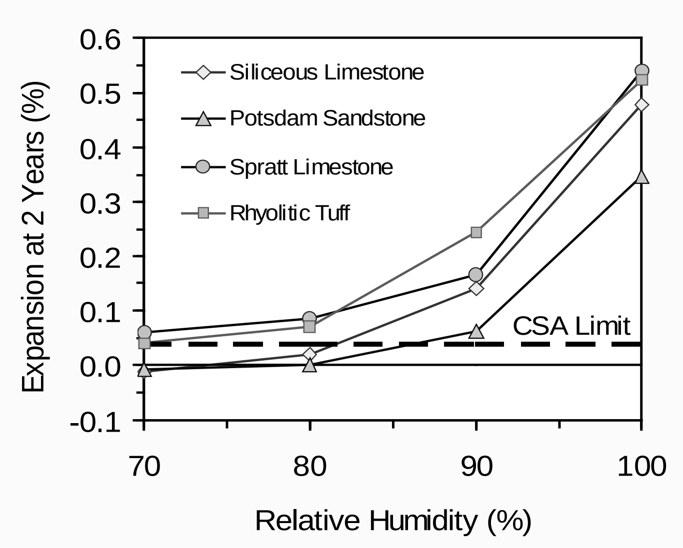

Figure 3. Effect of relative humidity on expansion of concrete prisms (Pedneault 1996).

In non-massive concrete elements [least dimension ≤ 0.9 m, (3 ft)] exposed to a dry environment (e.g. internal components of a building), the internal relative humidity will decrease with time and eventually fall below that necessary to sustain ASR before significant expansion can occur, unless a highly reactive aggregate is used. Consequently, the level of ASR risk is considered to be negligible (Level 1) if non-reactive or moderately-reactive aggregate (R0 or R1) are used, but the level increases with increased aggregate reactivity.

Massive concrete elements [least dimension > 0.9 m, (3 ft)] exposed to a dry environment are likely to retain sufficient moisture to sustain ASR for an extended period of time and, consequently, there is a risk of ASR if the aggregate is reactive, the risk increasing with the reactivity of the aggregate.

For concrete exposed to moisture during service, the internal humidity will likely remain sufficiently high to promote ASR regardless of the size of the element. In this type of exposure, there is a risk of ASR with all reactive aggregates, although the risk still increases with aggregate reactivity. It should be noted that the rate at which ASR proceeds will be a function of the degree of moisture exposure, and elements directly exposed to water will deteriorate more rapidly than sheltered elements exposed only to humid air (e.g. girders under a bridge deck). This is not taken into account in the prescriptive approach.

In AASHTO PP65, concrete elements that are exposed to alkalis in service (e.g. seawater or deicing salts containing sodium or potassium) are considered to be at a greater risk of ASR. The role of external alkalis is equivocal. For example, the effect of sodium chloride (NaCl) on expansion due to ASR is influenced by a number of factors including: the concentration of the salt, the composition of the cement (particularly C_3A and alkali), the mineralogy of the aggregate, and the permeability of the concrete (Sibbick and Page 1996). Alkali salts of formic and acetic acid (e.g. potassium acetate, sodium acetate, potassium formate, and sodium formate), now commonly used as anti-icing and deicing chemicals on airfield pavements, have been found to exacerbate ASR damage in laboratory tests on concrete containing reactive aggregates (Rangaraju and Olek 2007; Rangaraju 2007; Rangaraju et al. 2007) but the extent to which damage occurs in the field, where the depth of penetration is limited, has not been established. In view of the potential for external sources of alkali to aggravate ASR, it was considered prudent to increase the risk of ASR by one level for concretes exposed to alkalis in service compared with concretes exposed to moisture only. The increased level of risk only applies to concrete containing reactive aggregates (R1, R2, or R3).

Noticeably absent from the exposure conditions considered in AASHTO PP65 is the role of ambient temperature. It is well established that ASR, like most chemical reactions, is accelerated by an increase in temperature. Thus, for concrete of similar composition, the rate of reaction and the time to cracking will be faster in warm compared to cold climates. Fournier et al. (2009) compared the expansions in boosted concrete prisms and unboosted exposure blocks, which better correspond to the reality of concrete structures. They found that the failure ratio (defined as the time for blocks on outdoor exposure sites to reach an expansion of 0.040% divided by the time for prisms from the same mix to reach the same level of expansion under laboratory conditions) for concretes under cooler [e.g. Ottawa (Ontario, Canada) exposure site] and warmer [e.g. Austin (Texas, USA) exposure site] climatic conditions generally ranged between 10 to 16 and 2 to 6, respectively. This suggests that, on average, and although wide variations could be observed from one aggregate to another, if all conditions conducive to ASR are maintained, a structure incorporating a certain reactive aggregate could develop noticeable expansion and cracking in the field about 4 to 5 times faster under “warmer” conditions (such as those prevailing in Texas) compared to those found in “cooler” parts of the world (like those where colder winter and prolonged freezing conditions are prevailing). It can be argued that temperature does not influence whether ASR occurs or not (unless the concrete remains permanently frozen), just when it occurs; in other words, the “risk of ASR” is not affected by temperature. However, a counterargument would be that temperature does have an impact on the risk of ASR occurring within the service life of a structure. For example, neglecting the role of other deterioration mechanisms such as freeze-thaw, a concrete structure constructed with a reactive aggregate is more likely to suffer ASR damage during a 25-year service life in a warm environment than a similar structure constructed with the same materials in a cold climate. It is possible that future versions of AASHTO PP65 will take the temperature of the environment into consideration.

5.0 Classification of Structure

The approach to classifying structures on the basis of “the consequences should ASR occur” (see Table 4) was adopted from that used in RILEM TC 191-ARP Alkali-Reactivity and Prevention – Assessment, Specification, and Diagnosis of Alkali-Reactivity (Nixon et al. 2004). The approach presumes that a higher level of risk of ASR can be accepted in some types of structures. This approach provides a greater level of flexibility for the owner when selecting preventive measures. For example, when constructing a major bridge with a 100-year service life, ASR cannot be tolerated because ASR will likely jeopardize the service life and lead to premature rehabilitation. In such a case, if a reactive aggregate is used, it will be necessary to use a high amount of SCM and possibly even control the alkali content of the concrete. This could impact the constructability and cost of construction, but it is necessary to ensure durability. If the same aggregate was used in a sidewalk, a reduced level of SCM with no control of the alkali content may produce an acceptable level of risk because the consequences of ASR are less severe and are likely not to impact the service life of the concrete element.

Although examples are given in Table 4 of the type of structures that might fall into the different classes, it is probable that owners will reclassify structural types based on their own experiences dealing with the consequences of ASR.

6.0 Preventive Measures

6.1 Limiting the Alkali Content of the Concrete

Stanton's (1940) formative work on ASR indicated that expansive reaction is unlikely to occur when the alkali content of the cement is below 0.60% Na_2Oe. However, later research indicated that damaging ASR could occur both in the laboratory and in the field (e.g. Woolf 1952; Stark 1980; Blaikie et al. 1996) when low-alkali cements are used. Despite this, the 0.60% value has become the accepted maximum limit for cement to be used with reactive aggregates in the United States, and appears in ASTM C 150 Standard Specification for Portland Cement as an optional limit when concrete contains deleteriously reactive aggregate. Appendix X1 of ASTM C 33 Standard Specification for Concrete Aggregates includes “low-alkali cement” (meeting the ASTM C 150 limit of 0.60% Na_2Oe) as a measure “known to prevent excessive expansion.”

Limiting the alkali level of the cement takes no account of the cement content of the concrete which, together with the cement alkali content, governs the total alkali content of concrete, and is considered to be a more accurate index of the risk of expansion when a reactive aggregate is used in concrete. Figure 4 shows the relationship between alkali content and expansion for concretes produced with a range of cement contents and cements of varying alkali content (Lindgård et al. 2012). The figure was produced using previously unpublished data from the Building Research Establishment. The relationship clearly shows that it is the product of cement content and cement alkali level (i.e., the alkali content of the concrete) that controls the alkali content rather than the cement alkali level alone.

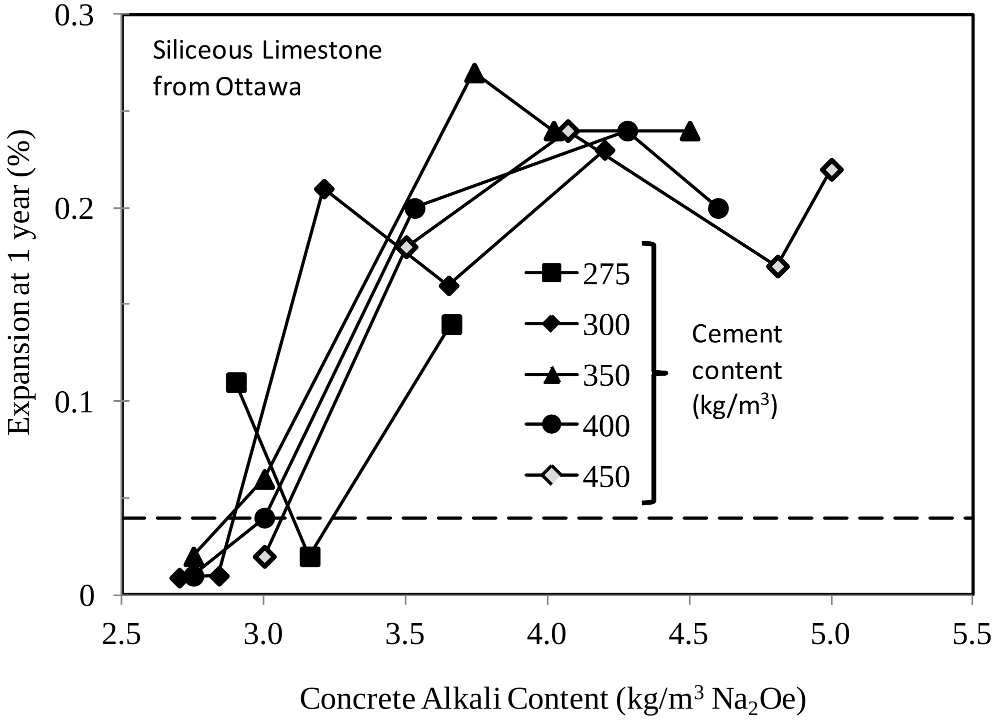

Figure 4. Expansion of concrete prisms as a function of alkali content for siliceous limestone (Spratt) aggregate (Lingaard et al. 2012).

Figure 5 shows expansion of concrete prisms plotted against the alkali content of the concrete for three different reactive aggregates (selected to demonstrate the range of behavior observed). It can be seen that the threshold alkali content required to initiate damaging expansion in the concrete prism test varies considerably between aggregates, with values ranging from approximately 3.0 kg/m^3 (5.0 lb/yd^3) Na_2Oe to more than 5.0 kg/m^3 (8.3 lb/yd^3) Na_2Oe for the aggregates shown in Figure 5. Figure 6 compares the 1-year concrete prism expansions (obtained in accordance with CSA A23.2-14A, equivalent to ASTM C 1293) and the 15-year exposure block expansions, using previously unpublished data from CANMET. The moderately and highly reactive aggregates in Figure 6 are classified in accordance with increasing 1-year expansion values in the Concrete Prism Test. The data show that, generally, the expansion increases with increasing total alkali content in the concrete; however, the use of the low-alkali cement/content in the exposure blocks was not equally effective for all reactive aggregates.

