



TRC9210

Durability of Bridge Decks

Margaret LeClair

Final Report

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***Planning and Research Division
Arkansas State Highway and Transportation Department
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Arkansas

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The opinions, findings, and conclusions expressed in this publication are those of the author and not necessarily those of the Arkansas State Highway and Transportation Department or the Federal Highway Administration.

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INTRODUCTION

In recent years, premature bridge deck cracking has become a major concern of the Arkansas State Highway and Transportation Department. Research Project TRC-9210, "Durability of Bridge Decks," was initiated to investigate the causes of this premature cracking and develop solutions to reduce or eliminate this problem. A literature review has been conducted on this subject. Its purpose was to examine the causes of cracking, define the different types of concrete cracking, and review past research on the subject of concrete cracking and bridge deck durability.

Since concrete exhibits inherent cracking tendencies due to relative weakness in tension, this research concentrated on the causes of premature cracking and processes to reduce, not eliminate the problem. The physical work on this project involved site selection and surveys, investigation of the concrete mixes and pour histories, investigation of repair methods, investigation of the use of synthetic fibers of preventive measures, and the petrographic examination of cores obtained from the bridge decks on the project.

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In reviewing the relevant literature on concrete durability, it was discovered that a large amount of literature existed on this subject. Since it would not be possible to review all of the literature on the general topic of concrete durability, efforts were concentrated on the following: specifically bridge deck durability, including cracking; preventive and repair methods, i.e. fiber reinforced concrete and the use of certain concrete sealers; and the use of fly ash and its effects on concrete. A review of this pertinent literature follows.

2.1 Bridge Deck Cracking

In considering bridge deck durability, the problem of cracking is quite evident. In some cases, the cracking may be the result of overloading, cyclic fatigue loading, or the interaction between the concrete materials and their surroundings. In other cases, the cracking is the result of thermal, drying, or plastic shrinkage. Thermal shrinkage occurs when temperature changes of the concrete are accompanied by volumetric changes. Plastic shrinkage occurs when water is evaporated from the surface at a faster rate than it rises to the surface. "Drying shrinkage occurs as hardened concrete loses moisture to the atmosphere." (3) Other problems that can occur are decreased durability resulting from corrosion of reinforcement, excessive deflections due to a decreased section, freeze-thaw damage, and increased sulfate attack. A list of intrinsic stresses that can result in cracking is shown in Table 2-1.

2.1.1 Thermal Cracking

The investigation into thermal shrinkage determined that cracking

occurs when the temperature changes associated with thermal shrinkage are accompanied by stresses and/or movements. These temperature changes can be the result of internal or environmental forces. These stresses generally occur after initial hardening. "While concrete is fluid, it can adjust to changing temperatures without developing stress." (3) Concrete has a tendency to set at temperatures ranging from 25 to 30° C (45 to 55° F) hotter than ambient. As this happens, shrinkage will occur and stresses will be generated and preserved as the concrete cools to ambient. These temperature variations should not create a sudden loss of strength. However, this could be the result in very extreme cases. "The primary detrimental effect from the temperature variation is the formation of unacceptable cracks in the concrete that reduce the serviceability of the bridge." (7) These cracks can contribute to accelerated deterioration which could lead to a strength loss. Other problems that can occur are decreased durability resulting from corrosion of reinforcement, excessive deflections due to a decreased section, freeze-thaw damage, and increased sulfate attack. Temperature gradients are a definite consideration while concrete is in its hardening stage. "In order to minimize the temperature gradient in the concrete, the time of casting and the concrete mix design should be determined such that the peak concrete temperature does not occur at the same time as the low air temperature of the day." (4) In addition to heat of hydration, another factor that needs to be considered is heat gain to exposed surfaces from solar radiation. The temperature gradient will be greater if the concrete is placed in the morning as opposed to the afternoon or evening.

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This is due to the fact that concrete placed in the morning has a longer exposure to the sun. "The higher concrete temperature resulted in a greater drop of the surface concrete temperature when exposed to the lower night time temperature, thus a greater temperature gradient developed in the concrete."(4) Tensile stresses are the result of the temperature difference between the hot interior and cooler exterior concrete. These stresses tend to occur whenever concrete is restrained

considered. Heat of hydration can be minimized by reducing the amount of cement in the mix design. A reduction of cement will, in turn, reduce the heat of hydration. "A popular rule of thumb for estimating the temperature rise in concrete, starting at moderate construction temperatures, include a 7 to 8°C (13 to 15°F) increase in peak temperature with each 45 kg (100 lb) of portland cement per 0.76 cubic meter (1 cubic yard)."(3) Utilizing a slowly hydrating cement or partial substitution

Table 2-1

(Fibermesh Specification Data)

| INTRINSIC STRESSES THAT CAN CAUSE CRACKING | | |
|--|--|-------------------------|
| TYPE | PRIMARY CAUSE (EXCLUDING RESTRAINT) | TIME OF APPEARANCE |
| Plastic settlement | excess bleeding | 10 min. - 3 hrs. |
| Plastic shrinkage | rapid early drying | 30 min. - 6 hrs. |
| Early thermal contractions | excess heat & temperature gradients | 1 day - 2 or 3 weeks |
| Long-term drying shrinkage | inefficient joints | several weeks or months |

from changing volume. Amount of concrete placed does not appear to have an effect on the development of tensile stresses. "The magnitude of these residual stresses is affected by the coefficient of thermal contraction/expansion of the concrete."(3) Several factors influence the coefficient of thermal contraction/expansion. These factors include aggregate type, cement content, water-cement ratio, magnitude of temperature change, concrete age, and relative humidity. Of these factors, the most influential is the aggregate type. This is due to two primary reasons: 70-80 percent of the concrete volume is aggregate, and their coefficient of thermal expansion/contraction differ greatly depending on the source.

In an effort to minimize thermal cracking, several procedures can be

with fly ash may minimize the rate of heat generation. "The decrease in the effectiveness of fly ash in decreasing the peak temperature as the casting volume increases indicates that fly ash hydrates at a slower rate than cement but may produce nearly the same amount of heat over a period of days."(4) The use of non-insulating formwork and the use of structural elements that have a high ratio of surface area to volume may minimize the potential for containing heat during the hydration process. In an effort to minimize the thermal cracking potential, keeping the concrete and its support girders at similar temperatures appears to be another solution. Initial heat gain may be minimized by cooling the concrete with a water spray. In the process of studying temperature variation in concrete, it is important to realize that high temperatures by

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themselves do not cause concrete to crack. The cracks result when stresses greater than the tensile strength of concrete are produced due to relative strain changes resulting from a temperature gradient. The ease in adjustment to changing temperatures while concrete is in the plastic state is due to a low elastic modulus and high creep properties for concrete at this early stage.

2.1.2 Drying Shrinkage Cracking

Cracking due to drying shrinkage is another concern of bridge deck durability. The cause of drying shrinkage appears to be the result of a decrease in moisture content of the calcium silicate gel component of the cement paste. The volume change due to the moisture loss from drying shrinkage is a characteristic of concrete. Drying usually results in about 0.04% to 0.08% of the total shrinkage. However, this can be reduced by up to one-half if restraint by reinforcement is present.

It is believed that if this shrinkage could occur without restraint, the concrete would not crack. The strains resulting from loss of moisture correspond to a thermal strain which would be generated by a temperature drop of 37.8°C (100°F). This moisture loss can result in shrinkage values as much as one percent per unit length. However, restraint usually reduced this number by at least one half.

Tensile stresses usually develop when the combination of shrinkage and restraint is present. This can include differential shrinkage between the surface and the interior concrete. The magnitude of these stresses is influenced by a variety of factors. These include the amount of shrinkage, degree of restraint, modulus of elasticity, and the

amount of creep. "the amount of shrinkage is only one factor governing the cracking."(6) Cracking occurs when the tensile strength of the concrete is exceeded. "Cracks may propagate at much lower stresses that are required to cause crack initiation."(5)

Drying shrinkage is influenced by several factors. These can include composition of cement, type of aggregate, water content, and mix proportions. "The rate of moisture loss or the shrinkage of a given concrete is greatly influenced by the size and shape of the concrete member, the environment, and the time of drying exposure."(6) It is important to remember the factors influencing shrinkage do not act independently of each other. The factors, if more than one is present, have a cumulative effect on the concrete. With an emphasis on early compressive strength gain and low slump, drying shrinkage has increased over the past few years. Water content and amount and type of aggregate appear to be the most influential factors of drying shrinkage. Reduction in the amount of shrinkage can be accomplished by reducing the water content, increasing the amount of aggregate or requiring a higher stiffness aggregate. "Since coarse aggregates serve as restraint, the shrinkage of concrete will decrease as the volume of coarse aggregate is increased."(3) If the maximum aggregate size cannot be increased, the mortar requirement and mixing water content will be reduced. The result of this is a reduction in shrinkage.

Reducing mixing water requirements should also result in shrinkage reductions. This factor appears to be the most controllable. Minimizing water contents could be

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controlled by the use of chemical admixtures to minimize water requirements, minimizing workability requirements, and avoiding practices that lead to increasing water requirements, such as high fresh concrete temperatures.

The composition of the cement used in a concrete mix can also effect the shrinkage potential of the mix. However, the effects are usually not large. High magnitudes of shrinkage are usually the result of high tricalcium aluminate (C_3A/SO_3) in cement. The C_3A/SO_3 ratio and alkali content generally determine the shrinkage potential. The proportion of gypsum used during production is considered to be a controlling factor of the C_3A/SO_3 . Taking this into consideration, control of early hydration reactions and decreased sensitivity to volume changes during drying could be the result if an optimum gypsum content can be determined.

2.1.3 Plastic Shrinkage Cracking

Plastic shrinkage occurs when water is evaporated from the surface at a faster rate than it rises to the surface. Plastic shrinkage cracks are considered to be a less severe form of deterioration than drying shrinkage cracks. However, plastic cracking is often confused with drying cracking. Cracking due to plastic shrinkage is usually a surface distress problem. These cracks are typically finer, shorter, and more closely spaced together—parallel to one another, 0.3 to 1m apart and 25 to 50 mm in depth. Drying shrinkage cracks are large, regularly spaced, and usually full depth. Plastic cracks have a tendency to occur within a few hours after the concrete has been placed. Plastic shrinkage cracking is generally the result of the difference in shrinkage between the surface and interior concrete. It usually occurs on

unprotected surfaces as they lose water because of evaporation. This is evident when the concrete loses its sheen. Any combination of high concrete temperature, high air temperatures, low humidity, and high wind velocity tend to accelerate the rate of evaporation. Cracks develop when, as a result of evaporation, the surface has become too stiff to flow, but not strong enough to withstand the tensile stress. The surface, as it dries, has a tendency to shrink, but is restrained by the interior concrete. The stresses that result from this restraint are too great for the surface to withstand.

Preventing moisture loss at early ages can control the development of plastic shrinkage cracks. "The use of proper curing procedures and favorable weather conditions (low wind and high relative humidity) are beneficial in achieving this goal." (8) Utilizing wind breaks and/or fog sprays during concrete placement has a tendency to reduce the occurrence of plastic shrinkage cracking. If the cracking should develop before the final set of the concrete, the cracks "can be closed by striking the surface on each side of the crack with a float." (1) The process of trowelling a slurry over the crack serves no lasting purpose.

The major cause of plastic shrinkage appears to be a low water-cement ratio. The risk of plastic shrinkage is increased if any action is taken to reduce the strength of concrete at early ages. Several simple precautions may be utilized in order to reduce the occurrence of plastic shrinkage: moistening the subgrade, forms, and any aggregate that is dry and absorptive; utilizing temporary windbreaks and/or sunshades; reduce fresh concrete temperatures by cooling the aggregate and/or mixing water; protect exposed concrete with temporary coverings, i.e. a

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polyethylene sheeting used between the time of placement and finishing; minimizing the time between concrete placement and commencement of standard curing procedures.

2.1.4 Settlement Cracking

Settlement cracking is the result of improper consolidation. Concrete has a tendency to continue to consolidate after placement, vibration, and finishing. "The degree of settlement cracking may be magnified by insufficient vibration or the use of leaking or highly flexible forms."⁽⁵⁾ Plastic concrete has a tendency to be locally restrained by reinforcing steel. Voids and/or cracks adjacent to the restraining element is the result of this local restraint. Bar size, slump, and the amount of cover have a direct bearing on the amount of settlement cracking. When bar size and slump are increased and cover is decreased, the amount of settlement cracking increases. If concrete is insufficiently vibrated, the amount of settlement cracking can be greatly magnified. The most obvious result of improper consolidation is honeycombing. This has a tendency to occur with extended vibration. The fine materials are brought to the surface during this extended vibration. It is apparent that proper mechanical vibration is of vital importance to the performance of concrete.⁽²³⁾

2.2 Crack Orientation

In addition to the causes, cracking can be defined according to its orientation with respect to the longitudinal axis of the bridge. Such cracking falls into one of the following categories: transverse, longitudinal, diagonal, pattern/map, "D", or random. Cracking on bridges is not limited to just

one type of cracking. Several types of cracking can exist on the same bridge. However, certain types of or location on bridge decks have a tendency to exhibit more of a certain type of cracking. Such as, transverse cracking is more evident on concrete and steel girder bridges, and longitudinal cracking is more evident on slab bridges. With respect to the location on the bridge, on concrete girder bridges, transverse cracking has more of a tendency to occur in the negative moment regions than in the positive moment regions.

2.2.1 Transverse Cracking

Transverse cracking appears to be the most predominant form of cracking. It is defined as "reasonably straight cracks perpendicular to the centerline of the roadway."⁽²⁴⁾ They are usually located over transverse bars in the top layer of reinforcement and can extend the full depth of the slab. These cracks can occur soon after the deck is finished and before it is opened to traffic. These cracks appear to be the result of tensile stresses that are caused by a variety of factors.

2.2.2 Longitudinal Cracking

Longitudinal cracking is defined as "fairly straight cracks, roughly parallel to the centerline of roadway."⁽²⁴⁾ These cracks have more of a tendency to appear in solid slab and hollow slab bridges. They can be full depth and vary in width, length, and spacing. The resistance that reinforcing and void tubes provide to the concrete appears to be the most significant contributing factor to longitudinal cracking.

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2.2.3 Diagonal Cracking

Diagonal cracking is defined as "roughly parallel cracks forming an angle other than 90 degrees with the centerline of the roadway." (24) One exception to this rule is if, on a skewed bridge, the concrete is placed at an angle other than 90 degrees, if the cracks are parallel to the reinforcing, they are defined as transverse cracks. These cracks vary in width, length, and spacing, and generally occur near the end of skewed bridges and over single-column piers. The cracking appears to be the result of stresses induced by restrained to load deformation and differential drying shrinkage. These cracks are usually shallow in depth and are not associated with the reinforcing.

2.2.4 Pattern or Map Cracking

Pattern or map cracking is defined as "interconnected cracks forming networks of any size and usually similar geometrically to those seen on dried mud flats." (24) These cracks are usually shallow and not associated with the reinforcing. Bleed channels and/or vertical separation caused by finishing appears to be one cause of pattern cracking. This cracking does not appear to have any detrimental effects on the performance of the deck.

2.2.5 "D" Cracking

"D" cracking is defined as "cracks usually defined by dark-colored deposits and generally located near joints and edges." (24) This type of cracking is generally associated with freeze-thaw deterioration. They appear as closely spaced crescent-shaped hairline cracks. This type of cracking appears to be a function of the pore properties of certain types of aggregate particles and the environment in which they are placed.

Cracking of the concrete occurs when the aggregate becomes saturated and freeze-thaw cycles begin. This type of deterioration generally begins near the bottom and progresses upward until it reaches the surface.

2.2.6 Random Cracking

Random cracking is defined as "cracks meandering irregularly on surface of slab, having no particular form, and not fitting other classifications." (24) The cause of random cracking vary from small imperfections in the concrete to local effects of wheel loads. These cracks can vary in width. While the cracks by themselves can be considered objectionable, their effect on overall deck durability is questionable.

2.2.7 Other Bridge Defects

Other bridge defects, in addition to cracking, can include scaling, spalling, and popouts. Scaling refers to a loss of surface mortar, or in severe cases, a loss of coarse aggregate. Spalling refers to a depression in the concrete surface caused by separation and removal of a portion of surface concrete. Popouts refer to conical fragments that break out of the surface of the concrete. The resulting holes may vary in size. In this study, the effort was concentrated on the cracking defect. However, these other defects were noted if there was any occurrence on the bridges selected for study.

2.3 Hot Weather Concreting

One item that appears to have an adverse effect on concrete is hot weather. Hot weather is defined as "any combination of high air temperature, low relative humidity, and wind velocity tending to impair the quality of fresh or

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hardened concrete or otherwise resulting in abnormal properties.”(1) A high air temperature does not always require strict precautionary measures. If air temperature is identical, precautionary measures will be more strict on a day that has low relative humidity and high wind velocity than a day that has high humidity and low wind velocity.

2.3.1 Adverse Effects

Adverse effects on concrete in the plastic state from hot weather may include the following: an increased water demand, increase rate of slump loss, and increased tendency for plastic shrinkage cracking. Adverse effects of hot weather on concrete in the hardened state may include the following: decreased durability, decreased uniformity of surface appearance, and increased tendency for drying shrinkage and differential thermal cracking.

Other factors, such as increased size of concrete delivery trucks, higher cement contents, increased use of pumping equipment, or the use of shrinkage compensating cement, can further complicate hot weather concreting operations. However, even if hot weather concreting is a necessity due to economic or construction requirements, it is possible to minimize some of the harmful effects hot weather exhibits on concrete. One important fact to remember is that “measures to alleviate or eliminate adverse effects of hot weather conditions must be planned in advance.”(1) Last minute improvisations are rarely successful.

2.3.2 Factors Affecting Concrete Temperature

One factor which demonstrates an influence on concrete temperature is the mixing water. If the temperature of

the mixing water is reduced, the concrete temperature as a whole will be reduced. Reducing water temperature is accomplished by utilizing one of the following measures: mechanical refrigeration, using ice as part of the mixing water, or using liquid nitrogen to cool the water and produce a slush. The use of ice is reported to be highly effective in reducing the concrete temperature. The reasoning behind this is that during the melting process the ice absorbs heat at the rate of 335 J/g (144 Btu/lb). The ice should be crushed, shaved or chipped, and be placed directly into the mixer. Mixing should continue until the ice is completely melted. In using ice, it is important to remember the quantity of ice cannot exceed the mixing water requirement.

