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Investigation of Differences in Deterioration of Two North Carolina Bridges Due to Alkali-Silica Reactivity (ASR)

by

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DEPARTMENT OF CIVIL ENGINEERING

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16. Abstract

A previous investigation of the extent of alkali silica reactivity (ASR) in North Carolina highway structures found a distinct difference in the extent of ASR in cores from different parts of two separate bridges which contained reactive aggregates.

In one bridge, differences were found to be directly related to the alkali content of the concrete. When the total equivalent alkali content of the cement was greater thatn 0.6%, deleterious expansion and significant cracking occurred. When the alkali content was less than 0.55%, no deleterious cracking occurred even though ASR was present. When the alkali content of the cement was less than 0.4%, essentially no ASR was found. More ettringite was found than expected in one location in concrete with very low alkali content (less than 0.4%).

In the other bridge, along the coast, the differences were due to the use of different coarse aggregates in the concrete, including a mixture of reactive an non-reactive aggregates along or in combination in varying amounts. An additional finding is that, due to the seawater exposure, the alkali content of the concrete is so high that ASR cannot be mitigated by controlling the alkali content of the cement. Ettringite, due to the seawater exposure, was found to have caused some cracking, but the damage was secondary in many cases and considerably less severe than that due to ASR.

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EXECUTIVE SUMMARY

General

A previous investigation of the extent of alkali silica reactivity (ASR) in North Carolina highway structures found a distinct difference in the quantity of ASR in cores from different parts of two separate bridges. In both structures, aggregates known to be reactive were used but some elements showed no signs of deterioration while other elements showed significant map cracking.

Cores were taken from both bridges by NCDOT. These cores were examined visually prior to additional testing. The dynamic modulus of all cores was determined using pulse velocity techniques. The splitting tensile strength of selected cores was determined, which also provided two approximately equal halves, split longitudinally.

One of these halves was used to determine the quantity of alkalies and, for some specimens, the quantity of cement in the concrete. The other half of the core was treated with uranyl acetate and examined under ultra-violet lamps. Image analysis techniques were used to determine the percent area of gel. Optical and scanning electron microscopes were used to examine both thin sections and fractured samples from selected cores. The residual expansion of selected cores was conducted to determine the potential for future expansion and to ensure that any potential reaction was identified.

Findings

In one bridge (83016), the differences between elements of the bridge were directly related to the alkali content of the concrete. The concrete was produced with a reactive phyllite but with two different cements. No deleterious ASR was found when the water soluble alkali content of the *concrete*, expressed as sodium oxide (Na_2O), was less than 1.5 kg/m^3. Additional analysis indicated that when the total equivalent alkali content of the *cement* was greater than 0.6%, expressed as sodium oxide, deleterious expansion and significant cracking occurred. When the alkali content of the cement was less than 0.55%, no deleterious cracking occurred even though ASR was present. When the alkali content of the cement was less than 0.4%, essentially no ASR was found. Potassium oxide appears to be more important in long term deterioration due to ASR than sodium oxide, at least for alkali reactive phyllite.

Slightly higher than expected quantities of ettringite were found in concrete with very low alkali contents (less than 0.4%). Minor cracking was associated with this concrete.

In the other bridge (15035), which is on the coast, the differences were due to the use of different coarse aggregates in the concrete. Reactive and non-reactive aggregates were used individually or in combination in varying percentages, probably due to blending at the concrete batch plant. An important conclusion of this study is that, due to the seawater exposure, the alkali content of the concrete is so high that ASR cannot be mitigated by controlling the alkali content of the cement. However, aggregate which may be classified as non-deleteriously reactive from petrographic analysis was not found to be reactive even in this severe exposure. Aggregate which had been shown to be reactive both in service at other sites and in laboratory tests was not deleteriously reactive in concrete with very low alkali contents (less than 0.4% total equivalent alkalies).

ASR in the concrete produced with low alkali portland cement may extend only a short distance (50 mm or 2 inches) in this bridge. Due to the cracking produced by the ASR, additional alkalies may accumulate at greater depths, eventually causing additional ASR. However, the process is likely to be relatively slow at this stage.

Ettringite in this bridge, also the result of seawater exposure, was found to have caused some cracking, but the damage was secondary in many cases and considerably less severe than that due to ASR.

Recommendations

These findings strongly suggest that optional AASHTO M 85 specifications regarding alkali content of cements are both needed and probably sufficiently restrictive to provide adequate protection against deleterious ASR in North Carolina for structures not exposed to external alkalies. A total equivalent alkali content of 0.55% of the cement, as sodium oxide, is a conservative estimate of the maximum alkali content permissible before additional measures are required. The study also found that additional preventive steps must be taken to protect concrete from ASR in concrete exposed to seawater, saltwater spray or significant deicing salts. That is, aggregates must be non-reactive or concrete must contain mineral admixtures in sufficient quantities to be effective in controlling ASR for concrete with salt water or significant deicing salt exposure.

More frequent inspections of structures with ASR is recommended, particularly of severely cracked elements directly exposed to water. It will probably be necessary to replace, repair or rehabilitate several elements prior to the end of the design service life. However, in many cases, the progression of ASR deterioration appears to have slowed significantly. Wholesale replacement, repair or rehabilitation is not indicated.

The minor cracking associated with ettringite found in one bridge suggests that additional information on secondary or delayed ettringite formation would be useful to the North Carolina Department of Transportation.

Investigation of Differences in Deterioration of Two North Carolina Bridges Due to ASR

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1. INTRODUCTION AND PURPOSE

1.1 Introduction

A previous investigation of the extent of alkali silica reactivity (ASR) in North Carolina highway structures found a distinct difference in the condition or extent of ASR in cores extracted from different parts of two bridges (1). In both structures, aggregates known to be deleteriously reactive were used. Since laboratory tests for ASR are typically severe, in order for results to be available in a relatively short time, the best method for confirming laboratory results is by comparison with long term performance of structures in service. The results from examination of concrete from these two bridges will assist in the development of fact-based material specifications.

1.1.1 Bridge #83016

In Bridge #83016, near Aquadale in Stanly county, parts of the deck, barrier rail and superstructure exhibit extensive map-cracking, however, other parts of the bridge deck and other members show no sign of ASR or limited potential for cracking due to ASR. Cores from the bridge had significant deposits of alkali-silica gel present in the deteriorated sections.

Exposure and service conditions are clearly the same for both the deteriorated sections and the sound sections of the bridge. The coarse aggregate in both sections was similar and was believed to be from the Gold Hill quarry, which is known to be deleteriously reactive. Therefore, differences in performance were suspected to be due to differences in paste composition; specifically, differences in alkali content in the cements used. The bridge was not apparently heavily salted, so external alkali exposure was expected to be slight.

1.1.2 Bridge #15035

In Bridge #15035 on US 70 along the coast past Morehead City in Cartaret county, differences in the conditions between sections of the bridge, while not nearly as conspicuous as in the Aquadale bridge, were nevertheless apparent. Tests in the previous investigation confirmed differences in behavior from concrete from different areas of the bridge. The aggregate used in this bridge was originally believed to be from the Princeton quarry, which is known to be deleteriously reactive. However, petrographic analysis indicated the aggregate was varied.

It was suspected that the differences in behavior at this site were due in large part to differences in exposure to external alkalies provided by the salt water environment. If this is the case, it should be possible to more accurately assess the effects of externally supplied alkalies on potential ASR. This is important not only along the coast from seawater exposure but in areas where pavements and bridges may be exposed to deicing salts. However, another factor in the differences in behavior could be related to differences in the aggregate. In this case, it was hoped that correlation of laboratory testing of various methods for identifying potentially reactive aggregates, currently in progress, with field results would be possible.

1.2 Problem Statement

The primary purpose of this research was to assess the differences in apparent ASR between different sections or elements of each of the two different bridges so that the effects of both internal alkalies (those supplied by the cement) and external alkalies (those supplied by either salt water or deicing salts) on alkali silica reactivity of aggregates known to be deleteriously reactive could be quantitatively determined. It was anticipated that a detailed examination of concretes in Bridge #83016 should provide a way to more accurately assess differences in ASR durability due to differences in cement alkali content, and that a detailed examination of concretes in Bridge #15035 should provide a way to more accurately assess differences in ASR durability due to differences in external (seawater exposure) alkali content, particularly with different aggregates.

This research therefore provides information to the NCDOT regarding limits on cement alkali content and on the role which external alkalies may be playing in ASR in North Carolina highway structures. The products of the research should facilitate decisions on alkali contents of cement which are based on firmly established facts derived from field studies and may be used to help develop or modify portland cement concrete specifications.

2. MECHANISMS OF ASR

This chapter provides a quick overview of ASR. It was drawn from the previous report to NCDOT (1) and a paper by Leming (2). It is included for completeness for the reader with limited familiarity with ASR.

Silica gel was first identified in material exuding from cracked concrete over fifty years ago, yet the reaction is neither completely predictable nor understood even today. This chapter offers a quick review of the mechanisms of the alkali silica reaction (ASR) and control of potential ASR problems through concrete specifications as understood today.

2.1 Silica

Silica (silicon oxide) exists in aggregates in a number of forms and in combination with a number of other elements. Completely crystalline silica, as in quartz, is chemically and mechanically stable. Quartz is relatively impermeable and reacts only on the surface of the crystal where the silicon and oxygen bonds are broken. Since the surface area per unit volume of most quartz is very low, reactivity is also very low. However, amorphous, that is, non-crystalline or "glassy", silica is more porous and very reactive chemically. The less crystalline the silica is, the more reactive it is. Silica which has melted and cooled quickly with little or no recrystallization, such as in certain volcanic aggregates, can be very reactive in an alkaline environment.

Another contributing factor to reactivity involves silica crystals which are either very small or have been highly stressed. Crystalline silica which has been transformed by heat and pressure may have a large amount of strain energy stored in the crystal structure. The higher energy makes the silica more reactive. The "strained quartz" found in many metamorphic aggregates means these aggregates are potentially susceptible to deleterious alkali silica reactivity, although the rate of reaction is generally much slower than with aggregates chiefly composed of glassy silica. Another problem may exist with aggregates in which the silica is primarily crystalline. In aggregates in which the silica exists as very fine crystals (crypto- or micro- crystalline) very high surface energies of the crystals will contribute to alkali sensitivity.

Therefore, the potential reactivity of an aggregate is partially a function of the degree of crystallization of the silica in the aggregate, and the amount of energy stored in the crystal structure, either due to a large quantity of micro-crystalline silica or a high strain energy stored in the crystals, or some combination of these factors. The surface area per unit volume of the reactive silica will also affect the rate of reaction since a larger surface area of reactive silica will have more opportunity to react. Obviously, the reactivity of the aggregate is also affected by the silica content. However, due to the other factors, the relationship is not straight forward.

Another type of alkali-aggregate reaction is alkali-carbonate reactivity (ACR) in which alkalies react with argillaceous dolomites. While this problem exists in Virginia and Tennessee, and deposits of dolomites are known to exits in western North Carolina, it does not appear to be a problem with any commercial quarries in North Carolina. The mechanism of ACR is quite similar to ASR; the argillaceous, or clayey, components of the aggregate contain microcrystalline silica which reacts to from alkali-silica gel.

2.2 Paste Characteristics

2.2.1 Cement

Hydrated portland cement is a very alkaline material. The pH of water in the pores is typically greater than 12. A moist, alkaline environment such as concrete provides an ideal place for noncrystalline, crypto-crystalline or strained silica to react. However, not all alkalies are equal in their effects. Calcium compounds react with glassy silica to form calcium silicate hydrate, commonly abbreviated C-S-H, a poorly crystalline material which can occur in several forms and chemical compositions. [Note: The expression C-S-H has its roots in conventional cement chemistry notation where C stands for calcium oxide, or CaO, S for SiO_2, H for H_2O, etc. C-S-H is approximately $C_3S_2H_3$, or $3\ CaO \cdot 2\ SiO_2 \cdot 3\ H_2O$.] C-S-H is, of course, the primary compound formed by hydration of portland cement, and accounts for many of the characteristics of concrete.

C-S-H is relatively stable. Although drying will cause some shrinkage, and re-wetting will cause some expansion, the volume stability of the C-S-H is very good compared to the volume stability of most alkali silica gels. Alkali silica gels with high sodium contents, for example, can swell and shrink a great deal compared to C-S-H.

All commercially manufactured portland cements contain some quantity of sodium and potassium, and C-S-H will actually be a combination of calcium and sodium/potassium silicate hydrate, or C-Na/K-S-H. The more calcium, the more stable the gel; the more alkali, the less stable the gel and the more likely the gel will be deleteriously expansive. Sodium and potassium ions are more mobile than calcium and will react with the silica in the aggregate unless they are tied up first by reacting with the silica from pozzolanic sources.

2.2.2 Pozzolans

A pozzolan contains reactive silica but is not cementitious in itself. Natural pozzolans, such as volcanic ash, were used by the Romans. Modern, commercially available, pozzolanic sources include Class F fly ash, ground granulated blast furnace slag or silica fume, which is also called fumed silica, microsilica or condensed silica fume. The silica is reactive because it has been melted and cooled quickly without crystallizing. The reactive silica in the pozzolan will, in the presence of moisture and calcium hydroxide, form C-S-H. Calcium hydroxide is available in the paste as a by-product of the hydration of portland cement.

Pozzolans act as "sacrificial silica" to react with the alkalies in the concrete before the silica in the aggregate can react. The gel produced by reaction with pozzolans is more stable due to the higher average calcium content of the paste and due to the smaller size and greater dispersion of the gels sites through the concrete. Since the gel is more dispersed, it is less likely to create deleterious expansive stresses even if it does swell slightly.

2.3 Mechanisms of Alkali Attack

Alkalies may attack reactive silica in aggregates by either or both of two mechanisms. With either mechanism, the first step is breaking up the existing silica structure. Alkali hydroxides attack the silicon-oxygen bonds opening the crystal to accommodate alkali ions and water. The hydroxyl ion breaks the bonds permitting additional reaction with the alkali ions.

2.3.1 Highly Reactive Aggregates

Very reactive aggregates, such as opal, chalcedony, chert or glassy volcanic aggregates, may create a large quantity of expansive gel which can cause cracking of the concrete or the aggregate or both. This gel is hygroscopic which means it wants to absorb water. The uptake in water causes the gel to swell. While the expansive forces are not very high, with formation of enough gel, there is sufficient force to cause the concrete to fail in tension and crack, and for the concrete itself to expand. The cracks will then permit more water to penetrate to the gel, which creates more swelling and, therefore, more cracking. Even without cracks, osmotic forces created by the tendency of the gels to take up water can create high stresses in pastes. Gels with a high sodium content are very hygroscopic.

This type of reaction occurs fairly rapidly and map cracking in the member may appear within several years. Rapid deterioration of the member or structure can occur. In a few situations, however, reaction has occurred with limited reduction in structure serviceability. In some dams which contain reactive aggregates, the weight of the dam itself was sufficient to overcome the expansive forces and to actually inhibit the formation of gel.

2.3.2 Moderately Reactive Aggregates

The other mechanism of deterioration is more typical of metamorphic aggregates, in which the silica is only moderately reactive, or in aggregates which do not contain a large quantity of reactive silica. In this case, the low quantity of reactive silica at the surface of the aggregate means that a more stable gel will be created due to the presence of significant quantities of calcium hydroxide at the paste aggregate interface. However, if the alkalies penetrate the aggregate and move into a region away from the calcium hydroxide, a less stable, low calcium gel is created. This softens the outside or rim of the aggregate.

The effect on the mechanical properties of the concrete are typically much slower and may be less obvious with this reaction. Expansion may be limited but since the aggregate can carry less of the load on the concrete, stresses in the paste are higher and cracks can develop through the paste and through the softened aggregate particle. Aggregates with seams of reactive matter may also behave in this fashion. Although service life may be longer compared to the highly reactive silica aggregates, some deterioration may be expected eventually, depending on moisture availability.

The role of silica content may be understood by considering a given alkali content in the concrete containing aggregate composed of various quantities of reactive silica. At very low reactive silica contents, not enough gel is produced to cause deleterious expansion or extensive softening. At very high reactive silica contents, the expansive gel may be dispersed over a large volume and the expansive forces generated therefore may be small enough to be accommodated by restraint of the concrete or by creep. Additionally, by being dispersed, the local calcium hydroxide content may be adequate to stabilize the gel. At some intermediate silica content, the worst possible combination of reactive silica for a given alkali content, termed the "pessimum" silica content, may produce rapid or extensive deterioration. One of the benefits of using a pozzolan is to increase the reactive silica content substantially above the pessimum content.

2.4 Consequences of ASR Attack

2.4.1 General

Typical deleterious features of ASR in concrete structures include cracking and expansion, with consequent misalignment of elements, frequently accompanied by the presence of gel in fractures or aggregate particles within the concrete. Reaction typically takes between 5 and 12 years to develop, though there are many exceptions.

Expansion due to ASR may cause cracking in both the cement paste and the reacting aggregate particles. Cracks in reacting aggregate particles may create cracks which propagate into the surrounding cement paste. If expansion is significant, the cracks can travel a considerable distance and can interconnect reacting aggregate particles. The cracks can sometimes pass through unreacted aggregate particles, but usually pass around them. The gel product is generally very mobile and may be found far from reacting particles, filling or partially filling voids, including entrained air voids, and occasionally filling cracks in unreacted aggregate particles. Obviously the cracks permit rapid infiltration of moisture.

Due to the relatively low swelling pressure of the gel, restraining pressures on the order of 2 MPa (approximately 300 psi) are sufficient to preclude expansion, but localized stresses at aggregate edges may easily exceed this value even when nominal average stresses are low, due to stress intensity concentration effect at crack tips. Therefore microcracks may exist in concrete which does not show any appreciable macroscopic deterioration.

Features visible generally associated with the presence of ASR include:

(1) Internal fracturing of aggregate particles, especially when the rock types are recognizably reactive or potentially reactive.

(2) Crack patterns which appear to be directly related to certain aggregate particles, or when the cracks are filled or lined with gel.

(3) Darkening of cement paste around aggregate particles or voids, or along

borders, perhaps (but not invariably) caused by gel impregnation.

(4) Dark rims or, alternately, white deposits immediately beneath the surface of freshly fractured aggregate particles.

(5) The formation of damp or "sweaty" patches on the surfaces of core samples. These are especially noticeable as a wet core dries.

2.4.2 Pavements and Bridges

Either expansion of concrete pavement or reduction in stiffness can cause cracking which creates higher internal stresses in pavements since the tire load cannot be spread over as large an area. This can result in additional cracking of the pavement, which can then lead to more water penetration which softens the subgrade unless *well* drained. The reduction in support, combined with higher pavement stresses and reduction in load transfer across cracks increases subgrade stresses. These factors can lead to differential settlement and a reduction in evenness of the surface as well as a more rapid deterioration of the pavement itself.

In reinforced concrete members in bridges, the greatest concern may be the increase in corrosion of the reinforcing steel due to the presence of large cracks. However, the softening of the concrete may also lead to reduced stiffness of the reinforced member resulting in greater deflection and increased stresses. In both cases, expansion of the member may close up joints such that subsequent heating of the member in the summer creates expansions which can no longer be accommodated and the concrete may rupture.

Due to the effects of restraint, cracks in pavements tend to be longitudinal at first with transverse, bridging cracks appearing later. In reinforced concrete members, restraint tends to create cracks parallel to the main reinforcement. Again, bridging cracks frequently appear later to create a map-cracked appearance but the main cracks can be quite wide.

Cracking is common in all structures and should not necessarily be taken to indicate problems related to either durability or structural capacity. For instance, barrier rails will typically exhibit vertical cracks at the half and quarter points or at the third points. This cracking can be related to both restrained expansion (on the bottom of the rail) during temperature cycling or to deflection, particularly of bridges with steel beams, with truck traffic. In addition, map cracking on the surface may be indicative of deterioration due to sulfate attack (formation of ettringite), frost attack, or, in some cases, fatigue failure. The cause of map cracking can only be conclusively identified through detailed examination in the laboratory of specimens removed from concrete in question.

2.6 Interaction with Other Mechanisms of Deterioration

Freezing and thawing damage and secondary or delayed ettringite formation can also cause map cracking which is very similar in appearance to ASR. Cracks associated with ettringite formation are frequently filled with products which can superficially resemble ASR. Frost damage and ettringite deterioration can often act in conjunction with ASR, causing difficulty in assessing the proximate cause of deterioration. Iron sulfides in the aggregate are typically associated with copious staining and can also cause expansion and map cracking,

ASR can create secondary effects in the frost resistance of concrete by filling the entrained air voids with gel, although to the knowledge of the Principal Investigator, no case has been reported in which there was sufficient alkali silica gel produced to create difficulties with the frost resistance of the concrete without having also created problems due to the ASR directly. However, ASR can reduce the frost resistance of concrete, especially concrete with only a marginally effective air void system.

Ettringite induced damage is also a paste phenomenon. Substantial "secondary" ettringite formation is normally associated with external sulfate attack, and "delayed" ettringite has been found in some precast elements exposed to elevated temperatures in curing. Ettringite is found in virtually all concretes and its presence should not be taken to indicate that deleterious expansion has or necessarily will occur in a specific member.

Significant secondary or delayed ettringite formation in hardened concrete will cause map cracking which superficially resembles ASR. The map cracking can develop into more extensive cracking leading to loss of serviceability of the member. Ettringite formation can also interfere with the frost resistance of concrete by filling entrained air voids.

It is typically difficult to distinguish between deterioration due to ettringite and ASR in the field. Microscopic examination and chemical analysis are required to confirm whether ettringite or ASR is the primary cause of damage in concrete. Ettringite is a strongly directional, crystalline product normally easily differentiated under a microscope from alkali silica gel. However ettringite has been reported to crystalize in micro-crystalline form which can easily be mistaken for alkali silica gel under an optical microscope and which is resolved only under a scanning electron microscope (SEM)(3).

It is common to find alkali silica gel and ettringite commingled in concrete, particularly in the transition zone. This is probably related to the fact that much of the soluble alkali in portland cement is in the form of alkali sulfates and ettringite formation requires sulfate ions.