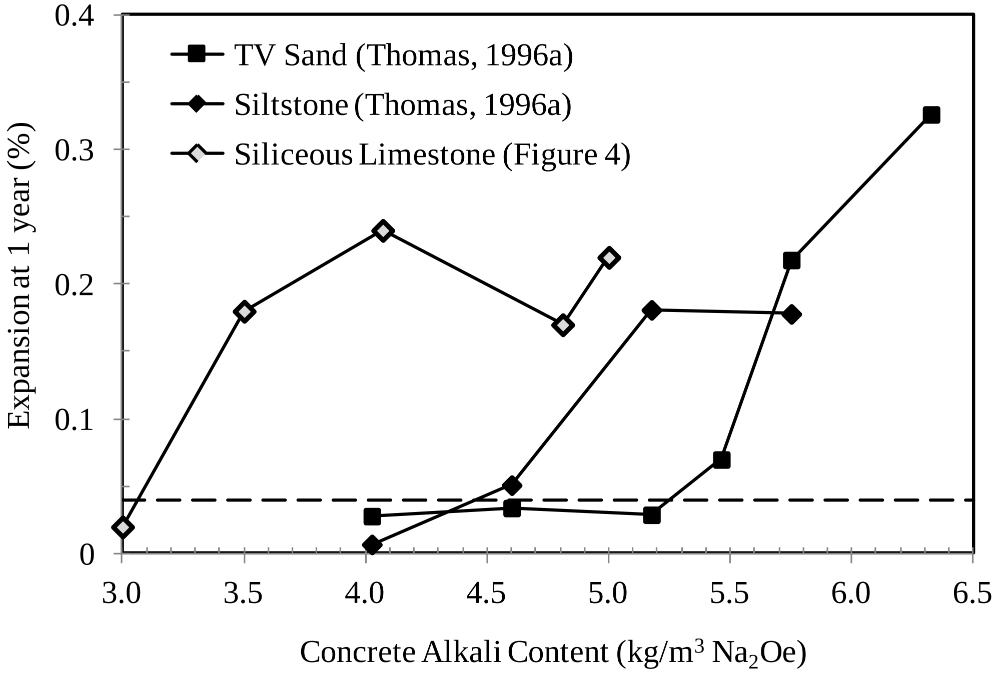

Figure 5. Effect of alkali content on expansion of concrete with different aggregates.

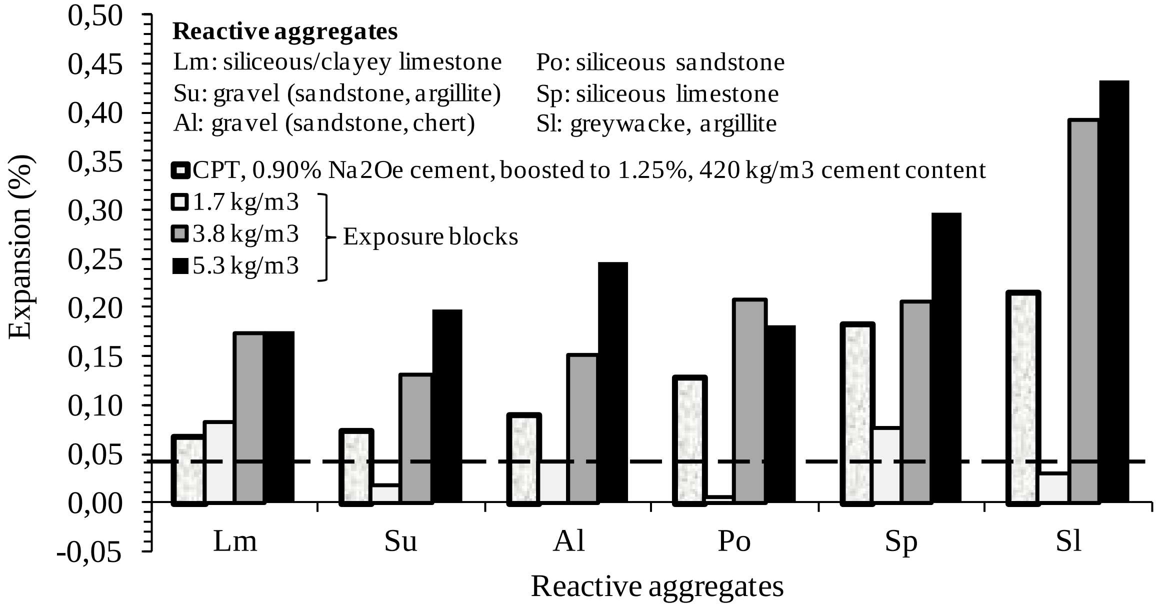

Figure 6. **Comparison of the one-year concrete prism (CPT) and the 15-year exposure block expansions for different moderately and highly reactive aggregates. The exposure blocks were made with a 420 kg/m^3 cement content and different "concrete" alkali contents obtained from the use of a low alkali cement (0.40% Na_2Oe;1.7 kg/m^3 alkali content), or a high-alkali cement (0.90% Na_2Oe) without (3.8 kg/m^3 alkali content) and with (5.3 kg/m^3 alkali content) added alkalis (NaOH).**

A number of specifications have employed a maximum concrete alkali content as an option to control expansion in concrete containing reactive aggregates. Nixon and Sims (1992) reported that maximum permissible alkali contents between 2.5 and 4.5 kg/m^3 (4.2 to 7.5 lb/yd^3) Na_2Oe have been specified by various countries and agencies, with the allowable alkali content sometimes varying depending on aggregate reactivity.

Unfortunately, traditional concrete prism tests tend to underestimate the threshold alkali content for aggregates as a result of alkali being leached from the concrete during exposure. This phenomenon has been discussed in the literature (Thomas et al. 2006; Lindgård et al. 2012) and a further example is shown in Figure 7. The figure shows the expansion of concrete blocks (0.38 x 0.38 x 0.71 m, 15 x 15 x 28 in.) containing reactive (Jobe) sand and various levels of alkali, and stored on an outdoor exposure site at the University of Texas in Austin, compared with the expansion of concrete prisms from the same mix stored over water (ASTM C 1293). It is evident that the concrete prism test will yield a higher threshold alkali content than may be observed for larger elements stored under field conditions.

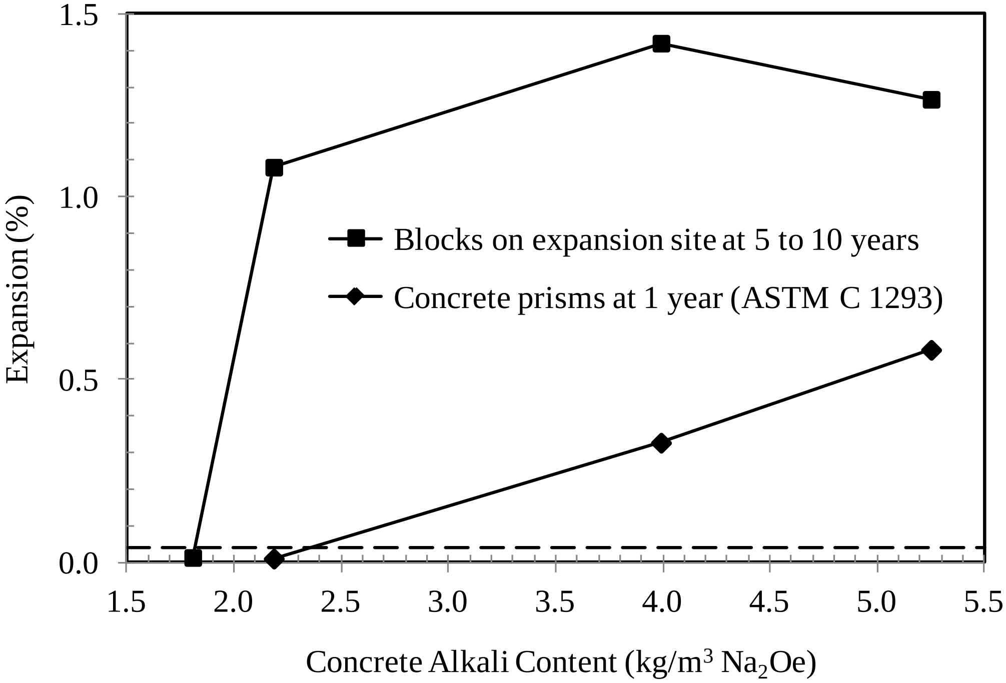

Figure 7. Effect of alkali content on the expansion of blocks stored outdoors and prisms stored over water (unpublished data from the University of Texas at Austin).

In 2000, the alkali limit used as a preventive measure in CSA A23.2-27A was modified from the single value of 3.0 kg/m^3 (5 lb/yd^3) Na_2Oe specified in the 1994 edition to a sliding scale from 1.8 to 3.0 kg/m^3 (3 to 5 lb/yd^3) Na_2Oe as shown in Table 9.

Table 9. Alkali limit in CSA A23.2-27A.

Prevention Level	Maximum Alkali Content kg/m^3 (lb/yd^3) Na_2Oe
V	No limit
W	3.0 (5.0)
X	2.4 (4.0)
Y	1.8 (3.0)
Z	1.8 (3.0) plus SCM

As the data available from laboratory tests at the time were considered unreliable for the purpose of establishing threshold alkali values, the maximum alkali content limits were based on published data from structures and field experience. Thomas (1996) had previously reported damaging ASR in concrete dams in the U.K. and Canada where the estimated alkali content was in the range of 2.0 to 2.4 kg/m^3 (3.3 to 4.0 lb/yd^3) Na_2Oe. Rogers et al. (2000) reported results from a study of 8-year-old concrete blocks (0.6 x 0.6 x 2 m, 2 x 2 x 6.6 ft) stored on an exposure site in Ontario, Canada. Specimens produced with high-alkali cement showed very significant expansion and cracking after eight years. Specimens with low-alkali cement (0.46% Na_2Oe) and a calculated alkali content of 1.91 kg/m^3 (3.18 lb/yd^3) Na_2Oe showed significantly less expansion; however, the expansion did exceed 0.04% at eight years and small cracks were evident. Subsequent investigations (Hooton et al. 2006; MacDonald et al. 2012) of these blocks at later ages (14 and 20 years) confirmed the presence of significant ASR-induced damage in the blocks produced with low-alkali cement and measured expansion in excess of 0.10% after 20 years (see Figure 8). Based on the information available, a maximum alkali limit of 1.8 kg/m^3 (3.0 lb/yd^3) Na_2Oe was selected for Prevention Level Y (e.g. concrete with highly reactive aggregate, exposed to moisture with a service life up to 75 years) and it was decided to require the same limit plus incorporate minimum levels of SCM for Level Z. This was supported by anecdotal evidence available to the committee preparing the guidelines, there being no known case of ASR in concrete structures with lower alkali contents.