Another factor for consideration in hot weather concreting is the cement itself. Heat is mechanically generated when the cement is ground. As a result, it is feasible for cement to be delivered at relatively high temperature. Because of this, it is possible for the cement to be 17 to 72°C (30 to 130°F) above the desired concrete temperature. This could ultimately have an adverse effect on the concrete even though cement is only 10 to 15 percent of the weight of the concrete mixture. This is due to the fact that temperature has a direct effect on the rate at which cement hydrates. As the concrete temperature increases, the rate of hydration increases. This results in an increased water demand which results in reduced strength and increased plastic shrinkage tendencies.

The use of chemical admixtures has been found to be beneficial in offsetting some of the undesirable characteristics of concrete placed during high ambient temperatures. Type B and Type D retarders which meet ASTM 494

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retard the set time of concrete, but not the slump loss. It is possible to use Type B, D and G admixtures in varying proportions or in combination with other admixtures as temperatures increase. The purpose of the application of admixtures is to obtain a uniform setting time. In offsetting the increase of water associated with high temperatures, the use of water reducing admixtures has been found to be beneficial. These admixtures reduce the water requirements and may increase the slump loss. An important fact to remember when considering the use of any admixture or combination of admixtures is that their performance is influenced greatly by characteristics of the cement and aggregate and their relative proportions, construction practices, and ambient conditions. It is possible that the same admixture or combination will perform differently with different concrete mixes or other applicable conditions. As a result, the use of admixtures needs to be evaluated with each different mix and conditions to fully understand their performance.

Another control measure that can be used to reduce concrete temperature is controlling the temperature of the ingredients. This can be accomplished at the point of mixing. In order for the concrete temperature to be reduced by 0.5°C (1°F), one of the following conditions must occur: Reducing the cement temperature about 4° C (8° F), reducing water temperature about 2° C (4° F), or reducing the aggregate temperature about 1° C (2° F). The most feasible of these suggestions appears to be the reduction of the aggregate temperature. Taking into consideration the fact that the greatest portion of concrete is aggregate, the greater the reduction of the aggregate temperature

results in a greater reduction of the concrete temperature. It would appear to be advantageous to keep the aggregate as cool as possible. This can be accomplished through relatively simple means: either shading the aggregate or sprinkling or fog spraying the coarse aggregate stockpiles. This would allow reduction in temperatures by evaporation and direct cooling. If the sprinkling method is utilized, it must be done in a uniform manner. Sprinkling in a non-uniform manner leads to excessive variation in surface moisture. This could lead to the impairment of the uniformity of slump.

Several very simple procedures exist in aiding the reduction of concrete temperature. These procedures can include, having batching facilities near the job site, keeping the mixing and agitation to a minimum, painting the mixer surface white, and possible continuously spraying the drum with water to obtain evaporative cooling. It is also possible to control the mixing and delivery of concrete during hot weather by simply coordinating the dispatching of trucks with the rate of placement. "The usual good concreting practices of minimizing mixing and delivery times and water content require increased diligence and coordination during hot weather."(1)

Preparations for placing and curing concrete during hot weather must be planned in advance. Transporting, placing, consolidating, and finishing the concrete should occur at the fastest possible rate. In transporting the concrete, the delivery should be scheduled so it will be placed promptly on arrival. During the placement process, the equipment available must have adequate capacity for performing the functions necessary. This could

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include having stand-by vibrators and having arrangements for another pump or crane. When considering preparation for placing concrete, the provision for proper curing should also be considered. All exposed surfaces should be promptly protected from drying. This is an attempt to avoid serious damage and cracking. Water curing seems to be the more preferred method. This method must be continuous and is more advantageous if all exposed areas are covered with a saturated material, such as burlap. However, if this method is used, the material must be kept in direct contact with the concrete surface and must be kept wet at all times. Pattern cracking has a tendency to develop if there are alternate cycles of wetting and drying. This should be avoided. In addition to water curing, the use of a white pigmented curing compound can also provide adequate curing. This method tends to be more practical for curing large areas of flatwork, i.e. highways and canal lining.

It is of extreme importance that during hot weather, the concrete not be placed faster than it can be properly consolidated. Cold joints, improper consolidation, and irregular surface finishes can be the result of this problem. When pouring a concrete deck in hot weather, this process should be confined to a small area. The use of a fog nozzle would be beneficial due to the fact that it would cool the air, steel, and forms. However, one factor to avoid would be the excessive fog spraying that results in causing water to stand on the surface during floating or trowelling. The main adverse effect of not utilizing the fog spray during hot weather is plastic shrinkage cracking. If plastic shrinkage cracking should occur before the final set of the concrete, the cracks can be

closed by striking the surface on each side of the crack with a float.

In order to assure good results with hot weather concreting, it is important to remember several items. It is extremely important that all precautions be prepared in advance. The concrete should be placed promptly and the temperature of the concrete should be as uniform as possible. The concrete should be protected from excessive drying. Finally, if it is at all possible, concrete placement during hot weather should be delayed or restricted to late afternoon or evening.

2.4 Concrete Microstructure

In order to examine possible causes and solutions to the bridge deck durability problem in Arkansas, the source of the problem needs to be considered—the concrete microstructure. The deterioration that leads to these durability problems originates at the microscopic or sub-microscopic level. The microstructure is the direct result of the mixing, placing, and curing processes. “The primary requirement of fresh concrete is that it should be of such consistency that it can be readily consolidated in forms and around the reinforcement without excessive bleeding or segregation.”(9) Microstructural development and hardened concrete properties are greatly influenced by the workability of fresh concrete. Difficulty in meeting workability requirements in fresh concrete leads to the inability to meet hardened concrete requirements. Workability is often increased with the use of chemical admixtures while hardened concrete properties are modified with the use of mineral admixtures. If the total surface area is

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increased by increasing the fine/coarse aggregate ratio, more water is required to maintain the required workability. Workability is decreased if the water content is kept constant. Another factor affecting the workability is size and composition of the aggregate. A spherical siliceous gravel tends to produce a higher slump concrete than an angular limestone of the same gradation. Slump is also increased when the aggregate size increases if the aggregate volume remains the same. When considering the workability aspects of fresh concrete, the state of hydration is an important factor. Hydration occurs when the anhydrous cement phases are mixed with water. The space between the solids, originally occupied by water, is filled with the developing hydrates. "The hydrates which form and fill space are poorly crystalline, foil-like calcium silicate hydrates, more crystalline hexagonal and cubic calcium aluminate and sulfoaluminate hydrates and crystalline calcium hydroxide." (9) As the hydration process continues, the following items have been noted: developing hydration products become denser, porosity decreases, and physical and mechanical properties performance increase. It has also been noted that the cracking of concrete may be an indicator of the inherent weakness of the interfacial hydrate-aggregate contact.

2.5 Use of Admixtures

In an effort to determine the causes of decreased bridge deck durability, the use of mineral and chemical admixtures is a factor that requires definite consideration. The purpose of utilizing admixtures is to customize the concrete for the climate, terrain, and application. The use of admixtures has been documented as

early in ancient Egypt and Greece. The concrete work performed by the Egyptians and Greeks has been noted to contain the intelligent use of color pigments. It has also been noted that the Romans added oxblood to their mortar. This proved to be an excellent air-entraining agent. Unfortunately, when the Roman Empire fell, all of this knowledge was lost. Admixtures did not make a reappearance until after portland cement was developed in 1824 and reinforced concrete in 1868. Even though calcium chloride, stearates, and color pigments were used, admixtures were considered a minor item as late as 1920. The stearates in the soap powders that farmers added to their concrete proved to be a good water-proofing agent. In 1938, the most important development is the use of admixtures was the "discovery that small amounts of well-dispersed, entrained air not only improved the workability of concrete but also increased its resistance to freezing and thawing." (10) Since that discovery, admixtures have gotten the reputation of being able to cure concrete of any problem it develops. Admixtures are divided into two categories: Mineral and chemical. Mineral admixtures include fly ash and silica fume. Chemical admixtures include accelerators, retarders, air-entraining agents, and water reducers. A list of commonly used admixtures is shown in Table 2-2.

2.5.1 Mineral Admixtures

The mineral admixture fly ash was a main consideration in this study. Fly ash is defined as "a by-product from the burning of pulverized coal in the generation of electricity. Fly ash refers to the fine particles that rise with the flue

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Table 2-2
(from *Better Roads*, October 1991)

| Some Common, Generally- Used Admixtures | | | | | |
|---|---|--|--|--|---|
| Type of Admixture | Effect on Concrete | Ingredient generally used | Advantages | Disadvantages | Major Use |
| Accelerators | Speeds up hydration of cement | Calcium Chloride | Speeds up setting time; develops strength earlier | Increase in expansion/contraction; increases corrosion of high-tension steel | In cold weather to accelerate setting time and strengthen |
| Air entraining agents | Minute air bubbles throughout the concrete | Resins, stearates, lauryl sodium sulfate, foaming agents | Increases plasticity, cohesiveness, resistance to freezing and thawing | Needs careful control; may require more frequent slump test; some loss of strength | For concrete exposed to freeze, thaw, and salt application |
| Latex (non-remulsifiable bonding type) | Improves adhesion and increases tensile and flexural strength | Organic polymer-type latex and air detaining agents | Increases water retention, adhesion to substrates, tensile strength, resistance to freeze/thaw | Difficult to finish, avoid steel-troweled finishing | Flash coat, topping, leveling, patching |
| Inert, finely divided powders | Corrects gradation of coarse aggregate | Powdered glass, sand (silica), stone dust, lime | Improves workability | Increases requirement for water and drying shrinkage; decreases strength | Improves workability |
| Water reducing agents (plasticizers) | Lowers water-cement ratio and lubricates aggregate | Polyhydroxylated polymers, lignosulfates, or hydroxylated carboxylic acids with calcium chloride | Reduces water content; increases plasticity and workability | Slows hydration and decreases early strength | Improves workability and plasticity |
| Pozzolanic finely divided powders | Reacts with free lime during hydration of cement | Volcanic ash, fly ash, calcined shale and clay, natural cement, some slag | Controls alkali-aggregate reaction; improves workability; reduces heat generation; expansion, contraction; increases strength after 30-day cure; increases resistance to sulfate attack and permeability | May cause excessive drying shrinkage; reduces durability and early strength | Controls alkali-aggregate reaction and increases resistance to sulfate attack |
| Retardant | Slows cement hydration | Zinc oxide, calcium lignosulfonate, derivatives of adipic acid | Slows setting; reduces heat due to hydration; reduces expansion and contraction | Loss of early strength; needs careful control, requires more frequent slump tests | For very hot weather and massive concrete |
| Waterproofing to reduce permeability | Reduces capillary attraction of voids in concrete | Stearic acid or compounds— calcium stearate | Decreases water absorption of concrete if no hydrostatic pressure | Reduces strength; does not make concrete waterproof | Concrete in contact with earth |

gases and are collected in the stack by particulate collectors such as electrostatic precipitators, mechanical collectors, or fabric filters rather than being discharged as an atmospheric pollutant.”(11) Fly ash is divided into two classes: Class C and Class F. Class C fly ashes have cementitious and pozzolanic properties. The high calcium oxide content of the Class C fly ash is the explanation for the cementitious properties. Class F fly ashes are generally the result of the burning of

anthracite, bituminous, or eastern, coals. These fly ashes are not self-hardening like Class C, but they do exhibit pozzolanic properties. “This means that in the presence of water, the fly ash particles react with calcium hydroxide (lime) to form cementitious products.”(12) Defining fly ash strictly by classes of coal is not considered to be accurate. Non-bituminous coals can produce Class F fly ash and Class C fly ash is not required to contain any CaO.

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Applications of fly ash include using it as an admixture or as an ingredient of blended cement. It is possible to use fly ash as a substitution for a portion of the fine aggregate. However, fly ash is generally used as a partial substitution for cement. Superior qualities of concrete at a lower cost can be obtained if the proper quality and amount of fly ash is used in a properly proportioned concrete mix.

Fly ash appears to have positive effects on concrete in both the plastic and hardened states. Most fly ashes, when used as a partial replacement of cement, improve the workability of concrete with equal water-cement ratios. Other effects of fly ash on concrete in the plastic state include reduced bleeding, increased pumpability, and a general increase of set time depending of characteristics and amount of fly ash used. Effects of fly ash on hardened concrete can include temperature rise and strength and rate of strength gain. "The initial impetus for using fly ash in concrete stemmed from the fact that at early ages Fly Ash Concrete develops less heat per unit of time than does similar concrete without fly ash." (12) Since more of the heat can be dispersed, the thermal gradient is reduced. Research conducted by the Louisiana Transportation Research Center concluded that a reduction of compressive strength results from increasing the amount of fly ash used in a concrete mix. (13) This research also concluded that the addition of an air-entraining agent further reduced the compressive strength of fly ash concrete. Another conclusion of this research indicated a maximum 40% replacement of cement with fly ash be allowed. However, a 25% maximum was recommended when possible strength

losses could not exceed 10%. It was also noted that in this research that the use of Class C fly ash in bridge decks exhibited rapid surface drying. This results in finishing problems and increases the risk of shrinkage cracks. Contradictory results were concluded by research conducted at the Center for Transportation Research and the University of Texas at Austin. (4) This research concluded that higher strengths at early ages are the result of the use of high calcium fly ashes. It was also concluded that the strengths of the concretes containing fly ashes are more dependent on the curing temperatures than the comparison to mixes without fly ash. These two types of mixes, with and without fly ash, however, are dependent on the range of temperatures associated with hot weather concreting.

2.5.2 Chemical Admixtures

Chemical admixtures can include retarders, accelerators, water reducers, and air-entraining agents. These types of admixtures can be used alone or in combinations, such as the use of a retarder with and air-entraining agent. The use of a retarding admixture slows the rate of setting of the concrete. Placing and finishing of concrete is often difficult due to the high concrete temperatures which cause the increased rate of hardening. The purpose of a retarding agent is not to reduce the initial temperature of the concrete, but to delay the initial set of concrete due to hot weather or unusual conditions of placement exist. Accelerating admixtures are used when accelerated strength development of concrete at early ages is desired. "Under most conditions, the common accelerators cause an increase in the drying shrinkage of concrete." (14) The most commonly

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used accelerator is calcium chloride. It is important to remember that calcium chloride is not an antifreeze agent and should not be used as such. The purpose of water reducing agents is to "reduce the quantity of mixing water required to produce concrete of a given consistency or to increase the slump of the concrete for a given water content." (14) The water reducers do not, however, reduce the rate of slump loss. The result of using this admixture is to increase the workability of the concrete. One category of water reducing agents is the superplasticizer. The superplasticizer tends to be more effective, but they are generally more expensive. Their effect usually lasts between 30 and 60 minutes. This is followed by a rapid loss in workability. Superplasticizers are generally limited to the use in flowing concrete and high early strength concrete.

"Air-entraining admixtures are used to purposely entrain microscopic air bubbles in concrete." (14) The size of the entrained air bubbles may range in size from 0.05 to 1.25 mm in diameter and are generally spherical in shape. Air bubbles out of this range or non-spherical in shape are usually referred to as 'entrapped' air. When air is entrained in concrete, the properties of the fresh and hardened concrete are altered. "Air-entrained fresh concrete is more plastic and workable than non-air-entrained concrete, while air entrainment in hardened concrete provides the durability necessary to resist unfavorable environmental conditions." (15) the one notable disadvantage to using air-entrained concrete is a 10 to 20% loss in strength for each 3 to 5% entrained air. However, when durability is of importance, the strength loss can be acceptable. To counteract the

disadvantage, there are several advantages to using air-entrained concrete: improved workability, decreased permeability, reduced water demand, and improved durability. The durability is improved by the air bubbles appearing to provide reservoirs to accommodate the expansion of water during the freeze-thaw cycles. If these air bubbles were not present, the expansion of the excess water could possibly rupture the concrete. Air-entrained concrete appears to be about ten times more durable than non-air-entrained concrete.