For this reason, even when the ettringite can be resolved under an optical microscope it is strongly recommended that chemical analysis be conducted to determine the composition of the various reaction products present.

Determining the proximate cause of damage when multiple, possibly inter-related factors are present is subjective to a certain extent. The presence of one factor may create conditions favorable for the formation of the other by cracking the concrete and allowing the ingress of water and water soluble salts. Cracks formed by one mechanism may permit the growth of another product in the cracks, complicating the determination of the cause of the deterioration. In addition, cracks caused by other mechanical loads or environmentally induced stress can also complicate the analysis. For example, warping and curling stresses in pavements due to thermal or moisture gradients can, particularly in conjunction with wheel loads, cause cracks which can be preferentially filled by ettringite or alkali silica gel.

A key to assessing the controlling factor(s) in deterioration is whether the damage appears to be associated primarily with the paste, as in frost damage and ettringite formation, or with the aggregate, as in ASR. While the presence and quantities of various compounds give an indication of the cause of deterioration, the lack of the compound is not necessarily indicative of the lack of problem. For example, the gel product in ASR may migrate through cracks, especially in thinner members, leaving relatively little gel in the concrete. Therefore, assessment of the probable cause or causes of deterioration should be conducted only after thorough examination by an experienced investigator using both microscopic techniques and chemical analysis.

2.7 Managing Potential Alkali-Silica Reactivity

The presence of ASR is not necessarily deleterious. However, in sufficient quantities, it can cause excessive expansion and disruptive cracking of the concrete. Managing the potential for deterioration due to ASR is similar to managing any other risk. Different alternatives or strategies have different costs and, most importantly, different levels of effectiveness. By selecting of an appropriate alternative the consequences of damage to a given structure may be minimized. There is neither a need nor the ability to eliminate ASR in all cases, but it can be controlled adequately and economically, particularly in the Carolina's, by intelligent choice and control of specifications.

In general, deterioration due to ASR may be controlled by one or more of the following options:

(1) keeping the concrete dry,

(2) use of a low alkali cement,

(3) use of a non-reactive aggregate, or

(4) use of an adequate quantity of an effective pozzolan.

In addition, lithium compounds have been known to control deleterious expansion due to ASR since the 1950's.

2.7.1 Service Exposure

ASR cannot occur without moisture and any concrete which is dry in service needs no additional protection. For example, elevated, interior slabs which will not be exposed to moisture after initial curing and which have been allowed to dry thoroughly do not require any special precaution. Concrete is not likely to suffer damage from ASR if it is never exposed to a relative humidity of more than 80 to 90%. This is not, of course, a practical consideration for pavements and bridges but may contribute to girder durability.

AASHTO pavement design guidelines place North Carolina in Region II which is a wet region. Pavements, approach slabs, abutments and other members in contact with the ground will clearly have adequate moisture available for the alkali silica reaction to proceed. Starke (4) and Janssen (5) report that pavements routinely have sufficient moisture to permit alkali silica reaction to proceed. Bridge decks and barrier rails will be drier, on average, therefore slowing the rate of deterioration, but the SHRP 343 report (4) indicates that moisture will be adequate for ASR in these members.

While pavements can be sealed on the surface with a waterproofing compound, moisture is available from the subgrade. In addition, "non-breathable" waterproofing compounds, such as high strength epoxies, will cause delamination of the sealer when moisture vapor cannot escape from the surface. Further, abrasion from traffic will eventually wear through the sealer, requiring periodic replacement.

For elevated sections of bridges, where the only source of moisture is precipitation, sealing the surface may be more effective. However, sections cast against the earth or where moisture is accessible will experience ASR, although typically at a somewhat slower rate.

2.7.2 Cement Controls

Limiting the alkali content in cement was one of the first strategies developed to control ASR. While generally successful, a number of potential problems exist with this approach.

Deleterious reactivity is reduced when low alkali cements are used. This is especially true with aggregates which are only moderately reactive. However, external alkalies, such as from deicing salts, or salts in seawater or certain waste waters, can invade even good quality concrete and cause local concentrations of alkalies which are sufficiently high that, in the presence of moisture, ASR can develop. In addition, the presence of chlorides may increase the rate of reaction.

Even when external alkali exposure is not anticipated, the maximum "safe" level of alkalies is subject to debate. The optional requirements of AASHTO M 85 require the total equivalent alkali, which is calculated as all of the sodium and about ⅔ of the potassium (both expressed as oxides), to be less than 0.6% of the mass of the cement for the cement to qualify as "low alkali."

A practical problem with trying to use portland cement to control ASR deterioration is that the alkali content of portland cement is controlled by the parent rock used in the raw feed and the sulfates used to regulate set. The alkali content of the raw materials used can vary significantly. However, cement manufacturers cannot consistently and economically reduce the average alkali content for a given source below any arbitrary level. In many parts of the country low alkali cement is not available in quantities sufficient to meet all demand.

Further, the use of percent alkalies to control reactivity is also challenged. The British Cement Association (6) suggests that an alkali content less than 3 kg/m^3 (somewhat over 5 pounds per cubic yard) is essentially innocuous for aggregates which do not contain opaline constituents. Another source (7) states that an alkali content greater that 4 kg/m^3 should be considered a high level. The rationale behind this type of specification is clear. A concrete mixture with 420 kg/m^3 of cement (about 7.5 sacks) with 0.4% total equivalent alkali will have the same alkali content as a mix with 280 kg/m^3 (about 5 sacks) of cement with 0.6% alkalies.

On the other hand, Johnston (8) argues that 0.05% alkalies by weight of the concrete is a limit for innocuous aggregates, while alkali contents greater than 0.10% can promote rapid deleterious expansion for reactive aggregates. These are considerably lower limits and correspond to 1.2 and 2.3 kg/m^3, respectively for a conventional concrete. A concrete mixture with 420 kg/m^3 of cement (710 pcy) with 0.6% alkalis will have approximately 2.5 kg/m^3 contributed by the cement alone. The same mix with 0.4% total equivalent alkalies will have 1.7 kg/m^3, not dramatically different from the limits proposed by Johnston (8).

However, these values are for acid soluble alkalis. The water soluble alkali content is considerably lower. The use of water soluble alkalies is preferred when testing hardened concrete in order to avoid alkalies contributed by the aggregate.

Water soluble alkalies in portland cement are predominately composed of the alkali sulfates in the cement. The remaining alkalies are tightly bound and will be released with acid extraction. Lea (9) reports that water soluble alkalies in cement will typically be about a quarter to a third of the total, acid soluble alkalies, but that percentage can exceed sixty percent. The ratio of water soluble alkalies to total (acid) soluble alkalies in the portland cement is partly a function of the total alkali content, with low alkali content cements typically having a higher average proportion of water soluble alkalies. The ratio of water soluble to acid soluble alkalies for a sample of portland cement from the Blue Circle Cement plant in Harleyville, South Carolina, with a 0.22% total equivalent (acid soluble) alkali content was found to be 82%.

In concrete, differences in solubility are somewhat more complex. Most of the alkalies will eventually be released as cement hydration progresses. However, some of the alkalies will be tied up in hydration products and will not contribute to ASR nor will they go into solution easily. Inderwick (31) notes that "active alkalies", that is, those which contribute to reactivity, constitute between 71% and 93% of the total, acid soluble, equivalent alkalies.

2.7.3 Aggregate Controls

The use of a non-deleteriously reactive aggregate is another possibility for eliminating ASR related deterioration. However, any silica based aggregate could, under the right conditions, show signs of alkali silica reactivity. In addition, non-deleteriously reactive aggregate may not be economically available. Therefore, two issues become important. First, how is deleterious reactivity determined and second, how deleterious does it have to be? These issues are also related to the concerns regarding the "safe" level of alkalies in the concrete and involve a review of the test methods used.

Reliable tests to determine whether an aggregate is deleteriously reactive have been under development since the problem was first identified, and continue to the present. ASTM C 1260 was just recently added to the list of ASTM standard test methods, for example.

A quick chemical test (ASTM C 289) was developed to provide a rapid assessment of aggregate and is conducted by determining how much silica will dissolve in a hot, highly alkaline solution overnight. This method has not been found to be entirely reliable due the presence of a number of false positives and false negatives in the data, that is, some aggregates pass which should not and some do not pass which should.

The standard petrographic method (ASTM C 295, which is a guide rather than a standard test method), in which thin sections of aggregate are examined under the microscope, is limited to the extent that it can only identify minerals or aggregates which have been shown to be reactive in practice. The practical impact of this is that few aggregates can be definitively classified as deleteriously reactive unless they already have service records. Many aggregates of practical interest can only be classified as "potentially" deleteriously expansive since they contain minerals which have been known to be reactive in other situations.

The mortar bar test (ASTM C 227) is conducted on aggregate which has been ground to a specific gradation resembling sand. Mortar bars containing an unrealistically high quantity of an alkaline portland cement are cast and kept warm and moist for periods of a least three months to six months and frequently in excess of twelve months. Acceptance according to ASTM C 33 is based on expansion of the mortar bars at three and six months. This test method suffers from two criticisms. The first is that it simply takes too long. The second is that it also appears to have a number of false negatives, particularly with aggregates which are only slowly reactive.

A test developed in South Africa (ASTM C 1260) is also a mortar bar test, although conducted at a higher temperature (80° C) with the bars stored in a highly alkaline environment. This test has shown considerable promise due to a much shorter time period for testing (14 days) and it also seems to be a better indicator of the reactivity of "slowly" reacting aggregates. However, there is still concern that the test does not adequately identify marginally reactive aggregates. Aggregate with an expansion less than 1000 microstrain ($\mu\varepsilon$) (0.1%) are considered innocuous while aggregate with an expansion greater than 2000 $\mu\varepsilon$

(0.2%) are considered potentially deleteriously reactive. For aggregate with an expansion between 1000 and 2000 $\mu\varepsilon$, the test is considered inconclusive.

2.7.4 Pozzolans

Due to the difficulty in determining reactivity of aggregate in all cases, the difficulty in obtaining low alkali cements in many locations and the uncertainties of exposure, many specifiers have begun to require the use of silicious mineral admixtures such as Class F fly ash, ground granulated blast furnace slag (GGBFS), or silica fume in order to control deleterious expansion. The quantity required of these products are usually determined by ASTM C 441, Standard Test Method for Effectiveness of Mineral Admixtures or Ground Blast-Furnace Slag ion Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction. However, C 441 is conducted using pyrex glass as the aggregate, so the results are very conservative and do not take into account the differences in aggregate used in a particular mix. While the requirements will vary widely depending on the specific characteristics of the mineral admixture, a generally conservative guide is:

- Class F fly ash will generally provide adequate control from deleterious expansion if used in quantities exceeding 30%; Class C fly ash will not generally provide protection and may contribute to ASR,

- GGBFS will generally provide adequate control from deleterious expansion if used in quantities exceeding 50%, and

- silica fume, or microsilica, will generally provide adequate control from deleterious expansion if used in quantities between 7% and 10%, when used in slurry form; the use of "densified" silica fume is not recommended to control ASR.

The costs of these different alternatives can be significant. In addition, costs and availability can vary significantly depending on the location and market. However, in many locations, the cost of Class F fly ash is typically about a third of the cost of portland cement. Where available, GGBFS may be about half to three quarters of the cost of portland cement. Silica fume can be very expensive, particularly since a high range water reducer, or superplasticizer, is also generally required due to the high water demand of silica fume.

Due to limitations in test methods and interpretation it would appear difficult, using currently accepted test procedures, to determine absolutely whether an aggregate is going to be reactive under the conditions encountered in service, and how effective any potential remedies might be. However, due to the extremely conservative nature of C 441, many Engineers feel comfortable with the use of mineral admixtures in quantities satisfying this test procedure.

3. RESULTS FROM PREVIOUS NCDOT RESEARCH

A previous study for the North Carolina Department of Transportation examined the extent of ASR in North Carolina (1). During the investigation, two bridges were identified in which considerable variation in the extent of ASR was found. The bridges, on US 70 along the coast (#15035) and at Aquadale (#83016), showed a difference in behavior depending on location within the structure. Analysis of variance (ANOVA) found that the coefficient of variation (CV) in residual expansion between duplicate samples was less than 10% for cores tested from all structures except these two. In these two bridges, the CV was in excess of 40%, indicating a high probability that the concrete in the companion cores was dissimilar.

3.1 Bridge #15035

Bridge #15035 is a two lane bridge, carrying two way traffic on US 70 over Ward Creek, along the coast near Otway, past Morehead City, in Carteret county. The bridge was constructed in 1977. Ward Creek contains salt water. The salt spray and low clearance of the bridge over the water also provide high moisture and readily available alkalies. This site was initially selected for investigation due to the presence of cracking which suggested ASR and because it reportedly contained coarse aggregate from the Princeton quarry, which was alkali reactive. In addition, because of the many structures in North Carolina exposed to salt water and salt spray, it was important to include a bridge along the coast in the investigation. This bridge was one of several bridges in the area built under the same contract.

Construction data were limited due to the age of the structure. The source of cement was believed to be Dixie cement. Dixie cement used at this location would probably have an average total equivalent alkali content of 0.56% as sodium oxide. This cement has been reported by NCDOT to have an alkali content as high as 0.75% and as low as 0.27%, with 90% chance of falling between 0.71% and 0.41%. Therefore this cement could be either low alkali or not, according to AASHTO M 85 optional specifications. The source of sand used in the structure was not known.

During the site visits for the previous investigation, significant differences in the extent and the nature of cracking were noted between several locations. Corrosion related cracking has been discussed in the previous report (1), but included cracking in the guardrail base and one deck slab. The abutment walls exhibited longitudinal cracking along the top and some map, or block, cracking on the face. The pile caps appeared to be in generally good shape, but there was directional cracking in a few locations.

Portions of the seawall exhibited significant deterioration, at least some of which was related to environmental exposure. While not a critical issue in terms of immediate bridge safety, these elements had the most severe exposure to brackish water, moisture content and wetting and drying, so they are important in terms of concrete behavior.

Cores were obtained from several locations, including areas with cracking and areas which appeared sound, for examination in the laboratory. The average water soluble alkali content of the concrete was found to be 5.89 kg/m^3, which was the highest value encountered in the previous research project and definitely constitutes a high alkali content. A slight alkali gradient was found, with very high alkali contents at the surface (5 to 10 mm). However, a relatively constant alkali content was found at deeper sample points. The alkali content is therefore almost certainly due to the seawater exposure.

Petrographic examination of a core from the base of the guard rail indicated that the coarse aggregate used was a combination of schist, gneiss and metaquartzite. This composition is different from aggregate obtained solely from the Princeton quarry. No indication of ASR and no unusual quantity of ettringite were found. No cracking was detected in the sample examined. The coarse aggregate in the core used for petrographic examination appears to be non-reactive. This finding was very surprising, given the presumed source of coarse aggregate and the service exposure. .

The air content of the hardened concrete was satisfactory. Mechanical properties indicated that the concrete tested was sound, although there was some variability. The concrete had a moderate permeability. Cores from this site had an expansion of 700 microstrain for one core and under 500 microstrain for the other core stored in the alkaline solution and just under 400 microstrain for the core stored in water at 90 days. Both of the cores stored in the alkaline solution were taken from the seawall. The 700 microstrain expansion indicates the concrete is likely to undergo alkali silica reactivity but that future expansion may not be rapid or extensive.

ANOVA indicated that the cores immersed in the alkaline solution have a different response to alkalies. The rates of expansion were different and the "reactive" core was continuing to expand at the end of the test while the other had stabilized. The findings of the petrographic report are consistent with the expansion in water and with the residual expansion test of the "non-reactive" core, but not with the remaining core. This also indicates there are significant differences in the concrete cores tested.

Additional scanning electron microscope examination of samples from the core with higher residual expansion was conducted after the residual expansion test was completed. Since this structure was exposed to seawater, a Type II cement was probably specified. This could help account for the lack of ettringite noted in the petrographic examination. However, the lack of ASR found in the petrographic examination was surprising and the SEM examination was conducted to help verify that the expansion of this core was due primarily to ASR.

While examination of the core revealed the presence of ettringite, there was no indication that ettringite formation was abnormally high nor did it appear to have caused any significant cracking. The ettringite was invariably located in voids, particularly air voids, where, if present, it frequently filled a considerable volume of the air void.

Alkali silica gel was found in substantial quantities. Significantly, the form of the alkali silica reaction product found was frequently partially crystallized, with the laminar and rosette form found in several locations. In one instance, the rod-like form of alkali silica reaction product was found. This structure has been reported to be commonly associated with the spongy form of alkali silica gel, and would imply that additional ASR products were formed from pre-existing gel during the residual expansion test.

3.2 Bridge #83016

Bridge #83016 is near Aquadale on NC 138 over Long Creek in Stanly county. It was constructed in 1980. This site was selected because the aggregate was the same as in the Bridge #8305, the James Garrison bridge, and is only a year different in age.

During the first site visit, the inspection team was prohibited from inspecting the bridge at close range by law enforcement officials in pursuit of a suspect in the area. Due to the need to visit other sites, selection of coring locations was left to the discretion of the coring crew. After residual expansion testing and petrographic examination of the cores, it was clear that the behavior of concrete from this bridge and the James Garrison bridge were dissimilar. Therefore, another site visit was scheduled to examine the bridge.

The inspection during the second visit revealed that the bridge had considerable map cracking, especially in the deck, but also in the barrier rails and the approach and departure slabs, although barrier rail cracking was not as severe. The ends of the bents or pile caps also showed significant pattern cracking suggesting ASR damage. Importantly, however, there were several large areas which did not have any substantial map cracking. Subsequent inspection at a period of low water found significant cracking in one of the footings.

Several of the deck sections were sound and showed no appreciable cracking. The color of these areas appeared to be different from the cracked areas, indicating that different cement sources or quantities were probably used. Discussions with NCDOT personnel indicated there may have been a change in contractors on the job.

The coarse aggregate used in this structure was reported to have come from the Gold Hill quarry, which is known to be very reactive. The source of sand was not known. The source of cement was not known but, in that location, may have been Roanoke cement. This cement has an average equivalent total alkali content of 0.67%, but can be higher, so it is not a low alkali cement.

The first set of cores, delivered to NCSU for testing, came from the sound concrete.

The results presented below are based on cores from the low reactivity section.

The average water soluble alkali content of the concrete was found to be 0.94 kg/m^3, which is only about 60% of the water soluble alkalies found in the James Garrison bridge. No alkali gradient was found, indicating little deicing salt usage.

Petrographic examination indicated that the coarse aggregate used was a phyllite, consistent with coarse aggregate from the Gold Hill quarry. No indication of ASR was found and only minor quantities of ettringite were noted.

The air content of the hardened concrete in the first set of cores was satisfactory. Mechanical properties indicated that the concrete tested was in excellent condition. The permeability of the concrete was good.

Cores from this site had almost identical average expansions in water (570 microstrain) and in the alkaline solution (580 microstrain) at 90 days and should therefore be considered only marginally expansive at most. However, analysis of variance of the residual expansion indicated that there was a significant difference between the expansions of the companion cores. This implies that the cores represented sections with fundamentally different responses.

Since one of the cores for the residual expansion test came from the deck and the other came from the approach slab, this could simply indicate the relative difference in degree of cracking between the two sections. However, on other structures, this type of difference in residual expansion was not found even with similar differences between the cores. It is more likely that some difference in composition existed between the cores.

The rate of expansion of cores in both exposures was steady and had not stabilized by the end of the test, although it was slow. These data indicate that future expansion will likely occur, but that the addition of deicing salts will have only a small effect on the results. These data also suggest that future expansion due to ASR will be slow.

Given the high reactivity of the aggregate, the small difference in expansion between cores in the water and the alkaline solution was surprising. The limited residual expansion values are consistent with the findings of the petrographic report, however, indicating little, if any, deterioration to date due to ASR.

Due to these inconsistencies, another set of cores were taken from a section with definite map cracking. The second set of cores exhibited extensive ASR. These cores also contained phyllite, again very closely resembling that in the James Garrison bridge, and consistent with the Gold Hill quarry.

The difference in performance between certain sections of this bridge is notable. If it is assumed that cores from the deteriorated sections would behave essentially as the cores from the James Garrison bridge, which is a reasonable assumption, and it is recognized that there is no difference in exposure between the different sections on the Aquadale bridge, it must be clear that another mechanism is at work significantly slowing progression of the alkali silica reactivity in at least certain sections of the Aquadale bridge.

The lower alkali content found in the sound sections of the Aquadale bridge compared to that found in the James Garrison bridge clearly contributed to the improvement in durability. Further, cracking in the James Garrison bridge may have been exacerbated by lower air contents.

4. RESEARCH METHODOLOGY

4.1 Research Outline

The project included an examination of the bridges, obtaining and inspecting cores from the bridges and determining the mechanical properties and residual expansion of the cores. An essential part of the research was microscopic and chemical analysis of the cores.

The mechanical properties tested included the dynamic modulus of all cores and splitting tensile strength of selected cores from locations within the bridge which appeared to have different ASR deterioration. The split cores were also used for uranyl acetate (UA) treatment and for chemical analysis (one split face for UA and one for chemical analysis).

Residual expansion, or immersion testing, of cores in water and in alkaline solution at 100 °F were conducted to determine the differences in possible future expansion of cores from various locations within a bridge. Following the immersion testing, the cores were split and treated with uranyl acetate. These were qualitatively assessed for comparison with the residual expansion values.

Limited, electrically based, rapid chloride permeability tests (RCPT) were conducted to evaluate the relative permeabilities of cores. However, previous work, both in the lab and in the field indicated that concrete with ASR will frequently have a very low RCPT reading (1). Therefore, only a few tests were conducted.

A major effort in this project involved microscopic and chemical analysis of cores. Optical microscopy was used to characterize both the condition of the concrete and the components including the paste and aggregate phases. Microscopic examination was required to detect the presence and contribution to deterioration of factors including ASR, ettringite and frost. Much of the optical microscopic work was conducted on fractured surfaces with a stereoscopic microscope using reflected light. In addition, work was conducted on thin sections from selected specimens using a petrographic microscope (transmitted light).