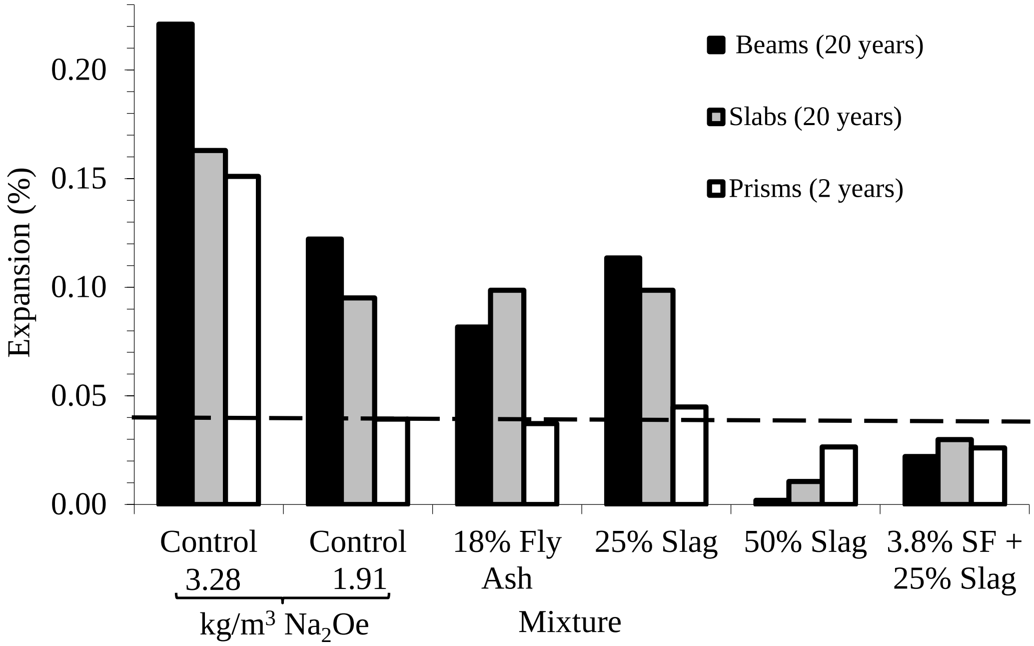

Figure 8. Expansion of concrete blocks on the Kingston exposure site (MacDonald et al. 2012).

In the absence of any other data, the same alkali limits were adopted in AASHTO PP65 for prevention levels V, X, Y, and Z. Prevention Level ZZ requires the same maximum alkali content limit as Level Z but a higher level of SCM. The option of combining a maximum alkali content limit with a minimum SCM replacement level will be discussed later.

Note that recent unpublished data from CANMET (see Figure 6) reports deleterious expansion (≥ 0.04%) of exposure blocks containing two sources of siliceous limestone (designated as aggregates Lm and Sp in Figure 6) after 15 years even at an alkali content of 1.7 kg/m^3 (2.9 lb/yd^3) Na_2Oe.

6.2 Using Supplementary Cementing Materials (SCM)

6.2.1 Background

The ability of pozzolans to control ASR expansion was reported in the same publication that first identified alkali-silica reaction as a cause of concrete deterioration (Stanton 1940). In this paper, Stanton (1940) reported that expansion due to the reaction could be reduced by the use of a pozzolanic cement containing finely ground shale or by replacement of 25% of high-alkali portland cement with pumicite. The pumicite replacement was found to be more effective in reducing expansion than an equivalent replacement with ground Ottawa sand, thus demonstrating that such materials have a role beyond mere dilution of cement alkalis. Subsequent tests by Stanton (1950) confirmed the beneficial effect of a wide range of natural pozzolans, and subsequent studies demonstrated the beneficial effects of fly ash and slag (Cox 1950; Barona 1951; Buck et al. 1953). The first report on the potential use of silica fume to control ASR in concrete appeared in 1979, following research studies at the Building Research Institute in Iceland (Asgeirsson and Gudmundsson 1979). In 1991, it was reported that metakaolin (produced from the thermal activation of relatively pure kaolin clay) was also highly effective in controlling ASR (Walters and Jones 1991).

Since these first studies, research on the use of SCMs to control ASR expansion proliferated and there are now literally hundreds of publications on the subject. Although there remain conflicting views regarding how SCMs control ASR and their precise efficacy in this role, there is a general consensus that most, if not all, SCMs can control deleterious expansion due to ASR provided they are present in sufficient quantity (Thomas and Folliard 2007). The question that remains is how much SCM is sufficient. The amount required varies widely depending on, among other things, the following (Thomas and Folliard 2007):

- The nature of the SCM (especially mineralogical and chemical composition)
- The nature of the reactive aggregate (generally, the more reactive the aggregate, the higher the level of SCM required)
- The availability of alkali within the concrete (i.e., from the portland cement and other sources such as mix water, aggregates, and chemical admixtures)
- The exposure conditions of the concrete (i.e., concrete exposed to external sources of alkali may require higher levels of SCM)

Figure 9 shows a conceptual relationship between the expansion of concrete and the amount of portland cement replaced by SCM. Generally, the amount of SCM required decreases as the reactive silica content of the SCM increases or as the calcium or alkali content of the SCM decreases. In other words, an SCM with high silica and low amounts of calcium and alkali, such as silica fume, tends to be effective at low levels of replacement (Thomas and Bleszynski 2001). Slag, on the other hand, is much less efficient due to its lower silica and higher calcium contents, and has to be used at much higher levels of replacement to control expansion (Thomas and Innis 1998).

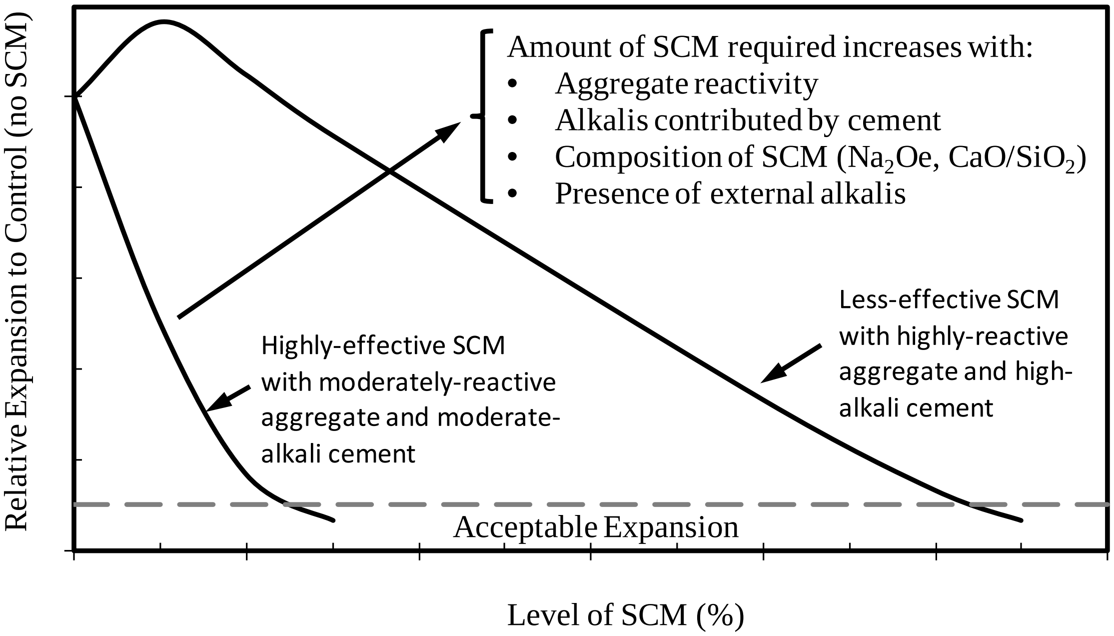

Figure 9. Conceptual relationship between expansion of concrete and level of SCM (adapted from Thomas and Folliard 2007).

Table 10 presents typical minimum replacement levels required for various SCMs to control expansion due to ASR; these values are based on a synthesis of published data from expansion studies conducted on concrete with highly reactive aggregate and high-alkali cement. Higher replacement levels than those given in Table 10 may be necessary in critical structures with an extended service life or where the aggregate reactivity or cement alkali levels are exceptionally high, or the concrete is exposed to alkalis in service. Conversely, lower replacement levels may be expected in structures with a relatively short service life or where moderately reactive aggregates or moderate alkali cements are used.

Table 10. Range of minimum replacement levels required for different supplementary cementing materials to control expansion due to ASR (Thomas and Folliard 2007).

Type of SCM	**Level Required (%)**
Low-calcium fly ash (< 8%)	20 – 30
Moderate-calcium fly ash (8-20% CaO)	25 – 35
High-calcium fly ash (> 20% CaO)	40 – 60
Silica Fume	8 – 12
Slag	25 – 65
Metakaolin	10 – 20

The minimum levels of SCM required in AASHTO PP65 to provide the various levels of prevention were selected largely on the basis of the values specified in the 2000 and 2004 editions of CSA A23.2-27A with some modifications. AASHTO PP65 includes an extra aggregate reactivity class (extremely reactive) and prevention level (ZZ), which were not included in the 2000 and 2004 edition of the CSA guideline[15] and additional preventive measures had to be developed for these cases.

In developing the prescriptive approach for CSA A23.2-27A, which first appeared in the 2000 edition of the specification, consideration was given to published data related to concrete expansion tests (laboratory and field exposure studies) and case histories rather than to data from studies using mortar bar tests such as ASTM C 227, AASHTO 303 (ASTM C 1260), or ASTM C 1567 because of concerns about the lack of correlation between these tests and field performance (Thomas et al. 2006).

It should be noted that neither the prescriptive approach in CSA A23.2-27A, nor that in AASHTO PP65, provides guidance for using natural pozzolans as this classification covers materials with an extremely broad range of physical, chemical, and mineralogical properties. When using natural pozzolans to control expansion due to ASR, the combination of the pozzolan and reactive aggregate must be evaluated using the performance-based methodology.

6.2.2 Minimum Replacement Levels for the "General Case"

This section presents data to support the minimum SCM replacement levels specified for Prevention Level Y in AASHTO PP65 and CSA A23.2-27A. Level Y is considered to represent the "general case" where a highly reactive aggregate (e.g. R2) is to be combined with a high alkali cement in concrete exposed to moisture (but not alkalis) for the construction of a structure with a typical service life of 40 to 75 years (i.e., Class S3). Subsequent sections provide evidence for increasing or decreasing the minimum level of SCM for different levels of aggregate reactivity of cement alkalis.

Figure 10 shows typical results from the concrete prism test (ASTM C 1293) with siliceous limestone (Spratt) aggregate and varying levels of low-calcium Class F fly ash, slag, and undensified silica fume (Shehata and Thomas 2000; Thomas and Innis 1998; Bleszynski 2002). It would appear from these data that 25% fly ash and 50% slag are sufficient to limit the expansion at 2 years to less than 0.040%. Numerous laboratory studies with Canadian reactive aggregates lend support to the efficacy of these levels of Class F fly ash and slag for controlling expansion in concrete with highly reactive aggregates and high-alkali cement (Duchesne and Berube 1994a; Fournier et al. 1995a; 1995b; 1996; Duncan et al. 1973a; 1973b; MacDonald et al. 2012; Bleszynski et al. 2002). Similar findings have been reported for similar laboratory concrete expansion tests with natural aggregates in the United States (Touma et al. 2001; Folliard et al. 2006) and worldwide (e.g. Nixon et al. 1986; Thomas et al. 1996; Connell and Higgins 1992; Shayan 1992).