2.6 Synthetic Fibers

In an effort to reduce the amount of cracking in concrete, the use of synthetic fibers has been investigated. While the use of synthetic fibers is a relatively new process, the use of fibers in general is not a new concept. Long before the advent of conventional fiber reinforcement, the use of straw in mud bricks is noted during Biblical times, and horse hair in mortar for reinforcement is evident during the Roman Empire. A wide variety of fibers are available. Table 2-3 lists several types of fibers and their associated properties. Since the fiber reinforced concrete (FRC) portion of TRC-9210 concentrated on polypropylene and nylon fibers, this portion of the literature review will discuss only these two types of fibers. The main advantage to FRC is the elimination of the welded wire fabric that is often utilized. The fibers in FRC are distributed throughout the concrete matrix. It is important, however, to remember that the fibers are to be used as secondary reinforcement only and not to be used as a structural element. Fiber reinforced concrete has a tendency to

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Table 2-3
(From *Fiber Reinforced Concrete*, 1991)

| Fiber Type | Diameter, 0.001 in. | Specific Gravity | Young's Modulus | Tensile Strength, ksi | Strain at failure, % |
|-----------------------|---------------------|------------------|-----------------|-----------------------|----------------------|
| Steel | | | | | |
| High tensile | 4.0-40.0 | 7.8 | 29000 | 50-250 | 3.5 |
| Stainless | 0.4-13.0 | 7.8 | 23000 | 300 | 3 |
| Glass | | | | | |
| E | 0.4 | 2.5 | 10440 | 500 | 4.8 |
| Alkali Resistant (AR) | 0.5 | 2.7 | 11600 | 360 | 3.6 |
| Polymeric | | | | | |
| Polypropylene | | | | | |
| Monofilament | 4.0-8.0 | 0.9 | 725 | 65 | 18 |
| Fibrillated | 20.0-160.0 | 0.9 | 50 | 80-110 | 8 |
| Polyethylene | 1.0-40.0 | 0.96 | 725-25000 | 29-435 | 3-80 |
| Polyester | 0.4-3.0 | 1.38 | 1450-2500 | 80-170 | 10-50 |
| Acrylic | 0.2-0.7 | 1.18 | 2600 | 30-145 | 28-50 |
| Aramid | | | | | |
| Kevlar 29 | 0.47 | 1.44 | 9000 | 525 | 3.6 |
| Kevlar 49 | 0.4 | 1.44 | 17000 | 525 | 2.5 |
| Asbestos | | | | | |
| Crocidolite | .004-0.8 | 3.4 | 28400 | 29-260 | 2-3 |
| Chrysotile | 0.0008-1.2 | 2.6 | 23800 | 500 | 2-3 |
| Carbon | | | | | |
| I (high modulus) | 0.3 | 1.9 | 55100 | 260 | 0.5-0.7 |
| II (high strength) | 0.35 | 1.9 | 33400 | 380 | 1.0-1.5 |
| Natural | | | | | |
| Wood cellulose | 0.8-4.7 | 1.5 | 1450-5800 | 44-131 | — |
| Sisal | <8.0 | — | 1890-3770 | 41-82 | 3-5 |
| Coir (coconut) | 4.0-16.0 | 1.12-1.15 | 2760-3770 | 17-29 | 10-25 |
| Bamboo | 2.0-16.0 | 1.5 | 4790-5800 | 51-73 | — |
| Jute | 4.0-8.0 | 1.02-1.04 | 3770-4640 | 36-51 | 1.5-1.9 |
| Akwara | 40.0-160.0 | 0.96 | 76-464 | — | — |
| Elephant grass | 17 | — | 716 | 26 | 3.6 |

increase tensile strength and improve toughness and durability. Crack reduction and propagation without affecting shrinkage properties or hydration is another advantage to using FRC.(16)

2.6.1 Polypropylene Fibers

In 1965, the U.S. Army Corps of Engineers used polypropylene fibers as concrete reinforcement in the construction of blast-resistant structures.(2) Polypropylene is a manmade hydrocarbon polymer. The extrusion process, material hot-drawn through a die, is utilized in the manufacture of these fibers. The fibers

used in this project were the fibrillated polypropylene. The fibers being fibrillated is interpreted to mean "the polypropylene film is slit so it can be expanded into an open network of fibers."(2) The Fibermesh™ fiber, the specific polypropylene fiber used, is produced specifically for the use in concrete. These fibers are chemically inert, lightweight, and very cost competitive. With the fibers being inert, it is not affected by either acids or alkalis found in concrete. The benefits of using Fibermesh™ fibers include inhibiting plastic cracking, providing impact and shatter resistance, lowering permeability, and imparting toughness.(17) These

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fibers are added to concrete at a rate of 0.68 Kg (1.5 lb.) per 0.76 cubic meter (1 cubic yard). No changes to the mix design are necessary since this is a mechanical process not chemical. If the Fibermesh™ Fas-Pak is used, there is no need to worry about measuring the correct amount of fibers or disposal of the bags. The Fas-Pak is designed in such a way that the entire bag, with a pre-measured 0.68 Kg (1.5 lb) of fibers, is added directly to the mix either at the plant or the jobsite. The bag is made of a material that disintegrates during the mixing process. The only requirement is an additional three to five minutes of mixing time. This is to ensure proper distribution of the fibers within the concrete matrix. The only disadvantage of the fibers is in the appearance of the concrete. These fibers are easily visible in the concrete. Over time, however, they do wear off the surface. This problem can be overcome with the use of another fiber developed by Fibermesh called the Fibermesh™ Stealth fiber. This fiber has the same properties of the regular fiber, only the fibers are not visible on the surface of the concrete.

2.6.2 Nylon Fibers

The nylon fiber utilized in this research was manufactured by Allied Signal in the form of Nycon™ fibers. These fibers are a 100% pure virgin nylon 6 fiber.(18) These are also used as secondary reinforcement as a replacement for welded wire fabric. These fibers also reduce cracking, increase impact resistance, and reduce permeability. The fibers are added at a rate of 0.45 Kg (1 lb) per 0.76 cubic meter of fibers (1 cubic yard) of concrete. About 34 million individual filaments are in this 0.45 Kg (1 lb) of fibers. These fibers are also added

directly to the concrete mix in a pre-measured pack. This bag also disintegrates during the mixing process. An additional three to five minutes mixing time is also required for proper distribution of the fibers. The Nycon fibers have a “no hairy concrete” claim. This is evident during and after the finishing process as the fibers are not easily visible.

2.6.3 SHRP Research

In a study done by the Strategic Highway Research Program (SHRP-C-366), the use of fibers in conjunction with High Early Strength concrete (HES) was examined.(19) The results were not very favorable towards the use of polypropylene fibers with HES. The results of this study showed deterioration in the compressive stress-strain response when compared to the control mix, lower strengths at early ages, and lower toughness indices when comparing the results of the polypropylene HESFRC and steel HESFRC. The conclusions of this research were that when HES was being used, the use of steel fibers is more appropriate than the use of polypropylene fibers.

The primary difference between polypropylene and nylon fibers is the nylon fibers are hydrophobic in nature. With this strong affinity to water, the nylon fibers bond chemically to the concrete matrix. Polypropylene fibers exhibit a mechanical bond only.

2.7 Investigated Solutions

In studying the causes of bridge deterioration, several solutions for minimizing these defects have been developed. One consideration in the quality of the concrete. The performance of the bridge deck is a direct result of the concrete quality. The

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Portland Cement Association (PCA) has developed requirements for bridge deck concrete. These requirements are shown in Table 2-4.(24) It has been determined that shrinkage will be reduced if the largest practical coarse aggregate is used. This minimizes the mixing water, which, in turn, has a tendency to reduce shrinkage. The lowest practical slump should be used, because, excessive slump appears to promote segregation, increase bleeding, drying shrinkage, and crack tendencies.

Concrete placement, finishing, and curing should also be done properly in order to minimize bridge deck defects. Extreme care should be taken in placement of concrete so cold joints are avoided, especially horizontal or sloping cold joints. Concrete should never be moved horizontally with vibration or other methods that would promote segregation. The concrete should never be overworked. Texturing should occur as soon as the concrete has hardened sufficiently. Curing should occur as soon as the concrete has hardened sufficiently to prevent surface damage. Fog sprays or soaker hoses should be used for at least the first 24 hours. Other curing methods such as the use of waterproofed curing paper, damp burlap, or curing compounds are satisfactory if properly applied. Waterproofed curing paper should be carefully lapped and taped at all joints and must be in constant contact with the concrete to ensure no damage is caused by the wind. Damp burlap must not be allowed to dry and must cover the entire concrete surface. Curing compounds should be white-pigmented and should be applied in two applications. The second application should be perpendicular to the direction of the first application. Concrete should be cured for a minimum

of five days if the mean concrete temperature during curing is 70°F or higher or seven days if the concrete temperature is 50°F.

Bridge deck design should also be considered. Cover dimensions are a major consideration. These dimensions are dependent on the use of de-icer chemicals. If these chemicals are used, the cover should be a minimum of two inches. One and a half inches should be the minimum if de-icer chemicals are not used.

2.8 Other Research

There has been ample research concentrated in the area of bridge deck durability. Two such studies-- the State of Florida with epoxy coated rebars and the State of New York with the white pigmented curing compound have been selected for inclusion in this literature review. Also included is the study performed by the National Cooperative Highway Research Program (NCHRP) on transverse bridged deck cracking. A synopsis of each research follows.

2.8.1-Florida-- Epoxy Coated Rebars

Several studies have been conducted to determine if the use of epoxy coated rebars is beneficial or detrimental to bridge deck durability.(20) In 1976, the Federal Highway Administration (FHWA) suggested that epoxy-coated rebar could be considered a protective measure on federally funded projects. Epoxy coated rebar became the primary protection method for bridge reinforcement in Florida after FHWA adopted the measure in 1981. Procedures for implementing the standards for inspection and control of the epoxy coated rebar have always been followed by the Florida Department of

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Transportation (FDOT). However, in 1986, one of the bridges selected for study, the Long Key Bridge in the Florida Keys, began to exhibit signs of corrosion only six years after construction. This led to the speculation

feasibility of use in aggressive environments.

2.8.2 New York-- Curing Compound

Another investigation into bridge deck durability occurred in New York

Table 2-4

(from *Durability of Concrete Bridge Decks*, 1970)

| MAX. SIZE OF COARSE AGGREGATE, IN. | MAX. WATER- CEMENT RATIO | | MIN. CEMENT CONTENT | | AIR CONTENT, PERCENT (±1 PERCENT) | MAX. SLUMP, IN. (±1/2 IN.) | MIN. 28- DAY COMPRESSIV E STRENGTH, PSI* |
|---|-----------------------------|-------------------|-----------------------------------|----------------------------------|---|-------------------------------|---|
| | GAL. PER BAG | LB. PER LB. | BAGS PER CU.YD. OF CONCRETE | LB. PER CY.YD. OF CONCRETE | | | |
| In areas where de-icers are used | | | | | | | |
| 1/2 | 5.00 | 0.44 | 7.60 | 714.00 | 8.00 | 2 1/2 | 4500.00 |
| 3/4 | 5.00 | 0.44 | 7.00 | 658.00 | 7.00 | 2 1/2 | 4500.00 |
| 1 | 5.00 | 0.44 | 6.60 | 620.00 | 6.00 | 2 1/2 | 4500.00 |
| 1 1/2 | 5.00 | 0.44 | 6.20 | 583.00 | 5.50 | 2 1/2 | 4500.00 |
| In areas where de-icers are not used | | | | | | | |
| 1/2 | 5.50 | 0.49 | 7.00 | 658.00 | 5.50 | 2.50 | 4000.00 |
| 3/4 | 5.50 | 0.49 | 6.40 | 601.00 | 5.00 | 2.50 | 4000.00 |
| 1 | 5.50 | 0.49 | 6.00 | 564.00 | 4.50 | 2.50 | 4000.00 |
| 1 1/2 | 5.50 | 0.49 | 5.60 | 526.00 | 4.00 | 2.50 | 4000.00 |

that epoxy coated rebar might not be all that effective. Investigations led to the discovery that spalling due to corrosion had progressed into sound concrete, and some of the epoxy coating had completely disbonded from the substrate while the coating itself appeared to be in good condition. The investigation led FDOT to the conclusion that epoxy coated rebar is inappropriate for marine corrosion protection. In December 1988, alternatives to epoxy coated rebar were implemented for bridge deck substructure. In July 1992, the use of epoxy coated rebar was discontinued in all construction. Investigations into the use of silica fume and calcium nitrite have been conducted to determine

State. This research investigated the use of white-pigmented curing compound and its effect on shrinkage cracking.(21) This was of such concern that the New York State Department of Transportation revised its specifications to eliminate the use of the white-pigmented curing compound, require the use of wet burlap, and the addition of certain provisions for hot- and cold- weather concreting (EI 86-24). Eighteen bridges were randomly selected for survey and evaluation-- nine placed before the revision, and nine placed after the revision. Ten of the bridges were cored, and these cores were later tested for unit weight, concrete percent absorption, shear strength of bonded concrete, and resistance to chloride ion penetration. Climatological

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data was also obtained and evaporation rates could be calculated. No correlation could be made between any of the factors, before or after the policy change, and the rate of concrete cracking. It was concluded that the construction practices should be studied more carefully in order to determine the effects on curing due to the policy change.

2.8.3 NCHRP Study

which appeared to influence the occurrence of early age bridge deck cracking.(25) This effort concentrated on the occurrence of transverse bridge deck cracking. The conclusions of this research indicated that this type of cracking was not limited to one

NCHRP project 12-37 made an attempt to identify and rank the factors

geographic location. It also indicated that possible causes of the transverse cracking are weather and curing, longitudinal stresses, the use of steel girders in construction, and thermal stresses from early hydration. Possible solutions to this problem include the use of shrinkage-compensating concretes, investigating the characteristics of the coarse aggregates and cements, and the time of placement (morning, afternoon, or evening). This research also indicates that since cracking is considered to be a major problem, standard repair procedures should be developed and included in contract specifications.

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CONCLUSION

The most important fact to remember is that it is not as important to *eliminate* concrete cracking as it is to *minimize* the cracking. Since concrete is relatively weak in tension, cracking will occur when tensile stresses develop and are exceeded. Measures to reduce shrinkage, which results in tensile stresses, should be taken. If shrinkage is

minimized, cracking will, in turn, be minimized. Many defects can be attributed to hot weather. So, when hot weather concreting is a necessity, precautions must be taken to minimize the effect. Otherwise, the integrity of the bridge deck could be compromised.

3.0 PHASE ONE

3.1 Site Selection and Evaluation

Phase One of TRC-9210 involved site selection, the evaluation of these sites, investigation of the concrete mixes, and a study of the pour histories of the selected sites. It was expected that this portion of the project would yield some insight into the causes of early age bridge deck cracking in Arkansas. It was then anticipated that these results would aid in developing solutions to this problem.

In 1991, the early age cracking problem became of such concern that this project was developed according to the guidelines set forth by the Research Section of AHTD. In the early stages of the project, efforts were concentrated on the effects of fly ash in Structural Air-Entrained (S-AE) concrete. The use of fly ash was of such concern that in August of 1991, the use of fly ash in bridge deck concrete was temporarily suspended. This action applied to projects let to contract after August 14, 1991. The memo stating this fact and the resulting Supplemental Specification are shown in Appendix A.

After the appropriate personnel were informed of this decision, the subcommittee for this project developed guidelines for the selection of bridges deemed appropriate for evaluation. It had been decided that a substantial number of bridges was necessary for an accurate evaluation. It had also been decided that only the "newer" bridges should be considered. The "newer" bridges were defined as bridges that had been constructed since 1985. It was also acceptable for a bridge under construction to be considered, provided it met the criteria for selection.

The actual selection occurred using one of two methods. The first method utilized an interoffice

memorandum. A copy of the memorandum is shown in Appendix A. This memorandum was submitted to each of the ten districts in the state. It requested information on bridges which met the criteria for the project. The districts then submitted a response which contained a list of appropriate bridges and the associated information. These bridges, if deemed acceptable, were then placed on the list of sites for evaluation. The second method was relatively simple. Information on bridge decks observed by various AHTD personnel was forwarded to Research. This communication occurred with a phone call, personal visit, or a brief memo describing the condition of the observed deck. Again, if the bridge was deemed appropriate, it was added to the list. At the end of the site selection process, a total of twenty-nine bridges had been chosen for evaluation. These were located throughout the state. A list of these bridges is shown in Table 3-1.

After a site had been selected, a survey of the bridge deck deterioration was necessary. a standard form for this was developed. The *TRC 9210 Bridge Deck Distress Survey* form is shown in Appendix A. This form was utilized on every bridge deck survey for this project. The survey process was simple, but, on occasion, proved to be time consuming.

The initial step in the survey consisted of a preliminary examination of the entire bridge deck surface by the survey team members. If the bridge was of sufficient length, such as the Moro Bay Bridge with a length of 836.5 m (2744.5 ft.), one team member would examine half of the bridge, and the other member would examine the remaining half. If the bridge were relatively short, such as the Brush Creek Bridge with a length of 53.4 m (175 ft.), each team

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member would examine the entire length of the bridge. After the preliminary examination was completed, the team members then determined which portion of the deck contained the greatest amount of cracking. This became the test section for the distress survey. The section, however, was limited to 30.48 m (100 ft.) in length. A few exceptions were allowed. One exception allowed was if the bridge was less than 30.48 m (100 ft.), the entire deck was evaluated. The other exception was if it was deemed necessary by the team members to increase the length of the test section to give a more accurate representation of the bridge deck distress. The first exception occurred more often than the second. This did not appear to alter any of the results.

After the test section was chosen, the mapping of the deck distresses was the next step to be completed. This step involved the efforts of each team member. One team member would locate the cracks and spray them with water. This made the cracks easily visible when the surface water evaporated. The second team member would then sketch the cracks and any other distresses on the Survey Form. The third team member, if applicable, would measure the length of each crack and keep a running total of the length. If necessary, the team member charged with the sketching could also measure the cracks. This occurred on many occasions. This process can be seen in Figures 3A-3C. Whenever possible, the widths of some of the cracks were measured using a Crack Comparator Card (Fig. 3D). After the survey had been completed, the condition ranking of the deck needed to be determined. As a standard, the team member who completed the sketching made this

determination. The amount of cracking versus the test area was estimated in a percentage and noted on the form in the appropriate area. The general condition of the deck was then decided. The deck could be rated Good, Fair, Poor, or Critical. This depended on the severity of the distress. It is important to emphasize the fact that this rating applied to the condition of the deck surface. It did not apply to the structural integrity of the bridge itself. When this rating was completed, the bridge deck survey was also complete. An example of a completed survey can be seen in Appendix A. This process occurred on each bridge deck that had been selected.

In addition to determining the linear feet of cracking, the cracking per square foot was also determined. It was presumed that this calculation could give a better comparison of the degree of distress over the linear feet of cracking only. If the linear feet was the only factor that determined the ranking, it was possible for false degrees of distress to be given. For example, Job #60301, Brush Creek, had an initial amount of cracking measured to be 44.5 m (146 ft.), and Job #R50013, Village Creek, had an initial amount of cracking measured to be 37.5 m (123 ft.). Using only this information, the assumption would be that Village Creek is less distressed than Brush Creek. However, after determining the survey area of Village Creek to be 111.5 m^2 (1200 ft^2) and Brush Creek to be 260.1 m^2 (2800 ft^2), the cracking per square foot revealed Village Creek to be more distressed than Brush Creek. Ideally, the survey areas of each bridge would have been equal, but due to traffic constraints, or other outside determining factors, this was not possible.

3.0 PHASE ONE

3.2 Pour History Evaluation

In addition to surveying the selected bridges, it had been decided to examine the pour histories of each bridge. This process involved a very time consuming search through both the job files and materials files on each bridge. Information needed from the files included the date of pour, daily temperature, air content, slump, fly ash content, and any other pertinent information. It had been expected that this information would develop some correlation between the environmental conditions and severity of deck cracking. The results of this search can be seen in Table 3-2. It is noted that the information listed is limited to the five bridge decks which are rated to be the least severely cracked and the five most severely cracked. The reason for this limitation occurred relatively late in the project. Time constraints were placed on obtaining certain test results. With this time constraint, it was almost impossible to perform the tests on all twenty-nine bridges. It was then decided to limit the testing to these ten bridges. From this evaluation, it appeared that the use of fly ash did not enhance the severity of cracking on the bridge decks. In fact, the use of fly ash is more prevalent in the five least severely cracked bridge decks.