Scanning Electron Microscopic (SEM) examination was conducted including energy dispersive X-ray analysis (EDX). The particular SEM used has enhanced EDX capabilities permitting conclusive identification of ettringite and semi-quantitative analysis of other features, including paste, aggregate and gel.

Chemical analysis included determination of water soluble alkali contents of the concrete by Atomic Absorption spectrophotometry (AA). Uranyl acetate treatment followed by UV inspection and image analysis were used to estimate the quantity of gel produced.

4.2 Purpose and Methodology of Tests

4.2.1 Mechanical Properties

Testing methods for determining mechanical properties of cores tend to provide optimistic results due to biased sampling. Cores are typically removed from concrete which is fairly sound and not extensively cracked, in order to be useful in testing procedures. This selection methods leads to an optimistic assessment of the condition of the concrete. However, these data can provide a general indication of quality of the concrete in place.

4.2.1.1 Dynamic Modulus

The dynamic modulus of elasticity is non-destructive and can provide an indication of the extent of cracking in a concrete specimen. A deteriorated specimen will have a reduced dynamic modulus. Changes in dynamic modulus can provide an indication of changes in the internal structure of concrete before visible cracking has taken place (11). Other research has suggested a correlation between dynamic modulus and degree of deterioration of concrete from the effects of ASR (12). However, dynamic modulus in the previous NCDOT research was found to be significantly reduced only after extensive expansion of the specimens (11).

The dynamic modulus of concrete cores was determined using pulse velocity measurements. The cores were trimmed, the length of the core measured, and the specific gravity determined similarly to the method used in AASHTO T 85. The pulse velocity was determined in accordance with ASTM C 597 using a calibrated Pundit™ gage.

4.2.1.2 Strength

Splitting tensile strength was determined for several reasons. Typically, compressive strength is insensitive to the effects of cracking within the concrete because it pushes the cracks back together. In addition, ASR causes debonding between the coarse aggregate and the paste. The splitting tensile test provides the best estimate of the extent of change in mechanical properties and degree of debonding. However, the test was conducted primarily to obtain two equal halves concrete for use in chemical analysis and with uranyl acetate treatment. The test was conducted in accordance with AASHTO T 198.

4.2.1.3 Chloride Permeability

The "rapid chloride permeability test" (RCPT) or Electrical Indication of the Relative

Permeability of Concrete, provides a relative indication of the susceptibility of concrete to the ingress of solutions such as deicing salts and sea water. The test measures the ability of a concrete to pass ions from one cell to another, under a constant electrical field. The test has been criticized for its inability to discriminate between concretes with high permeability and concretes with a lower permeability which also contain water soluble salts such as calcium chloride.

In addition, since this test requires vacuum saturation of the concrete, the possibility exists that any ASR gel present might absorb water and fill the pores, reducing the measured permeability. Therefore, low coulomb values may not indicate a low permeability as much as an indication of voids in the concrete being filled with ASR gel. Testing was conducted in accordance with AASHTO T 277.

4.2.2 Residual Expansion

4.2.2.1 Purpose

Residual expansion of the cores was determined by measuring length changes over time of cores immersed in both highly alkaline solutions and water solutions. The residual expansion, or modified alkali immersion test, is useful in gauging the potential for future expansion likely to occur in concrete due to alkali-silica reaction with severe alkali exposure. The results should be treated with caution since the potential expansion can overestimate that of the concrete on site because the extracted cores are not restrained from expansion. Extracted cores will also expand for reasons other than ASR, such as shrinkage recovery.

Results in the alkaline solution provide information regarding the reactivity of the aggregate and potential expansion under extremely severe conditions and at a considerable age. This permits an estimate of the amount of expansion likely to occur if both additional external alkalies and moisture penetrate the concrete, while results in water provide information regarding the inherent reactivity and expansion potential of the concrete as a whole, without external alkalies. Specimens containing non-reactive aggregate which are stored in water will expand approximately 400 to 500 microstrain due to shrinkage reversal or recovery. This is a natural occurrence not related to ASR.

4.2.2.2 Specimen Preparation and Test Parameters

All specimens were air dried at least three days in order to have a standard initial moisture condition prior to the test, to accelerate the water or alkaline solution uptake in the cores and to permit attachment of gage studs. Three pairs of stainless steel gage studs, each pair with an approximate gage length of 100 mm (4 inches), were attached to each specimen with chemically stable, alkali resistant epoxy. Each pair was placed parallel to the long axis of the core, spacing equally around the core. After allowing the epoxy to harden for at least 8 hours, initial lengths were determined to the nearest 0.0025 mm (0.0001 inch).

Specimens were then placed in either an alkaline solution or in tap water containing a small amount of calcium hydroxide to prevent lime leaching from the concrete. The alkaline solution used was selected from the British Cement Association's alkali immersion test (6). The alkaline solution of 1-normal concentration consisted of 20 grams per liter (g/l) $NaOH$, 28 g/l KOH, and saturated with lime by adding 0.2 g/l CaO and using the solution after the excess lime had settled.

The specimens were completely covered with solution and stored up to 90 days. Samples were placed in plastic tubs and covered to avoid evaporation of the solution. Flexible plastic spirals were used as supports to prevent cores from touching the tub walls or bottom and being restrained.

Previous tests (1) had been conducted at room temperature. Since these previous results were available and since the need was to clearly discriminate between the potential reactivity of concrete with different components, the tests in this investigation were conducted at a higher temperature. Jones and Poole (13) note that the latent reaction at temperatures greater than 38 °C is questionable. More reaction will occur than expected under service conditions. Therefore the test was conducted at 38 °C ± 2 °C (100 °F) in order to accelerate reaction without creating "false positives".

4.2.2.3 Length Measurements

Measurements were taken with a hand-held, spring-loaded, mechanical length comparator with probes which fit into centering holes in the stainless steel studs. This process was performed at least twice per set of studs for accuracy. Before each measurement, the instrument was calibrated with an INVAR bar standard. The temperature of the solution was measured to ensure that thermal length changes were minimized.

4.2.3 Microscopic Examination

4.2.3.1 Optical Microscopy

Microscopic analysis included examination of fractured surfaces from selected specimens using a Nikon stereoscopic microscope, with reflected light, at 10x to 20x magnification. This permits easy identification of general aggregate characteristics and qualitative assessment of the extent of cracking, ASR deposits, paste quality and obvious anomalies in composition.

Petrographic examination included comparison of aggregate types, qualitative characterization of the concrete air void system, and identification of ASR and abnormal amounts of ettringite. Petrographic analysis was used to help distinguish between map cracking caused by ASR, ettringite, and frost cycles, which look similar macroscopically, and identify the probable deterioration mechanism of the concrete. Petrographic examination was conducted using a Vickers petrographic microscope and standard thin section slides prepared from selected cores.

4.2.3.2 Scanning Electron Microscope

A scanning electron microscope (SEM) with energy dispersive x-ray (EDX) analysis was used to examine specimens for the presence of ASR and abnormal amounts of ettringite. The SEM investigation was conducted using a state-of-the-art, Hitachi S 3200 N, Environmental SEM which does not require gold or carbon sputtering for conductance and which can be operated at normal high vacuum pressures as well as pressures exceeding the saturated pressure of water vapor, allowing the specimen to remain wet during observation and analysis. This type of SEM is the most recommended for examination of concrete.

The higher air pressure mitigates changes in morphology of the components which can sometimes occur with older SEM's. The higher pressure also allows dissipation of electrical charge of non-conductive specimens. The great advantage of being able to do elemental analysis without an interfering conductive coating is further enhanced by the use of an ultra-thin window to the detector making light element detection possible down to beryllium. Since resolution is not dependent on the sputtering, concerns with differences in structure sometimes noted between gold and carbon sputtering are eliminated.

Quantification of ceramic samples is optimized with corrections using internal mineral compound standards. The enhanced EDX capabilities of the Hitachi S 3200 N therefore permits reasonably accurate, although still not truly quantitative, elemental analysis of the concrete components, thus allowing, among other properties, conclusive identification of ettringite. It also provides semi-quantitative analysis of other features, including paste, aggregate and gel. Although the traditional limitations of EDX analysis for lighter elements such as sodium and potassium have been significantly mitigated and quantitative analysis improved significantly, analysis of alkalies is still best accomplished using other techniques.

4.2.4 Chemical Analysis

4.2.4.1 Aggregate

X-ray diffraction (XRD) was used to determine whether the aggregate contained significant quantities of amorphous silica. These tests were conducted using fresh samples from the Gold Hill and Princeton quarries. EDX analysis was also conducted with these samples to provide background information for use during SEM investigations.

4.2.4.2 Alkali Contents

4.2.4.2.1 Purpose: Determining the sodium and potassium contents of the concrete samples was critical to this study. Many researchers believe that total equivalent alkalies, expressed in kilograms per cubic meter of concrete (kg/m^3), provide the most logical method to assess the potential for deleterious ASR. Water-soluble alkalies were used instead of acid-soluble alkalies since they are more representative of the amount of alkalies which can be readily provided in service. Further, water soluble alkalies will not include any alkalies contributed by the aggregates or any which are not available for reaction with aggregates.

It was also necessary to determine alkali contents expressed as a percent of cement mass since the results from this study will be used to develop material specification limits or criteria. Therefore concrete cylinders containing known quantities of cement with different alkali contents were prepared in the laboratory. This permitted water soluble alkalies to be determined from samples with known total alkali contents. Alkali contents of the cements used in these specimens were determined by the NCDOT M&T Unit using standard methods.

4.2.4.2.2 Methodology: Split core halves, obtained from the splitting-tensile test, were placed in plastic bags to prevent contamination until sampling. A steel hammer was used to break the core fragment into small particles, no larger than approximately 10 mm (less than ½ inch). The particles included coarse aggregate as well as cement paste in order to provide a sample which was representative of the concrete itself. The hammer was rinsed with distilled water before repeating with subsequent samples to prevent contamination.

Approximately 100 to 200 grams of concrete particles from each structure were placed in 500 ml plastic containers. After recording the particle mass, approximately 150 to 300 ml of deionized water was added to each sample. Quantities of concrete and water were recorded. After 7 days of soaking at 38°F, solution samples were extracted from each container for individual alkali determination. The minimum soaking period was determined empirically based on the time required to come to equilibrium alkali content.

The alkali content, in kg/m^3, was determined using Atomic Absorption spectrophotometry (AA) test methods on the water samples. This provided sodium and potassium contents in parts per million which could be converted to percent by mass of the specimen after correcting for dilution of the extraction water.

The sodium and potassium contents were converted to oxide equivalents and the total equivalent alkali content calculated as indicated in AASHTO M 85. Total equivalent alkalies were then determined by percent mass of the concrete and percent mass of cement. These values were calculated from concrete specific gravity and cement contents.

4.2.4.3 Cement Content

Cement contents were estimated based on silica and calcium contents of selected, representative samples obtained from splitting the samples for alkali content. Due to the small sample size available and the variability inherent in the test method, results are not more accurate than about 30 kg/m^3 (½ sack per cubic yard) of cement.

4.2.5 Fluorescence Testing

4.2.5.1 Purpose:

Ultraviolet (UV) light inspection of a fractured concrete surface treated with uranyl acetate (UA) provides a relatively simple means for estimating the amount of ASR gel present and the pattern of ASR gel dispersion in concrete. Using this method, it has also been shown that residual expansion can be directly related to ASR gel production (1, 14).

UA treatment of the concrete marks the ASR gel deposits by replacing cations previously adsorbed on the ASR gel with the uranyl ion (UO_2^{++}) which is easily identified under a short-wave UV lamp. By using an electronic camera connected to a computer with image-processing capability, it is possible to quantitatively determine the amount of ASR gel present on the fractured surface of the core.

4.2.5.2 Sample Preparation and Procedure:

Uranyl acetate solution was applied to the freshly broken face of one core from each structure in accordance with the proposed method of testing SHRP 2013, Standard Test for Rapid Identification of Alkali Silica Reaction Products in Concrete. After drying, the specimens were placed in a wooden cradle to stabilize the cores for photographing. Two short-wave (254 nm) ultraviolet lights were used to illuminate the specimens, at an approximate distance of 300 mm (6 inches), aligned longitudinally on each side of the core to reduce shadows and illuminate any possible ASR gel deposits.

With other lights dimmed, a Sony CCD-IRIS / RGB color video camera (model DXC-151A) was used to capture an digital image of the specimen under ultraviolet light. This image was captured on a computer using Image-Pro Plus (v. 1.1) software, an image-processing program. A program function available in Image-Pro Plus uses video-averaging to produce sharper, more distinct specimen images.

The individual images were calibrated by using an 11 mm (about ⅜ inch) diameter metal disk, attached to the wooden cradle, which was coated in a pink, fluorescing material in order to appear on the electronic image. The fluorescing gel areas are a bright yellow to green color. A typical fluorescing area was selected manually and a mask created based on these colors. The mask feature reproduces the original computer image using only the specified colors and intensity levels selected, in effect masking all features except the ASR gel. Both color and intensity must be used in determining the extent and severity of ASR.

Once the computer generated mask was developed, the total area of fluorescing material (AFM) was determined, and converted to percent area after measuring the gross area of fractured surface. This method typically underestimates the actual values slightly by removing smaller fragments of fluorescing material from consideration.

4.3 Location of Cores

The fundamental strategy in selection of core locations at each site was to take cores from sections which appeared sound and from sections which were undergoing significantly more map cracking, and therefore indicative of possible ASR. Sites were also selected representing a variety of exposures, particularly at Bridge #15035 were some elements are in direct contact with seawater and others exposed only to salt spray.

Cores were extracted from by NCDOT personnel in coordination with the research team and delivered to the Department of Civil Engineering at North Carolina State University for testing. The core identification code was established by NCDOT. All cores were logged in and given a preliminary examination which included a visual inspection and general description noting the presence or extent of cracking, gel formation or powder deposits, and aggregate rims, if any. The length and diameter of the cores was measured, and the dynamic modulus determined for all cores.

5. RESULTS AND DISCUSSION

The general conclusions of the report on Project 23241-94-6, Investigation of Alkali-Silica Reactivity in North Carolina Department of Transportation Structures, were confirmed in that ASR and the extent of cracking were found to vary significantly with location at both bridges. The cause of variation in cracking was found to be different for the two bridges.

5.1 Bridge #83016, NC 138 over Long Creek, near Aquadale, in Stanly County

5.1.1 Site Examination (83016)

5.1.1.1 Visual Examination (83016)

The precast girders, believed to have come from a South Carolina source, appear to be in excellent condition. Girders from South Carolina are unlikely to contain Gold Hill aggregate and probably also contain very low alkali cement. However, significant cracking was noted in other sections of the bridge, as reported in previous investigations.

One footing, resting in the creek bed and exposed to view during the coring operations, had a large crack in it, clearly visible from the bridge deck. The pier caps and abutment walls also had clearly visible cracking as did a number of sections of the barrier rails. Map cracking was evident in the sections of bridge deck in several spans as well. However, as reported previously, there were a number of barrier rail sections and portions of the deck with considerably less cracking.

Visual examination of the cores immediately after removal indicated that the coarse aggregate in all of the cores taken was similar, and appearance was consistent with the Gold

Hill phyllite. Abundant deposits of a white gel, consistent with ASR, was noticed in a number of cores. These deposits were later confirmed to be ASR. The quantity of ASR in the cores was consistent with the extent of cracking in the different bridge sections.

A list of cores obtained, along with the size, specific gravity, dynamic modulus, general condition and results of visual examination of the cores are received are provided in Table 5.1.1. The visual examinations do not necessarily provide a definitive analysis of the quantity of gel or the extent of cracking. A schematic of the site is provided in Figure 5.1.1.

5.1.1.2 On-Site Testing

A Troxler gage measuring moisture content and density was used to try to estimate moisture penetrability as water was ponded on the deck over periods of up to 30 minutes. Surprisingly, no significant penetration was detected with this device, even when used on sections which contained significant visible cracking.

The initial moisture content was substantially higher than anticipated, however, typically 8 to 9% by weight. A reasonably dry bridge deck will typically have moisture contents closer to 4%. It is possible that the higher moisture content indicates the presence of (hygroscopic) ASR gel. The presence of the gel, which would fill the voids and cracks, is consistent with low penetrability and low RCPT test results found in other research.

Air permeability tests were conducted by members of the Materials and Tests Unit of NCDOT, using a device on loan from SHRP. Results are believed to have indicated a generally low permeability.

5.1.1 Core Characteristics and Condition, Bridge #83016 (Long Creek, near Aquadale, Stanley County)

ID #	Location	Size (cm)		SpG	E_D (modulus)		Cracks (Comments)	Gel
		Dia	L		(GPa)	(Mpsi)		
1na	N approach slab (1)	9.5	6.0	2.36	43.5	6.30	surface	none noted
2na	N approach slab (2)	9.5	14.8	2.37	42.7	6.18	minor, surface	none noted
3na	N approach slab (3)	9.5	6.5	2.31	36.0	5.21	none noted	none noted
4na	N approach slab (4)	9.5	13.3	2.31	36.3	5.27	minor	none noted
5na	N approach slab (5)	9.5	13.4	2.31	39.0	5.65	minor	none noted
6na	N approach slab (6)	9.5	13.6	2.35	40.6	5.89	surface	none noted
1nc	N pier cap, N face (1)	5.7	10.8	2.39	42.3	6.13	significant	yes
2nc	N pier cap, N face (2)	5.7	9.8	2.40	36.7	5.32	surface and internal	yes
3nc	N pier cap, N face (3)	5.7	6.3	2.39	40.4	5.86	significant	significant
1nw	NW abutment (1)	5.7	9.7	2.35	36.7	5.33	internal (porous surface)	yes
2nw	NW abutment (2)	5.7	6.8	2.34	41.0	5.94	internal	yes
4nw	N pier, W end (top)(4)	5.7	11.7	2.37	35.5	5.15	significant	significant
1sw	Rail, SW end (1)	5.7	11.3	2.39	45.6	6.60	minor, surface	none noted
2sw	Rail, SW end (2)	5.7	13.7	2.32	38.9	5.64	minor, surface	none noted
3sw	Rail, SW end (3)	5.7	15.0	2.32	36.8	5.33	none noted	none noted
4sw	Rail, SW end (4)	5.7	6.5	2.39	46.7	6.77	minor, surface	none noted
5sw	Rail, SW end (5)	5.7	14.3	2.37	45.0	6.52	none noted	none noted
6sw	Rail, SW end (6)	5.7	16.5	2.37	43.7	6.34	minor, surface	none noted

Notes: "N", "S", "E", "W" (North, South, East, West) "cm" (centimeter) "SpG" (Specific Gravity) "Dia", "L" (diameter, length) "GPa" (gigapascal) "Mpsi" (million pounds per square inch)

Table 5.1.1 (continued) Core Characteristics and Condition, Bridge #83016 (Long Creek, near Aquadale, Stanley County)

ID	Location	Size (cm)		SpG	E_D (modulus)		Cracks (Comments)	Gel
		Dia	L		(GPa)	(Mpsi)		
1sa	S approach slab (1)	9.5	10.5	2.37	40.6	5.89	minor	none noted
2sa	S approach slab (2)	9.5	11.1	2.37	41.9	6.07	minor	none noted
3sa	S approach slab (3)	9.5	11.1	2.40	44.5	6.45	minor, surface	none noted
4sa	S approach slab (4)	9.5	10.5	2.38	45.9	6.65	none noted	none noted
5sa	S approach slab (5)	9.5	12.6	2.37	44.5	6.45	minor, surface	none noted
6sa	S approach slab (6)	9.5	6.4	2.39	46.6	6.75	surface	none noted
1A	Span A, NBL, N end (1)	9.5	13.4	2.39	39.4	5.71	none noted	none noted
2A	Span A, NBL, N end (2)	9.5	12.8	2.36	41.2	5.97	minor, surface	none noted
3A	Span A, NBL, N end (3)	9.5	11.2	2.35	38.9	5.64	minor, surface	none noted
3Ab	Span A, NBL, N end (3b)	9.5	4.8	2.28	32.4	4.70	minor, surface	minor
1Ans	Span A, SBL, S jt (1)	9.5	9.4	2.35	42.3	6.13	minor	minor
2As	Span A, SBL, S end (2)	9.5	11.6	2.30	39.6	5.74	none noted	minor
3Am	Span A, SBL, mid, R whl(3)	9.5	10.6	2.34	38.0	5.51	none noted	minor
4A	Span A, SBL, mid (4)	9.5	9.9	2.33	41.7	6.05	none noted	no
5A	Span A, SBL, mid, L whl(5)	9.5	11.6	2.32	39.4	5.71	none noted	minor
3B	Span B, E rail (3)	9.5	11.7	2.39	51.3	7.43	significant	yes

Notes: "N", "S", "E", "W" (North, South, East, West) "cm" (centimeter) "SpG" (Specific Gravity) "Dia", "L" (diameter, length) "GPa" (gigapascal) "Mpsi" (million pounds per square inch)

Table 5.1.1 (continued) Core Characteristics and Condition, Bridge #83016 (Long Creek, near Aquadale, Stanley County)

ID	Location	Size (cm)		SpG	E_D (modulus)		Cracks (Comments)	Gel
		Dia	L		(GPa)	(Mpsi)		
1D	Span D, E rail (1)	9.5	6.7	2.44	50.7	7.35	surface	yes
2D	Span D, E rail (2)	9.5	6.1	2.42	52.1	7.55	significant	yes
4D	Span D, E rail (4)	9.5	12.8	2.39	44.1	6.38	significant	yes
5D	Span D, E rail (5)	9.5	12.5	2.38	38.1	5.51	surface	yes
6D	Span D, E rail (6)	5.7	16.4	2.37	40.8	5.91	none noted	yes
1Dsdj	Span D, SBL, S jt, R whl(1)	9.5	14.1	2.41	41.6	6.02	significant	yes
2Dj	Span D, SBL, S jt (2)	5.7	12.1	2.38	37.9	5.49	minor, surface	minor
3D	Span D, SBL, S jt, L whl(3)	9.5	7.9	2.36	36.1	5.24	minor, surface	minor
1E	Span E, NBL, S end (1)	9.5	7.5	2.28	27.7	4.01	surface and internal	significant
2Es	Span E, NBL, S end (2)	9.5	12.3	2.29	25.2	3.66	surface and internal	significant
3Ea	Span E, NBL, S end (3a)	9.5	9.0	2.34	31.6	4.58	surface and internal	yes
3Eb	Span E, NBL, S end (3b)	9.5	12.7	2.33	34.5	4.99	surface and internal	significant
1Em	Span E, SBL, mid, R whl(1)	9.5	14.6	2.36	36.4	5.28	minor, surface	yes
2Em	Span E, SBL, mid (2)	5.7	13.1	2.32	39.2	5.68	none noted	yes