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[15] The 2009 edition of CSA A23.7-27A does include these additional categories for aggregate reactivity and prevention level.

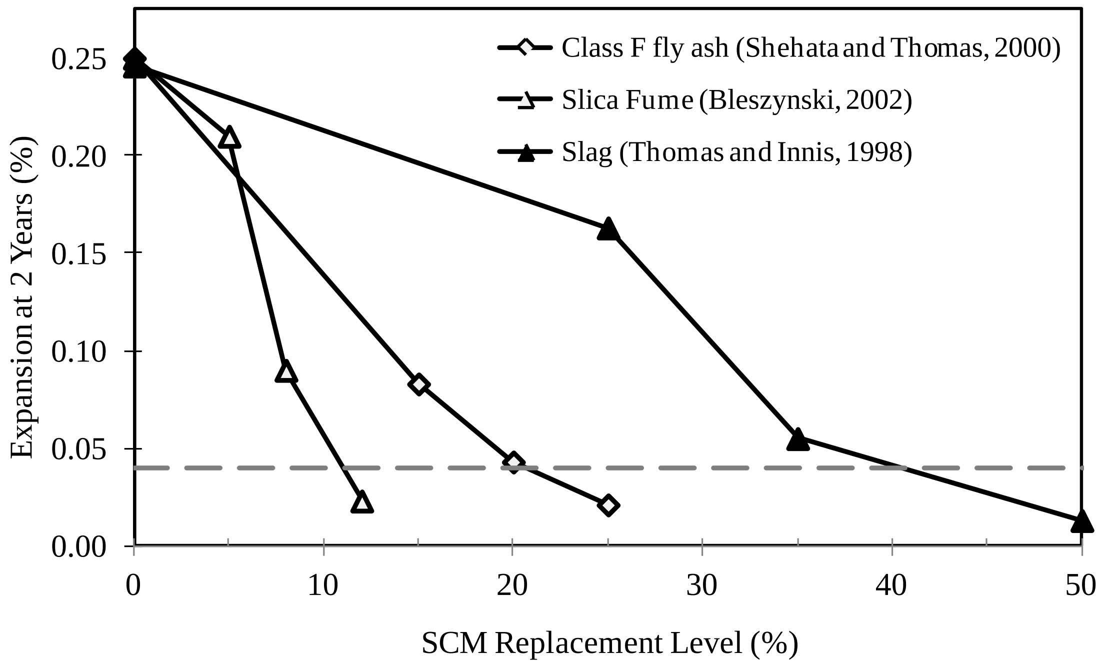

Figure 10. Typical results from concrete prism tests (ASTM C 1293).

Further evidence of the effectiveness of 25% Class F fly ash or 50% slag for controlling ASR has been provided from exposure site studies in Canada (MacDonald et al. 2012; Fournier et al. 2004), the U.K. (Thomas et al. 2011), and South Africa (Oberholster 1989).

The data from the Kingston exposure site (MacDonald et al. 2012; Rogers et al. 2000; Hooton et al. 2006) are shown in Figure 8. After 20 years exposure, concrete beams (0.6 x 0.6 x 2 m, 2 x 2 x 6.6 ft) and slabs (0.2 x 1.2 x 4 m, 0.66 x 3.94 x 13.1 ft) containing high-alkali cement and either 50% slag or a combination of 3.8% silica fume plus 25% slag have exhibited no deleterious expansion (i.e., expansion ≤ 0.040%) or cracking, whereas expansion and cracking is observed in all beams and slabs without SCM even when low-alkali cement (0.46% Na_2Oe) was used. It is noteworthy that although 25% slag or 18% fly ash reduced expansion compared to the high-alkali cement control, these replacement levels were not sufficient to completely suppress damaging expansion.

CANMET operates a concrete exposure site in Ottawa and selected data from this site are presented in Figure 11 (from Fournier et al. 2004 and unpublished data). The data selected are for blocks (0.38 x 0.38 x 0.71 m, 15 x 15 x 28 in.) produced with high-alkali cement (0.90% Na_2Oe), varying amounts of different fly ashes (all Class F), and six different reactive aggregates (note Su = Sudbury and Sp = Spratt). Mixes were also cast with the addition of NaOH to raise the cement alkalis to 1.25% Na_2Oe and slabs were also produced; the performance of these mixes will be discussed later. The data in Figure 11 show that 20 to 30% fly ash is required/sufficient to control expansion at the age of 10 years.

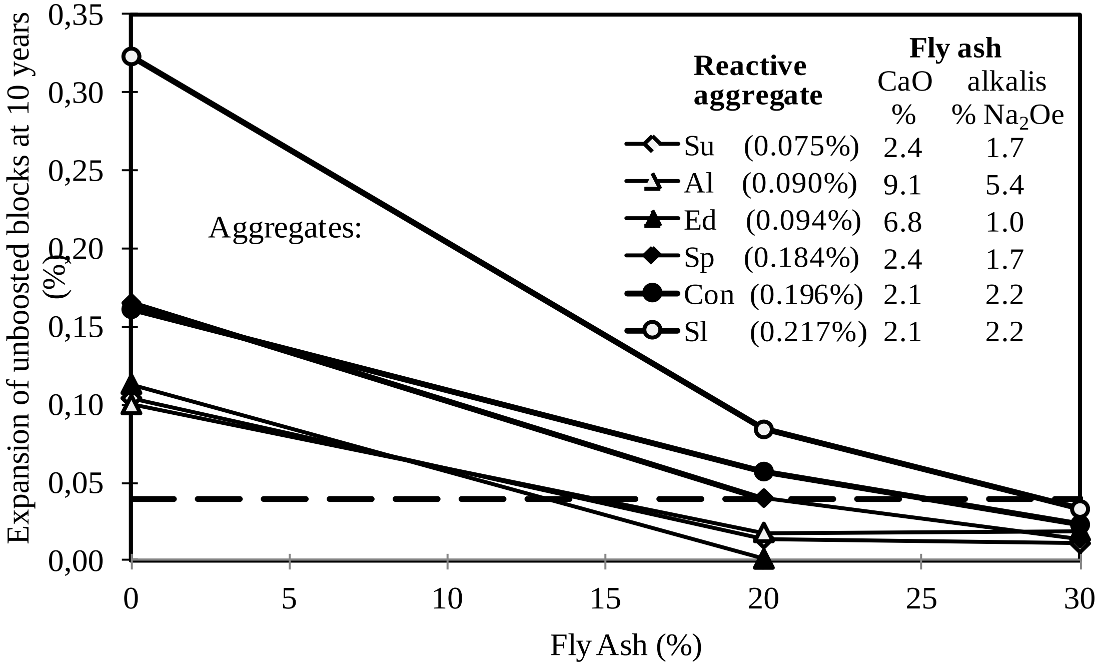

Figure 11. **Expansion of (unboosted) blocks after ten-year exposure on the Ottawa exposure site (Fournier et al. 2004 and unpublished data). The expansions obtained for each aggregate in the Concrete Prism Test (420 kg/m^3 and boosted alkalis to 5.25 kg/m^3 Na_2Oe) are given in brackets. Also, the CaO and alkali (Na_2Oe) contents of the fly ash used with each aggregate are provided.**

In 1987, expansion results for large beams (1000 x 450 x 300 mm, 39 x 18 x 12 in.) and cubes (300 mm, 12 in.) produced with a reactive hornfels (Malmesbury) aggregate and stored under field conditions in South Africa were reported by Oberholster and Davis (1987) and later updated by Oberholster (1989). The mixes used and the results are summarized in Table 11. The active alkalis expressed in Table 11 are based on the sum of the "available" alkalis (ASTM C 311) in the portland cement and SCM, and the additional alkali hydroxides added to the mix. The additional alkalis were added to increase the portland cement "active" alkalis to 1.18% and 1.10% for beams and cubes, respectively, and to compensate for the effects of alkali dilution due to replacement of the portland cement in a few mixes containing SCMs (marked + in Table 11). The latter was achieved assuming the SCM to contain no alkali. The results in Table 11 show that 50% slag (by volume) and 15% fly ash are both effective in reducing expansion due to ASR in concrete containing reactive aggregate.

Table 11. Effect of SCMs on expansion of field exposed concretes (from Oberholster and Davies 1987; Oberholster 1989).

		Active alkalis in concrete[16] (kg/m^3)	Reported expansion			Time to exp.>0.05% (days)
			1987		1989	
Mix series	SCM[17]		1450 days	2100 days	2700 days	
Beams	0%	4.37		0.26		174
370 kg/m^3	15% ash	4.07		<0.01		-
total cement	50% slag	2.48		<0.01		-
Cubes	0%	3.85	0.17		0.32	950
350 kg/m^3	48% slag	1.74	<0.01		<0.01	-
total cement	48% slag$^+$	3.85	<0.01		0.02	-
	3.5% SF$^+$	3.87	0.51		0.26	1400
	7.0% SF$^+$	3.88	<0.01		<0.01	-
Cubes	0%	4.95	0.17		0.28	870
450 kg/m^3	48% slag	2.23	<0.01		<0.01	
total cement	48% slag$^+$	4.95	<0.01		0.02	
	3.5% SF$^+$	4.97	0.08		0.15	1200
	7.0% SF$^+$	4.99	<0.01		0.09	1300

$^+$Alkalis added to mix to compensate for dilution by SCM, assuming SCM to contain no alkalis.

Thomas et al. (2011) reported the results from a study of concrete blocks (915 x 915 x 815 mm, 36 x 36 x 32 in.) and cubes (350 mm, 14 in.) stored for 14 to 18 years on an exposure site at the Building Research Establishment (BRE) near Watford in the U.K. The blocks contained a variety of reactive flint sands or reactive greywacke coarse and fine aggregates, together with fly ash from two different sources. In most cases, 25% fly ash was effective in eliminating damaging expansion even when the alkali contribution from the portland cement in the fly ash concrete was at a level that produced very high levels of expansion (> 1.0%) in concrete without fly ash. The only exception were two mixes produced with 25% of a relatively coarse fly ash (28% ash retained on a 45-microns sieve) and very high levels of alkali (4.74 and 5.18 kg/m^3 Na$_2$Oe, 7.90 and 8.63 lb/yd^3 Na$_2$Oe), which showed a small amount of expansion (0.044% and 0.061%) after 17 years.

Yet, more evidence of the beneficial effect of fly ash in terms of controlling ASR can be found in the continued good performance of structures that were constructed with reactive aggregates and fly ash. One such example is the Lower Notch Dam in Ontario, which was completed in 1969. The aggregate used was known to be reactive at the time of construction and has been implicated as the cause of ASR in numerous hydraulic and highway structures in the same vicinity (Thomas 1996). Fly ash was used at a replacement level of 20% and 30% in the structural and mass

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[16] Active alkalis are calculated from the "available" alkalis of cement and admixture by ASTM C 311 and include alkalis added at the mixer.