3.3 Mix Evaluation

The first series of tests in evaluating the concrete mix designs did not yield any useful results. A total of fourteen laboratory mixes were executed for the purpose of performing shrinkage tests. Originally, it was perceived that the shrinkage tests would show which mix had more of a tendency to shrink and develop cracking. However, as the mixing progressed, several conditions which had a negative impact developed.

The main problem was the time constraint with the concrete remaining in the molds. The concrete had to remain in the molds for twenty-four hours before the molds could be stripped. Any shrinkage that occurred during this time was not taken into consideration. Unfortunately, this early shrinkage was one of the main concerns of the project. According to research performed by other agencies, the shrinkage which occurs within the first twenty four hours is the main concern with developing a solution to early age bridge deck cracking. The other major problem that developed involved the pins used for seating the sample on the measuring device. A majority of the pins loosened as the measuring progressed. They were repaired, but it could not be shown how they affected the results. To try and solve this problem, special care was taken when placing the concrete in the molds to make certain that enough concrete was surrounding the pins to guarantee an adequate seating of the pins in the concrete. Results of this shrinkage testing can be seen in Table 3-3. After the sample measuring had ended, it had been decided to perform regression analysis using the STATGRAPHICS software program. This regression analysis showed a slight correlation between time and shrinkage. It also showed very little correlation between air content and shrinkage. These results were not deemed to be significant due to the fact that the data necessary for the regression analysis was of a questionable nature.

Information had been discovered on a test which determined the cracking potential of concrete. It was then decided to perform the Kraai Test for Crack Potential. Two sets of mixes were performed using this test method. These

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tests appeared to give better results than the previous testing method. Four samples were consecutively mixed on the same day. These were nicknamed "mega-mixes) because they were larger than any of the other laboratory mixing that had been performed. The original mix design was utilized as well as alterations to this mix design to include no fly ash, nylon fibers, and polypropylene fibers. The first "mega-mix" was performed by Heavy Bridge Maintenance. It had been decided to alter some of the conditions to encourage cracking in these samples. A computer program was utilized to determine the specific conditions necessary to encourage cracking. This program inquired as to certain conditions, such as air temperature, wind velocity, and relative humidity. The evaporation rate was then calculated and a recommendation for preventive measures against cracking was stated. With this information, it was then possible to attempt to alter the conditions to allow for cracking. Since ambient temperature and relative humidity could not easily be regulated, it had been decided to attempt to alter the concrete temperature and wind velocity. Altering the concrete temperature was accomplished by heating the aggregate prior to mixing. However, one drawback to this was an increase in the amount of water absorbed by the aggregate. This, in turn, affected the consistency of the mix by changing the water/cement ratio. This was easily resolved by calculating the amount of extra water added to the mix, and determining an accurate water/ cement ratio. Additional measures for increasing the concrete temperature were also utilized. After the concrete had been placed in the molds, heat lamps were

positioned so that they would increase the concrete temperature during the curing process. The attempt at altering the wind velocity was as simple as using fans to create wind across the surface of the samples. Standard box fans were used for this. they appeared to sufficiently perform the task necessary. Unfortunately, the humidity on this particular day was higher than had been predicted. With this high humidity, the conditions of the slabs did not encourage cracking. three of the four slabs did not develop any amount of cracking. The fourth sample did develop a small amount of cracking, but the amount of cracking was not considered significant. Since these results were not what had been expected, it had been decided to attempt this procedure again at a later date.

With the second attempt, some changes were made in the way the slabs were to be cured. The primary change was the slight elevation of the slabs. Since this test was supposed to simulate bridge deck samples, it had been decided that the slabs needed to be elevated. This was accomplished by placing the slabs on wooden blocks. Enough support was gives as to ensure the stability of the slabs during the curing process. There was no concern that the slabs would fall apart due to the weight of the concrete in the molds. The other change was an additional set of heat lamps placed over the samples. This was to try and increase the surface temperature of the concrete once it had been placed in the molds.

Environmental conditions were monitored using the National Weather Service's extended forecasts. Since these mixes were to be performed when acceptable conditions existed, pre-planning was a necessity. All of the

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materials were measured before a date for the next "mega-mix" had been set. A low temperature, low humidity day had been predicted, and the "mega-mix" was set for that day. All of the mixing was performed by Research personnel since Heavy Bridge Maintenance was not available. These mixes were altered in the same manner as the previous "mega-mix". This series of mixes proved to give the expected results. Each slab, with the exception of the mix with fly ash, developed some type of cracking within six hours of the time of placement. These slabs were monitored for several days with the expectation that more cracking would develop. One of the

cracked slabs did develop some additional cracking. The remaining two cracked slabs did not develop any additional amount of cracking. The additional cracking in the one slab was relatively small in amount, but it was noted that the additional cracking did develop. With the absence of cracking in the slab that contained fly ash, the conclusion from this portion of the laboratory testing appeared to demonstrate no correlation between fly ash and the occurrence of early age bridge deck cracking. Photographs of the mixing processes can be seen in Figures 3E-3P.

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Table 3-1

| Job Number | Job Name | District | Route | Section | Completion Date |
|------------|--------------------------------------|----------|-------|---------|-----------------|
| R10008 | West Memphis Interchange | 1 | 40 | 52 | 1992 |
| R10014 | Missouri St. Overpass Rehab. | 1 | 40 | 52 | 1992 |
| 110104 | Miss. Rvr Bridge Deck Rehab @ Helena | 1 | 49 | 11 | 1991 |
| R20063 | Hwy 65 W Terminal Interchange | 2 | 65 | 14 | 1988 |
| R30029 | Hwy 29B @ Hope | 3 | 29 | 4 | 1988 |
| 1566 | Red River Bridge | 3 | 82 | 1 | 1987 |
| 1616 | Boughton Rd. | 3&7 | 30 | 13 | 1989 |
| 1616 | Gurdon Rest Area | 3&7 | 30 | 13 | 1989 |
| R40032 | I-40 Interchange @ Alma | 4 | 71 | 15 | 1989 |
| 40014 | Jenny Lind & Fresno | 4 | Jenny | Lind | 1991 |
| R40086 | Gregory Chapel | 4 | N71 | 15 | 1991 |
| R40075 | Frog Bayou | 4 | N71 | 15 | 1993 |
| 5869 | Edgemont | 5 | 16 | 11 | 1985 |
| R50054 | White River | 5 | 58 | 0 | 1990 |
| 5831 | Spring River | 5 | 62 | 17 | 1990 |
| R50013 | Village Creek | 5 | 67 | 14 | 1986 |
| R60030 | Taylor Loop | 6 | 10 | | 1991 |
| 60319 | Carpenter Dam | 6 | 128 | 10 | 1991 |
| 60301 | Brush Creek | 6 | 222 | 2 | 1988 |
| R60043 | Hwy 270/Hwy 70 | 6 | 270 | 5 | 1991 |
| R60044 | Rickey Branch | 6 | 40 | 41 | 1990 |
| 6852 | I-430 AR Rvr Bridge | 6 | 430 | 21 | 1971 |
| 60184 | I40/I440 Interchange Phase II | 6 | 440 | 1 | 1988 |
| 60502 | New Hwy 7- Ouachita Rvr | 6 | 7 | 9 | 1991 |
| 60350 | Rock Creek | 6 | Kanis | | 1991 |
| R70070 | Moro Bay | 7 | 15 | 2&3 | 1992 |
| 100184 | Tyronza Bayou | 10 | 14 | 16 | 1987 |
| R00070 | Hwy 181 Interchange | 10 | 55 | 12 | 1991 |
| 100237 | Hwy 18 Overpass | 10 | 18 | 7 | 1991 |

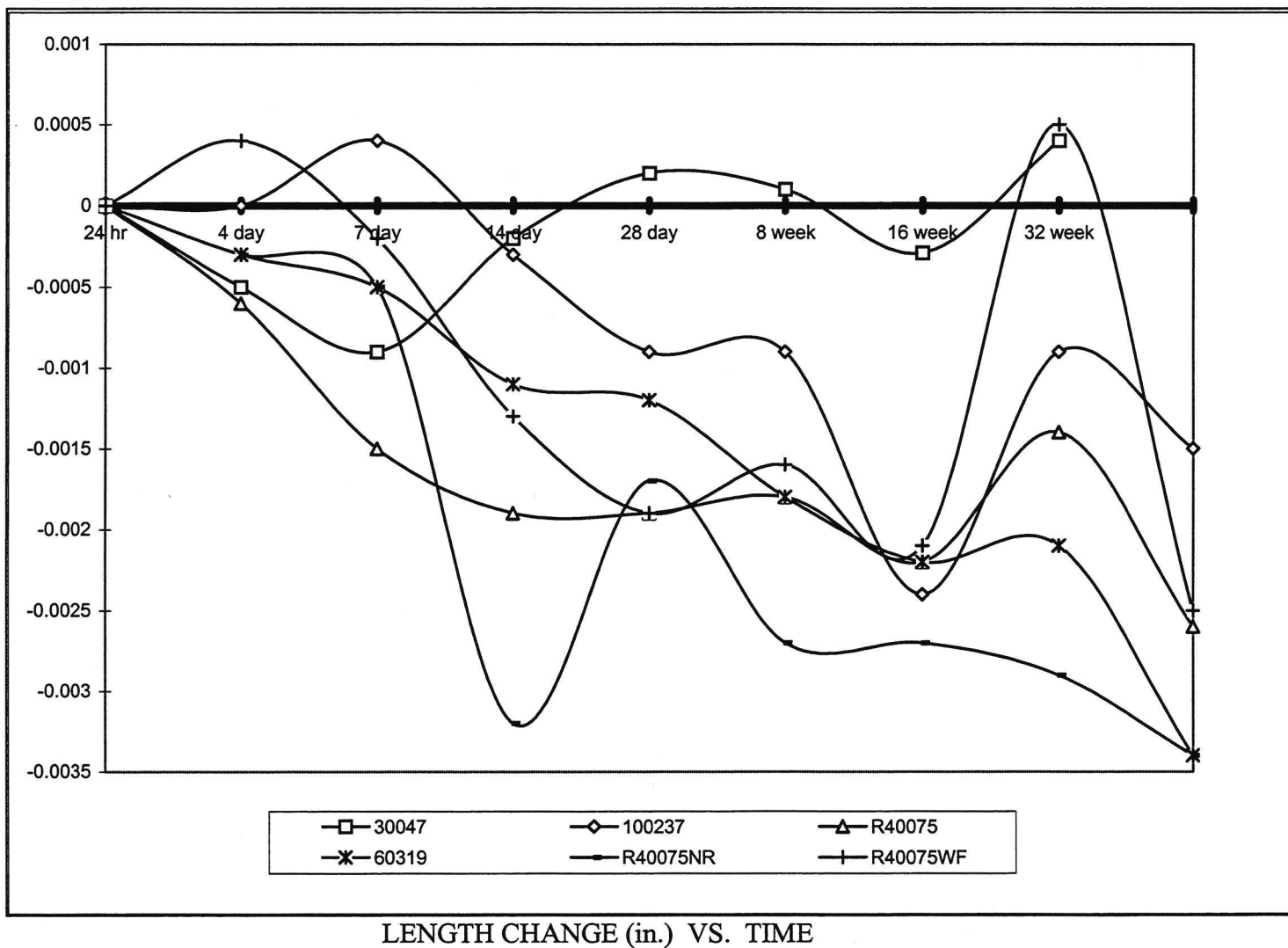
3.0 PHASE ONE

Table 3-2

| Pour Histories | | | | | | | |
|--------------------------------|-------------------------------|--------------------------------------|------------------------------|---------------------|---------------|----------------|-------------------|
| Least Severely Cracked Bridges | | | | | | | |
| Job Number | Job Name | Section Location | Date of Pour of Test Section | Daily Temp. °C (°F) | Air Content % | Slump cm (in.) | Fly Ash Content % |
| R10008 | 7 th St. Overpass | 95 ft. from South end of Bridge | 7/20/89 | 26/19 (79/67) | 3.5 | 10.16 (4) | 25 |
| 60350 | Rock Creek | 50 ft. from East end of bridge | 6/28/91 | 32.8/22.8 (91/73) | 5.2 | 3.81 (1.5) | 25 |
| 40014 | Jenny Lind & Fresno | Entire Bridge | 8/12/89 | 33.3/21 (92/70) | 6.15 | 7.62 (3) | 0 |
| 100237 | Hwy 18 Underpass | 38 ft. from West end of bridge | 3/5/91 | 17.8/3.3 (64/38) | 6.6 | 9.525 (3.75) | 25 |
| 60301 | Brush Creek | East end of bridge | 9/30/87 | 25.5/8.3 (78/47) | 3.6 | 9.525 (3.75) | 0 |
| Most Severely Cracked Bridges | | | | | | | |
| R20063 | Hwy 65 W Terminal Interchange | 342 ft. from Southwest end of bridge | 5/17/90 | 27.7/15 (82/59) | 4.0 | 7.62 (3) | 25 |
| R50013 | Village Creek Bridge | 181 ft from North end of bridge | 3/31/89 | 15.5/5.5 (60/42) | 4.0 | 9.525 (3.75) | 0 |
| R70070 | Moro Bay | 1035 ft. from north end of bridge | 5/21/92 | 32.2/19.4 (90/67) | 5.0 | 10.16 (4) | 0 |
| 110104 | Miss. Rvr. Bridge @ Helena | 971 ft from West end of bridge | 5/7/90 | 26.7/10.5 (80/51) | N/A | N/A | 0 |
| 6852 | I-430 AR Rvr Bridge | 662 ft. from North end of bridge | 4/5/74 | 17.2/2.8 (63/37) | N/A | N/A | N/A |

N/A-- information was not available

Table 3-3
Shrinkage Data



3.0 PHASE ONE



Fig. 3A
searching for cracks



Fig. 3B
cracks sprayed with water

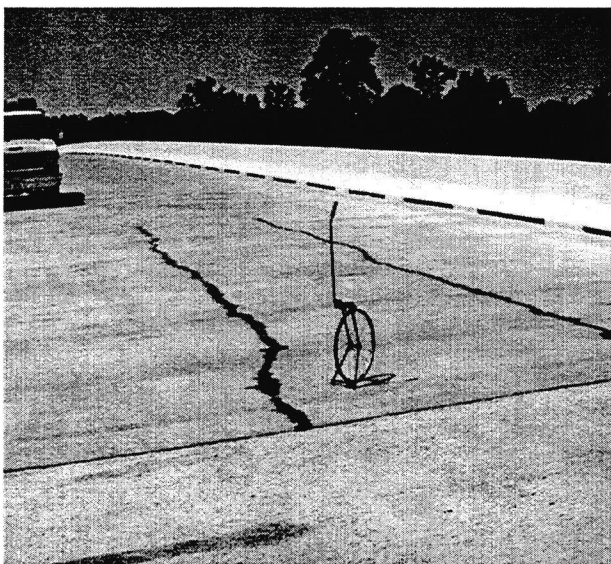


Fig. 3C
Measuring wheel



Fig 3D
Crack Comparator Card

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Fig 3E
Performing an Air Content Test



Fig. 3F
Performing a Unit Weight Test



Fig 3G
Slump Test



Fig. 3H
Preparing Shrinkage Test
Samples and Test Cylinders

3.0 PHASE ONE

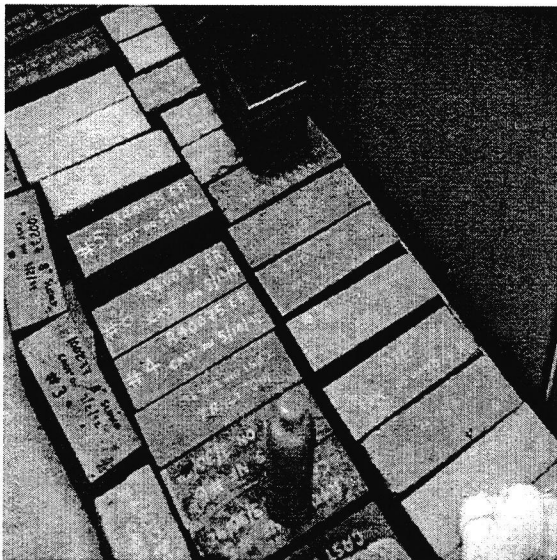


Fig 3I
Shrinkage samples

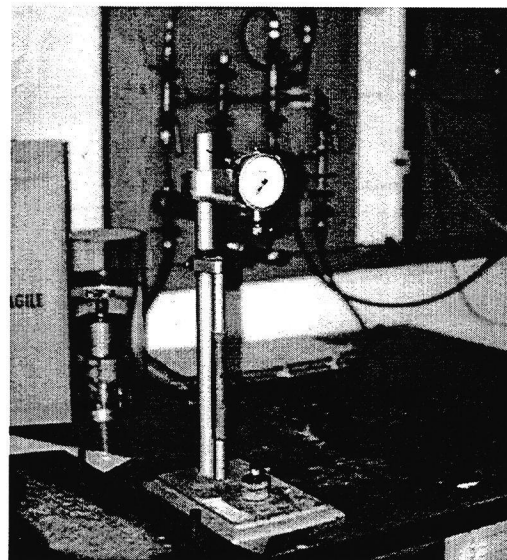


Fig. 3J
Shrinkage measuring device

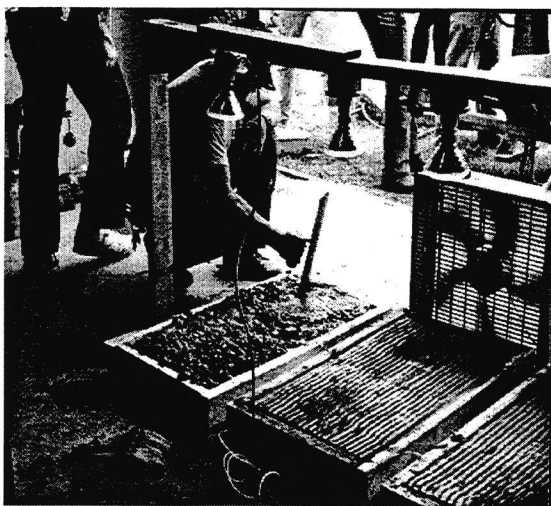


Fig. 3K
preparing Kraai test slabs



Fig. 3L
mixing concrete for "mega-mix"