Notes: "N", "S", "E", "W" (North, South, East, West) "cm" (centimeter) "SpG" (Specific Gravity) "Dia", "L" (diameter, length) "GPa" (gigapascal) "Mpsi" (million pounds per square inch)

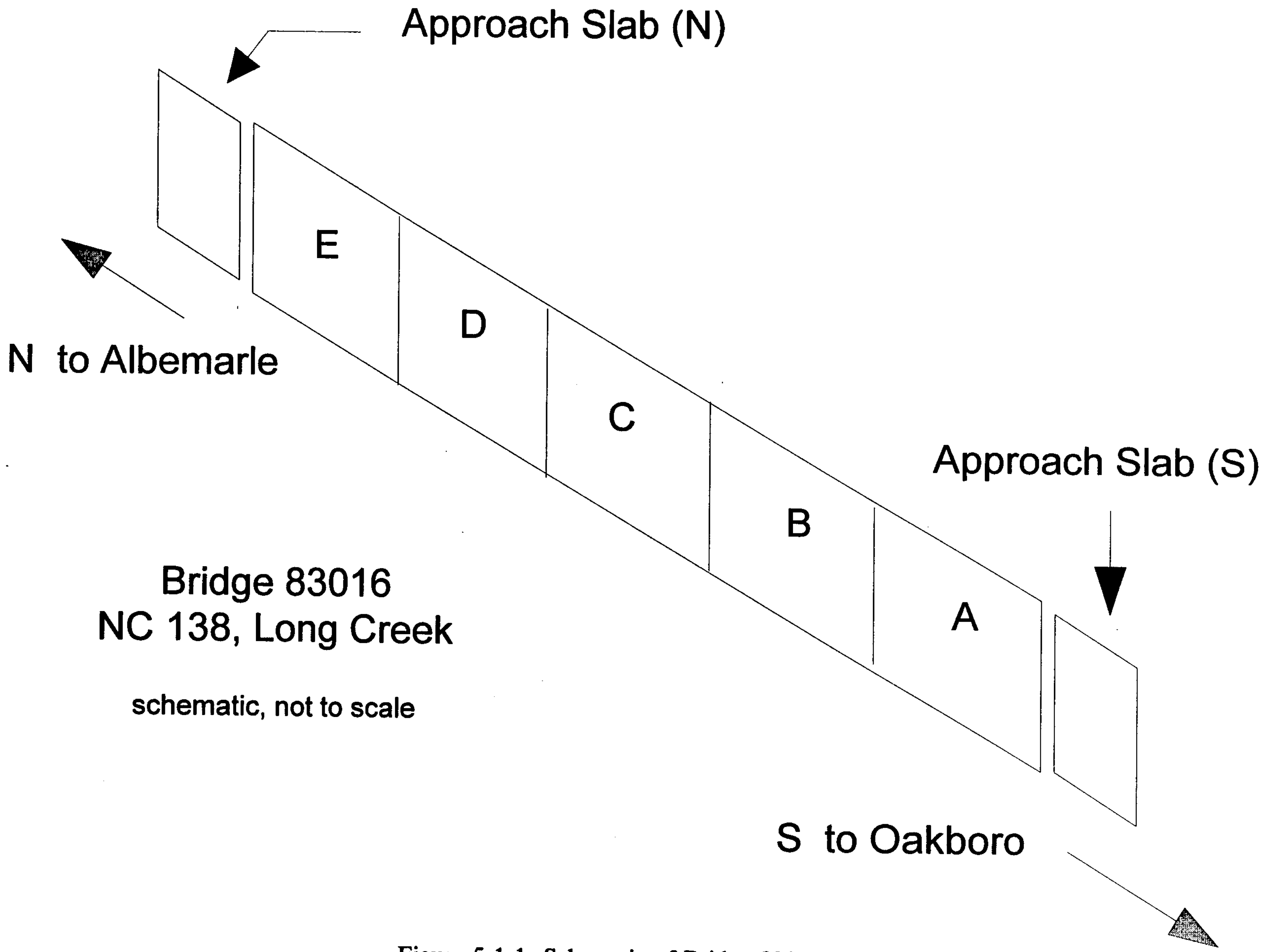

Figure 5.1.1 Schematic of Bridge 83016

5.1.1.3 Construction Review (83016)

The possibility of two different contractors and/or two different concrete mixes was discussed with Mr. Tom Hearne, of NCDOT (Albemarle). He provided the names of two individuals, Mr. Dusty West of Dean's Ready Mixed Concrete, and Mr. Bill Strickland, of NCDOT, who were actively involved in the construction of the bridge.

Mr. West confirmed that Gold Hill aggregate had been used in conjunction with Lonestar (Roanoke) cement on the job. He believed the concrete contained 639 pcy of cement. Mr. West stated that he did not deliver any other type of concrete to the job, but that, due to differences with the contractor, he did not supply the job for a short period, leaving open the possibility that the general contractor used another supplier for at least a portion of the job. He stated that, as he remembered, his company did finish out the job.

Mr. Bill Strickland, an assistant resident engineer on the bridge during construction, stated that he thought only one supplier was used on the job. However, he also stated that it was possible that another ready mix contractor could have supplied concrete from the other (Monroe) side of the project at one point due to construction scheduling and access limitations from the Albemarle side.

According to sources in the local cement industry, at that point in time and in that part of the state, only two cements were likely to have been used; Roanoke and Blue Circle. The Blue Circle cement likely to have been used is a very low alkali cement. The Roanoke likely to have been used is not low alkali.

It was impossible to obtain definitive boundaries or even confirm conclusively that some concrete used in the structure contained a low alkali cement while most did not, although any concrete supplied by another contractor would probably have been placed in only a limited area of the superstructure. This is consistent with observations regarding areas with and without significant cracking.

5.1.2 Mechanical Properties and Residual Expansion (83016)

Mechanical properties of the cores indicate that the concrete in most locations is reasonably sound. Only a few cores had marginal dynamic modulus (see Table 5.1.2). Splitting tensile strengths of the specimens tested were adequate. The dynamic modulus and splitting tensile strength were generally correlated (see Figure 5.1.2). However, as found in previous research, neither was correlated with the extent of ASR; some cores exhibited significant cracking and many cores showed evidence of ASR.

Specific values of the residual expansion test were only marginally useful in this research but the test results did confirm conclusions from site observations and other laboratory tests. Residual expansion of cores from cracked sections was moderate to high, regardless of whether the cores were stored in water or in alkaline solution.

Residual expansion of cores from uncracked sections had acceptable expansion when stored in water, indicating that future expansion is likely to be limited as long as exposure to external alkalies is minimal. Residual expansion of cores from uncracked sections which were stored in alkaline solution were less than those from cracked sections indicating a difference in composition between the cracked and uncracked sections. Based on residual expansion testing, cracking of the sound sections due to ASR may occur in the future but will probably be minimal. Cracking in already cracked sections will continue (see Table 5.1.3).

Table 5.1.2 Mechanical Properties, Bridge #83016

Core ID	Dynamic Modulus (E_D)		Splitting Tensile Strength		In-Situ Cracking
	(GPa)	(Mpsi)	(MPa)	(psi)	
3D	36.1	5.24	3.7	530	yes
2Es	25.2	3.65	3.3	480	yes
2Em	39.2	5.69	3.5	510	yes
3Ea	31.6	4.58	3.6	525	yes
1E	27.6	4.01	2.5	360	yes
1D	50.7	7.35	3.9	570	yes
1Ans	42.3	6.13	3.0	440	minor
3na	36.0	5.22	3.1	445	no
3sw	36.8	5.34	4.4	640	no
4A	41.7	6.05	3.0	435	no
3Ab	32.4	4.70	2.8	410	minor

Notes: "GPa" (gigapascal) "Mpsi" (million pounds per square inch) "MPa" (megapascal) "psi" (pounds per square inch)

Table 5.1.3 Residual Expansion, Bridge #83016

Core ID	Immersion Solution	Expansion (microstrain)	In-Situ Cracking	coarse agggregate	Potential Expansion
6D	alkaline	3400	yes	slate	significant
1Dsdj	alkaline	3200	yes	slate	significant
1A	alkaline	2900	no	slate	significant
5D	alkaline	2800	yes	slate	significant
2sw	alkaline	2400	no	slate	deleterious
5sw	alkaline	2400	no	slate	deleterious
1Em	alkaline	2100	yes	slate	deleterious
2A	alkaline	1800	no	slate	marginal
2Dj	water	2400	yes	slate	deleterious
3Am	water	2000	minor	slate	marginal
2As	water	1600	minor	slate	marginal
4D	water	1500	yes	slate	minor

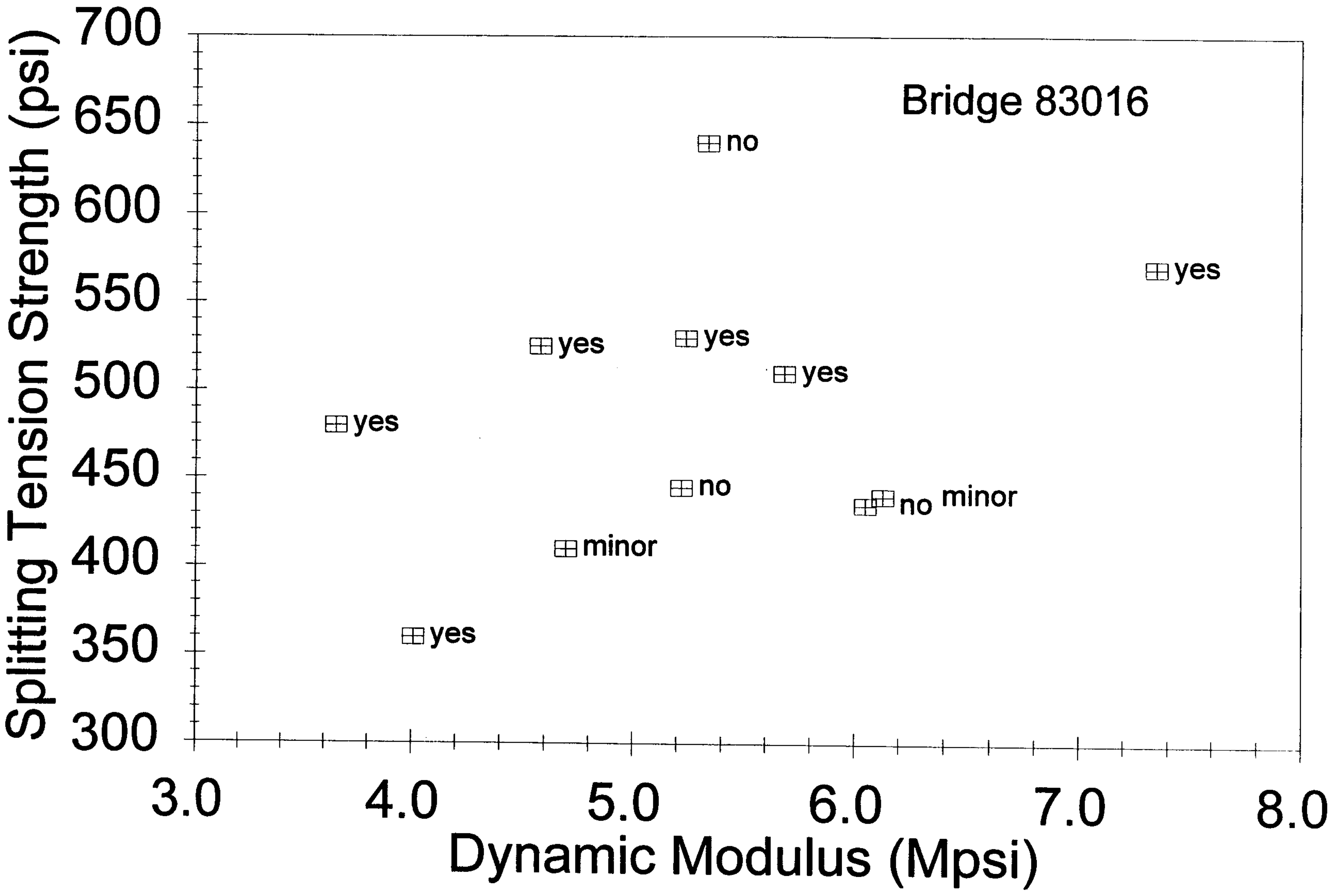

Figure 5.1.2 Mechanical Properties, Bridge 83016

Labels indicate the presence or extent of in-situ cracking

The electrical indication of chloride permeability indicated that concrete from both the cracked and uncracked sections, approximately 50 mm (about 2 inches) below the surface, was sound.

5.1.3 Microscopic Examination and Related Analysis (83016)

5.1.3.1 Constituents (83016)

No amorphous silica was found with X-ray diffraction of aggregate from the Gold Hill quarry, which indicates that the alkali-silica reaction is typically only moderately rapid and is rarely associated with extremely high quantities of gel. This is consistent with observations from the field.

Iron sulfide was found in the coarse aggregate using EDX techniques. This material has been known to cause deterioration resembling ASR. In the specimens examined, no evidence of deterioration associated with the iron sulfide was found.

Optical microscopic examination was conducted on thin sections of cores from both cracked and uncracked sections of the bridge. The paste in specimens from all sections was dense, indicating a concrete with a high cement content which had been adequately consolidated and cured. The coarse aggregate is the same in both sections. The coarse aggregate contains significant amounts of strained, micro-crystalline silica. This type of aggregate is prone to ASR and is consistent with the fine textured phyllite, or slate, from the Gold Hill quarry.

Fine aggregate used in this bridge does not show any evidence of reactivity. The fine aggregate was predominantly quartz composed of large crystals and therefore non-reactive.

No ASR gel was associated with the fine aggregate.

In Figure 5.1.3, a photograph of a thin section taken from core 5, the extremely fine grain size in the large piece of coarse aggregate in the upper left half of the photograph is evident, particularly in comparison with the large crystal size of the fine aggregate in the lower right quadrant of the photograph. A preferred direction in the grain alignment of the coarse aggregate is also evident. These features are typical of reactive aggregate.

This cores was taken from a cracked/uncracked section of the bridge. A rim of gel along the outer edge of the coarse aggregate is visible in the photograph. Others cores from this section were found to contain ASR gel.

5.1.3.2 Air Content (83016)

Optical microscopic examination of the concrete found a low to moderate air content in many of the cores. The air voids were typically fine, mitigating the lower air contents. The frost durability of the concrete appears to be acceptable. Internal cracking did not appear to be associated with frost durability in general.

Most importantly, air content was not related to the degree of cracking noted in different sections of the bridge. Air contents were varied both in those sections exhibiting significant cracking and those sections of the bridge without significant visible cracking. Therefore, cracking in the bridge is not due primarily to frost action. However, a possible interaction between frost and ettringite was seen in one area.

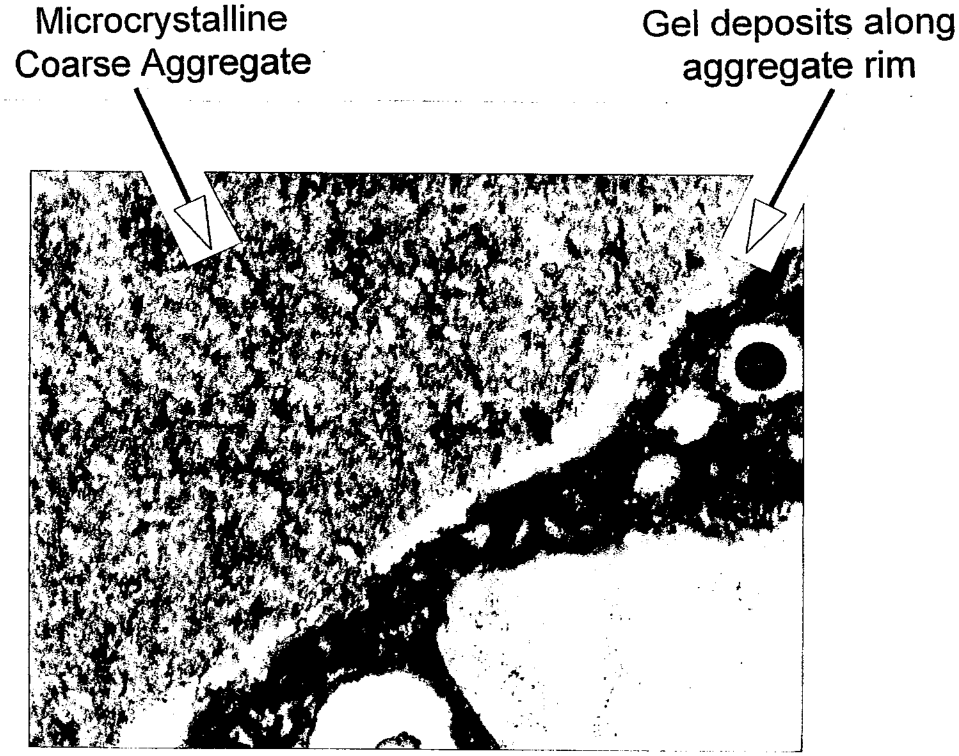

Figure 5.1.3 Thin Section, Core 5A, Bridge #83016

5.1.3.3 Presence of Ettringite and ASR (83016)

Ettringite was found in the air voids in some sections of the structure in somewhat higher contents than originally anticipated, however, the quantity of ettringite was still relatively minor. Ettringite was not found in the aggregate paste interface. Due to the relatively small quantity, ettringite cannot be said to be the primary factor in deterioration of this specimen. ASR was also found in limited amounts, but it too cannot be said to be the primary factor in deterioration. However, since the ettringite was found in entrained air voids, the map cracking found in Span A may be due to the combined effects of the small amounts of ettringite with the slightly low total air contents also found in Span A. Figure 5.1.4 is a SEM photograph of a sample from Span A showing the ettringite partially filing several air voids.

Microscopic examination confirmed the presence of significant ASR deposits in certain cores and the lack of significant ASR in others. The presence of significant ASR was limited to cores which came from sections of the bridge with visible surface cracking. In these specimens, matrix softening and microcracking were clearly visible with the SEM.

Cracks which had been in-filled after formation in a specimen from Span E were examined with both the petrographic microscope and the SEM. The cracks were found to contain aged ASR gel. The in-fill material was also gel. This is indicative of continuing alkali-silica reactivity occurring over time, between periods of drying.

Figure 5.1.4 Photograph Showing Ettringite in Air Voids, Scanning Electron Microscope, Core 1Ans, Bridge #83016

5.1.4 Chemical Analysis and Area of Fluorescence (Uranyl Acetate Treatment) (83016)

5.1.4.1 Chloride Content

The chloride content was tested at two locations and indicates minimal deicing salt applications at this structure.

5.1.4.2 Water Soluble Alkali Content and Percent Area of Fluorescence

The water soluble alkali content, expressed as sodium oxide (Na_2O), of specimens from various locations on the bridge was compared to the percent area of fluorescence of the specimens after treatment with uranyl acetate and viewed under ultra-violet light. The results are presented in Figure 5.1.5A. A summary of the chemical analysis, including water soluble alkali content, percent area of ASR gel as indicated by fluorescence under ultra-violet light after treatment with uranyl acetate, estimates of cement content and cement alkali levels, whether or the core was from a cracked section, and aggregate type are provided in Table 5.1.4.

It is clear that a higher alkali content results in a greater amount of ASR. It is also apparent that there is a minimum alkali content required for deleterious reactivity with this aggregate. In Figure 5.1.5A, a cluster of cores with relatively high alkalies appear which did not have a very large percent area of gel. It was tempting to view the data as forming two lines, one extended through 1D-1E-3D, while the other extended through 3Ea-2Em-2Es. These sets of cores were examined for common features or conditions since that information could be used to identify other important factors in controlling ASR. Factors such as air content, moisture exposure, core size and ettringite content were considered.