[17] In the original paper, the replacement levels for the cubes are expressed as percent by volume; these values have been recalculated to percent by mass using the reported specific gravities for the materials.

concrete, respectively. The portland cement used had an alkali content that was reportedly as high as 1.08% Na_2Oe. An inspection of the structure after approximately 40 years revealed no evidence of ASR (Thomas et al. 2012). Thomas (1996) also reports on the use of 25% fly ash in the Nant-y-Moch Dam in Wales which was constructed between 1957 and 1962 as part of the Cwm Rheidol Hydro-Electric Scheme. The Dinas Dam, which forms part of the same system and was constructed at the same time with the same reactive greywacke aggregate, but without fly ash, exhibited severe symptoms of ASR after 30 years (Thomas 1996) which necessitated some measure of remediation after about 40 years (Thomas et al. 2012). An inspection of the Nant-y-Moch Dam 50 years after construction revealed no evidence of ASR, the concrete being in generally excellent condition (Thomas et al. 2012). Another interesting case history that permits the comparison of the performance of concrete structures with and without fly ash is the Gordon River Power Development (Blaikie et al. 1996) constructed between 1967 and 1977. This hydraulic generating station receives water from Lake Gordon and the concrete intake tower was observed to be cracking due to ASR approximately 15 years after construction. The same aggregate and cement was used in the construction of Gordon Dam (which forms Lake Gordon); however, fly ash was used in the dam to replace between 20 to 25% (and sometimes up to 35%) of the cement. ASR damage has not been observed in the concrete dam.

A final consideration is that, to the authors' knowledge, there is no reported case of ASR in a concrete structure than contains 25% or more Class F fly ash or 50% or more slag, despite extensive use of these materials for many decades.

Figure 10 indicates that the amount of silica fume required to control the 2-year expansion of concrete containing Spratt aggregate to a value below 0.040% is in the range of 8 to 12%. Table 12 shows data from other investigators (Fournier et al. 1995b; Bérubé and Duchesne 1992; Duchesne and Bérubé 1994a; Durand 1993) working with Canadian aggregates, which shows 10% silica fume is usually effective. Note that the silica fume with a low silica content and high alkali content tested by Bérubé and Duchesne (1992; Duchesne and Bérubé 1994a; 1994b) is much less effective than the high-silica, low-alkali silica fume.

Table 12. Expansion of silica fume concrete prisms at 24 months - Canadian studies.

Authors	Aggregate Source	Cement Alkalis (kg/m^3 Na_2O_e)	Silica Fume			Expansion at 24 Months (%)
			% SiO_2	% Na_2O_e	Level	
Fournier et al. 1995b	Greywacke	5.25	Control		0	0.267
		4.9	97.5	0.72	7.5	0.079
		4.7			10	0.039
		4.6			12.5	0.016
Bérubé & Duchesne 1992 Duchesne & Bérubé, 1994a	Rhyolitic Tuff	4.4	Control		0	0.245
		4.2	74.6	3.63	5	0.370
		3.9			10	0.177
		4.2	94.2	0.77	5	0.370
		3.9			10	0.033
	Siliceous Limestone (Spratt)	4.4	Control		0	0.287
		4.2	74.6	3.63	5	0.239
		3.9			10	0.078
		4.2	94.2	0.77	5	0.275
		3.9			10	0.019
Durand et al. 1990 Durand, 1993	Chloritic Schist	3.9	Control		0	0.077(1)
		3.7	91.85	0.64	5	0.023(1)
		3.5			10	0.009(1)
		3.3			15	0.001(1)
	Siliceous Limestone (Spratt)	3.9	Control		0	0.137(1)
		3.7	91.85	0.64	5	0.047(1)
		3.5			10	0.016(1)
		3.3			15	0.006(1)
	Trois-Riviéres Limestone	3.9	Control		0	0.175(1)
		3.7	91.85	0.64	5	0.044(1)
		3.5			10	0.000(1)

(1) - These values have been estimated from figures presented in the original papers and are not absolute values as determined by experimentation.

The different studies shown in Table 12 used different cementitious material contents and cement alkali loadings, but otherwise the procedures were generally the same and reasonably consistent with the current concrete prism test. These data indicate that the performance of silica fume is dependent on both the nature of the reactive aggregate and the alkali content in the concrete. Lower levels of silica fume (e.g. 5 to 7.5%) may be effective in controlling expansion in concretes containing moderately reactive aggregates or relatively low alkali contents [e.g. < 4 kg/m^3 (6.7 lb/yd^3) Na_2Oe]. However, as the aggregate reactivity and the alkali content of the concrete increase, so too does the level of silica fume required to control expansion. When a highly reactive aggregate is combined with a high content of alkali [e.g. > 4.5 kg/m^3 (7.5 lb/yd^3) Na_2Oe], then the level of silica fume required may be much higher (e.g. 10 to 12.5%). The effect of alkali content and aggregate type on the expansion of silica fume concrete can be further illustrated by examining the expansion data for the mixes containing 10% silica fume as shown in Figure 12. This level of silica fume (10%) appears to provide long-term protection from damaging expansion (e.g. < 0.040%) with certain aggregates and relatively low alkali contents, but is clearly not sufficient in systems with the more reactive aggregates and higher contents of alkalis.

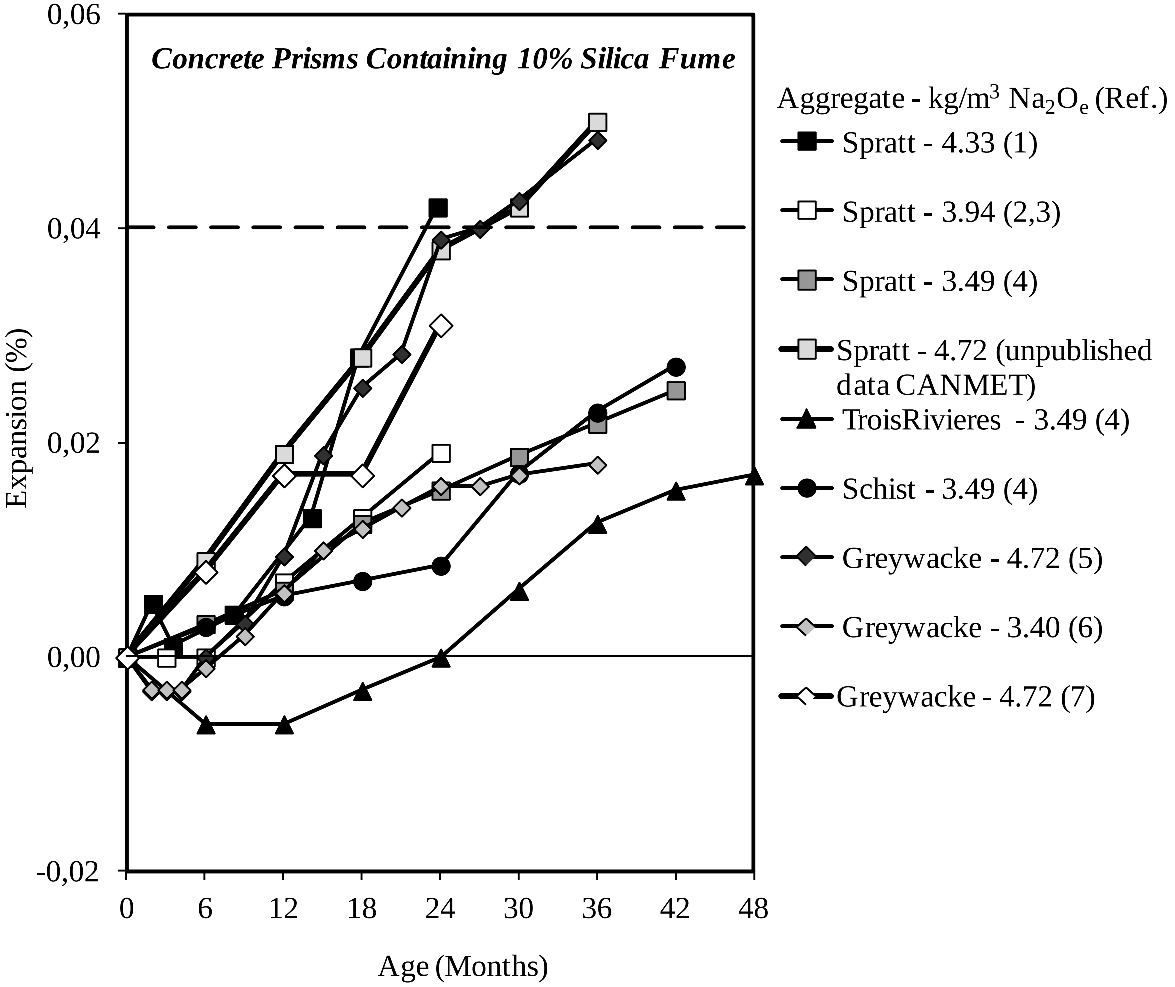

Figure 12. **Results of concrete prism tests with silica fume and Canadian aggregates (1 - Grattan-Bellew 1991; 2 - Bérubé and Duchesne 1992; 3 - Duchesne and Bérubé 1994a; 4 - Durand 1993; 5 - Fournier et al. 1995b; 6 - Fournier and Malhotra 1999; 7 - Langley et al. 1995).**

Further evidence of the importance of the alkali content in determining the level of silica fume required can be seen in the results from the exposure site in South Africa (Oberholster and Davis 1987; Oberholster 1989). Details of this study were discussed above and expansion results up to just over seven years of age are given in Table 11 and Figure 13. The use of 3.5% or 7% silica fume delayed the onset of expansion and time to cracking in all cases. However, only the mix at the lower cement content with 7% silica fume failed to expand after just over seven years field exposure. These data clearly show the effect of alkali content on the efficacy of silica fume in controlling expansion. A replacement level of 7% silica fume appears to have been sufficient to provide long-term prevention of expansion when the "active alkalis" of the mix were just less than 4 kg/m^3 (6.7 lb/yd^3) Na_2Oe, but not at the higher alkali content of 5 kg/m^3 (8.3 lb/yd^3) Na_2Oe.