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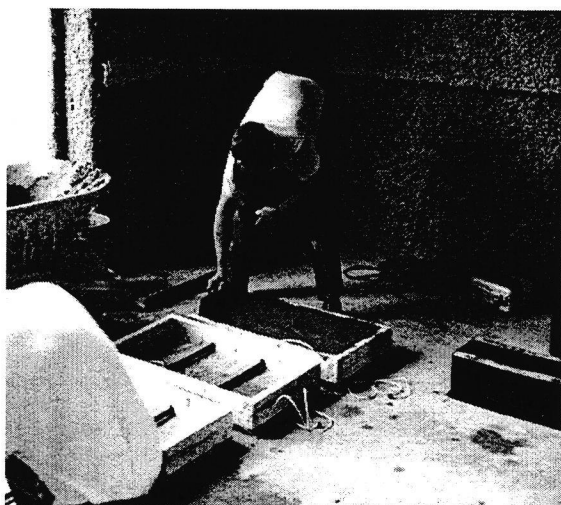


Fig. 3M
finishing a Kraii test slab

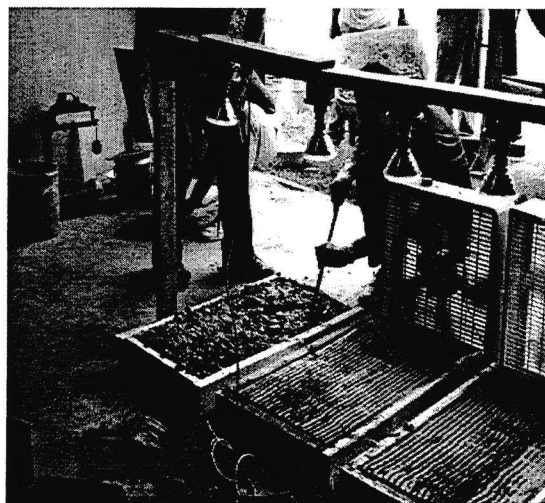


Fig 3N
tined Kraii test slabs

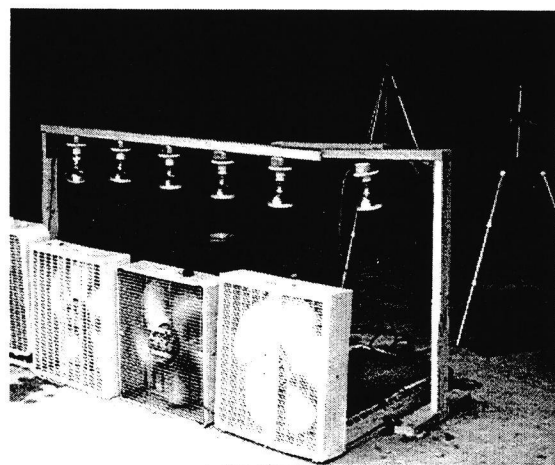


Fig. 3O
"mega-mix" test setup

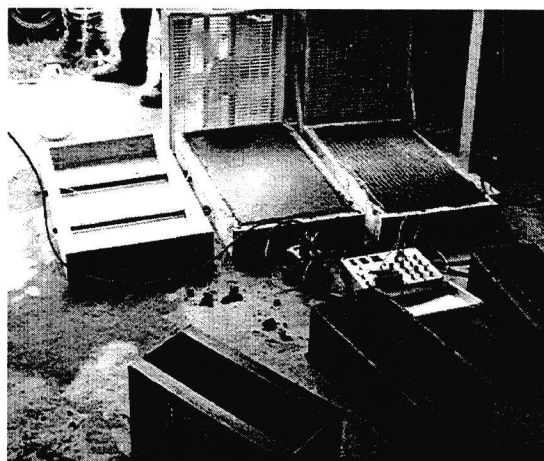


Fig. 3P
test slabs, beam molds, and temperature
measuring device

4.0 PHASE TWO

In order to obtain more detailed information on the selected bridges, it had been decided that petrographic examination of samples obtained from the bridges would be beneficial. After this decision had been made, a testing agency needed to be chosen. After receiving a bid for this series of tests, it was the decision of the subcommittee to attempt to develop a petrographic system for AHTD. Through a subcontract, a petrographic system was developed. This system cost the same as sending the samples to be tested at another agency. After the system was developed, and all associated equipment purchased, classes were held on the proper operation of this equipment. A more detailed discussion of this apparatus will occur later in this chapter.

4.1 Bridge Deck Coring

The first order of business in preparing for the petrographic examination was to obtain core samples from each bridge. It had been decided to take two core samples from each bridge. One sample was to be retrieved from the test section, "bad" concrete, and one sample retrieved from outside the test section, "good" concrete. However, due to certain restrictions, not every bridge could be cored, or only one core could be obtained. This did not appear to create a problem. Enough samples were obtained to give an accurate comparison of "good" concrete versus "bad" concrete.

Before the coring had taken place, it had been decided to retrieve 4 inch (10.16 cm) diameter core samples. The depth had a tendency to vary. The depth varied from 4 inches (10.16 cm) in to 18 inches (45.72 cm) in length. After the core had been obtained, the problem of the remaining hole had to be resolved.

It was the desire of Research to patch these holes in such a manner as to make it difficult to distinguish between the patch and the original concrete. This included the tined finish on the surface. A rapid-set mixture was prepared using a rapid-set cement, fine aggregate, and water. The mixture was then placed in the hole. When it had sufficiently set, a wire brush was run over the surface to put the tined finish on the patch. This process can be seen in Figures 4A-4D. This process was repeated on all possible bridges on the list.

4.2 Sawing the Samples

After the cores were obtained, the task of preparing them for examination needed to be completed. The first item on the list was the sawing of the samples. The samples were to be sawed in half, lengthwise. A diamond blade saw unit (Fig. 4E) was purchased for this. Unfortunately, sawing these samples was extremely difficult. Several blades had to be purchased along with other materials. When the unit was operating at its capacity, the sawing process proceeded rapidly. The only other resulting problem was when an attempt was made to cut through the reinforcing on some of the samples. This task could not be accomplished using this particular saw. The only option for this problem was to not prepare these samples and forego the petrographic analysis on these samples. Fortunately, this did not occur on very many samples.

4.2 Polishing the Samples

After the samples had been cut, the polishing process was the next task to occur. A lapidary unit (Fig. 4F) was purchased for this task. The samples were polished to remove any marks left

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Fig. 4A
Coring the bridge deck



Fig. 4B
Patching the deck



Fig. 4C
"Tining" the patch

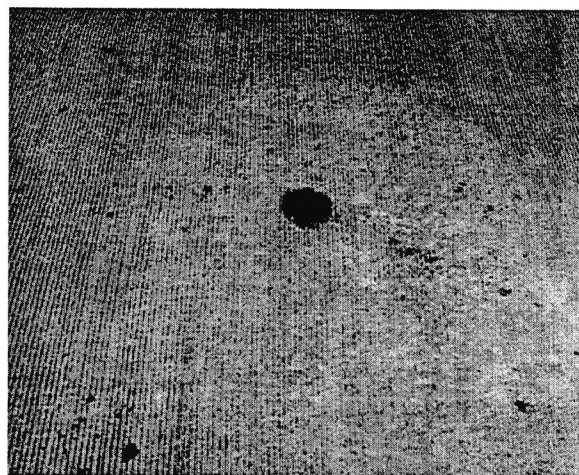


Fig. 4D
Final product

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from sawing. After these marks were removed, the samples were polished to achieve a smooth, mirror-like surface on the sample. This smooth surface would allow for ease in running the petrographic analysis. The polishing occurred in incremental phases using a finer grit for each phase. The ultimate goal of the polishing was to have a sample smooth enough to make it difficult to distinguish aggregate and paste when feeling the surface of the core. On occasion, obtaining this smooth surface proved to be quite difficult. During the polishing, paste and fine aggregate would loosen from the sample and fall away. This created either a surface which required more polishing or a surface which contained what became known as "false voids." The false voids appeared as voids during the petrographic examination. However, since these were not voids which were already existing in the sample, the voids should not have been counted towards the total air void content of the sample. This problem was rectified only through experience in running the test. The false voids had a different general appearance than the true voids. These voids were then ignored if they occurred in a position to be counted.

4.4 Petrographic Examination

The actual petrographic examination was a relatively simple yet time consuming process. Running this test on one sample required between three and four hours time. The sample had to be properly situated on the apparatus (Fig. 4G). The length, width, and maximum aggregate size of a sample were entered, and calculations were made on the number of passes and spacing between the passes. Aside from inputting this data, the only other requirements of the operator were

depressing the appropriate mouse button and the possible refocusing of the microscope. The left mouse button was designated for counting air voids, and the right mouse button was designated for counting aggregate. If neither of these mouse buttons were depressed, the material was considered to be cement paste. The middle mouse button was designated for restarting the apparatus after it had stopped due to pausing, or at the beginning or end of a pass. The system was designed for a three-button mouse, but it could be run using a two button mouse. If this occurred, the operations of the middle mouse button were accomplished by depressing any key on the keyboard. The computer totaled the number of air voids and aggregates in each designated range. The size was determined by the length of time each mouse button was depressed and the velocity of the apparatus. At the conclusion of the test, the air void content, aggregate content, and air/paste ratio was computed for each pass and overall. A sample of how the core appeared under the microscope can be seen in Fig. 4H. The petrographic analysis was run on a total of thirty-nine samples. As stated earlier, cores were not taken from every bridge or were unable to be prepared properly. Appendix B shows the results of the petrographic examination of the thirty-nine properly prepared cores.

When the test is being run, all of the information is saved after completion of the first pass. It is also possible to run a partial test, then finish it at a later time. This usually occurred when a sample was started late in the day and was unable to be finished the same day. Each portion is given a slightly different name, and the two files are later merged together to give the results for the one

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sample. It is important to remember that this can be performed on *two files only*. It is extremely important to remember to name the files differently. If the files are given the same name, the second file will overwrite the first, and data will be lost.

A short time after the petrographic examination had begun, it had been decided to attempt to purchase a camera for the apparatus. The camera would allow better working conditions. The operator would not be required to examine the samples through the microscope eyepieces. Fortunately, the microscope had been equipped in such a manner as to allow for a photographic camera to be attached. This same provision would also allow for a video camera to be attached. The only additional equipment needed would be an adapter which had already been purchased. After the camera had been

purchased and was operating, a test was run to determine the variance in results with and without the camera. When this test had been completed, it had been determined that an approximate 1% difference in results existed. Knowing this information, when personnel began training in the use of the apparatus, a variance of 1/2% was the largest that could be tolerated. With this small margin of error, not all personnel received qualification for testing samples. When a person qualified on the apparatus, his/her name was added to the list. When the list was of sufficient length, a schedule was developed to achieve completion of this phase as soon as possible. The testing process had a duration of approximately four months. Testing was sporadic in the beginning, but later became more continuous. Table 4-1 shows the comparison of the air contents of the "good" samples versus the "bad" samples.

4.0 PHASE TWO

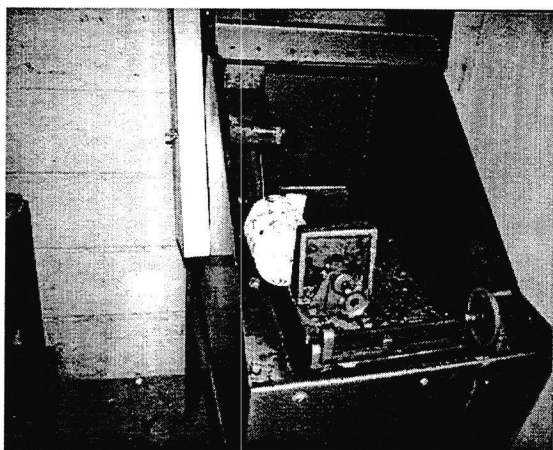


Fig 4E
Saw Unit

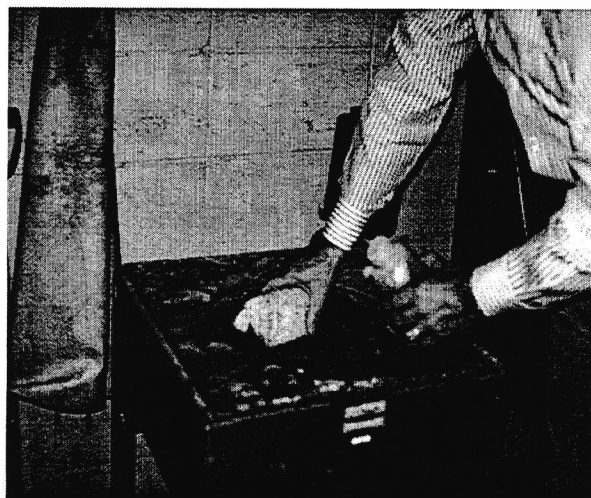


Fig. 4F
Lapidary Unit

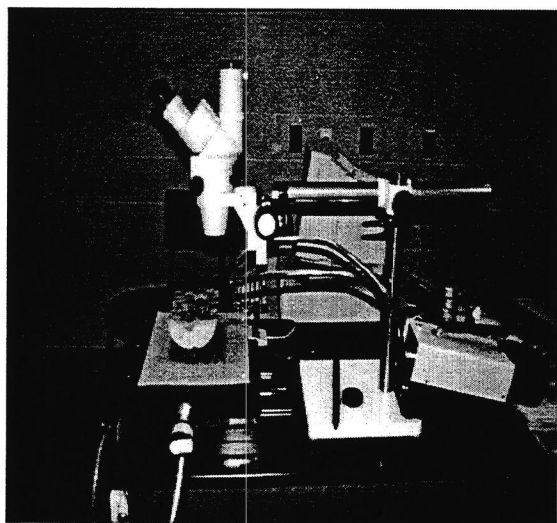


Fig. 4G
Petrographic Apparatus

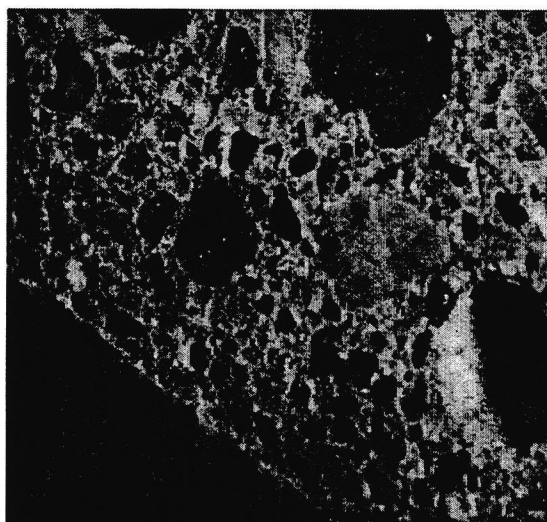


Fig. 4H
Sample under the microscope

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Table 4-1

Air Void Contents

| “Good” Samples | | “Bad” Samples | |
|-----------------------|--------------------|----------------------|--------------------|
| Job Number | Air Content | Job Number | Air Content |
| | | 60184 | 8.2 |
| | | 10008 | 6.5 |
| | | 1616 | 3.9 |
| | | 1616 | 3.9 |
| 60301 | 9.5 | 60301 | 6.4 |
| 5869 | 6.3 | 5869 | 9.99 |
| | | R40075 | 7.6 |
| R40086 | 6.1 | R40086 | 5.8 |
| R00070 | 6.7 | R00070 | 9.4 |
| R30029 | 5.4 | R30029 | 8.3 |
| R20063 | 4.0 | R20063 | 5.7 |
| 60184 | 9.0 | | |
| | | R40032 | 6.2 |
| 6852 | 4.5 | | |
| 1616 | 5.8 | | |
| R70070 | 7.8 | | |
| R60044 | 7.5 | R60044 | 7.9 |
| 60350 | 8.8 | 60350 | 9.1 |
| | | 1566 | 7.1 |
| 5831 | 6.1 | 5831 | 8.3 |
| 60030 | 9.4 | 60030 | 9.8 |
| 100184 | 7.2 | 100184 | 7.5 |
| R50013 | 7.1 | R50013 | 6.5 |
| R50054 | 12.3 | R50054 | 6.6 |
| AVG | 7.3 | AVG | 7.2 |

5.0 PHASE THREE

After discovering the extent of deterioration in some of the bridge decks, it was decided to examine repair methods to increase the life of these decks. It had been decided to examine the use of the High Molecular Weight Methacrylate sealers for this portion of the project. The two sealers selected for evaluation are the 3M CP&R and the Sika Pronto 19. Both of these products are very similar in nature, and it was felt that these products would give an accurate evaluation of their performance. This section contains the specifications for each product and the evaluation of these products.

5.1 3M CP&R

The 3M CP&R (Concrete Protector and Restorer) is a two component, rapid curing, solvent free, modified high molecular weight methacrylate. It is designed to reconsolidate and waterproof cracked portland cement concrete. The 3M CP&R penetrates and seals both the large (macro-) and micro-cracks. It also impregnates sound concrete surfaces to form a protective layer. It is designed to protect the concrete from freeze-thaw damage, deterioration due to cracking, corrosion, scaling, delamination, and spalling. The low viscosity liquid penetrates fine cracks and cures within a few hours. Minimum temperature during application should be at least 10°C (50°F), and the maximum temperature should not exceed 54°C (130°F). The surface and cracks to be sealed should be clean and free of any standing water. If the cracks to be sealed are greater than 3 mm (1/8 inch), the crack should be filled with dry 8-20 sand before applying the sealer. The product is water soluble and should not be used in wet conditions. After the surface has been prepared, the two components are mixed for one

minute. Within fifteen minutes of the mixing process, the sealer is to be placed using a gravity type flow device (crack sealing) or poured directly onto the concrete and spread using a paint roller, broom, or squeegee (topical treatment). For the topical treatment, after the sealer has been applied, dry 8-20 sand should be broadcast over the area for anti-skid purposes. The sealer should be allowed to cure until it is tack free. Curing time is usually six to eight hours at a constant 22°C (72°F). Traffic should be kept off the sealed area until this time. The product does have a strong odor and is not recommended for use in enclosed areas. However, there is a "Low Odor" version of this product that is recommended for enclosed areas. Since the sealer is water soluble, cleaning the equipment can be accomplished using soap and water if the sealer has not cured. If the sealer has cured, cleaning can be rather difficult. The product is available in both 3.8 liter (one gallon) and 18.9 liter (five gallon) packages. 3.8 liter (one gallon) will treat approximately 9.3 sq.m. (100 sq.ft.)