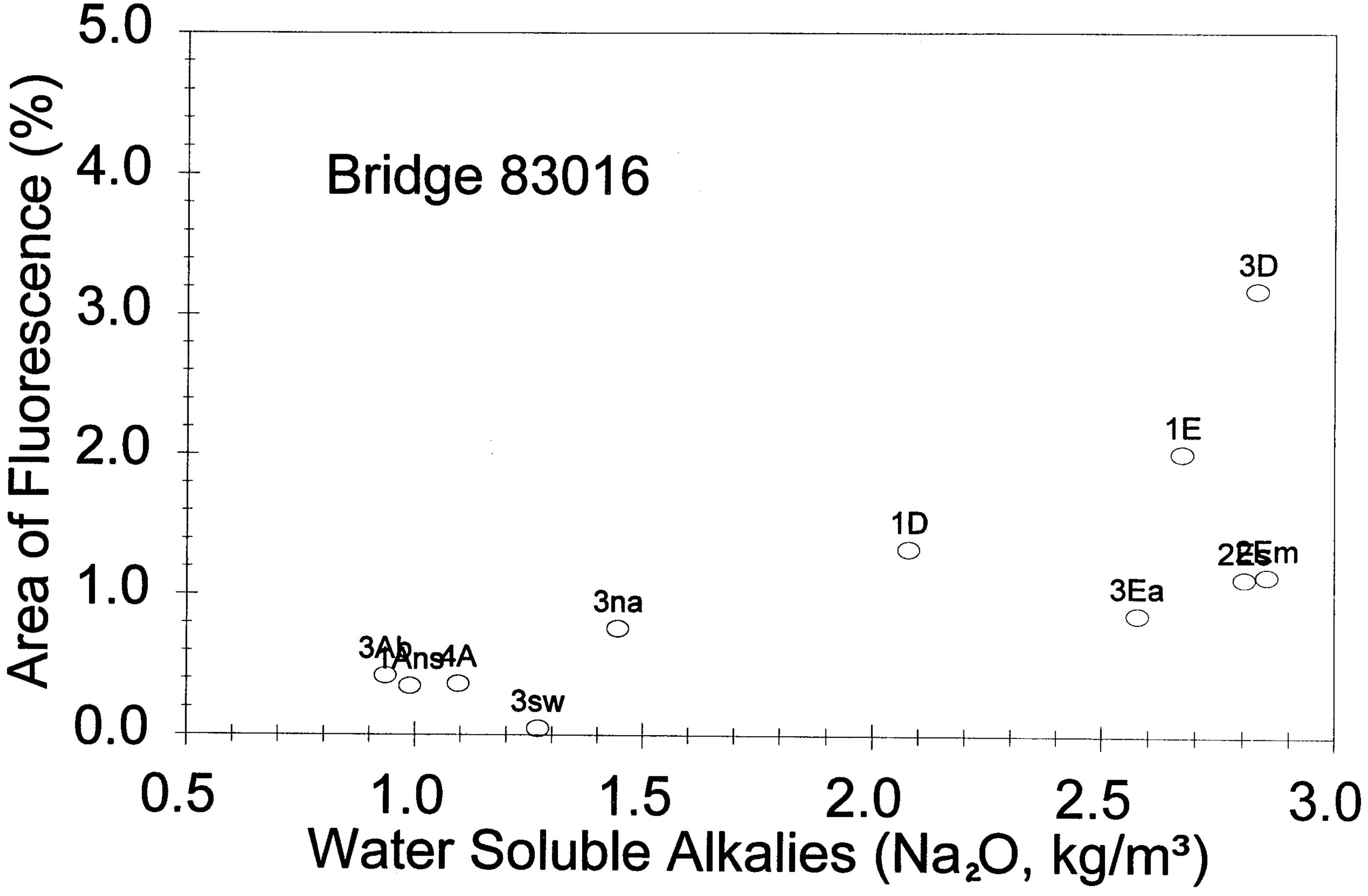

Figure 5.1.5A Relationship Between Concrete Alkali Content and Quantity of ASR Gel, Bridge #83016

Table 5.1.4 Chemical and Gel Analysis, Bridge #83016

Core ID	% AFM	Alkalies		K / Na	Cement Content			In-Situ Cracking	Aggregate Type
	adj for length	TEA, % cmt	Concrete, kg/m^3	(mass)	kg/m^3	pcy	method		
3D	3.75	0.91	2.83	2.2	380	640	type (AA)	minor	phyllite
1E	2.26	0.86	2.67	1.6	380	640	Si	significant	phyllite
1D	1.33	0.63	2.08	1.9	400	670	Si and Ca	yes	phyllite
2Em	2.24	0.92	2.85	2.3	380	640	type (AA)	yes	phyllite
2Es	2.06	0.90	2.80	2.2	380	640	Si and Ca	significant	phyllite
3Ea	1.16	0.83	2.58	2.0	380	640	Si and Ca	yes	phyllite
3na	0.74	0.53	1.45	1.9	335	560	type (A)	no	phyllite
3Ab	0.30	0.30	0.94	0.7	380	640	type (AA)	minor	phyllite
4A	0.54	0.35	1.10	0.5	380	640	type (AA)	no	phyllite
1Ans	0.49	0.30	0.99	0.9	400	670	Si and Ca	minor	phyllite
3sw	0.11	0.41	1.27	0.9	380	640	type (AA)	no	phyllite

"AFM" is the Area of Fluorescing Material; "TEA" is Total Equivalent Alkalies, expressed as Na_2O (TEA = $Na_2O + 0.657 \cdot K_2O$) as percent by mass of portland cement; "Soluble" refers to water soluble; "kg/m^3" is kilograms per cubic meter; "pcy" is pounds per cubic yard; "method" refers to the method of estimating portland cement content, either by "Si" (silica content), "Si and Ca" (silica and calcium content), "type (A) or (AA)" (estimated based on class of concrete probably used, Class A or AA).

Although no obvious ASR gel gradients were noted, such as were found in cores from Bridge 15035 (see section 5.2.4.2), re-examination indicated that virtually no ASR gel was found near the bottom of these cores indicating a likely difference in ASR with depth. Further examination also revealed that all three cores were much longer than the others examined with uranyl acetate. A difference in ASR with depth is reasonable for a bridge deck which will only be exposed to moisture on one side.

These factors suggested that the simple measure of the area of fluorescing material is not an accurate assessment of the extent of ASR with cores of widely varying lengths. Therefore , an adjustment was made to the measured area of fluorescing material, based on length. The percent area of fluorescing material was multiplied by the quantity 1 - (1 - 6.7)/6.7 to provide the percent fluorescing material adjusted for length, allowing more meaningful quantitative comparison of cores of varying length.

In Figure 5.1.5B, the percent fluorescence, adjusted for length, is again plotted against the quantity of water soluble alkalies, but the core identification has been replaced with labels indicating the extent of in-situ cracking noted at the bridge during coring operations. This graph clearly shows the relationship between the degree of cracking, the extent of ASR and the quantity of alkalies in the concrete. In reviewing this graph, it is worth noting that "non-deleteriously reactive" does not mean either "non-reactive" or that the presence of any amount of ASR gel is necessarily problematic.

It is clear from Figure 5.1.5B that the extent of cracking in the bridge is directly related to the alkali content of the concrete. Concrete in spans D and E exhibited significant cracking and had both a higher alkali content and much higher quantities of ASR gel. Other areas had

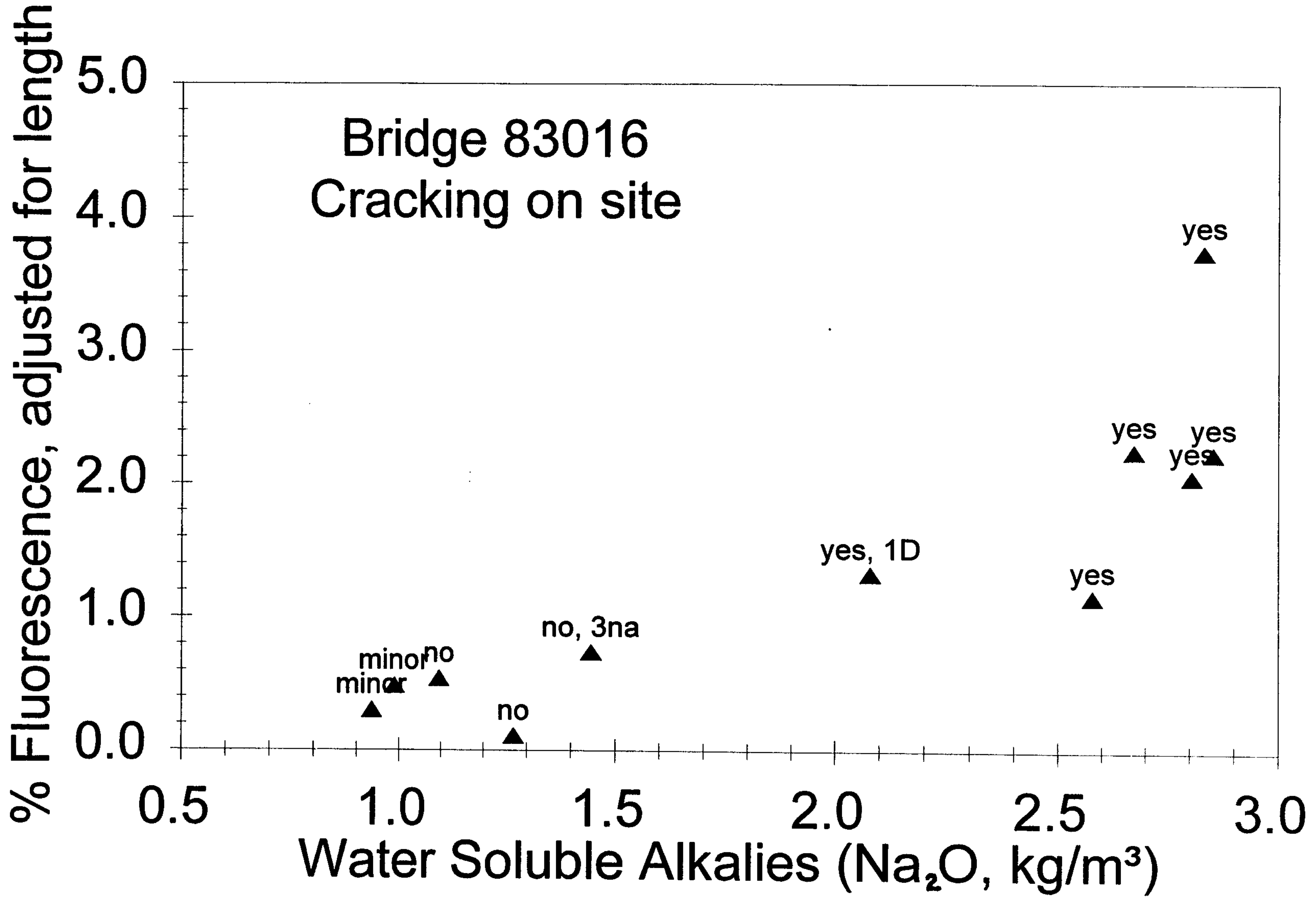

Figure 5.1.5B Relationship Between Concrete Alkali Content and Quantity of ASR Gel, adjusted for length, Bridge #83016

no or only minor map cracking and cores from these sections had lower alkali contents and only minor quantities of ASR gel. As noted above in section 5.1.3.3, cores from Span A had a slightly higher ettringite content, located in the air voids, and a slightly low air content as well. The minor cracking noted in this location may be related to a combination of frost action and ettringite.

In Figure 5.1.6, the ratio of potassium to sodium is plotted against the percent area of fluorescence. The potassium to sodium ratio indicates that two different cements were apparently used. Both Roanoke (Lonestar) and Blue Circle cements are believed to have been used in the construction of this bridge.

It is interesting to note that core 3na, with a relatively low alkali content and relatively little evidence of ASR, was apparently produced from the same cement source as that found in cores with significant ASR and higher alkalies. It is likely that this is the result of routine variation in the alkali content of the cement source.

Since other mechanisms of deterioration are minimally active at best, it may be concluded that the differences in deterioration of sections of the bridge are due to differences in alkali content of two cements used in the construction of the bridge.

5.1.4.3 Alkali Content as a Percent of Cement (83016)

Although alkali-silica reactivity may have occurred, no deleterious ASR was generated when the equivalent water soluble alkali content, expressed as sodium oxide (Na_2O), of the concrete was less than 1.5 kg/m^3, including alkalies from all sources, including admixtures. When the equivalent water soluble alkali content was greater than 2.0 kg/m^3, deleterious

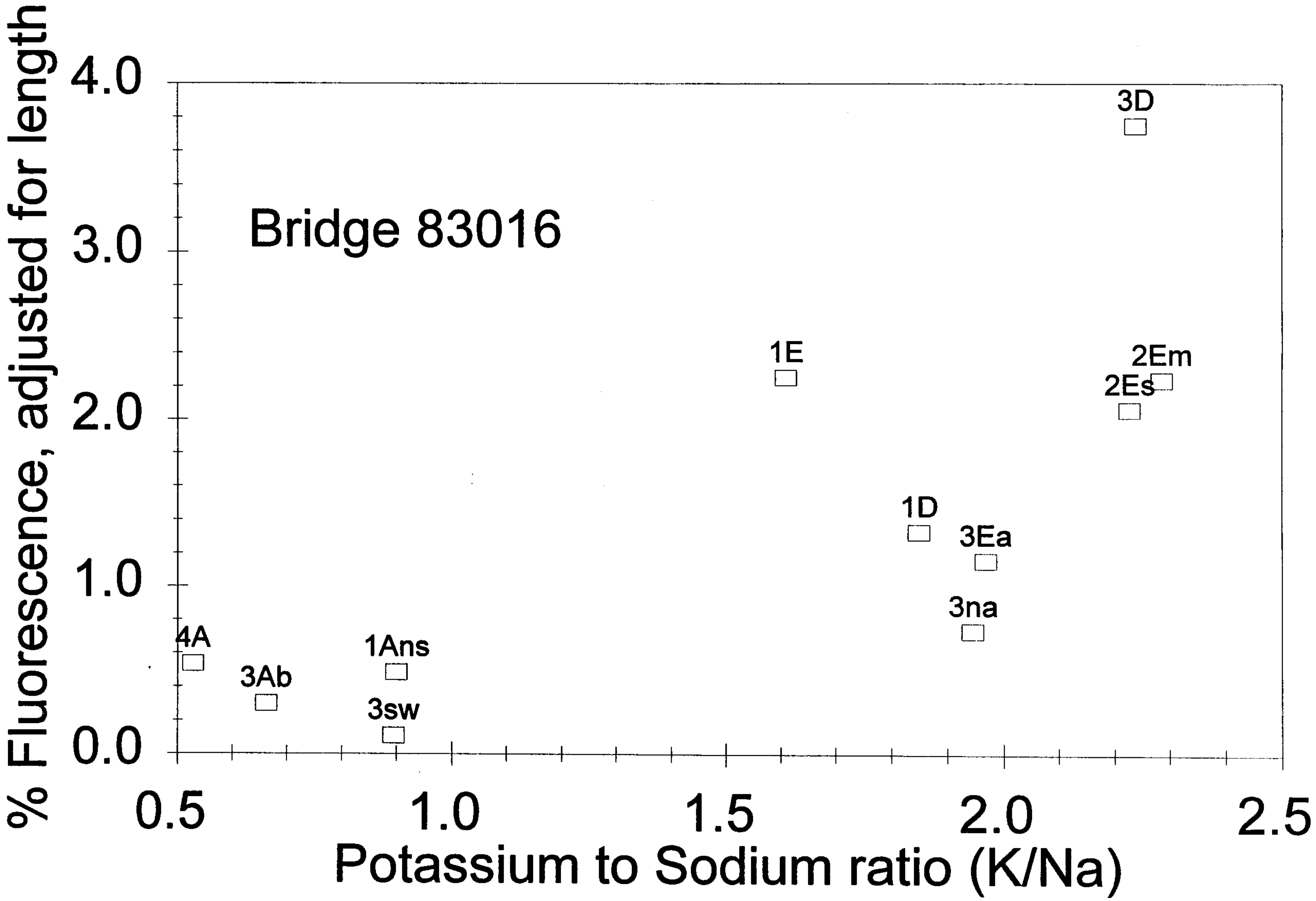

Figure 5.1.6 Relationship Between Potassium to Sodium Ratio of Concrete and Quantity of ASR Gel, Bridge #83016

alkali-silica reactivity occurred.

Compliance with these limits can be readily determined with simple tests and can therefore be used in specifications. While the use of total alkali content of the concrete has been strongly recommended for specification limits by some, limiting alkalies in the portland cement is the most common method currently used in the US. Therefore, estimates of the total alkali content as a percent mass of the cement were needed.

Since relatively small quantities of material were available for testing, the precision of the cement content was not optimal. The inherent accuracy of portland cement content of hardened concrete is typically no better than about 30 kg/m^3 (about ½ sack per cubic yard). Therefore, cement alkali limits cannot be established with more precision than 2 significant figures. However, it is believed that the conclusions and recommendations regarding cement alkali limits are accurate enough to establish specifications. This problem is also being investigated in another NCDOT sponsored investigation.

The cement content of selected samples was determined from silica and calcium content, except for sample 1E in which only the silica content was used due to a problem which occurred during testing. Since estimates based on silica are preferred, results are considered acceptable. The cement content of other samples was estimated from a combination of location within the structure, specifications and the results of chemical analysis.

The cement contents determined experimentally were slightly higher than anticipated. The lower air contents encountered would account for approximately 7 kg/m^3 (13 pcy) or about half the difference found. It is also possible that slightly higher cements contents were required to maintain the specified w/c ratio with the angular and somewhat flat Gold Hill aggregate. It is also possible that the sample contained a higher paste content than a more representative sample due to the limited amount of material available for all of the tests.

The alkali contents reported for cements in the M 85 optional specifications are acid soluble, or total alkalies, while the alkali contents reported in this study were water soluble to avoid interference from any alkalies dissolved from the aggregate. A correction factor is required to account for this difference. This factor was determined experimentally.

In order to convert water soluble alkalies into total equivalent (acid soluble) alkalies, two concrete mixes were produced containing Gold Hill coarse aggregate, a conventional concrete sand, Raleigh city water, and a portland cement with a known alkali content. The alkali content of the cement actually used in the mixes was determined by the NCDOT Materials and Tests Unit. One mix was made with a moderate alkali content cement and one with a low alkali content cement. Proportions conformed to NCDOT specifications (AA).

The concrete was kept moist for 7 days at room temperature and then stored in lab air for 21 days to simulate curing conditions in the field and still provide 28 day old concrete for testing. Water soluble alkali contents were then determined as with other specimens. The water soluble alkali contents were compared to the theoretical total alkali contents based on cement content, density of the concrete and cement alkalies.

The ratio of water soluble alkali content to total theoretical alkali content based on mass of cement was 83.2% for the low alkali cement concrete and 80.5% for the moderate alkali concrete. Since they were so close, a value of 82% was used for all concrete samples.

This is a slightly higher ratio than anticipated. The reason for the high ratio may be related to the curing regime. However, the high ratio means that the conclusions reached below regarding acceptable cement alkali levels are conservative.

Figure 5.1.7 shows the relationship between percent area of fluorescence, adjusted for length, and total equivalent alkalies (TEA), expressed as Na_2O, as a percent mass of the cement. According to these data, a TEA of less than 0.55% is innocuous, with reactive phyllite aggregate, at least for concrete not greater than 17 years old. A TEA greater than about 0.60% will result in deleterious expansion of concrete containing reactive phyllite aggregate in less than 16 years. These findings suggest that specifications limiting the total equivalent alkalies, expressed as Na_2O, to less than between 0.55% to 0.60% would be effective in controlling deleterious alkali-silica reactivity of concrete containing reactive phyllite or shale aggregates *in service*, in North Carolina.

It is useful to consider the location of the two cores which form the boundary of these recommended limits, that is, cores 3na and 1D. Core 3na is from an approach slab, which is in contact with the ground and will have a higher average moisture content than concrete in or on the bridge itself. Core 1D is from the barrier rail of Span D, which will have the lowest average exposure to moisture content. Core 1D had the higher alkali and ASR gel contents. Therefore, while the higher average moisture content of an approach slab contributes to reaction at an earlier age, it may be concluded that the average moisture content of any portion of the bridge is sufficient to sustain deleterious alkali-silica reactivity, over the span of its service life, as long as the alkali content is high enough.

The alkali content of several of the cores showing considerable ASR is very high when expressed as a percent by mass of the cement. In specimen 2Es, for example, cement content was determined from chemical analysis, so the values should be reasonably accurate.

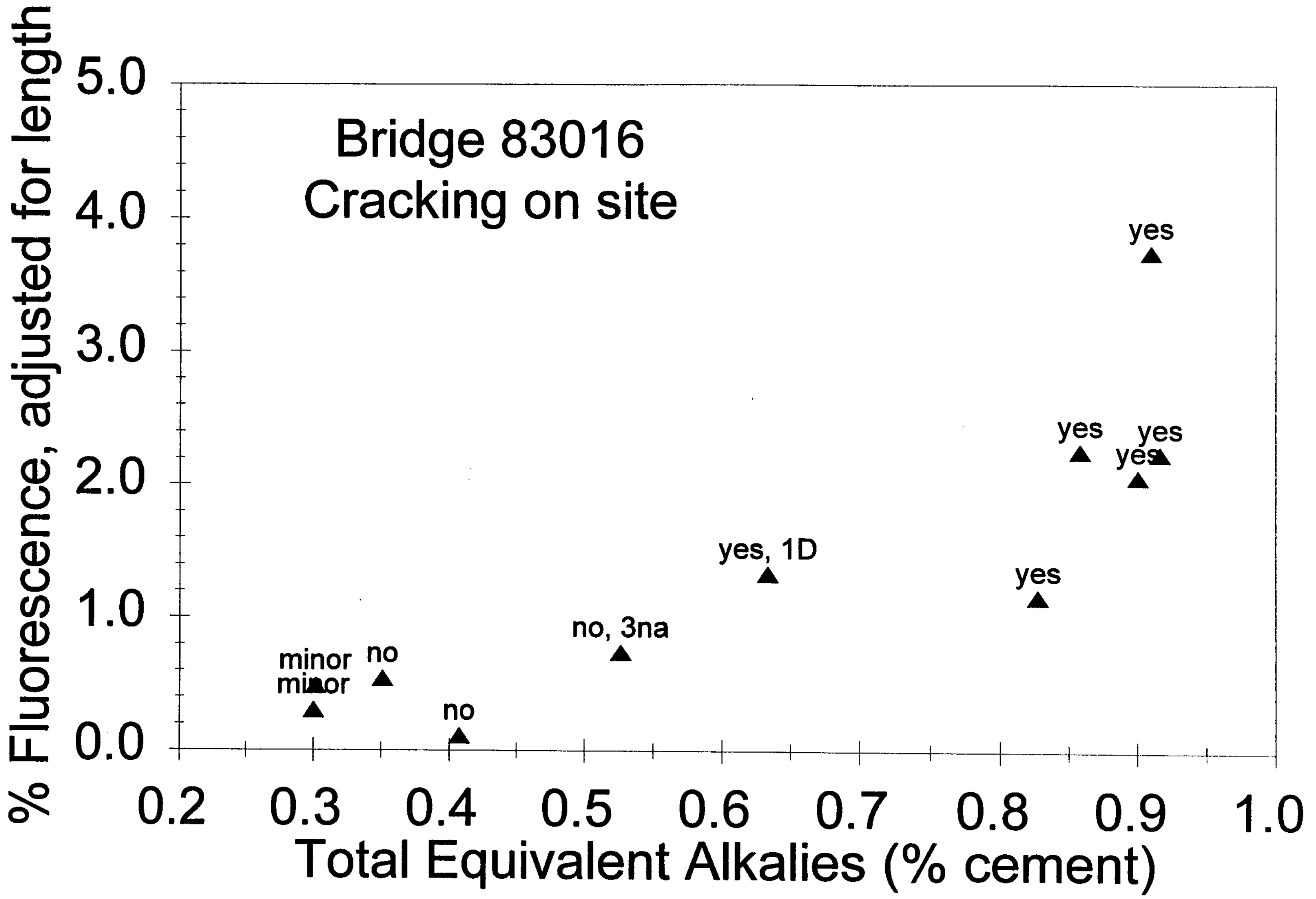

Figure 5.1.7 Relationship Between Cement Alkali Content and Quantity of ASR Gel, Bridge #83016

Since the soluble alkali content includes the alkalies available from all sources, it is possible that part of the alkali content is due to admixtures in the original batch. Another source is deicing salts. Although the chloride content did not indicate a significant amount of salting, any alkalies associated with deicing salts would be in a water soluble form, and therefore contribute to the overall water soluble alkali content of the concrete. A third possible source is that alkalies may have been liberated from the aggregate during the alkali-silica reaction. This is speculative, but the phyllite does contain significant potassium.