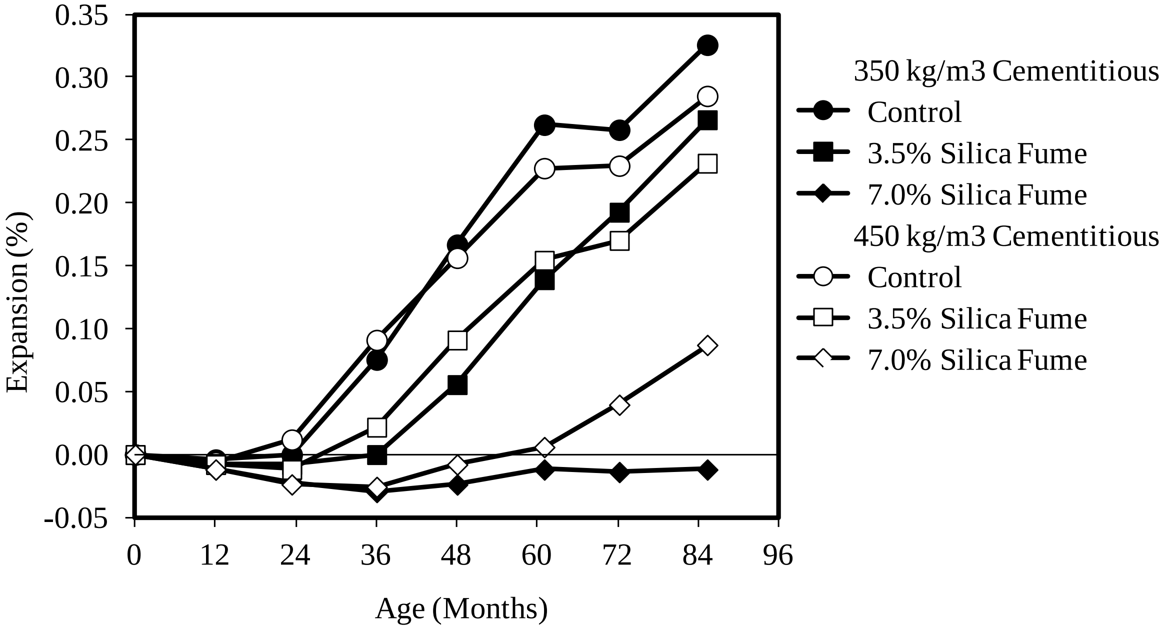

Figure 13. **Effect of silica fume on the expansion of cubes (350 mm, 14 in.) on outdoor exposure site in South Africa (Oberholster and Davies 1987; Oberholster 1989).**

Thomas and Bleszynski (2001) presented a synthesis of the published data on silica fume and ASR, including a substantial database on the effect of silica fume on the alkalinity of the pore solution. It was demonstrated that in order to control expansion and to maintain a low concentration of alkali hydroxides in the pore solution, the level of silica fume must increase as the level of alkali in the portland cement increases. The following relationship was proposed (Thomas and Bleszynski 2001):

$$SF = 2.5 \times AL \qquad \text{Eqn. 1}$$

where SF is the minimum level of silica fume (% by mass of cementitious material) and AL is the alkali content contributed by the portland cement (kg/m^3 Na_2Oe). This results in a minimum replacement level of 12.5% silica fume when the alkalis in the concrete are 5 kg/m^3 (8.3 lb/yd^3) Na_2Oe, which is in close agreement with the observations of Fournier et al. (1995b) for concrete containing greywacke. This proposal was adopted by CSA in the 2000 edition of CSA A23.2-27A and subsequently by AASHTO PP65.

Silica fume has been used to control ASR in concrete in Iceland since 1979 (Asgeirsson and Gudmundsson 1979) and after 20 years it was reported that silica fume "has virtually eliminated all alkali-silica reactions (ASR) in concrete" (Gudmundsson and Olafsson 1999) and the satisfactory performance of silica fume concrete in Iceland was confirmed more recently (Gudmundsson and Olafsson 2004). Initially the silica fume was interground with the cement at a 5% replacement level, but this level was later increased to 7.5%. Portland cement produced in Iceland is very high in alkali (1.65% Na_2Oe) and some of the volcanic aggregates are very reactive, so the Icelandic field experience provides a solid testament to the efficacy of silica fume in decreasing the risk of ASR.

6.2.3 Effect of Fly Ash Composition

Studies by Shehata and Thomas (2000) showed that the efficacy of fly ash depends on the composition of the fly ash, particularly the calcium (CaO) and alkali (Na_2Oe). Figure 14 (from Thomas 2011) summarizes the expansion data for mixes with 25% fly ash from Shehata and Thomas (2000; 2002) and subsequent testing. Most fly ashes with calcium contents less than 20% CaO are able to control damaging expansion with Spratt aggregate when used at a 25% replacement level. As the calcium content of the fly ash increases above 20% CaO, the efficacy of the fly ash in terms of controlling ASR decreases. Fly ashes with high alkali contents (> 5% Na_2Oe) are not effective when used at a 25% replacement level. Figure 15 (from Shehata and Thomas 2000; 2002) shows a comparison of the performance of low-calcium (Class F) and high-calcium (Class C) fly ashes, and the data indicate that Class C fly ashes can control expansion provided they are used at a higher level of replacement; the required level of replacement may exceed 50% for some fly ashes. The figure also shows data for a Class F fly ash (designated BD) with a moderate calcium content (12.3% CaO) and a high alkali content (8.45% Na_2Oe), which is not effective in controlling expansion even at a 40% replacement level. Unpublished data from the same source indicates that high-alkali fly ashes may be effective if used at much higher replacement levels.

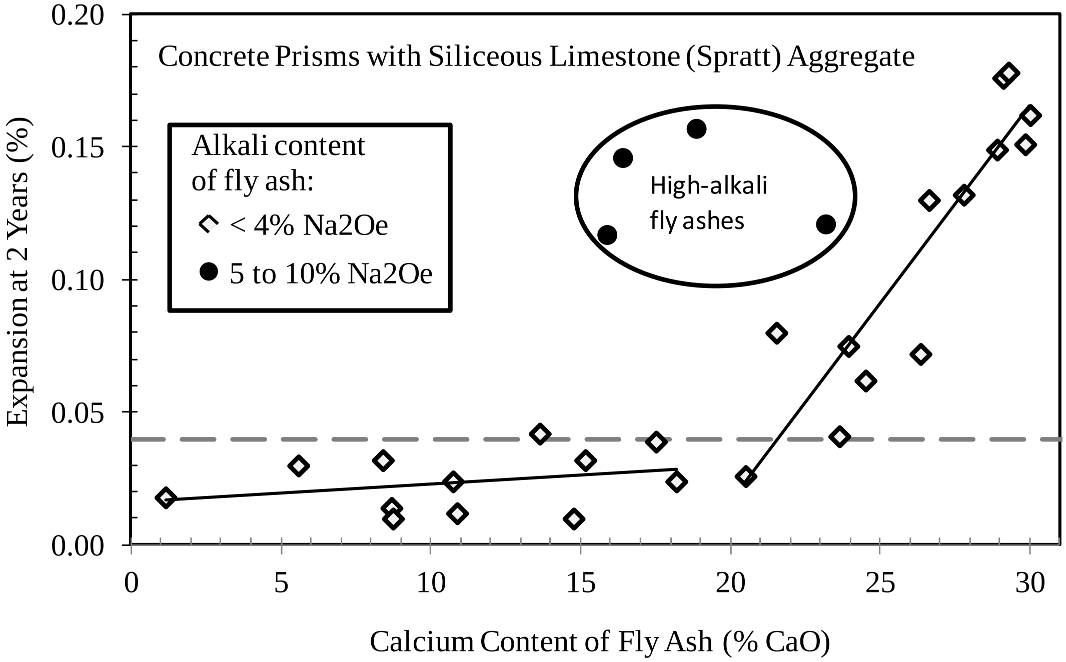

Figure 14. Effect of fly ash composition on expansion of concrete (ASTM C 1293) containing 25% fly ash and Spratt aggregate (Shehata and Thomas 2002; Thomas 2011).

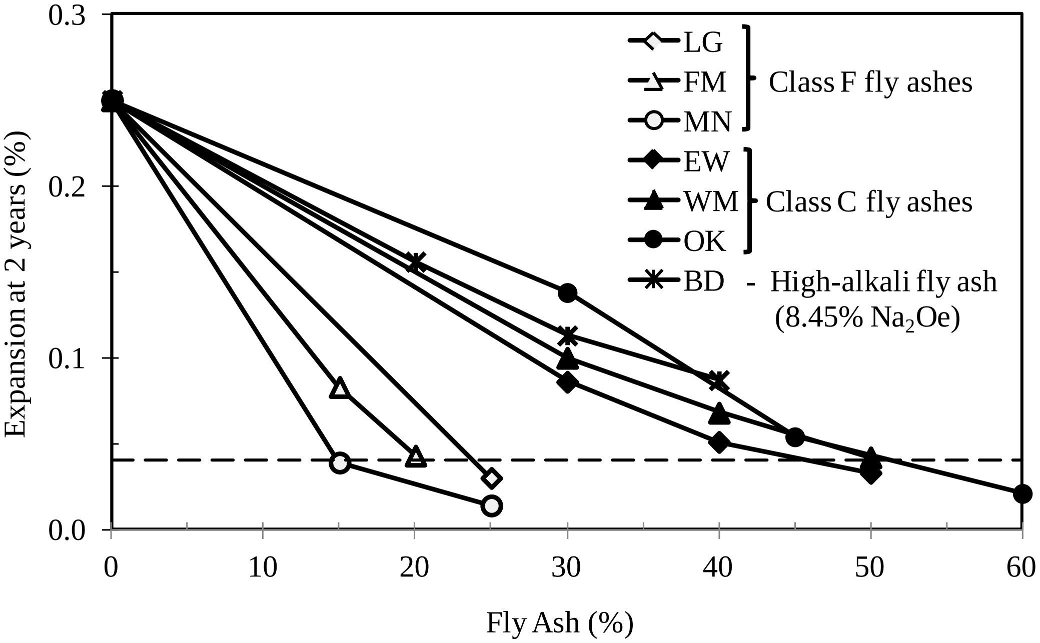

Figure 15. Expansion of concrete prisms incorporating the Spratt limestone (ASTM C 1293) and varying amounts of different fly ashes (Shehata and Thomas 2000; 2002).

In CSA A23.2-27A, Prevention Level Y requires a minimum fly ash replacement level of 25% for low-calcium (< 8% CaO) Class F fly ash with an alkali content less than 3% Na_2Oe. This level is increased by 5% if either the calcium content increases (above 8% CaO) or the alkali content increases (above 3% Na_2Oe), or by 10% if both calcium and alkali increase. Fly ashes higher in calcium (> 20% CaO) or alkali (> 4.5% Na_2Oe) are not included in the standard practice.

In the prescriptive approach of AASHTO PP65, the use of fly ash is limited to those with calcium contents less than or equal to 18% CaO. The value of 18% CaO was selected to ensure that all fly ashes meeting the criteria for ASTM C 618 Class F fly ash were included. A minimum replacement level of 25% is specified regardless of the calcium content (provided it does not exceed 18% CaO); this is reasonably consistent with the data shown in Figure 14, as there is little impact of the calcium content of the fly ash for ashes with calcium contents in the range from 1 to 18% CaO. The level of fly ash is increased to 30% for fly ashes with moderate alkali (3.0 to 4.5% Na_2Oe). The prescriptive approach does not cover fly ashes with calcium contents in excess of 18% CaO or alkali contents in excess of 4.5% Na_2Oe; such materials must be tested with the reactive aggregate being used following the performance-based approach in PP65.

6.2.4 Use of Ternary Cement Blends

Laboratory testing of concrete has shown that ternary blends containing combinations of silica fume with either fly ash (Shehata and Thomas 2002; Fournier et al. 2008) or slag (Bleszynzki 2002; Bleszynski et al. 2002) are effective in controlling expansion due to alkali-silica reaction.