5.2 Sika Pronto 19

The Sika Pronto 19 is a two component, rapid curing, solvent free, high molecular weight methacrylate. It is designed to structurally repair cracked concrete and seal concrete surfaces from water and chlorides. The low viscosity liquid is easy to use in topical applications and penetrates the cracks by gravity. It is a two component product that has a high bond strength, and structurally improves the concrete surface. It is designed to prolong the life of cracked concrete. Minimum temperature during application should be 1.7°C (35°F). The surface to be sealed should be clean, sound, and free of

5.0 PHASE THREE

surface moisture. For optimum application, the substrate should be dry. However, a saturated surface dry condition is acceptable. After the surface has been prepared, the two components are mixed together for three minutes using a low speed drill (400-600 rpm). The product must then be placed within twenty minutes of the mixing process. The product is applied to horizontal surfaces by roller, squeegee, or broom. It is then spread over the area and allowed to pond over the cracks. The material should penetrate into the cracks and substrate. Any excess sealer should be removed, leaving no visible surface film. Cracks greater than 3mm (1/8 inch) should first be filled with oven dried sand before applying the Sika Pronto 19. To prevent leakage and when it is accessible, cracks should be sealed from the underside. On very porous substrates, a second application could be necessary. This application should occur before sand is broadcast over the surface. Broadcasting dry 8-20 sand over the surface should occur approximately 20 minutes after application of the sealer. The sand should be distributed at a rate of 6.8-9.1 Kg (15 to 20 pounds) per 9.29 sq.m. (100 sq.ft.). Any loose sand should be removed before the sealed area is opened to traffic. Curing time is approximately 12 to 16 hours at 22.8°C (73°F). Uncured material can be removed from the mixing equipment with water. However, cured material can only be removed mechanically.

A chart showing properties of each sealer is shown in Table 5-1

5.3 Site Selection

In selecting a site for this assessment, the bridges from research project TRC-9210, "Durability of Bridge Decks," were evaluated. After

examining the bridges on this project, the John Lipton Moro Bay Bridge in District 7 was selected for this test. At the time of this application process, this bridge had exhibited the greatest amount of cracking among the twenty nine bridges on the project. Because of this, and the fact that the cracking appeared to be evenly distributed between the lanes, it was perceived that this bridge was a good candidate for both the actual sealing process and the evaluation of the two different sealers.

The first survey of the Moro Bay Bridge deck occurred on August 17, 1992. At this time, the bridge was not opened to traffic. The linear feet of cracking was measured at 203.3 m (667 ft.). The cracking per square foot was calculated to be .625 m./ sq.m. (.191 ft/sq.ft.). A second survey of the bridge deck occurred on November 4, 1992. The bridge had been opened to traffic approximately one month at the time of the second survey. The linear feet of cracking at this time was measured to be 371.9 m. (1220 ft.). The cracking per square foot was calculated to be 1.144 m/sq.m. (.349 ft/ sq.ft.). The cracking has almost doubled from the time before the bridge was closed to traffic and the time it was opened to traffic. It had later been discovered that a majority of the cracks on this bridge deck were full depth cracks.

It had been decided that the bridge deck was in need of some type of sealing due to the fact that most of the cracks were full depth cracks. Corrosion probability was one of the concerns with this deck. It had also been decided that since the 3M product and the Sika product were very similar in nature, sealing with both of these products would allow for a valid comparison test on this bridge deck.

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5.4 Sealer Application

On August 2, 1994, one half of the Moro Bay Bridge Deck was sealed using the 3M CP&R. 75.7 liters (twenty gallons) of the product was purchased for this project. It had been decided to do an individual crack sealing instead of a topical treatment. It was felt that this amount would be sufficient for sealing half of the bridge deck. A representative of Bridge Maintenance was present to supervise the application of the sealer. A representative of the Research section was also present to observe. One problem was noted with this sealer. It had been shipped in the 18.9 liter (five gallon) containers. Since this was to be an individual crack sealing, the question of placing the entire 18.9 liter (five gallons) within the fifteen minute time period allowed before the curing process began was raised. A technical representative of the 3M Corporation, was contacted about this problem. The directions concerning this were to divide the Part A component into five 3.8 liter (one-gallon) portions and divide the Part B component into five equal portions. For the Part B component, the equal portions measured out to be approximately 68 ml (2.3 ounces). These proportions were measured and mixed for approximately one minute. In some of the batches of sealer, a dye was added during the mixing process. This would allow for the penetration ability of the sealer to be seen if a core were taken. After the sealer was mixed, it was placed in individual squeeze bottles for applications. These bottles had an approximate capacity of 236.6-295.7 ml (8-10 ounces). The sealer was then applied directly to the cracks on the bridge deck. This process continued for the entire north bound lane of the bridge. Other than the initial problem of

dividing the sealer into equal proportions, there were no problems associated with the application of this sealer.

On August 3, 1994, the Sika Pronto 19 was applied to the south bound lane of the Moro Bay Bridge Deck. Weather conditions were the same as the day before, hot and dry, so the comparison of the two sealers should be accurate. 75.7 liter (twenty gallons) of the Sika Pronto 19 was purchased for this portion of the test. These were shipped in 3.8 liter (one gallon) containers. The Part B component was mixed with the Part A component for approximately three minutes. Again, the sealer was placed in the individual squeeze bottles. Then, the sealer was applied directly to the crack. There were problems associated with the application of this sealer. One problem occurred when the first gallon was opened. It was noted that the opening the container of Part A component was extremely difficult. After the container was finally opened, it was discovered that the material had crystallized in the container and was unusable. This was discovered on two additional gallons of the sealer. This was mentioned to the District 7 Maintenance Superintendent. He decided to contact the Sika representative about the three bad gallons and obtain either a refund or a replacement. Another problem occurred during the application of the sealer. The "pot life", length of time from mixing to start of curing, is twenty minutes, according to the specifications supplied with the material. Two members of the maintenance crew applying the sealer commented that the sealer had begun to set up and begin curing in the squeeze bottle. This occurred approximately 10 to 15 minutes after the mixing process.

5.0 PHASE THREE

Aside from these problems, the application of the Sika Pronto 19 Concrete Sealer/Healer progressed well. No dye was available for placement in this sealer. Any evaluation on the penetration ability of this sealer will be quite difficult.

Photographs of the application can be seen in Figures 5A - 5J.

5.5 Evaluation

Preliminary results showed that both sealers did penetrate the cracks. It was then decided that it would be advantageous to allow the bridge to undergo one winter season, then render further evaluation. This would allow for the evaluation to include the performance of the sealer under adverse conditions, such as the use of de-icing chemicals.

On April 18, 1995, the first evaluation after the winter occurred. This evaluation indicated several particulars. The first was the appearance of the sealer. The 3M product had a brownish appearance and appeared to be performing according to specifications. The Sika product had the brownish color, but parts of the sealer had a "cloudy" appearance. The other difference was noticed upon closer evaluation of each sealer. The 3M sealer did not fill in the tines on the deck surface. The Sika product, at different locations on the deck, did fill in the tines, creating a smooth surface instead of a rough surface. This did not occur on all of the Sika product. It is possible that the Sika product, at these locations during application, was close to the end of its "pot life" and was starting to cure before it had time to penetrate the cracks.

An evaluation of this bridge occurred on October 11, 1995. During this evaluation, the Surface Air Flow Meter was run. The results of this test showed that neither sealer decreased the permeability to the level of the sound concrete. The sealers did, however, decrease the permeability significantly over the permeability of unsealed concrete. In appearance, the 3M product had not changed since the previous evaluation. The Sika product did have a couple of changes in its appearance. The "cloudy" appearance of the sealer had disappeared. The entire Sika product had the brownish color. The other change in the appearance of the Sika product was that it no longer filled in the tines on the deck surface. This could be attributed to the daily traffic and/or environmental conditions wearing the excess sealer off the bridge deck surface. At this evaluation, if it was not known ahead of time which sealer had been placed in each lane, it would have been very difficult to determine which sealer occupied each lane.

The final evaluation of this bridge deck occurred on April 9, 1998. The cracks that had been sealed were still visible. However, some sealer still remained on the surface. This was limited to very small amounts and appeared to be a portion of the sealer that had settled into the tines of the concrete during application.

It was noted during this evaluation that several new cracks had developed. Most of these new cracks appeared to be extensions of cracks that had been sealed. The additional cracking contained within the 100 ft. (30.48 m) test section was measured and found to be 1313 lin.ft. (400.2 m). The previous survey indicated 1292 lin. ft.

5.0 PHASE THREE

(393.8 m) of cracking within the test section.

5.6 Recommendations

In discussing this trial with personnel from the Maintenance Division, several observations concerning the performance of the sealers were topic for discussion. The observation that both of the sealers seemed to work quite well was noted. An additional observation resulted in a discussion that concluded the Sika Pronto 19 product would be more advantageous in cooler weather, and the 3M CP&R would be more advantageous in warmer weather. This appears to be a fair assessment due to the rapid set up of a portion of the Sika Pronto 19 during application. Each sealer seemed to perform according to the specifications supplied with each product. As stated earlier, the permeability is decreased with the cracked concrete that has been sealed. While the permeability is not that of uncracked concrete, it can be concluded that it would be that much more difficult for water and chemicals to penetrate into the deck. This will not

prohibit the intrusion of water and chemicals, but it will hinder their advancement. This will, in turn, possibly delay repair/rehabilitation work that would eventually become necessary due to bridge deck deterioration.

It can be perceived that the use of either the 3M CP&R or the Sika Pronto 19 would be advantageous on a bridge deck surface that has exhibited cracking. The temperature during application of the sealer should be the only determining factor in deciding which sealer to use. The Sika Pronto 19 should be utilized when the average temperature is cooler, and the 3M CP&R should be utilized when the average temperature is warmer. According to the technical data on each product, the minimum temperature for the application of the Sika Pronto 19 is 1.7°C (35°F), and the minimum temperature for the application of the 3M CP&R is 10°C (50°F). If either product is used according to the specifications, the durability of the bridge deck should be increased. It is important to realize that this increased durability is the increase in durability over the unsealed, cracked concrete, not the sound concrete.

5.0 PHASE THREE

Table 5-1

SPECIFICATION DATA¹

| | 3M CP&R | Sika Pronto 19 |
|-----------------|---|--|
| Property | | |
| Color | Amber | Amber |
| Pot Life | 15 minutes | 20 minutes |
| Bulk Cure Time | 1-2 hours@ 73°F | 6 hours maximum * |
| Traffic Time | 6-8 hours @ 73°F | 12 hours maximum* |
| Viscosity | 10-20 cps | 25 cps maximum |
| Shelf Life | 1 year** | Component 'A'--3 months** Component 'B'--6 months** |
| Mixing Ratio | Preportioned-- mix entire unit | Preportioned-- mix entire unit |
| Flash Point | 221°F (105°C) | 220°F (105°C) |
| Coverage | 100 sq.ft./gal (9.3 sq.m./3.8 liter) | 100 sq.ft./gal (9.3 sq.m./3.8 liter) |
| Minimum Temp. | 50°F (10°C) | 35°F (1.7°C) |
| Mixing time | 1 minute | 3 minutes |
| Bond Strength | 1500 psi | 2100 psi |
| Packaging | 1 gallon (3.8 liter) and 4.5 gallon (17 liter)units | 1 gallon (3.8 liter) and 5 gallon (18.9 liter) units |

*Times vary based on temperature, humidity, and exposure to sunlight

**In original unopened container

¹ information combined from 3M CP&R specification data, 1993, and Sika Pronto 19 specification data, 1993.

5.0 PHASE THREE

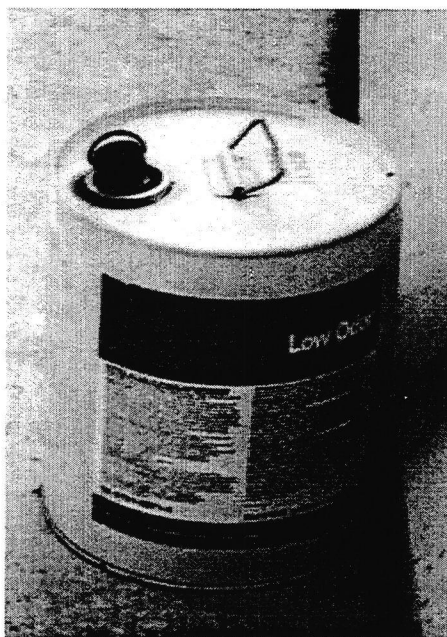


Fig 5A
3M 5741 Sealer



Fig. 5C
Mixing dye into the sealer



Fig. 5B
Sika Pronto 19 Sealer



Fig 5D
Pouring the sealer into application
containers

5.0 PHASE THREE



Fig. 5E
Applying the sealer



Fig. 5G
Sealer Application

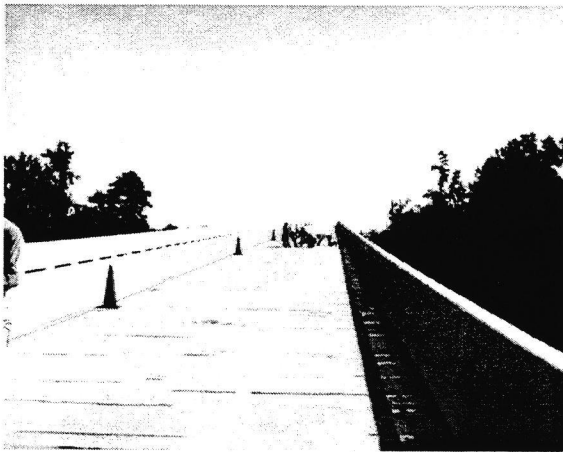


Fig. 5F
Sealer application



Fig. 5H
View of bridge after sealer application

5.0 PHASE THREE

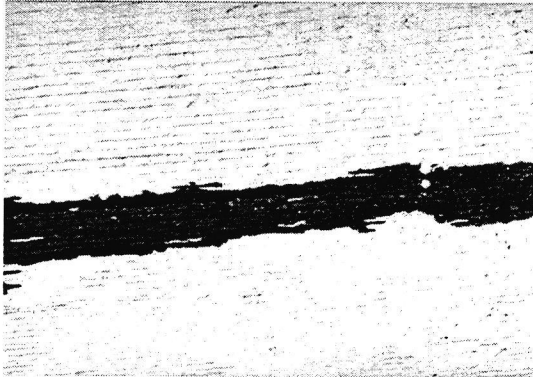


Fig. 5I
Close-up of sealer applied to a crack



Fig. 5J
Close-up of sealer

6.0 Phase Four

One possible solution to the premature bridge deck cracking problem is the use of synthetic fibers in the concrete. Two such fibers are the polypropylene fiber and the nylon fiber. The polypropylene fiber used in this portion of the project was manufactured by the Fibermesh™ company. The nylon fiber is manufactured by Allied Signal, Inc. under the name Nycon™ brand fiber. Both of these fibers can be added to the concrete without any change to the mix design. They are also designed to reduce the macro-cracking tendencies in concrete. The fiber manufacturers report that the fibers are designed to interrupt the formation of micro-cracks that eventually develop into the macro-cracks. The fibers are reported to be used as a replacement for welded wire fabric when it is being used for crack control purposes. Information on the Nycon™ and Fibermesh™ fibers can be seen in Chapter 2 of this report.

6.1 Site Selection

The suggestion was made to use both types of fibers in a bridge deck to test the performance of each set of fibers. The two sections, each containing a different type of fiber, would be compared to a "control" section which did not contain fibers. The Ingram Boulevard Overpass in West Memphis, Arkansas was chosen for this test. The Resident Engineer for this job spoke with the contractor about using the fibers in this bridge deck. The contractor granted approval to use the fibers with the condition that they be added at the jobsite. The Fibermesh™ Company and Allied Signal both agreed to donate the fibers needed for the test. Each donated enough fibers to treat 23.7 cubic meters (31 cubic yards) of concrete. This was the amount estimated to be in each section of the bridge deck.

The original plan for utilizing the fibers was to add both types of fibers on the same pour. One section would contain the Fibermesh™ fibers and the other section would contain the Nycon™ fibers. A problem with this plan developed when it was discovered that the concrete was going to be placed using a pump. It would be very difficult to determine the exact ending of each fibered section. It had then been suggested to add the fibers on two different days. This plan met with approval and the dates were set.

6.2 Fibermesh™ Application

The first phase of the fiber testing occurred on September 13, 1994. This section is located 18.9 m (62 ft) from the north end of the bridge. Fibermesh™ fibers were added at the jobsite as per the contractor's instructions. The concrete trucks had a capacity of 3.2 cubic meters (5 3/4 cu.yd.). Six bags of fibers were added to each truck. The Fas-Paks were used, so no disposal of the bags was necessary. After the addition of the fibers to the truck, the concrete was mixed for approximately three to four additional minutes to ensure proper distribution of the fibers within the concrete matrix. The weather conditions on this day were clear and sunny with a high temperature of approximately 33.3°C (92°F). Only a trace of wind was noted on this day. No changes were necessary in either the placing or finishing of the concrete. The concrete was placed using a pump located on the access road underneath the bridge deck. The president of the contracting company attended a portion of the bridge deck pour. He noted that the fibers did not cause the workers to deviate from the normal finishing processes. The Fibermesh™ fibers were quite noticeable in the uncured concrete.