Since continued cracking of the sections already exhibiting deterioration due to ASR is expected, a frequent inspection of critical sections of the bridge which are already cracked, particularly the substructure, is recommended.

5.1.4 Sodium and Potassium Effects (83016)

The total equivalent alkali content is calculated as all of the Na_2O plus about 2/3 of the K_2O. The multiplier for the potassium oxide is the ratio of the molecular weights of the Na_2O and K_2O, which provides an alkali content in terms of equivalent number of alkali ions. However, the ratio of potassium to sodium ion mobilities is approximately 3/2, or the inverse of the ratio of sodium oxide to potassium oxide molecular weights.

The relationship between cracking and total water soluble alkali oxides, that is Na_2O + K_2O, without the molecular weight correction, expressed as percent mass of the cement, was examined in order to consider any possible effects of ion mobility as well as number of ions. In Figure 5.1.8, there is a more pronounced difference between the cracked and uncracked

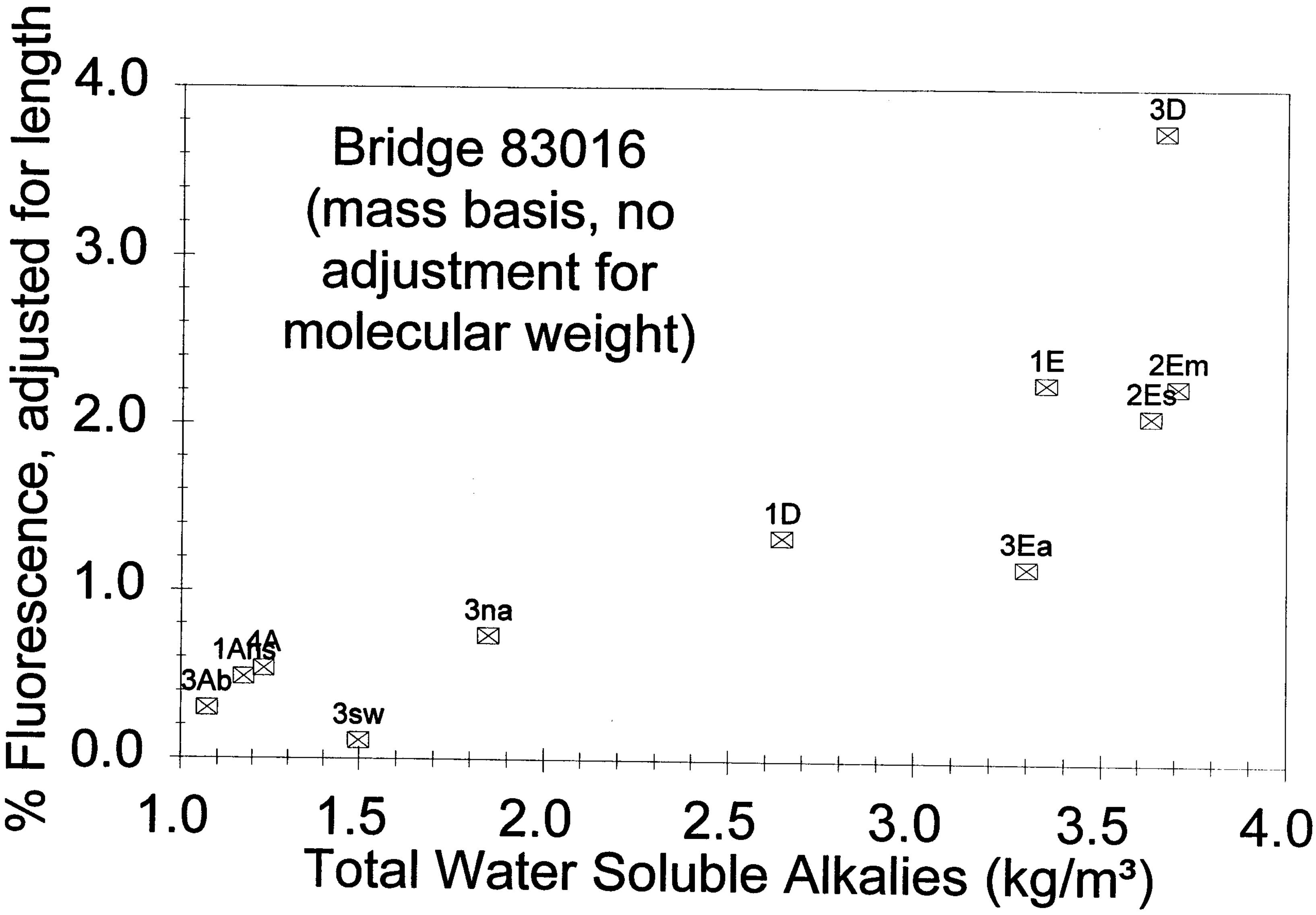

Figure 5.1.8 Relationship Between Total, Unadjusted Alkali Content and Quantity of ASR Gel, Bridge #83016

sections, suggesting that the role of the potassium is more important in long term deterioration of phyllite due to ASR.

5.1.5 Conclusions (83016)

5.1.5.1 Deterioration of sections of the concrete deck, barrier rail and substructure of Bridge #83016 exhibiting clearly visible pattern cracking are the result of ASR.

5.1.5.2 No deleterious ASR was generated when the water soluble alkali content, expressed as sodium oxide (Na_2O), of the concrete was less than 1.5 kg/m^3. Compliance with this limit can be readily determined with simple tests and can therefore be used in specifications, if desired.

5.1.5.3 A. Based on the results of this study a total equivalent alkali (TEA) content, expressed as sodium oxide (Na_2O), of less than 0.55% of the portland cement is innocuous with highly reactive phyllite, at least for concrete in service not more than 17 years.

B. A TEA content greater than 0.60% will result in deleterious expansion of concrete containing highly reactive phyllite in less than 16 years.

5.1.5.4 Based on residual expansion testing, cracking of the sound sections due to ASR may occur in the future but will probably be minimal. Cracking of the previously cracked sections will continue.

5.1.5.5 Frequent inspection of critical sections of Bridge 83016, particularly the substructure, where continued deterioration due to ASR is expected, is highly recommended.

5.1.5.6 Potassium oxide appears to be more important in long term deterioration due to ASR than sodium oxide, at least for alkali reactive phyllite.

5.2 Bridge #15035, US 70 over Ward Creek, past Morehead City, in Carteret County

5.2.1 Site Examination (15035)

5.2.1.1 Visual Examination (15035)

The pile caps and the precast hollow core panels and piles appeared to be in generally good condition, except for the corrosion and spalling of one panel noted in the report for Project 23241-94-6, Investigation of Alkali-Silica Reactivity in North Carolina Department of Transportation Structures. There was no indication of ASR in the precast elements. Minor distress previously noted on the end of several caps was found to be related to a concrete lip added to each end of the pile caps after the original construction. Partial debonding of several of the lips was noted; ASR appeared to be a factor. There was little other indication of ASR in the pile caps.

Exposed aggregate in the precast hollow core panels was rounded gravel, clearly different than the crushed stone supplied by the Princeton quarry, which was reportedly used in the cast-in-place sections of this bridge. Subsequent examination of cores from the pile caps indicated that the coarse aggregate in these members was also different from that supplied by the Princeton quarry. Some members primarily contained a coarse grained, cubical to angular, crushed, pink granitic stone, with numerous inclusions.

Cracking was noted in several sections, particularly the abutments, where several large, horizontal cracks were found, and along the seawall, where map cracking was evident in many locations. Several cores showed evidence of ASR, significant in a few cases. Visual examination of cores found considerable variability in the coarse aggregate, again indicating that some of the concrete delivered contained aggregate other than from the Princeton quarry.

A list of cores obtained, with the size, specific gravity, dynamic modulus and type of aggregate found are provided in Table 5.2.1A. The general condition and results of visual examination are provided in Table 5.2.1B. Table 5.2.1B also contains an alternate identification code which was used in the laboratory due to the complexity of the primary code. The conditions and observations are based on conditions as received. They do not necessarily provide a definitive analysis of the quantity of gel or the extent of cracking. A schematic of the site is provided in Figure 5.2.1.

Considerable variation was found in the extent of cracking and the presence of visible deposits. Much of the variation is related to location but not necessarily exposure. Elements in the tidal zone had the most severe exposure, but the extent of cracking in these elements was varied and was not always the most severe. Considerable variation was seen even within the same area or member. A significant amount of variation appeared to be due to variation in the aggregate and mixing of aggregate within batches of concrete.

5.2.1.2 On-Site Testing (15035)

Since the Troxler moisture gage had not been able to detect an appreciable amount of water penetration in the Aquadale bridge, it was not used at Bridge #15035. Air permeability tests were conducted at Bridge #25035 by members of the M&T Unit using the device on loan from SHRP. Results of the tests are believed to have been inconclusive.

Table 5.2.1A Core Characteristics, Bridge #15035 (Ward Creek, US 70 past Morehead City, Carteret County)

ID	Location	Aggregate	Size (cm)		SpG	E_D (modulus)	
			Dia	L		(GPa)	(Mpsi)
1	1st pilecap, SW (top lip)	schist-gneiss + granite	5.7	10.5	2.41	35.6	5.16
2	1st pilecap, SW (top, curb)	schist-gneiss + granite	5.7	13.6	2.31	42.9	6.21
3	Pilecap, end abutment, SW (top)	schist-gneiss + granite	5.7	14.5	2.43	27.6	3.99
4	Seawall, SW (top)	schist-gneiss	9.5	13.2	2.36	30.4	4.41
5	Seawall, SW (top)	schist-gneiss	9.5	15.8	2.36	30.6	4.43
6	Pilecap, 1st Bent, (outside, horiz)	granite	5.7	11.5	2.24	32.9	4.77
7	Pilecap, 1st Bent (inside, horiz)	granite	5.7	12.7	2.27	25.1	3.64
8	Seawall, SW, (outside, horiz, below tide)	schist-gneiss + quartzite	5.7	12.3	2.37	20.4	2.96
9	Seawall, SW (inside, horiz)	schist-gneiss	5.7	14.4	2.40	40.6	5.89
10	Bent #8, SE (top lip)	schist-gneiss	5.7	13.7	2.34	42.1	6.10
11	Top (lip) SE abutment	schist-gneiss + quartzite	5.7	13.9	2.34	29.1	4.21
12	Seawall, SE (horiz)	schist-gneiss + quartzite	5.7	13.1	2.38	42.6	6.18
13	Seawall, SE (horiz)	schist-gneiss + granite	5.7	20.4	2.34	34.2	4.97
14	Seawall, SE (top)	schist-gneiss	9.5	15.3	2.28	32.9	4.77
15	Seawall, SE (top)	schist-gneiss + quartzite	9.5	17.7	2.28	33.2	4.81

Notes: "N", "S", "E", "W" (North, South, East, West) "cm" (centimeter) "SpG" (Specific Gravity) "Dia", "L" (diameter, length) "GPa" (gigapascal) "Mpsi" (million pounds per square inch)

Table 5.2.1A (continued) Core Characteristics, Bridge #15035 (Ward Creek, US 70 past Morehead City, Carteret County)

ID	Location	Aggregate	Size (cm)		SpG	E_D (modulus)	
			Dia	L		(GPa)	(Mpsi)
EaNw(h)	East abutment, North wall (horiz)	granite	5.7	8.1	2.34	44.2	6.41
EaNw(v)	East abutment, North wall (vert)	granite	5.7	17.3	2.32	45.4	6.58
EaSw(h)	East abutment, South end (horiz)	granite	5.7	14.7	2.30	40.6	5.88
EaSw(v)	East abutment, South end (vert)	granite	5.7	16.4	2.31	41.2	5.97
EeNw(h)	East end, North end wall (horiz)	granite	5.7	15.9	2.33	43.1	6.25
EeNw(v)	East end, North end wall (vert)	granite	5.7	14.8	2.33	38.9	5.64
EeSc(1)	East end, South curb #1	schist-gneiss + granite	5.7	13.3	2.35	33.8	4.90
EeSc(2)	East end, South curb #2	schist-gneiss + granite	7.0	11.2	2.37	43.3	6.28
EeSw(vm)	East end, South end wall (vert, main)	granite	7.0	9.4	2.30	40.7	5.90
EeSw(vt)	East end, South end wall (vert, tail)	schist-gneiss + granite	5.7	13.5	2.36	39.4	5.71
WaBw(1)	West abutment, back wall, #1	schist-gneiss	7.0	15.1	2.32	20.6	2.99
WaBw(2)	West abutment, back wall, #2	schist-gneiss	7.0	9.1	2.36	37.0	5.36
WaSw(h)	West abutment, South wall (horiz)	schist-gneiss	7.0	13.1	2.28	25.0	3.62
WaSw(v)	West abutment, South wall (vert)	schist-gneiss	7.0	13.0	2.26	35.7	5.17
WeNc(1)	West end, North curb #1	schist-gneiss	5.7	16.5	2.33	43.0	6.23
WeNc(2)	West end, North curb #2	schist-gneiss + granite + quartzite	5.7	13.2	2.30	28.7	4.16
WeSc(2)	West end, South curb #2	gneiss	7.0	16.2	2.30	46.0	6.67
WeSw(h)	West end, South end wall (horiz)	schist-gneiss	5.7	32.2	2.30	24.3	3.52
WeSw(v)	West end, South end wall (vert)	schist-gneiss	7.0	6.2	2.28	3.8	0.55

Notes: "N", "S", "E", "W" (North, South, East, West) "cm" (centimeter) "SpG" (Specific Gravity) "Dia", "L" (diameter, length) "GPa" (gigapascal) "Mpsi" (million pounds per square inch)

Table 5.2.1B Core Condition, Bridge #15035 (Ward Creek, US 70 past Morehead City, Carteret County)

ID (both)	Location	Aggregate	Cracks (Comments)	Gel
1	1st pilecap, SW (top lip)	schist-gneiss + granite	surface and internal	yes
2	1st pilecap, SW (top, curb)	schist-gneiss + granite	none noted	none noted
3	Pilecap, end abutment, SW (top)	schist-gneiss + granite	surface and internal	yes
4	Seawall, SW (top)	schist-gneiss	significant	significant
5	Seawall, SW (top)	schist-gneiss	none noted	yes
6	Pilecap, 1st Bent, (outside, horiz)	granite	none noted	none noted
7	Pilecap, 1st Bent (inside, horiz)	granite	none noted	none noted
8	Seawall, SW, (outside, below tide)	schist-gneiss + quartzite	none noted (porous surface)	minor
9	Seawall, SW (inside, horiz)	schist-gneiss	internal (porous surface)	none noted
10	Bent #8, SE (top lip)	schist-gneiss	significant	yes
11	Top (lip) SE abutment	schist-gneiss + quartzite	surface and internal	none noted
12	Seawall, SE (horiz)	schist-gneiss + quartzite	internal (salt deposits)	minor
13	Seawall, SE (horiz)	schist-gneiss + granite	none noted	none noted
14	Seawall, SE (top)	schist-gneiss	minor, surface	minor
15	Seawall, SE (top)	schist-gneiss + quartzite	none noted	yes

Notes: "N", "S", "E", "W" (North, South, East, West) "cm" (centimeter) "SpG" (Specific Gravity) "Dia", "L" (diameter, length) "GPa" (gigapascal) "Mpsi" (million pounds per square inch)

Table 5.2.1B (continued) Core Condition, Bridge #15035 (Ward Creek, US 70 past Morehead City, Carteret County)

ID	Short ID	Location	Aggregate	Cracks (Comments)	Gel
EaNw(h)	E1	East abutment, North wall (horiz)	granite	none noted	none noted
EaNw(v)	E2	East abutment, North wall (vert)	granite	none noted	none noted
EaSw(h)	E5	East abutment, South end (horiz)	granite	none noted	none noted
EaSw(v)	E6	East abutment, South end (vert)	granite	none noted	none noted
EeNw(h)	E9	East end, North end wall (horiz)	granite	none noted	none noted
EeNw(v)	E10	East end, North end wall (vert)	granite	none noted	none noted
EeSc(1)	E3	East end, South curb #1	schist-gneiss + granite	significant	yes
EeSc(2)	E4	East end, South curb #2	schist-gneiss + granite	minor surface and internal	yes
EeSw(vm)	E8	East end, South end wall (vert, main)	granite	none noted	none noted
EeSw(vt)	E7	East end, South end wall (vert, tail)	schist-gneiss + granite	minor surface and internal	minor
WaBw(1)	W1	West abutment, back wall, #1	schist-gneiss	surface and internal	yes
WaBw(2)	W2	West abutment, back wall, #2	schist-gneiss	minor	yes
WaSw(h)	W7	West abutment, South wall (horiz)	schist-gneiss	fine cracks throughout	none noted
WaSw(v)	W8	West abutment, South wall (vert)	schist-gneiss	surface and internal	yes
WeNc(1)	W3	West end, North curb #1	schist-gneiss	none noted	none noted
WeNc(2)	W4	West end, North curb #2	all	none noted	none noted
WeSc(2)	W6	West end, South curb #2	gneiss	none noted	minor
WeSw(h)	W9	West end, South end wall (horiz)	schist-gneiss	internal	yes
WeSw(v)	W0	West end, South end wall (vert)	schist-gneiss	minor	yes

Notes: "N", "S", "E", "W" (North, South, East, West) "cm" (centimeter) "SpG" (Specific Gravity) "Dia", "L" (diameter, length) "GPa" (gigapascal) "Mpsi" (million pounds per square inch)

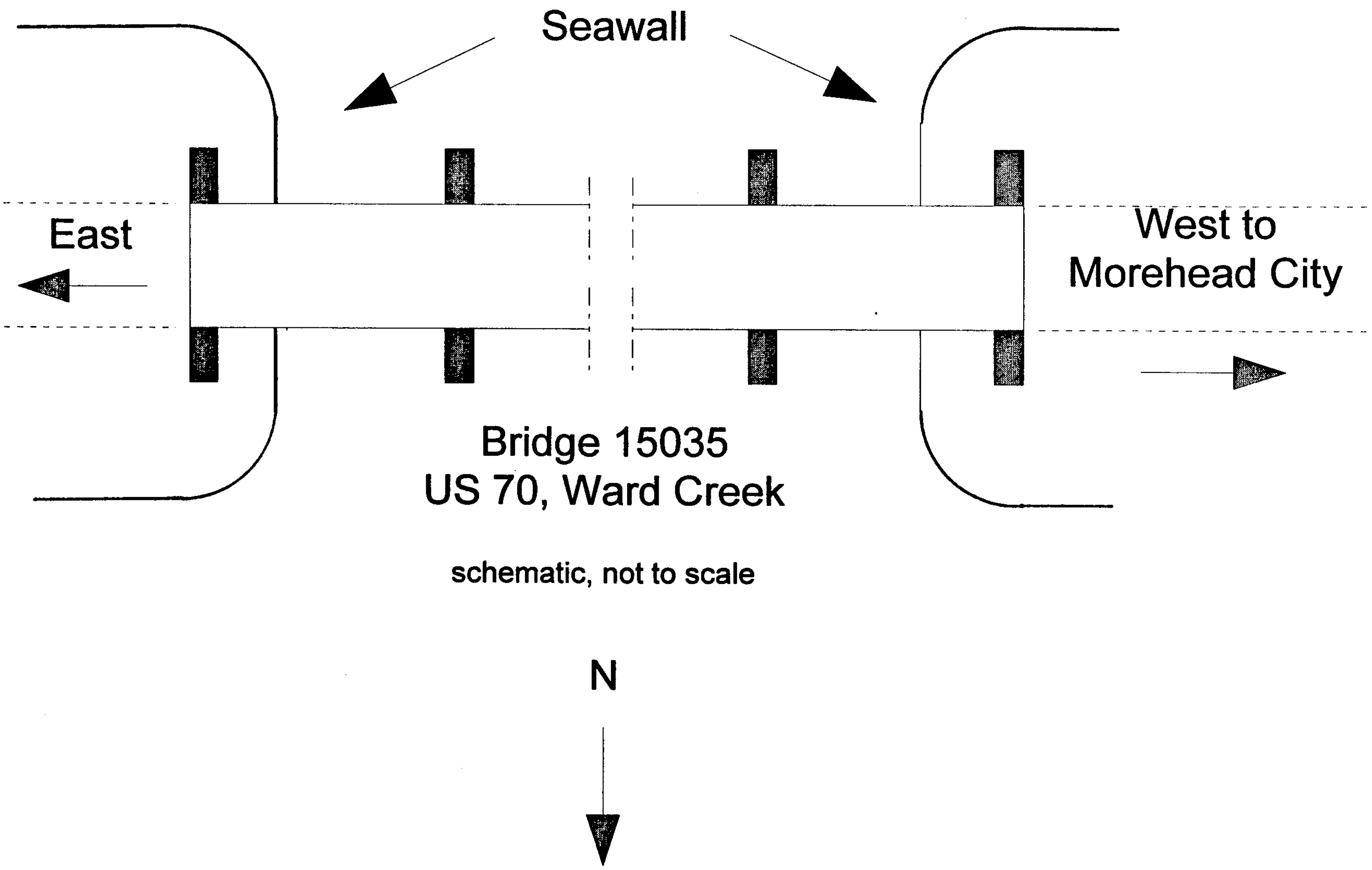

Figure 5.2.1 Schematic of Bridge #15025

5.2.2 Mechanical Properties and Residual Expansion (15035)

Mechanical properties of the cores indicated that much of the concrete is reasonably sound. However, a number of cores exhibited significant cracking and many cores from both sites showed evidence of ASR.