Figure 16 shows data for binary blends of portland cement plus fly ash and ternary blends of portland cement plus silica fume and fly ash (Shehata and Thomas 2002). The data show that the effects of using two SCMs are accumulative. For example, the amount of silica fume required to control expansion with Spratt aggregate is approximately 10% when used as the sole SCM, whereas approximately 50 to 60% of the high-calcium fly ash designated OK in Figure 16 is required. If half these amounts are used together, that is 5% silica fume and 20 to 30% OK fly ash, the expansion at 2 years is acceptable.

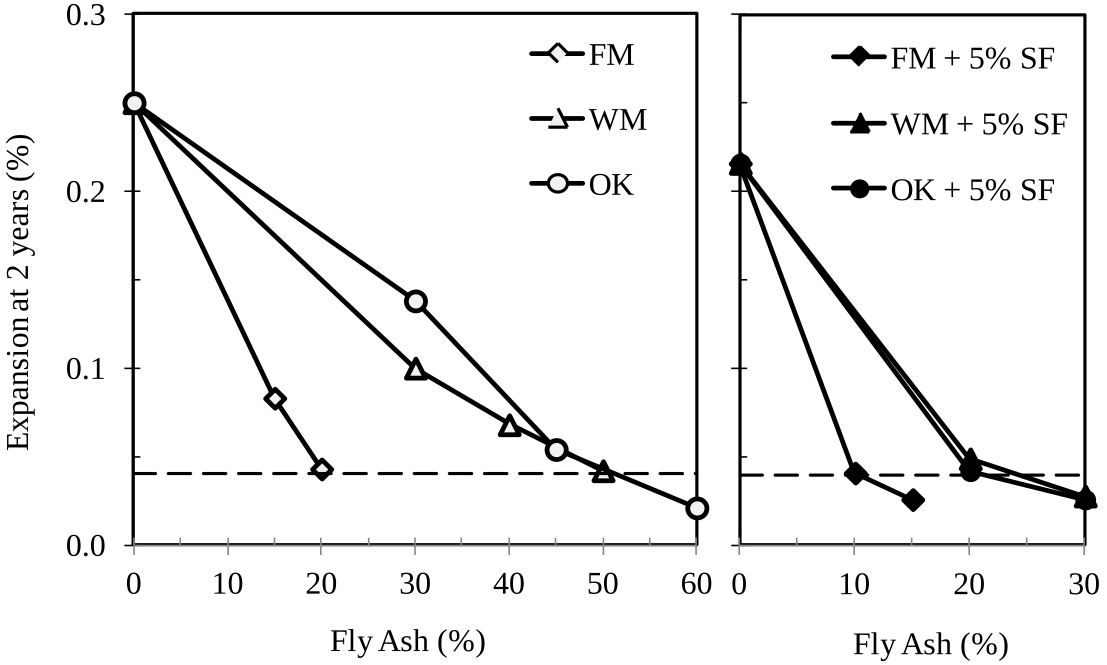

Figure 16. Expansion of concrete (ASTM C 1293) with fly ash only (left) and with 5% silica fume + fly ash (right) (Shehata and Thomas 2002).

On the basis of these data, both CSA A23.2-27A and AASHTO PP65 permit the use of combinations of SCM provided the following condition is met:

$$\frac{fa}{FA} + \frac{sg}{SG} + \frac{sf}{SF} \geq 1 \qquad \text{Eqn. 2}$$

where *fa*, *sg*, and *sf* are the replacement levels of fly ash, slag, and silica fume, respectively, that are to be used in combination, and *FA*, *SG*, and *SF* are the minimum amounts of fly ash, slag, and silica fume, respectively, required according to Table 6 when these materials are used on their own.

6.2.5 Effect of Aggregate Reactivity

Figure 17 shows 2-year expansion data for concrete prism tests containing various reactive aggregates and different amounts of slag (Thomas and Innis 1998). It can be seen that the amount of slag required to limit expansion below 0.040% at 2 years varies between 35% and 50% depending on the aggregate type.

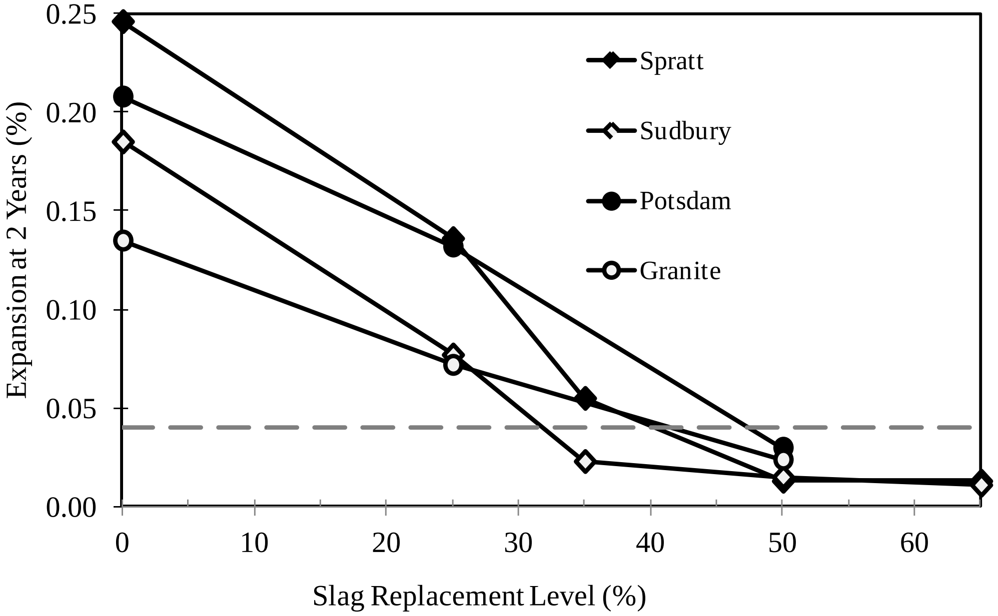

Figure 17. Effect of aggregate type on the amount of slag required to control ASR expansion (Thomas and Innis 1998).

Figure 18 shows expansion data for blocks exposed on an outdoor exposure site in Ottawa (Fournier et al. 2004; unpublished data); the blocks contain high-alkali cement (0.90% Na_2Oe), either low-CaO Class F fly ash or silica fume, and aggregates of varying reactivity. For the moderately reactive aggregates designated as Su and Al in the figure (CPT expansions > 0.040%, < 0.120%), which produced exposure block expansions of 0.120% and 0.100% when tested with 100% portland cement, a fly ash replacement level of 20% or a silica fume replacement level of 7.5% was sufficient to reduce 10-year expansions below 0.040%. For the highly-reactive aggregates designated as Sl and Con in the figure, that produced exposure block expansions of 0.326% and 0.161% when tested with 100% portland cement, 30% fly ash was required to reduce the expansion below 0.040%. The use of 7.5% silica fume reduced the exposure block expansions below 0.040% with the highly reactive Sl aggregate after 10 years exposure; however, expansions higher than the above limit were obtained at 13 years. In this figure, it would appear that the amount of fly ash and silica fume required increases as the reactivity (as determined by the expansion when tested with portland cement alone) of the aggregate increases.

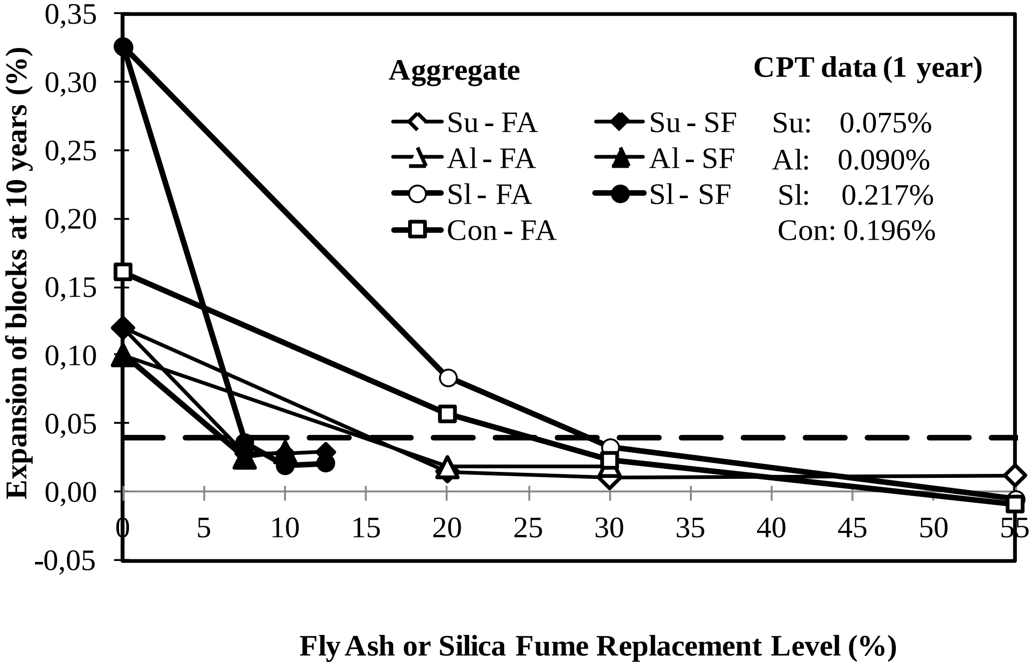

Figure 18. **Effect of aggregate type on the amount of silica fume (SF) or fly ash (FA) required to control ASR expansion (Fournier et al. 2004 and unpublished data). NM, Con, Su, Al, and Sl are different aggregates. FA and SF are Class F fly ash and silica fume, respectively.**

By including aggregate reactivity in the determination of the Risk of ASR, both CSA A23.2-27A and AASHTO PP65 require lower amounts of SCM for moderately reactive aggregates and higher amounts with very highly (or extremely) reactive aggregates.

6.2.6 Effect of Cement Alkalis

One of the drawbacks of the concrete prism test is that significant alkali leaching occurs during the test, which means it cannot generally be used to determine the threshold alkali content required to initiate expansion with a specific aggregate or to determine how the minimum amount of SCM required varies as the alkali content of the cement changes (Thomas et al. 2006). Larger samples exposed under natural conditions should be relatively immune from the effects of alkali leaching, however, much longer testing periods are required.

The importance of the alkali content of the concrete on determining the minimum level of silica fume required was discussed above (see Section 6.2.2). The alkali content also has an effect on the minimum level of fly ash or slag required although the impact is much less marked. Figure 19 shows how the alkali content contributed by the portland cement impacts the expansion of large blocks on the exposure site at BRE in the U.K. (Thomas et al. 2011). For concrete with less than 4.5 kg/m^3 (7.5 lb/yd^3) Na_2Oe, 25% fly ash was sufficient to control damaging expansion at 17 years, but at higher levels of alkali, the expansion of concrete with 25% fly ash exceeded

0.040% (for fly ash DX only) and very fine cracks were observed. An increased level of this fly ash would be required to eliminate damaging expansion.