6.0 Phase Four

It was also noted that the concrete was quite stiff, although it did not seem to hamper the finishing process. After the concrete was finished, it was then tined, sprayed with curing compound, and covered with wet burlap. Tining occurred approximately an hour and a half after the concrete had finished. During the tining process, it was noticed that excessive "balling" occurred. The contractor was not concerned with this. He stated that once the concrete had cured, the balls could be swept away. The curing compound was applied approximately forty-five minutes after the tining process. Covering the section with wet burlap occurred approximately three hours after the concrete had been finished.

6.3 Nycon™ Application

The second phase of this evaluation occurred on September 15, 1994. The Nycon™ fibers were added to the concrete on this date. The location of this section is 59.4 m (195 ft.) from the north end of the bridge. The weather conditions were similar to the previous pour. These fibers were added to the concrete trucks at a rate of six bags per truck load. The entire bags were added to the concrete. The concrete was then mixed an additional three to four minutes to ensure proper mixing of the fibers in the concrete. The concrete was then, again, placed using a pump, and no deviation from the placement and finishing process was necessary. This concrete was not as stiff as the concrete with the Fibermesh™ fibers. The main difference between the two types of fibers was that the Nycon™ fibers were not as visible in the uncured concrete. This is considered to be one of the advantages of using the Nycon™ fibers. This advantage is mainly due to aesthetics. The Nycon™ "No Hairy

Concrete" claim is more pleasing to the eye than a concrete slab that appears "hairy." Tining of the deck occurred approximately an hour and a half after the concrete was finished. Curing compound was applied approximately an hour after the tining process. The covering of the section with wet burlap occurred an hour after the curing compound had been applied. It was observed that the "balling" had occurred again.

6.4 Evaluation

On September 23, 1994, a follow up visit was made to the Ingram Blvd. Overpass. Both of the sections containing the fibers were uncovered. These sections were closely examined for any type of cracking. No cracking was noticed on either of these sections. The Fibermesh™ fibers were easily visible, but the Nycon™ fibers were not.

Another visit to this bridge occurred on November 3, 1994. The bridge was not under traffic at this time, and again, no cracking was noted in either fiber section.. It was anticipated that after the bridge had been under traffic for a brief amount of time, it could develop some amount of cracking.

Another evaluation occurred on April 11, 1995. At the time of this evaluation, the bridge had been under traffic for several months. Cracking was noticed in all three sections. The Nycon™ section exhibited about 6.096 m (20 lin. ft.) of transverse hairline cracking. The Fibermesh™ section exhibited about 6.096 m (20 lin. ft.) of hairline map cracking. The control section exhibited 12.192 m (about 40 lin. ft.) of longitudinal and transverse hairline cracking.

The final evaluation of this bridge occurred on April 8, 1998. It was noted that since the previous evaluation,

6.0 Phase Four

the Fibermesh™ section had developed one new, transverse, full width crack. The Nycon™ section also had additional cracking. It was also transverse but was not full width. This cracking was two separate cracks, but it appeared that they could eventually join and become a full width crack. The control section also exhibited some new cracking. This was both transverse and longitudinal. The cracking in the control section did appear to be a little more severe than in the two fibered sections.

The claims of each fiber manufacturer appear to be valid. The evaluation that first revealed the cracking, showed that the fibered sections did not crack as much as the unfibered sections.

The only change in the mixing process is an additional three to four minutes of mixing time to ensure proper

fiber distribution. The only adverse effect was the balling during the tining process. This did not appear to be an item that would be detrimental to the bridge deck. In fact, this problem was considered to be extremely minor. The environmental conditions during the concrete pours were such that cracking would most likely occur. This was apparent during the final evaluation. Each fiber appeared to reduce the amount of cracking which occurred in each section. Since the purpose of this phase was to determine the effectiveness of the fibers in concrete, these results were positive toward developing a solution of the premature bridge deck cracking. Pictures of the fibers and this application procedure can be seen in Figures 6A - 6H.

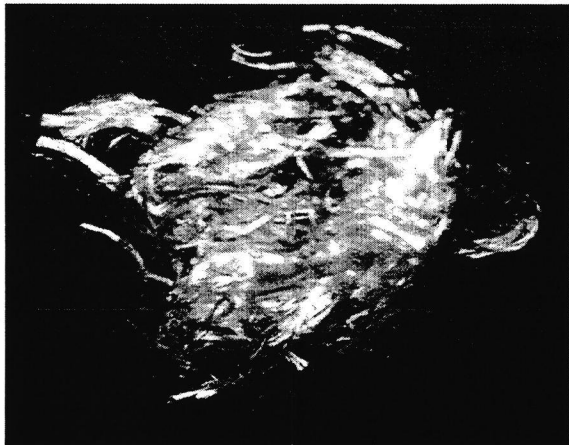


Fig. 6A
Nycon Fibers

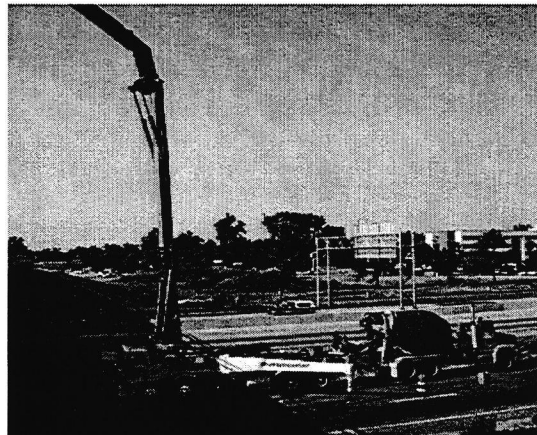


Fig 6C
Pump set-up

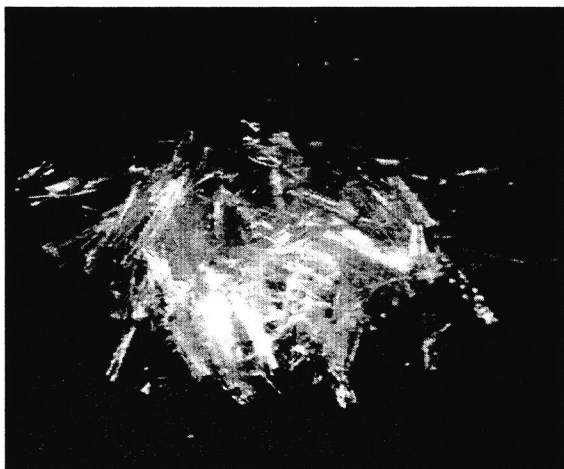


Fig. 6B
Fibermesh Fibers



Fig. 6D
Adding fibers to the mixer

6.0 CONCLUSIONS

While it is generally accepted that concrete will crack, attempts are being made to minimize this occurrence. From the literature review, the following facts on concrete cracking are known:

1. High ambient temperatures, high wind velocity, and low humidity encourage cracking.
2. Not all cracking results in structural deficiency.
3. Proper curing procedures are a necessity to minimize cracking.
4. Different types of cracking are caused by various factors.
5. The concrete mix design, including admixtures, is an important factor in determining cracking tendencies.

The current investigation of bridge decks throughout Arkansas has led to the following conclusions:

1. Fly ash does not appear to be a major contributing factor towards cracking.
2. A majority of cracking appears to be full depth.
3. With a few exceptions, high air temperature and high air content increases cracking.
4. Increase in bridge length appears to increase the amount of cracking.
5. The results of the regression analysis show a correlation between shrinkage and time. This shrinkage has the potential for crack propagation.

The preventive methods tested during this project appear to be promising. The use of the fibers appears to be beneficial in reducing the amount of cracking. While

cracking does occur with the use of each type of fiber, this is not as great as the amount of cracking in the concrete without the fibers.

The use of the high molecular weight methacrylate sealers also appears to be promising. Since these materials are penetrating sealers, it can be expected that they could penetrate the entire depth of the crack, even in the case of a full depth crack. Care should be taken not to apply the sealers when the conditions do not meet the specifications. Each of the materials tested during this project should be approved for use in bridge deck crack sealing.

While the scope of this project changed, information gathered has been quite useful. Items as simple as changing the time of day for pouring concrete or as complicated as changing curing procedures altogether can have beneficial results. The ultimate goal was to determine ways to increase the durability of the bridge decks. This goal has been accomplished. It is possible that at a future date, more processes will be examined. If this should occur, data gathering will take place. It may be possible in the future to not be concerned with the problem of premature bridge deck cracking because it will not exist.

REFERENCES

References

1. *Hot Weather Concreting*; ACI Committee 305; April 1983.
2. *Fiber Reinforced Concrete*; Portland Cement Association; 1991.
3. *Implementation of Database Management Techniques for Determining Probable Causes of Full-Depth Transverse Cracks in Reinforced Concrete Bridge Decks*, Department of Civil Engineering, Mississippi State University; November, 1994.
4. Barrow, Richard S.; Carrasquillo, Ramon L.; *The Effects of Fly Ash on the Temperature Rise in Concrete*; Center for Transportation Research, University of Texas at Austin; February 1988.
5. *ACI 224.1R-90 — Causes, Evaluation, and Repair of Cracks in Concrete Structures*, ACI Committee 224; 1990.
6. *ACI 224R-90 — Control of Cracking in Concrete Structures*; ACI Committee 224; 1990.
7. *Thermal Effects in Concrete Bridge Superstructures*; AASHTO Guide Specifications; 1989.
8. Schmitt, Tony; Darwin, David; *Cracking in Concrete Bridge Decks*; Department of Civil Engineering; University of Kansas, Kansas Department of Transportation; 1995.
9. Roy, D.M.; *Concrete Microstructure*; Strategic Highway Research Program, National Research Council, 1993.
10. "What You Need to Know About Admixtures"; *Better Roads*; October, 1991.
11. Barrow, R.S.; Carrasquillo, R.L.; *The Effects of Fly Ash on the Temperature Rise in Concrete*; Center for Transportation Research, The University of Texas at Austin; February, 1988.
12. "Use of Fly Ash in Concrete"; *NCHRP Synthesis of Highway Practice #127*, 1986.
13. Rasoulia, M.; *Fly Ash in Concrete*; Louisiana Transportation Research Center, Louisiana Department of Transportation and Development; December 1991.
14. *Design and Control of Concrete Mixtures*; Portland Cement Association; 1979.
15. Tikalsky, P.J.; Carrasquillo, R.L.; *Durability of Concrete Containing Fly Ash*; Center for Transportation Research, University of Texas at Austin; May 1986.

REFERENCES

16. Rabalais, N.; *Evaluation of Fiber Reinforced Concrete*; Louisiana Transportation Research Center; May 1991.
17. *Fibermesh* Specification Data; 1991.
18. *Nycon* Specification Data, May 1991.
19. Naaman, A.E., Alkhairi, F.M., Hammoud, H.; *High Early Strength Fiber Reinforced Concrete--SHRP-C-366*; University of Michigan; National Research Council; 1993.
20. Smith, L.L.; Kessler, R.J.; Powers, R.G.; "Corrosion of Epoxy Coated Rebar in a Marine Environment"; *Transportation Research Circular-- Number 403*; March 1993.
21. Lorini, R.A.; Hossain, M.M.; *Effects of Curing on Bridge-Deck Concrete Shrinkage Cracking*; Transportation Research and Development Bureau; New York State Department of Transportation; March 1995.
22. Laning, A; "Synthetic Fibers"; *Concrete Construction*; July 1992.
23. "Consolidation of Concrete for Pavements, Bridge Decks, and Overlays"; *NCHRP Synthesis #44*; Transportation Research Board, 1977.
24. *Durability of Concrete Bridge Decks*; a cooperative study-- Portland Cement Association; Bureau of Public Roads; ten state highway departments; 1970.
25. Wiss, Janney, Elstner Associates, Inc., *Transverse Cracking in Newly Constructed Bridge Decks, NCHRP project 12-37*, National Cooperative Highway Research Program, September 1995.

APPENDIX A

✓
A. Meaders

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

LITTLE ROCK, ARKANSAS

August 5, 1991


M E M O R A N D U M

TO: Mr. Bob Walters, Assistant Chief Engineer-Design
SUBJECT: Fly Ash in Bridge Decks

Due to concerns with widespread cracking in bridge decks, I have asked for a research project to investigate the cracking and the possible causes.

The use of fly ash in bridge decks will be a part of the investigation. In the interim, we want to discontinue allowing the use of fly ash in bridge decks on all projects let to contract after the August 14 letting.

I have asked Operations to prepare a Supplemental Specification to address this matter.


C. E. Venable
Chief Engineer

cc: Assistant Chief Engineer-Operations
Bridge Engineer
Construction and Maintenance Engineer
Materials and Research Engineer

RECEIVED
AUG 6 1991

MATERIALS AND
RESEARCH DIVISION



ARKANSAS STATE HIGHWAY COMMISSION



RON HARROD, CHAIRMAN
PRESCOTT

RODNEY E. SLATER, VICE CHAIRMAN
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L.W. "BILL" CLARK
HOT SPRINGS

BOBBY HOPPER
SPRINGDALE

HERBY BRANSCUM, JR.
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P. O. Box 2261

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TELEPHONE No. (501) 569-2000

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MAURICE SMITH
DIRECTOR OF
HIGHWAYS AND TRANSPORTATION

DAN FLOWERS
ASSISTANT DIRECTOR OF
HIGHWAYS AND TRANSPORTATION

CHARLES VENABLE
CHIEF ENGINEER

August 20, 1991

Mr. H. C. Wieland
Division Administrator
Federal Highway Administration
3128 Federal Office Building
Little Rock, Arkansas 72201

Dear Mr. Wieland:

By this letter, we are advising your office that, effective August 15, 1991, the Department is temporarily suspending the use of fly ash in bridge decks.

Several recent projects have exhibited widespread early cracking of bridge decks. This situation is of significant concern since cracking may affect the durability of the deck and the rate of corrosion of reinforcing steel.

The Department has a research subcommittee currently established to investigate bridge deck durability. Fly ash and other additives will be evaluated as to their effect on the concrete. Literature reviewed thus far gives evidence that fly ash may be a contributor to this type of cracking. It is for this reason that we have elected to discontinue use of fly ash in bridge decks until further research results are obtained. Attached is a copy of our Special Provision implementing this decision.

At the conclusion of the research effort, use of fly ash in bridge decks will be re-evaluated.

Yours truly,

A handwritten signature in cursive script, reading "Charles E. Venable".
Charles E. Venable
Chief Engineer

Attachment

08-15-91

Page 1 of 1

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB NO _____

NO FLY ASH IN BRIDGE DECK CONCRETE

Section 802 of the Standard Specifications for Highway Construction, Edition of 1991, is hereby amended as follows:

The following is added to subsection 802.05(d):

Fly ash shall not be used in bridge deck concrete.

TRC-9210

BRIDGE DECK DISTRESS SURVEY

BRIDGE NO: _____ DIRECTION: _____ DATE: _____

DISTRICT: _____ COUNTY: _____

ROUTE: _____ JOB #: _____ RATER: _____

| TYPE | | 1 | 2 | 3 | 4 | 5 | REMARKS |
|-------------------------------------|--------|---|---|---|---|---|---------|
| E X T E N T % | 0-10 | | | | | | |
| | 11-30 | | | | | | |
| | 31-50 | | | | | | |
| | 51-100 | | | | | | |

SKETCH /

LOCATION OF DISTRESS

⊖ CORE

□ BLOCK

OVERALL
CONDITION

- 1. GOOD
- 2. FAIR
- 3. POOR
- 4. CRITICAL

| SEVERITY | 1.SPALLING | 2. CRACKING | 3. DELAMINATION | 4. PUNCHING | 5. CRACKING LENGTH OF CRACKING PER 100' |
|----------|------------|------------------------|------------------|-------------------------|--|
| LOW | 1" x 2" | HAIRLINE UP TO 1/8" | 1' x 1' | 3 CRACKS UP TO 1/4" | |
| MODERATE | 2" x 6" | 1/8"-1/4" | 1/4 LANE WIDTH | 5 CRACKS UP TO 1/2" | |
| HIGH | > 6" | > 1/4" | > 1/2 LANE WIDTH | OVER 5 CRACKS > 1/2" | |

TRC-9210 BRIDGE DECK DISTRESS SURVEY

1989 Rehab

BRIDGE NO: 2899R DIRECTION: WEST DATE: 10/25/95
 DISTRICT: 1 COUNTY: Phillips
 ROUTE: 49 JOB #: 110104 RATER: TH, LA, JW

MISSISSIPPI RIVER BRIDGE APPROACH

| TYPE | | 1 | 2 | 3 | 4 | 5 | REMARKS |
|----------------------------|-------|---|---|---|---|---|---------------------------------------|
| E X T E N T | 0-10 | | | | | | West Bound lane only ARKANSAS SIDE |
| | 11-30 | | | | | | |
| | 31-50 | | | | | | |
| | % | | | | | | |

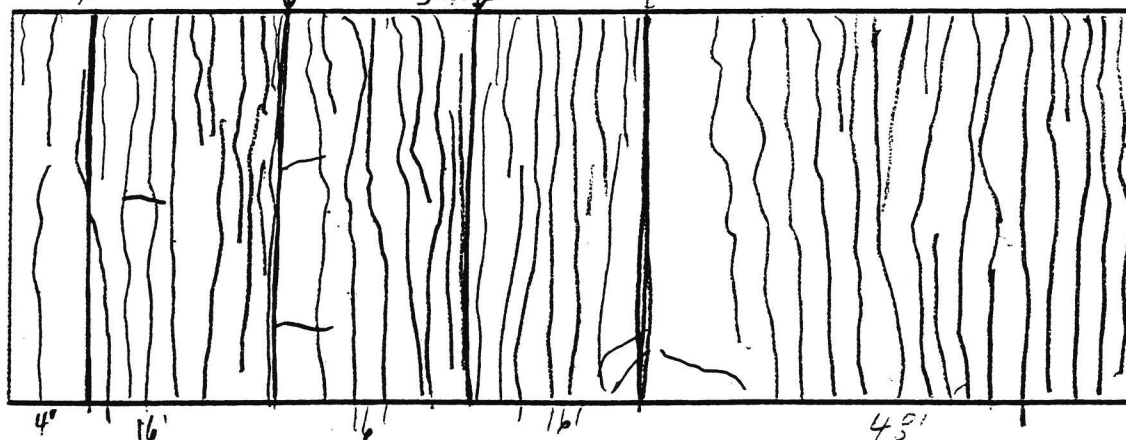
SKETCH /

⊖ CORE

LOCATION OF DISTRESS

□ BLOCK

971' FROM END



OVERALL
CONDITION

- 1 GOOD
- 2 FAIR
- 3 POOR
- 4 CRITICAL

| SEVERITY | 1. SPALLING | 2. CRACKING | 3. DELAMINATION | 4. PUNCHING | 5. CRACKING LENGTH OF CRACKING PER 100' |
|----------|-------------|---------------------|------------------|----------------------|---|
| LOW | 1" x 2" | HAIRLINE UP TO 1/8" | 1' x 1' | 3 CRACKS UP TO 1/4" | 592' |
| MODERATE | 2" x 6" | 1/8" - 1/4" | 1/4 LANE WIDTH | 5 CRACKS UP TO 1/2" | |
| HIGH | > 6" | > 1/4" | > 1/2 LANE WIDTH | OVER 5 CRACKS > 1/2" | |