Dynamic modulus was acceptable to good except for one core. Splitting tensile strengths of the specimens tested were typically very good. However, the dynamic modulus and splitting tensile strength were not correlated (see Figure 5.2.2), indicating the composition of the concrete varied considerably. The mechanical properties are provided in Table 5.2.2.

As with the results of cores from Bridge #83016, specific values of the residual expansion test were only somewhat useful. However, the residual expansion tests did result in one very important finding. Cores containing the crushed, pink aggregate did not expand deleteriously regardless of the storage solution, indicating that this aggregate is not deleteriously reactive under any reasonable exposure (see Table 5.2.3). Residual expansion of cores containing other material, especially those from cracked sections, was moderate to high, regardless of whether the cores were stored in water or alkaline solution.

5.2.3 Stereo-, Petrographic and Scanning Electron Microscopic Examination (15035)

5.2.3.1 Constituents (15035)

X-ray diffraction of aggregate from the Princeton quarry found no amorphous silica, indicating that the alkali-silica reaction is not particularly rapid and is rarely associated with extremely high quantities of gel. This is consistent with observations from the field.

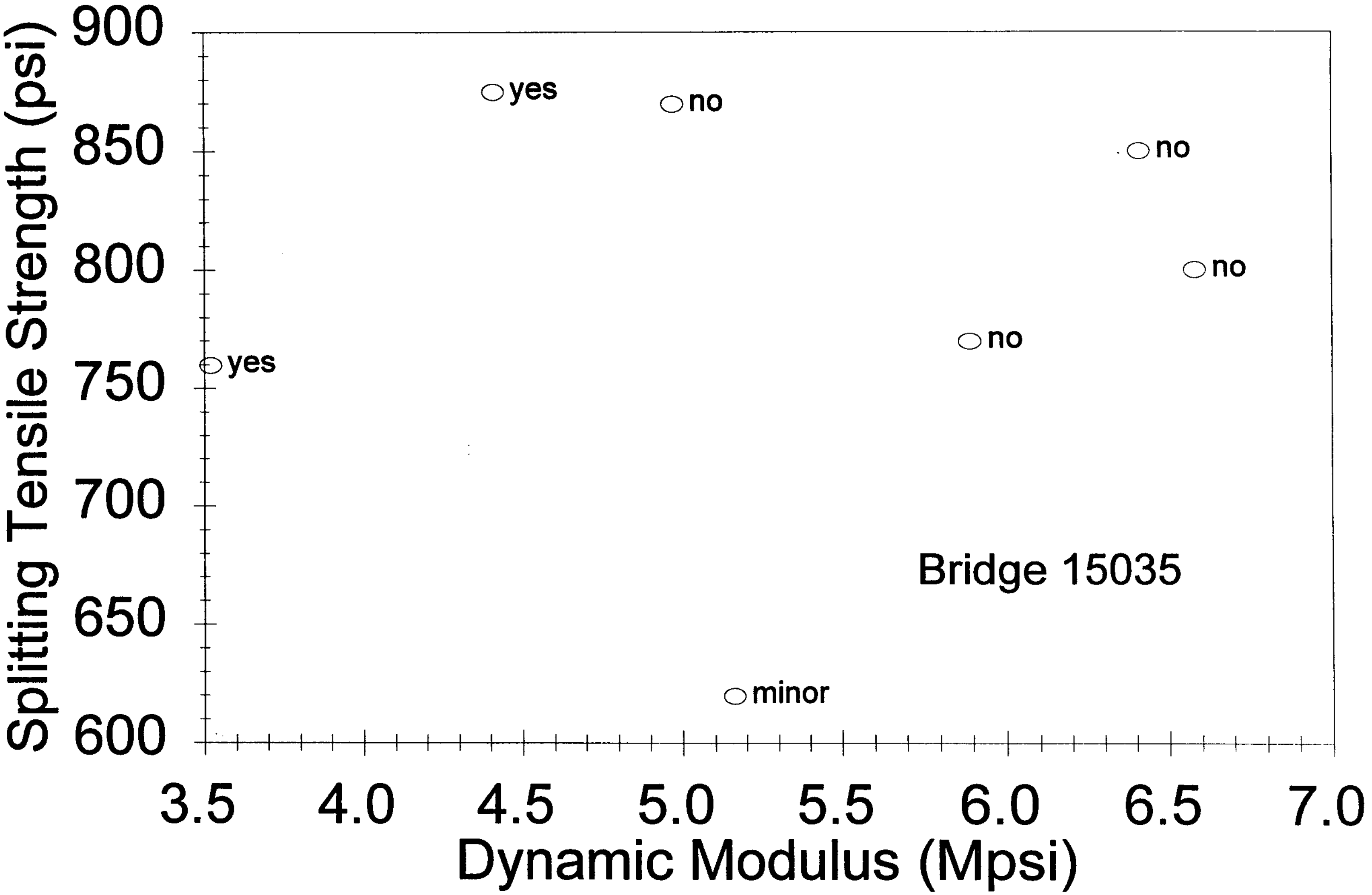

Figure 5.2.2 Mechanical Properties, Bridge 15035

Labels indicate the presence or extent of in-situ cracking

Table 5.2.2 Mechanical Properties, Bridge #15035

Core ID	Dynamic Modulus (E_D)		Splitting Tensile Strength		Aggregate
	(GPa)	(Mpsi)	(MPa)	(psi)	
E1	44.2	6.41	5.9	850	granite[1], mix
E4	30.4	4.41	6.1	875	mix
13	34.3	4.97	8.5	870	mix
W9A	24.3	3.52	5.3	760	schist-gneiss[2]
9	40.6	5.89	5.4	770	schist-gneiss[2]
E2	45.4	6.58	5.6	800	granite
1	35.6	5.16	4.3	620	mix

Notes: [1] Mostly granite with one large piece of reactive aggregate near the top, [2] schist and gneiss are fine grained with strained crystals "GPa" (gigapascal) "Mpsi" (million pounds per square inch) "MPa" (megapascal) "psi" (pounds per square inch)

Table 5.2.3 Residual Expansion, Bridge #15035

Core ID	Immersion Solution	Expansion (microstrain)	In-Situ Cracking	Predominant coarse agggregate	Area - Exposure
E3	alkaline	4500	yes	schist-gneiss	curb
10	alkaline	3500	yes	schist-gneiss	bent
W2	alkaline	2700	yes	schist-gneiss	end/abut
15	alkaline	2700	no	schist-gneiss	sea/pile
5	alkaline	2700	yes	schist-gneiss	sea/pile
2	alkaline	2200	no	mix	sea/pile
W8	alkaline	2000	yes	schist-gneiss	end/abut
W4	alkaline	1500	no	mix	curb
14	alkaline	1500	minor	schist-gneiss	sea/pile
11	alkaline	1200	no	schist-gneiss	end/abut
6	alkaline	700	no	granite	bent
E9	alkaline	700	no	granite	end/abut
E10	alkaline	600	no	granite	end/abut
12	water	2700	(salt)	schist-gneiss	sea/pile
3	water	1400	yes	mix	end/abut
W6	water	1200	no	schist-gneiss	curb

Notes: "(salt)" massive salt deposits, "end/abut" end wall or abutment wall, "sea/pile" seawall or pile

Optical microscopic examination found that the coarse aggregate varied considerably within the structure. The coarse grained, crushed, pink aggregate found in a number of cores suggested that for some period of time concrete delivered to the construction site was produced exclusively with this aggregate. This type of aggregate is not expected to be reactive and was not found to be associated with ASR.

Schist and gneiss were found in other cores. These types of aggregate are expected to be deleteriously reactive with alkalies and cores containing these aggregates invariably contained ASR. This aggregate is consistent with material from the Princeton quarry. In several samples, the characteristic "greenstone" appearance of aggregate from this part of North Carolina was evident.

In many samples, a combination of the non-reactive pink, granitic aggregate and reactive schist and gneiss were found in the same core. The combination of aggregates found in some cores is probably due to mixing at the ready mixed concrete plant. Several pre-approved sources may have been used by the concrete supplier over time depending on market forces and availability, with little attempt to segregate the stockpiles.

A white quartzite was also occasionally found mixed with the schist and gneiss, and never found alone. This material was not considered likely to be deleteriously reactive with alkalies. It is believed to be part of the deposit containing the schist and gneiss.

Fine aggregate used in this bridge did not indicate reactivity. No deposits of ASR gel were associated with the fine aggregate. The fine aggregate was predominantly quartz composed of large crystals and therefore non-reactive.

Figure 5.2.2A, a 50x of a thin section from core #4, obtained from a seawall, shows the very fine grained, strained silica in the schist. Some apparent micro-cracking in the aggregate is also visible. The photograph also shows the large crystals typical of the fine aggregate. One aggregate particle, which is clearly multi-crystalline, can also been seen.

Figure 5.2.2B shows a another section of coarse and fine aggregate from the same core in which the simple crystal structure of the fine aggregate is seen under cross-polarized light. Figure 5.2.2C is also from core #4 and shows a section of the pink aggregate. The differences in crystal size between the two coarse aggregates are evident. There was little indication of straining in the pink aggregate. Ettringite was found in the air voids.

5.2.3.2 Air Content (15035)

Optical microscopic examination found the air content of the cast in place concrete was typically good. Due to the location, frost durability of the concrete is not a concern, even though ettringite fills many of the air voids. Air content was not related to the degree of cracking noted in different sections of the bridge.

5.2.3.3 Presence of Ettringite and ASR (15035)

Ettringite was found in the air voids of much of the concrete, particularly that close to the surface. Very large quantities of ettringite were found in cores from the seawall or other areas with significant direct exposure to salt. Ettringite was also found in the aggregate paste interface of two cores (#1 and W9A), both with significant exposure to salt. In cores from the pile caps, which contained pink aggregate, ettringite was not uncommon, but was not

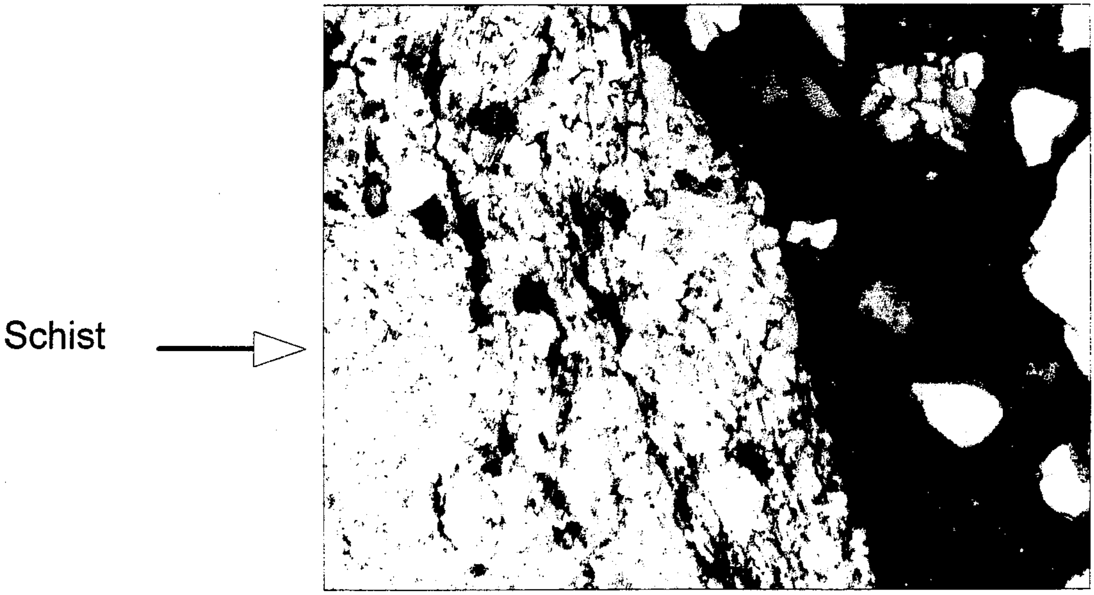

Figure 5.2.2A Thin Section, Schist in Core # 4, Bridge #15035

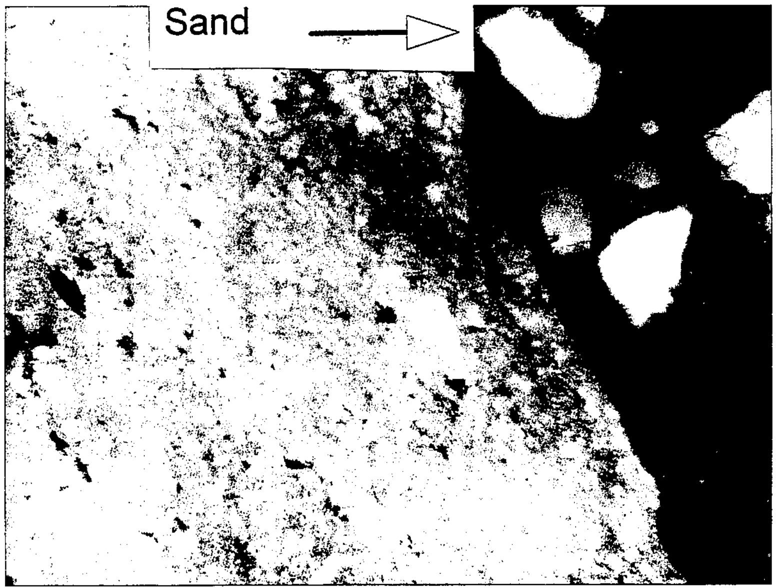

Figure 5.2.2B Thin Section, Schist in Core # 4, Cross Polarized, Bridge #15035

Figure 5.2.2C Thin Section, Non-Reactive Aggregate in Core # 4, Bridge #15035

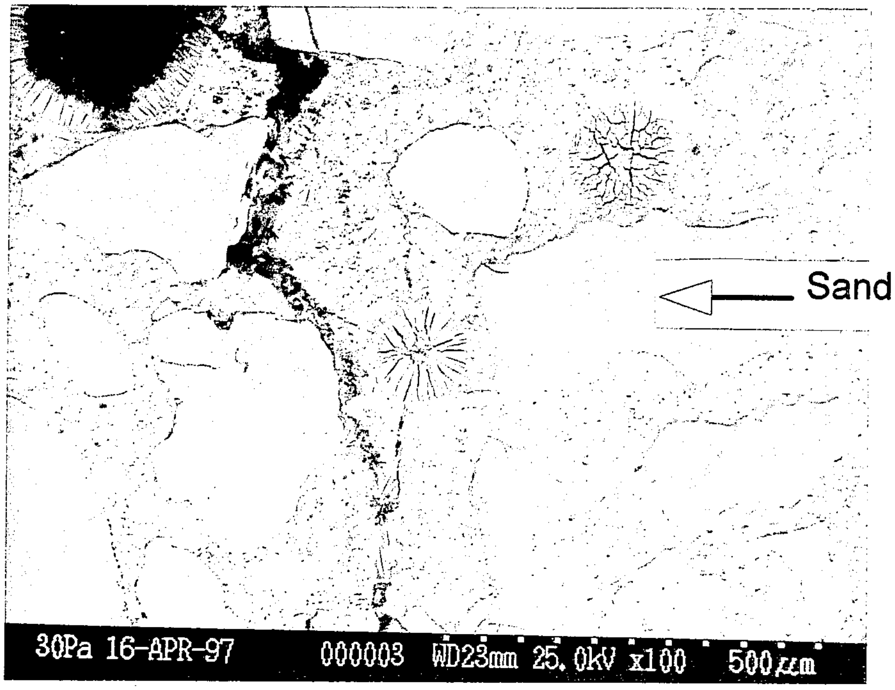

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Figure 5.2.3A Cracking - Ettringite, Thin Section, SEM, Core # 4, Bridge #15035

found in excessive quantities. Ettringite and ASR was frequently commingled in cores containing schist and gneiss. The pile caps were sound, with no significant cracking and no ASR.

The quantity of ASR found in microscopic examination varied widely. Some cores contained significant ASR deposits while there was a lack of significant ASR in others. The extent of deterioration also varied widely with some sections showing little surface deterioration to date. In a number of specimens with ASR, however, surface cracking was evident and matrix softening and microcracking were clearly visible with the SEM.

ASR and ettringite were frequently found in close proximity in cores from deteriorated sections. In several cases both ASR and ettringite were found close to the original surface of the member, however, the quantity of each varied significantly. No strong correlation was found between the quantity of ASR and the quantity of ettringite.

In some specimens, there is evidence that damage occurred due to ettringite. In Figure 5.2.3A, copious quantities of ettringite were found in a thin section from a seawall. All of the small air voids are filled with ettringite and the large void has a distinctive rim of ettringite crystals along the inside edge. While the "mud crack" appearance of the material in the air void in the top right looks suspiciously like ASR, EDX indicated that it is apparently a combination of ettringite and thaumasite, an ettringite relative.

Cracks emanate from the ettringite filled void in the approximate center of the photograph and appear to be the result of the ettringite. However, the large crack running from the top to the bottom of the photograph, just to the left of center, is the result of ASR deterioration. The small, ettringite induced cracks formed after the large ASR induced crack.

The cracked aggregate at the top of Figure 5.2.3A is seen at a higher magnification in the top right corner of Figure 5.2.3B. The deterioration associated with ASR is more significant than the deterioration found with ettringite. This was a typical finding.

In Figure 5.2.3C, ettringite filled air voids are clearly visible with the larger air voids being only partially filled. No significant cracking is associated with the ettringite. However, the coarse aggregate particle in the top of the photograph shows significant deterioration along the edge associated with matrix softening and cracking. This deterioration is associated with the formation of ASR gel.

In core #1, ASR deposits were noted within fractured aggregate. In core W9A, the presence of aged ASR was found along with the ettringite. An aggregate socket contained both ASR and ettringite but the quantity of ASR was significant and the quantity of ettringite was much smaller. An air void in close proximity to the socket was filled with ettringite indicating the ettringite was preferentially deposited in the air void.

These factors all indicate that while some of the cracking is certainly related to ettringite, the primary cause of concrete deterioration in the more heavily cracked sections at Bridge 15035 was due to ASR. Therefore, although significant quantities of ettringite were found in cores from cracked concrete, it appears that most of the deterioration is related to ASR rather than ettringite.

5.2.4 Chemical Analysis and Area of Fluorescence (Uranyl Acetate Treatment) (15035)

A summary of the chemical analysis, including water soluble alkali content, percent area of ASR gel as indicated by fluorescence under ultra-violet light after treatment with

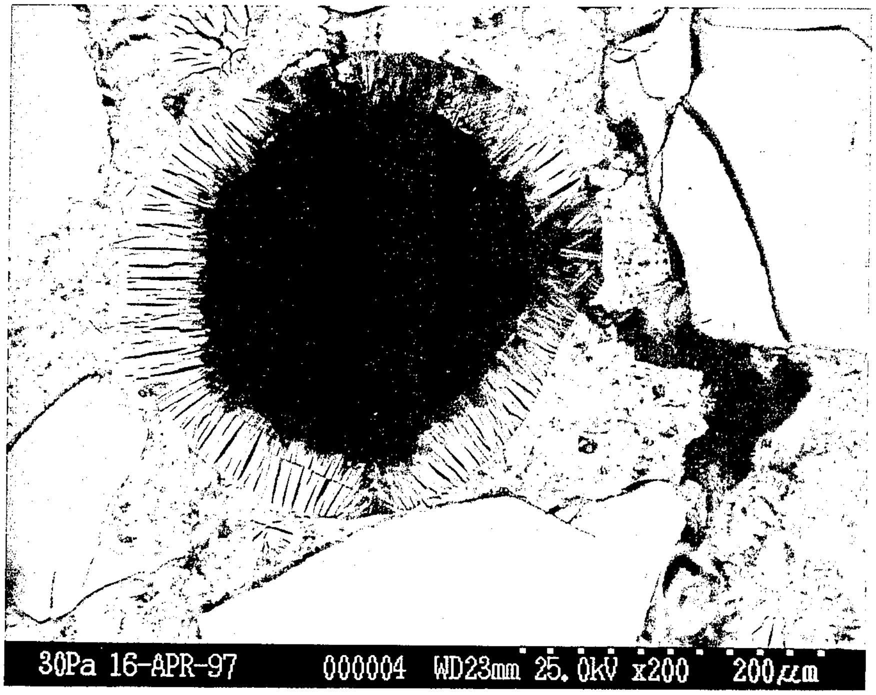

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Figure 5.2.3B Ettringite in Air Void, Thin Section, SEM, Core # 4, Bridge #15035

Figure 5.2.3C Ettringite in Air Voids, Thin Section, Core #4, Cross Polarized, Bridge #15035

uranyl acetate, estimates of cement content and cement alkali levels, whether or the core was from a cracked section, and general aggregate type are provided in Table 5.2.4.

5.2.4.1 Chloride Content (15035)

The chloride content of the concrete from both the cracked and uncracked sections of the bridge was high, consistent with sea salt exposure. A gradient exists.

5.2.4.2 Water Soluble Alkali Content and Percent Area of Fluorescence (15035)

In Figure 5.2.4, the water soluble alkali content of specimens from various locations on the bridge was plotted against the percent area of fluorescence of the specimens, after treatment with uranyl acetate and viewed under ultra-violet light. Two facts are clear from this graph. First, the alkali content of all of the cores tested except one was extremely high and reasonably consistent. Second, no correlation exists between alkali content and the amount of ASR gel in the core.

The one specimen in this graph (#1, interior) with virtually no fluorescence and a low alkali content was a section from the inside of a core taken from the lip on a pile cap, approximately 50 mm (about 2 inches) from the surface. The outside end of the core showed evidence of significant ASR and cracking, indicating that a significant source of the alkalies is seawater. This core contains both reactive and non-deleteriously reactive aggregate, and indicates that ASR extends only partly into thicker members.