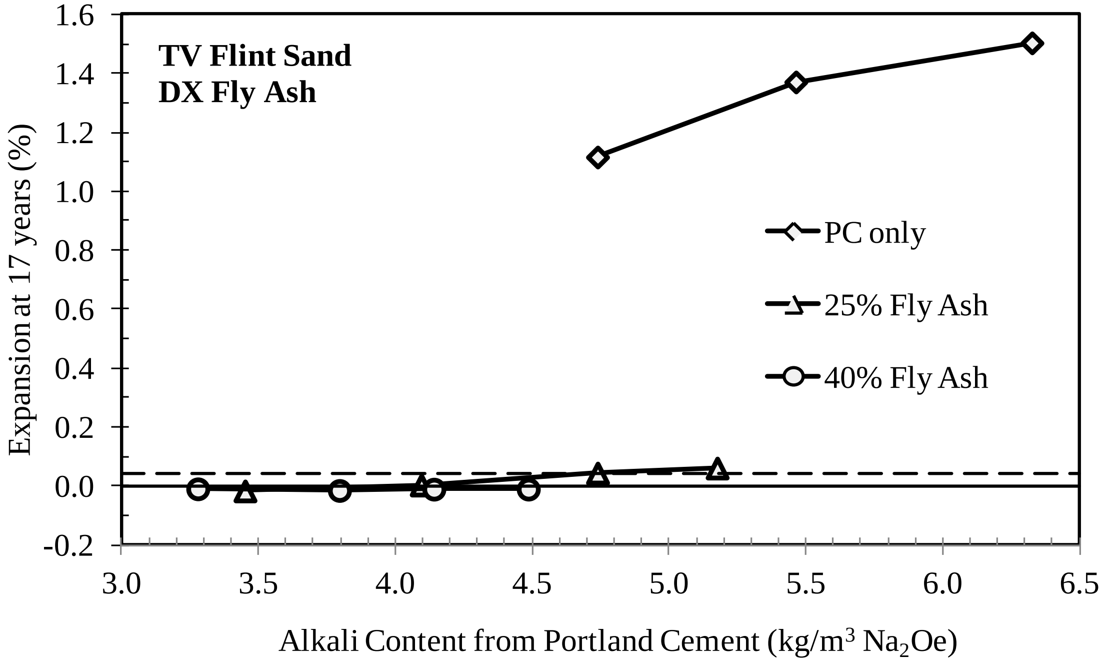

Figure 19. Effect of alkali contributed by the portland cement on expansion of concrete blocks stored on the BRE exposure site (Thomas et al. 2011).

Figure 20 shows data from Fournier et al (2004; unpublished data) for concrete blocks stored outdoors in Ottawa, Canada. The blocks contain two moderately-reactive (Su and Al) and two highly-reactive (Con and Sl) coarse aggregates and 420 kg/m^3 (700 lb/yd^3) of cementing material. High-alkali portland cement with 0.90% Na$_2$Oe was used to manufacture the blocks, and in a companion series of blocks, the alkali content of the portland cement component of the concrete was boosted to 1.25% Na$_2$Oe by the addition of NaOH to the mix water, as per ASTM C 1293. The data in Figure 20 show that the differences in expansion between boosted and unboosted blocks can sometimes be significant, and that increased amounts of SCM are clearly needed to control expansion to acceptable levels when the alkali content of the mixture is increased. It can also be noted that the limited efficacy of the fly ash (FA2) with the moderately-reactive aggregate Al, at the 20% replacement level, is likely to be partly related to its fairly high alkali content (5.40% Na$_2$Oe).

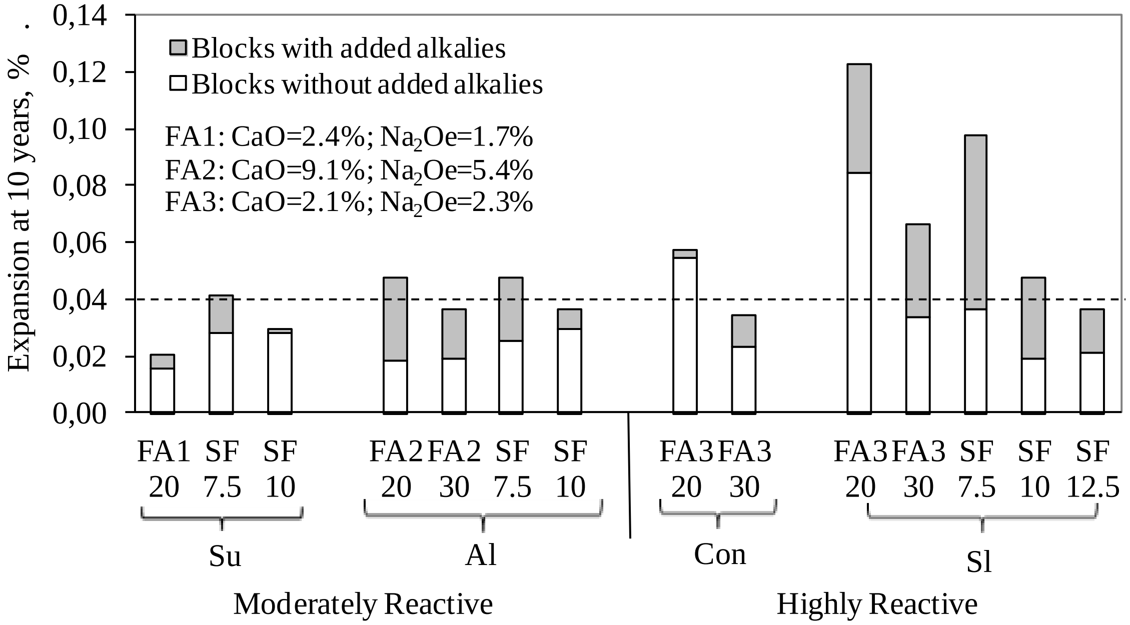

Figure 20. **Effect of cement alkalis on expansion of concrete containing fly ash or silica fume (Fournier et al. 2004 and unpublished data).**

The minimum replacement levels provided in Table 6 of AASHTO PP65 are intended for use with moderate to high alkali cements (0.70 to 1.00% Na_2Oe). If the job cement has an alkali level below or above this range, the minimum SCM replacement levels are reduced or increased, respectively, as shown in Table 8. Note that the adjustments to the minimum SCM level shown in Table 8 are intended for use with fly ash and slag only, as the alkali content of the cement is already considered when determining the minimum level of silica fume.

6.3 Limiting the Alkali Content of the Concrete and Using SCMs

When the level of risk is high and the concrete has an extended design service life, extreme measures are required to prevent damaging ASR. In such cases, it may be necessary to both control the alkali content of the concrete and to use a minimum level of SCM. AASHTO PP65 includes this option in Table 8.

For prevention level Z, there is an option to either use the minimum levels of SCM for level Z given in Table 6 without limiting the alkali content of the concrete, or to limit the alkali content of the concrete with a reduced level of SCM. For prevention level ZZ, both the maximum alkali content and minimum SCM level must be met.

7.0 Summary

The AASHTO PP65-11 "Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction" provides approaches for evaluating aggregate reactivity (ASR and ACR) and for selecting preventive measures (for ASR only). This document provides the background information that was used in the development of the prescriptive approach for selecting preventive measures, which is to: (i) control the alkali content of the concrete to a maximum allowable level, (ii) use a minimum level of supplementary cementing material (SCM) or combination of SCMs, or (iii) use a combination of these two options (that is controlling the alkali content of the concrete and using SCM).

The prescriptive approach in AASHTO PP65 uses a step-by-step analysis that was developed considering that the level of ASR risk is directly influenced by the reactivity of the aggregate, size of the structure, expected service life of the structure, the availability of moisture, and exposure of the structure to alkalis during the service life. As a result, a series of recommended preventive actions/levels (V to ZZ) has been developed for new structures and pavements.

Deleterious ASR can be prevented solely by limiting the alkali content of the concrete. The threshold alkali content primarily depends on the reactivity of the aggregate and whether or not the concrete is exposed to alkali compounds in service. The maximum allowable concrete alkali content ranges from 3.0 kg/m^3 (5.0 lb/yd^3) for a structure with lower risk level to 1.8 kg/m^3(3.0 lb/yd^3) for a structure with higher risk level requiring strong preventive action.

Deleterious ASR can be prevented by using supplementary cementing material (SCM), or a combination of SCMs. The amount of SCM required to prevent deleterious ASR depends upon a number of factors, including, but not limited to, the reactivity of the aggregate, the composition of the SCM (particularly CaO, SiO_2, and Na_2Oe), the alkali provided to the system by the portland cement, and the presence of alkali compounds in service.

In the prescriptive approach of AASHTO PP65, the use of fly ash is limited to those with a calcium content less than or equal to 18% CaO. The specified replacement level ranges from 15% (low risk level/preventive action required) to 40% (high risk level/preventive action required). The level of fly ash increases as the alkali content in the fly ash increases from the < 3.0% Na_2Oe group to the 3.0 - 4.5% Na_2Oe group of ashes. The prescriptive approach does not cover fly ash with a calcium content in excess of 18% CaO or an alkali content in excess of 4.5% Na_2Oe; such materials must be tested with the reactive aggregate being used following the performance-based approach in PP65.

Laboratory and field investigations have shown that silica fume (SF) can be used to prevent deleterious ASR in concrete. The minimum SF content required increases as the level of alkalis in the portland cement increases. However, a minimum replacement level of 7% is required for controlling deleterious ASR when SF is the sole source of SCM used for prevention.

Similar to silica fume (SF), laboratory and field investigations have shown that ground granulated blast furnace slag (GGBFS) can be used to prevent deleterious ASR in concrete. The minimum slag content required ranges from 25% (low risk level/preventive action required) to 65% (high risk level/preventive action required).

Laboratory testing of concrete has shown that ternary blends containing combinations of silica fume with either fly ash (Shehata and Thomas 2002; Fournier et al. 2008) or slag (Bleszynzki

2002; Bleszynski et al. 2002) are effective in controlling expansion due to the alkali-silica reaction.

Testing has shown that the effects of blending SCMs (e.g. FA and SF, FA and slag, slag and SF) are accumulative and can provide an effective solution when higher proportions of fly ash or slag may be difficult to incorporate for other reasons (e.g. durability issues).

The specified minimum replacement levels provided in the AASHTO PP65 are intended for use with moderate to high alkali cements (0.70 to 1.00% Na_2Oe). If the job cement has an alkali level below or above this range, the minimum SCM replacement levels are reduced or increased.

Finally, when the level of risk is high and the concrete has an extended service life, extreme measures are required to prevent damaging ASR. In such cases, it may be necessary to control the alkali content of the concrete while also using a minimum amount of SCM.

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PP65-11, Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction.

T 303, Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction.

ASTM Standards:

C 33, Standard Specification for Concrete Aggregates.

C 150, Standard Specification for Portland Cement.

C 227, Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method).

C 1157/C 1157M, Standard Performance Specification for Hydraulic Cement.

C 1260, Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method).

C 1293, Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction.

C 1567, Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method).

Canadian Standards:

A23.1, Concrete materials and methods of concrete construction.

A23.2-26A, Determination of Potential Alkali-Carbonate Reactivity of Quarried Carbonate Rocks by Chemical Composition.

A23.2-27A, Standard practice to identify degree of alkali-reactivity of aggregates and to identify measures to avoid deleterious expansion in concrete.

RILEM Recommendations:

RILEM TC 191-ARP, Alkali-Reactivity and Prevention—Assessment, Specification, and Diagnosis of Alkali-Reactivity.

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