In Figure 5.2.5, the uranyl acetate treated, split face of core W9A is shown, as seen under UV. This photo show that the vast majority of the gel found in this core exists near

Table 5.2.4 Chemical and Gel Analysis, Bridge #15035

Core ID	% AFM	Alkalies		K / Na	Cement Content			In-Situ Cracking	Aggregate Type
	adj for length	TEA, % cmt	Concrete, kg/m³	(mass)	kg/m³	pcy	method		
E4	7.25	1.03	3.21	2.5	380	640	type (AA)	yes	mix
E1	3.27	0.87	2.84	2.5	400	670	Si and Ca	no	granite[1], mix
13	5.65	0.86	3.51	0.3	335	560	type (A)	no	mix
W9A	2.29	1.29	4.03	2.0	380	640	type (AA)	yes	schist & gneiss[2]
9	1.84	1.41	3.89	0.8	335	560	type (A)	no	schist & gneiss[2]
E2	1.18	1.38	3.79	2.8	335	560	type (A)	no	granite
1	0.44	0.32	0.74	0.8	280	470	Si and Ca	minor	mix

[1] *aggregate is almost exclusively granite, with no indication of gel; however, one large piece of strained gneissic material was found near the top of the core which accounted for all of the fluorescence*

[2] *schist and gneiss are very fine grained and contain strained crystals*

"AFM" is the Area of Fluorescing Material; "TEA" is Total Equivalent Alkalies, expressed as Na_2O (TEA = $Na_2O + 0.657 \cdot K_2O$) as percent by mass of portland cement; "Soluble" refers to water soluble; "kg/m³" is kilograms per cubic meter; "pcy" is pounds per cubic yard; "method" refers to the method of estimating portland cement content, either by "Si" (silica content), "Si and Ca" (silica and calcium content), "type (A) or (AA)" (estimated based on class of concrete probably used, Class A or AA).

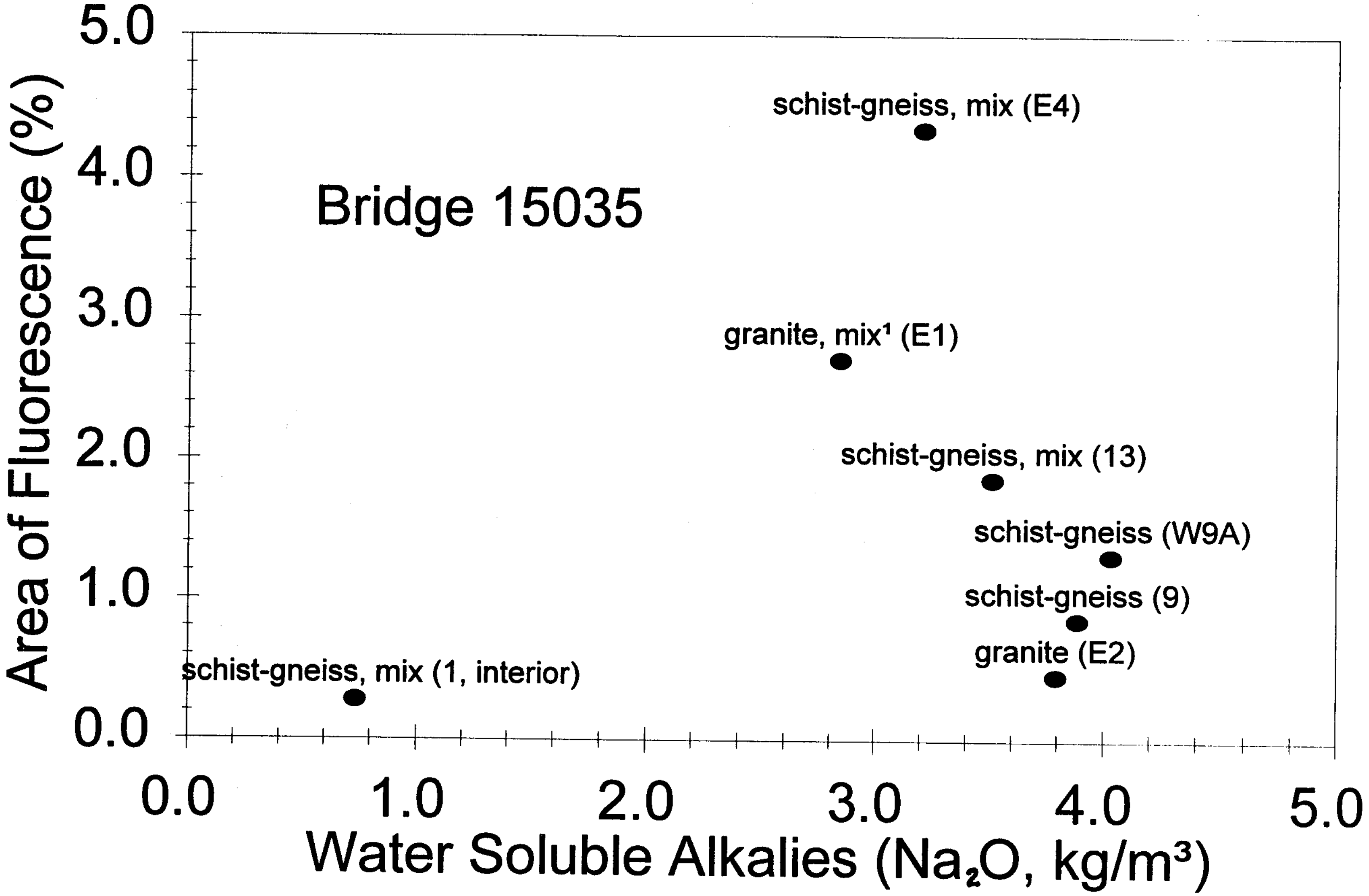

Figure 5.2.4 Relationship Between Concrete Alkali Content and Quantity of ASR Gel, Bridge #15035

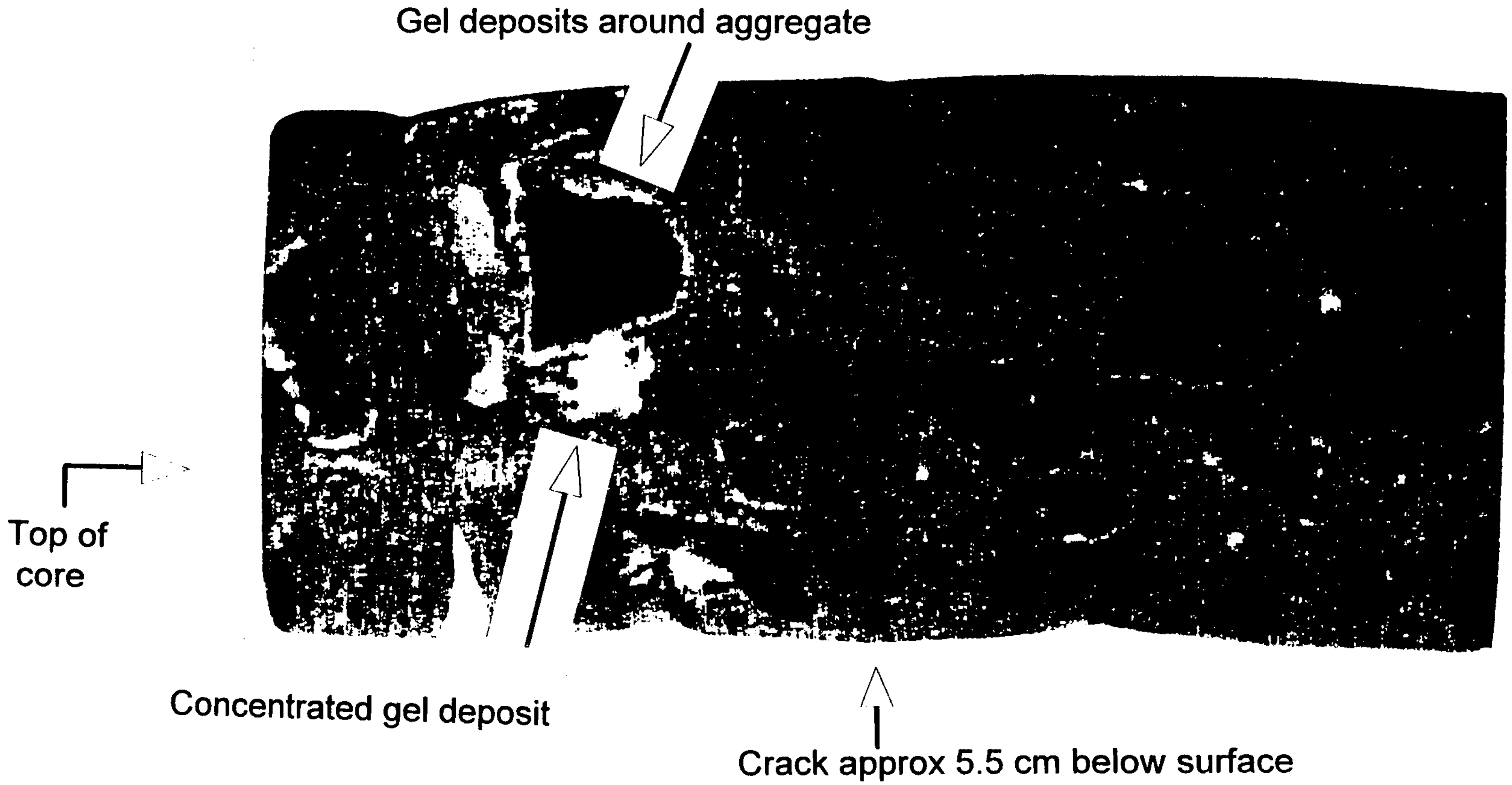

Figure 5.2.5 Photograph of Core Treated with Uranyl Acetate Showing ASR Gradient, Bridge #15035

the surface, and in close proximity to reactive aggregate particles. A crack extends transversely across the core at a depth of approximately 5.5 cm below the surface. The crack is small and difficult to see in the photograph but it marks the approximate boundary between massive ASR associated with significant cracking and ASR which is present in only small quantities with no associated deterioration. Comparing the results of cores W9A and #1 (interior), the deleterious ASR appears to extend to a depth of approximately 5 to 6 cm below the surface of the member.

Although there is a definite gradient of ASR, the percent of fluorescing material was not adjusted for length in these figures since the level of the alkalies is so high and the blending of aggregates minimizes the usefulness of quantitative assessment.

5.2.4.3 Alkali Content as a Percent of Cement (15035)

As with Bridge #83016, the alkali content was estimated as a percent of mass of the portland cement. Results are provided in Figure 5.2.6. The cement alkali levels calculated are much greater than the maximum possible from cements typically used in this part of the US, which implies that the high alkali content is due to seawater or salt spray exposure. It is not possible to control deterioration due to ASR by controlling only the alkali content of cement in concrete exposed to salt water or salt spray.

There is sufficient alkali in cores other than #1(interior) to cause alkali-silica reactivity in conjunction with reactive aggregates. It is significant to note that a high alkali content was present in parts of the structure, such as curbs on the deck, not routinely in direct contact with seawater. Therefore, exposure to salt spray provides alkalies sufficient to

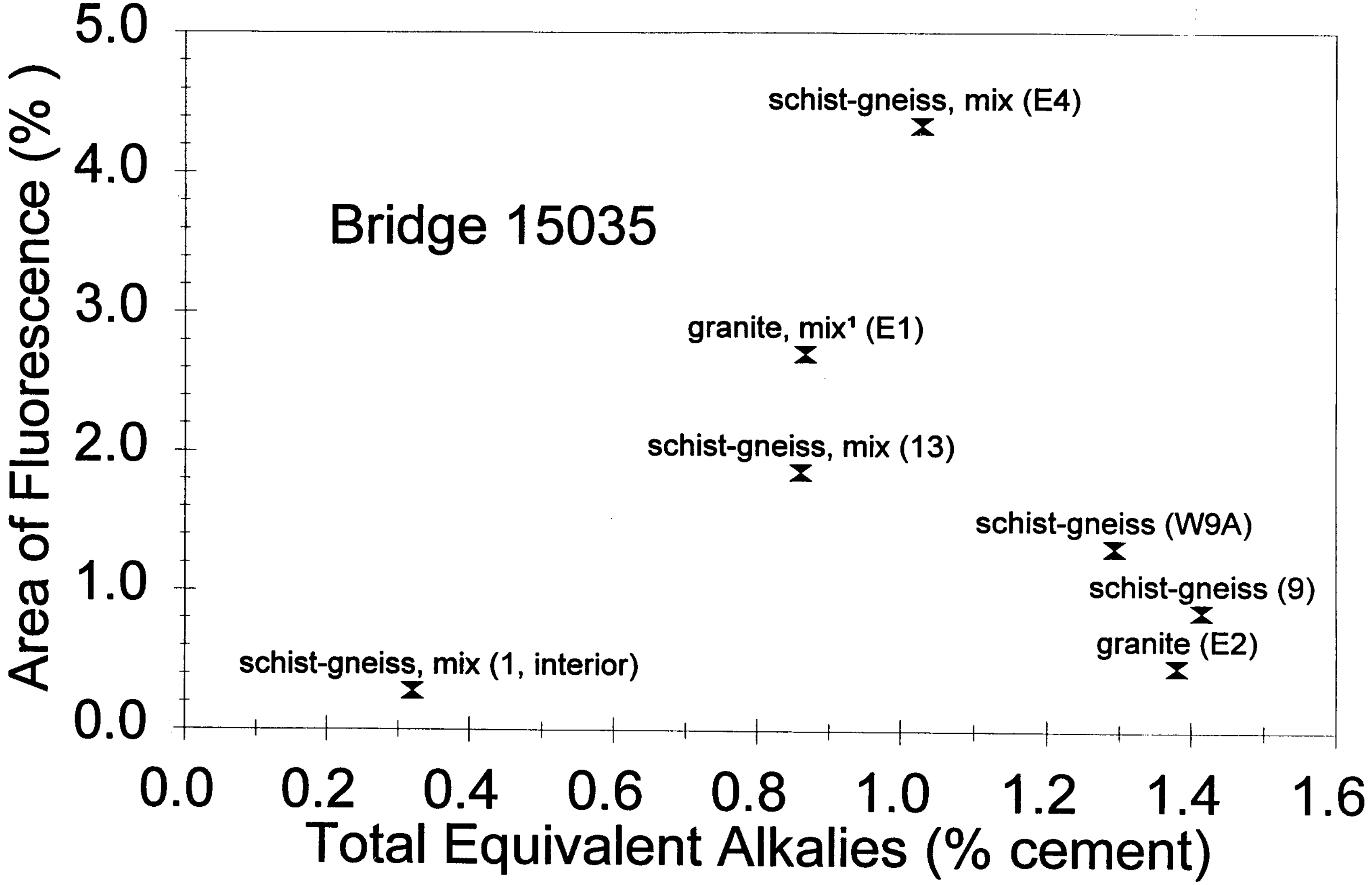

Figure 5.2.6 Relationship Between Cement Alkali Content and Quantity of ASR Gel, Bridge #15035

produce ASR, at least in the near surface region. Figure 5.2.6 also shows that aggregate which is classified as non-deleteriously reactive based on petrographic analysis was not found to be reactive even in a severe exposure.

5.2.5 Conclusions (15035)

5.2.5.1 Blending of coarse aggregates and use of multiple sources of coarse aggregate are evident in the bridge.

5.2.5.2 The differences in deterioration of sections of Bridge #15035 are primarily a function of aggregate reactivity.

5.2.5.3 ASR in the concrete produced with low alkali portland cement may extend only a short distance (50 mm or 2 inches) in this bridge. Due to the cracking produced by the ASR, additional alkalies may accumulate at greater depths, eventually causing additional ASR. However, the process is likely to be relatively slow at this stage.

5.2.5.4 ASR cannot be controlled by limiting the alkali content of portland cement in concrete structures along the coast or otherwise exposed to significant external alkalies.

5.2.5.5 Aggregate which may be classified as non-deleteriously reactive based on petrographic analysis was not found to be reactive even in a severe exposure.

5.2.5.5 Aggregate which had been shown to be reactive both in service at other sites and in laboratory tests was not deleteriously reactive in concrete with very low alkali contents (less than 0.4% total equivalent alkalies).

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

6.1.1 No deleterious ASR was generated in Bridge #83016 when the water soluble alkali content, expressed as sodium oxide (Na_2O), of the concrete was less than 1.5 kg/m^3. Compliance with this limit can be readily determined with simple tests and can therefore be used in specifications, if desired.

6.1.2 Alkali Limits

A. Based on the results of this study, a total equivalent alkali content, expressed as sodium oxide (Na_2O), of less than 0.55% by mass of the portland cement is apparently innocuous with reactive phyllite, for concrete in service not more than 17 years.

B. Based on the results of this study, a total equivalent alkali content, expressed as sodium oxide (Na_2O), greater than 0.60% by mass of the portland cement will result in deleterious expansion of concrete containing reactive phyllite in less than 16 years.

6.1.3 Based on residual expansion testing, cracking of the sound sections of Bridge #83016 due to ASR may occur in the future but it will probably be minimal. Cracking of the previously cracked sections will continue.

6.1.4 Potassium oxide appears to be more important in long term deterioration due to ASR than sodium oxide, at least for alkali reactive phyllite.

6.1.5 Blending of coarse aggregates and use of multiple sources of coarse aggregate are evident in Bridge 15035.

6.1.6 The differences in deterioration of sections of Bridge #15035 could be traced to aggregate reactivity in the presence of extremely high alkali contents due to the exposure to salt water or salt spray. ASR cannot be controlled by exclusively limiting the alkali content of portland cement in concrete structures along the coast or otherwise exposed to significant external alkalies.

6.1.7 ASR in concrete produced with low alkali portland cement may extend only a short distance (50 mm or 2 inches) into thick members in Bridge #15035. Due to the cracking produced by the ASR, additional alkalies may accumulate at these or greater depths, eventually causing additional ASR. However, the process is likely to be relatively slow.

6.2 Recommendations

6.2.1 Frequent inspection of critical sections of Bridge 83016, particularly the substructure, where continued deterioration due to ASR is expected, is highly recommended.

6.2.2 Determination of water soluble alkali, cement and ASR gel contents, determined by the M&T Unit, of additional cores from the north approach slab and from the barrier rail and slab of Span D, will provide additional sample points which may provide narrower limits on maximum permissible alkali content.

6.2.3 Deterioration due to ASR in Bridge #15035 primarily occurs in the seawalls, the abutments and end-walls, and elements of the superstructure such as curbs. Of these elements, the abutments are the most critical from structural considerations. These may need to be replaced or reinforced prematurely.

6.2.4 Deterioration of one of the hollow core panels was related to corrosion of the mild steel rather than ASR. The significance of this finding must be evaluated separately.

IMPLEMENTATION AND TECHNOLOGY TRANSFER PLAN

The scope of technology transfer involves providing findings, conclusions and recommendations to the Materials and Tests Unit and the Maintenance Division of the North Carolina Department of Transportation.

The methods of technology transfer of the results of this investigation include:

1) providing a Final Report to the Technical Advisory Committee,

2) presentation of the findings to the Technical Advisory Committee,

3) presentation of the findings to selected NCDOT personnel, chosen by the Technical Advisory Committee, if desired, and

4) delivery of a sample of an alternative to uranyl acetate, developed in this study, to Mr. Bill Medford of NCDOT Materials and Tests Unit for further evaluation.

The findings of this research may be considered for implementation in at least two areas. Interim specifications regarding the alkali limits of cement or concrete may be reviewed and bridge or pavement maintenance strategies, including inspection schedules, may be affected.

The new material developed in this study provides an alternative to uranyl acetate. This new material permits visual identification of ASR gel similarly to the uranyl acetate. However, the new material is not radioactive and poses no disposal problems for the concrete after use. A minor drawback is that the new material is effective for a shorter period of time (several days to a week). This material may be useful to both M&T Unit and to other NCDOT crews during inspections.

M&T Unit Implementation (Specifications): The findings of this study strongly suggest that optional AASHTO M 85 specifications regarding alkali content of cements are both needed and may be sufficiently restrictive to provide adequate protection against deleterious ASR in North Carolina for structures not exposed to external alkalies. The maximum total equivalent alkali content permitted may be reduced from 0.60% to 0.55% by mass of the cement in the absence of other controls.

The findings of this study also indicate that ALL concrete structures and pavements exposed to seawater or saltwater spray will have a high enough alkali content over time that ASR cannot be controlled by limiting cement alkalies. This implies that additional preventive steps must be taken to protect concrete from ASR in concrete exposed to seawater or seawater spray. Aggregates must be non-reactive or concrete must contain mineral admixtures such as fly ash, ground granulated blast furnace slag or silica fume (slurry only) in sufficient quantities to be effective in controlling ASR.

The precise boundaries of the region where this exposure is critical is unknown but it may be possible to tentatively identify them from the study conducted by Mr. Bill Medford of the NCDOT Materials and Tests Unit regarding chloride exposure. Areas with significant deicing salt application, such as Interstate pavements and structures in the piedmont and mountain areas, are also likely candidates for this restriction.

Other findings of this study indicate the continued need for separate approval by M&T Unit for each unique set of raw materials or significant change in concrete proportions on each project.

Maintenance Implementation: The impact of this and related NCDOT research on changes in inspection or maintenance strategies to reflect the influence of ASR may be summarized in three statements:

1. If no problem with ASR has appeared within 15 - 18 years, any ASR which does occur will be limited in effect and consequences. Structures with potentially serious ASR problems should experience noticeable map cracking within about 5 - 9 years in most cases.

2. In those structures in which ASR has occurred, deterioration is more rapid once cracking has been initiated. A more frequent inspection schedule should be considered for structures with ASR induced map cracking, at least for any affected, critical sections, segments or members with significant map cracking and readily available moisture. However, the rate of deterioration of many structures with map cracking due to ASR may still be relatively slow where moisture availability is limited, such as well drained bridge decks and superstructures.

3. Repair or maintenance options are unfortunately few and sensitive to the precise nature of the affected concrete structure or pavement. Replacement may be required for critical members far earlier than the end of the original design life in structures with significant ASR and available moisture.

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