APPENDIX B

Tester Jennifer
Bridge: 7th Street
Date: 7/10/95
Job #: 10008
Comment:

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 699 |
| 2-4 | 358 |
| 4-6 | 94 |
| 6-8 | 48 |
| 8-10 | 29 |
| 10-12 | 18 |
| 12-14 | 20 |
| 14-16 | 14 |
| 16-18 | 6 |
| 18-20 | 7 |
| 20-30 | 34 |
| 30-40 | 7 |
| 40-50 | 4 |
| 50+ | 16 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 2976 |
| 1-2 | 708 |
| 2-3 | 170 |
| 3-4 | 69 |
| 4-5 | 27 |
| 5-10 | 74 |
| 10-20 | 66 |
| 20-40 | 65 |
| 40-60 | 25 |
| 60-80 | 2 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|---------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| Overall | 0.0047 | 845.4165 | 6.471 | 74.6986 | 18.8304 | 34.3647 |
| Stddev | 0.0017 | 0.0017 | 1.6087 | 3.2064 | 2.9678 | 12.3085 |

Tester Jennifer
Bridge: Black Creek
Date: 6/19/95
Job #: 1616
Comment: Bad sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 151 |
| 2-4 | 169 |
| 4-6 | 88 |
| 6-8 | 44 |
| 8-10 | 20 |
| 10-12 | 7 |
| 12-14 | 5 |
| 14-16 | 4 |
| 16-18 | 5 |
| 18-20 | 4 |
| 20-30 | 12 |
| 30-40 | 4 |
| 40-50 | 3 |
| 50+ | 16 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|-------|
| 0-1 | 12227 |
| 1-2 | 469 |
| 2-3 | 107 |
| 3-4 | 37 |
| 4-5 | 35 |
| 5-10 | 97 |
| 10-20 | 85 |
| 20-40 | 42 |
| 40-60 | 25 |
| 60-80 | 4 |
| 80-100 | 3 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0069 | 548.4731 | 3.8929 | 71.7443 | 34.363 | 15.9781 |
| <i>Stddev</i> | 0.0027 | 0.0027 | 1.5229 | 6.6661 | 6.0884 | 7.1358 |

Tester Jennifer
Bridge: Black Creek
Date: 6/19/95
Job #: 1616
Comment: Bad Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 151 |
| 2-4 | 169 |
| 4-6 | 88 |
| 6-8 | 44 |
| 8-10 | 20 |
| 10-12 | 7 |
| 12-14 | 5 |
| 14-16 | 4 |
| 16-18 | 5 |
| 18-20 | 4 |
| 20-30 | 12 |
| 30-40 | 4 |
| 40-50 | 3 |
| 50+ | 16 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 1227 |
| 1-2 | 469 |
| 2-3 | 107 |
| 3-4 | 37 |
| 4-5 | 35 |
| 5-10 | 97 |
| 10-20 | 85 |
| 20-40 | 42 |
| 40-60 | 25 |
| 60-80 | 4 |
| 80-100 | 3 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| | 0.0069 | 578.4731 | 3.8928 | 71.7443 | 24.363 | 15.9781 |
| <i>Stddev</i> | 0.0027 | 0.0027 | 1.5229 | 6.6661 | 6.0884 | 7.1358 |

Tester Margaret
Bridge: Brush Creek Bridge
Date: 7/24/95
Job #: ml60301g
Comment: Good Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 122 |
| 2-4 | 631 |
| 4-6 | 234 |
| 6-8 | 133 |
| 8-10 | 85 |
| 10-12 | 50 |
| 12-14 | 31 |
| 14-16 | 30 |
| 16-18 | 21 |
| 18-20 | 6 |
| 20-30 | 43 |
| 30-40 | 13 |
| 40-50 | 4 |
| 50+ | 10 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 1353 |
| 1-2 | 387 |
| 2-3 | 99 |
| 3-4 | 39 |
| 4-5 | 33 |
| 5-10 | 85 |
| 10-20 | 65 |
| 20-40 | 58 |
| 40-60 | 17 |
| 60-80 | 6 |
| 80-100 | 1 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0067 | 601.2302 | 9.5926 | 62.1049 | 28.3025 | 33.893 |
| <i>Stddev</i> | 0.0018 | 0.0018 | 2.6747 | 7.0694 | 4.9241 | 6.7594 |

Tester Margaret
 Bridge: Brush Creek
 Date: 7/10/95
 Job #: b60301ml
 Comment: Bad Sample

Air Void Frequencies

| <i>Air Void Diameters .001 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-2 | 140 |
| 2-4 | 349 |
| 4-6 | 122 |
| 6-8 | 51 |
| 8-10 | 30 |
| 10-12 | 18 |
| 12-14 | 17 |
| 14-16 | 10 |
| 16-18 | 8 |
| 18-20 | 8 |
| 20-30 | 15 |
| 30-40 | 7 |
| 40-50 | 8 |
| 50+ | 20 |

Aggregate Frequencies

| <i>Aggregate Diameters 0.01 in.</i> | <i>Total</i> |
|-------------------------------------|--------------|
| 0-1 | 1625 |
| 1-2 | 440 |
| 2-3 | 87 |
| 3-4 | 29 |
| 4-5 | 38 |
| 5-10 | 96 |
| 10-20 | 83 |
| 20-40 | 59 |
| 40-60 | 13 |
| 60-80 | 0 |
| 80-100 | 4 |
| 100+ | 0 |

Overall Analysis

| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|----------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.0076 | 527.3393 | 6.4455 | 67.9995 | 25.5551 | 25.2218 |
| <i>Stddev</i> | 0.0041 | 0.0041 | 3.1179 | 5.053 | 3.7246 | 12.8292 |

Tester Margaret
Bridge: Edgemont Bridge
Date: 4/3/95
Job #: 5869
Comment: Good sample. Run with the camera

Air Void Frequencies

Air Void Diameters 0.001 in.

Total

| | |
|-------|-----|
| 0-2 | 152 |
| 2-4 | 278 |
| 4-6 | 190 |
| 6-8 | 113 |
| 8-10 | 36 |
| 10-12 | 30 |
| 12-14 | 19 |
| 14-16 | 12 |
| 16-18 | 9 |
| 18-20 | 4 |
| 20-30 | 9 |
| 30-40 | 3 |
| 40-50 | 4 |
| 50+ | 8 |

Aggregate Frequencies

Aggregate Diameters 0.01 in.

Total

| | |
|--------|------|
| 0-1 | 1185 |
| 1-2 | 667 |
| 2-3 | 180 |
| 3-4 | 72 |
| 4-5 | 44 |
| 5-10 | 53 |
| 10-20 | 56 |
| 20-40 | 46 |
| 40-60 | 11 |
| 60-80 | 0 |
| 80-100 | 2 |
| 100+ | 1 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0067 | 594.534 | 6.3403 | 62.6342 | 31.0255 | 20.4358 |
| <i>Stddev</i> | 0.0031 | 0.0031 | 2.5658 | 6.0782 | 5.2056 | 8.6599 |

Tester Margaret
 Bridge: Edgemont
 Date: 5/19/95
 Job #: 5869nc
 Comment: Bad sample, run without the camera

Air Void Frequencies

| <i>Air Void Diameters</i> | <i>Total</i> |
|----------------------------------|---------------------|
| 0-2 | 30 |
| 2-4 | 211 |
| 4-6 | 162 |
| 6-8 | 200 |
| 8-10 | 109 |
| 10-12 | 56 |
| 12-14 | 46 |
| 14-16 | 24 |
| 16-18 | 26 |
| 18-20 | 12 |
| 20-30 | 47 |
| 30-40 | 21 |
| 40-50 | 4 |
| 50+ | 19 |

Aggregate Frequencies

| <i>Aggregate Diameters 0.01 in.</i> | <i>Total</i> |
|--|---------------------|
| 0-1 | 924 |
| 1-2 | 654 |
| 2-3 | 188 |
| 3-4 | 66 |
| 4-5 | 20 |
| 5-10 | 75 |
| 10-20 | 53 |
| 20-40 | 36 |
| 40-60 | 20 |
| 60-80 | 8 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|-----------------------|----------|------------------|-------------|---------------|----------------|------------------|
| <i>Overall</i> | 0.0101 | 395.8325 | 9.9967 | 61.6068 | 28.3965 | 35.2042 |
| <i>Stddev</i> | 0.0026 | 0.0026 | 2.874 | 6.318 | 4.3605 | 8.5876 |

Tester Margaret
 Bridge: Frog Bayou
 Date: 6/21/95
 Job #: R40075
 Comment: South bound, bad sample (only sample) run without camera

Air Void Frequencies

| <i>Air Void Diameters .001 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-2 | 17 |
| 2-4 | 105 |
| 4-6 | 189 |
| 6-8 | 76 |
| 8-10 | 73 |
| 10-12 | 40 |
| 12-14 | 43 |
| 14-16 | 26 |
| 16-18 | 23 |
| 18-20 | 16 |
| 20-30 | 29 |
| 30-40 | 13 |
| 40-50 | 3 |
| 50+ | 14 |

Aggregate Frequencies

| <i>Aggregate Diameters .01 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-1 | 927 |
| 1-2 | 448 |
| 2-3 | 1231 |
| 3-4 | 37 |
| 4-5 | 23 |
| 5-10 | 63 |
| 10-20 | 35 |
| 20-40 | 62 |
| 40-60 | 22 |
| 60-80 | 5 |
| 80-100 | 1 |
| 100+ | 0 |

| <i>Overall Analysis</i> | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|-------------------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.0107 | 373.765 | 7.5536 | 62.8911 | 29.5553 | 25.5576 |
| <i>Stddev</i> | 0.003 | 0.003 | 3.2898 | 8.1477 | 6.0326 | 9.3378 |

Tester Richard
 Bridge: Gregory Chapel
 Date: 6/21/95
 Job #: R40086
 Comment: Good sample

Air Void Frequencies

Air Void Diameters .001 in.

| | <i>Total</i> |
|-------|--------------|
| 0-2 | 60 |
| 2-4 | 225 |
| 4-6 | 142 |
| 6-8 | 77 |
| 8-10 | 33 |
| 10-12 | 23 |
| 12-14 | 18 |
| 14-16 | 8 |
| 16-18 | 5 |
| 18-20 | 5 |
| 20-30 | 13 |
| 30-40 | 5 |
| 40-50 | 5 |
| 50+ | 18 |

Aggregate Frequencies

Aggregate Diameters .01 in.

| | <i>Total</i> |
|--------|--------------|
| 0-1 | 825 |
| 1-2 | 420 |
| 2-3 | 133 |
| 3-4 | 56 |
| 4-5 | 33 |
| 5-10 | 84 |
| 10-20 | 67 |
| 20-40 | 74 |
| 40-60 | 18 |
| 60-80 | 1 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|----------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.0095 | 420.4763 | 6.075 | 62.8771 | 31.0479 | 19.5665 |
| <i>Stddev</i> | 0.006 | 0.006 | 3.1154 | 5.2313 | 4.3043 | 12.2255 |

Tester Jennifer
Bridge: Gregory Chapel Bridge
Date: 7/26/95
Job #: R40086B
Comment: Bad Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 79 |
| 2-4 | 114 |
| 4-6 | 74 |
| 6-8 | 42 |
| 8-10 | 23 |
| 10-12 | 10 |
| 12-14 | 10 |
| 14-16 | 5 |
| 16-18 | 7 |
| 18-20 | 3 |
| 20-30 | 15 |
| 30-40 | 5 |
| 40-50 | 5 |
| 50+ | 17 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 1135 |
| 1-2 | 572 |
| 2-3 | 145 |
| 3-4 | 63 |
| 4-5 | 27 |
| 5-10 | 58 |
| 10-20 | 45 |
| 20-40 | 40 |
| 40-60 | 22 |
| 60-80 | 6 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0114 | 349.9596 | 5.8072 | 71.4904 | 22.7024 | 25.5799 |
| <i>Stddev</i> | 0.0065 | 0.0065 | 4.5756 | 7.0338 | 3.9502 | 17.9466 |

Tester Margaret
Bridge: Hwy 181 Interchange
Date: 7/26/95
Job #: R00070B
Comment: Bad Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 11 |
| 2-4 | 456 |
| 4-6 | 133 |
| 6-8 | 115 |
| 8-10 | 43 |
| 10-12 | 18 |
| 12-14 | 19 |
| 14-16 | 15 |
| 16-18 | 8 |
| 18-20 | 8 |
| 20-30 | 20 |
| 30-40 | 8 |
| 40-50 | 3 |
| 50+ | 18 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|-----|
| 0-1 | 810 |
| 1-2 | 547 |
| 2-3 | 151 |
| 3-4 | 49 |
| 4-5 | 25 |
| 5-10 | 60 |
| 10-20 | 44 |
| 20-40 | 24 |
| 40-60 | 16 |
| 60-80 | 7 |
| 80-100 | 2 |
| 100+ | 2 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0086 | 462.4953 | 9.4008 | 65.3063 | 25.2929 | 37.1678 |
| <i>Stddev</i> | 0.0037 | 0.0037 | 5.1847 | 7.1607 | 4.3009 | 19.1688 |

Tester Richard
 Bridge: Hwy 181 Interchange
 Date: 6/20/95
 Job #: R00070
 Comment: Hwy 181 Interchange Modification, good sample

Air Void Frequencies

| <i>Air Void Diameters .001 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-2 | 266 |
| 2-4 | 399 |
| 4-6 | 237 |
| 6-8 | 111 |
| 8-10 | 41 |
| 10-12 | 28 |
| 12-14 | 17 |
| 14-16 | 28 |
| 16-18 | 7 |
| 18-20 | 3 |
| 20-30 | 20 |
| 30-40 | 4 |
| 40-50 | 5 |
| 50+ | 9 |

Aggregate Frequencies

| <i>Aggregate Diameters .01 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-1 | 973 |
| 1-2 | 528 |
| 2-3 | 173 |
| 3-4 | 63 |
| 4-5 | 31 |
| 5-10 | 56 |
| 10-20 | 44 |
| 20-40 | 49 |
| 40-60 | 26 |
| 60-80 | 8 |
| 80-100 | 4 |
| 100+ | 0 |

| <i>Overall Analysis</i> | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|-------------------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.0058 | 694.4526 | 6.7175 | 65.1413 | 28.1412 | 23.8709 |
| <i>Stddev</i> | 0.0021 | 2.0612 | 8.6498 | 7.6797 | 7.6797 | 9.7484 |

Tester Margaret
Bridge: Hwy 29B- Hope
Date: 7/25/95
Job #: R30029G
Comment: Good Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 47 |
| 2-4 | 434 |
| 4-6 | 97 |
| 6-8 | 68 |
| 8-10 | 42 |
| 10-12 | 26 |
| 12-14 | 19 |
| 14-16 | 18 |
| 16-18 | 8 |
| 18-20 | 6 |
| 20-30 | 12 |
| 30-40 | 5 |
| 40-50 | 2 |
| 50+ | 6 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 1072 |
| 1-2 | 286 |
| 2-3 | 62 |
| 3-4 | 36 |
| 4-5 | 18 |
| 5-10 | 50 |
| 10-20 | 38 |
| 20-40 | 42 |
| 40-60 | 20 |
| 60-80 | 12 |
| 80-100 | 11 |
| 100+ | 4 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0062 | 641.3047 | 5.4148 | 72.6261 | 21.9591 | 24.6585 |
| <i>Stddev</i> | 0.0036 | 0.0036 | 1.9345 | 8.4052 | 7.1961 | 11.6404 |

Tester Richard
 Bridge: Hwy 65W Interchange
 Date: 6/29/95
 Job #: R20063
 Comment: Good

Air Void Frequencies

| <i>Air Void Diameters .001 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-2 | 82 |
| 2-4 | 303 |
| 4-6 | 141 |
| 6-8 | 75 |
| 8-10 | 26 |
| 10-12 | 14 |
| 12-14 | 9 |
| 14-16 | 7 |
| 16-18 | 5 |
| 18-20 | 2 |
| 20-30 | 8 |
| 30-40 | 5 |
| 40-50 | 0 |
| 50+ | 8 |

Aggregate Frequencies

| <i>Aggregate Diameters .01 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-1 | 988 |
| 1-2 | 580 |
| 2-3 | 179 |
| 3-4 | 102 |
| 4-5 | 40 |
| 5-10 | 71 |
| 10-20 | 55 |
| 20-40 | 32 |
| 40-60 | 19 |
| 60-80 | 10 |
| 80-100 | 1 |
| 100+ | 0 |

| <i>Overall Analysis</i> | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|-------------------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.006 | 669.564 | 4.012 | 61.5424 | 34.4456 | 11.6473 |
| <i>Stddev</i> | 0.0029 | 0.0029 | 1.4746 | 4.5171 | 4.5171 | 5.691 |

Tester Margaret
 Bridge: Highway 29B Hope
 Date: 7/25/95
 Job #: R30029B
 Comment: Bad Sample

Air Void Frequencies

Air Void Diameters .001 in.

| | <i>Total</i> |
|-------|--------------|
| 0-2 | 18 |
| 2-4 | 407 |
| 4-6 | 174 |
| 6-8 | 90 |
| 8-10 | 66 |
| 10-12 | 40 |
| 12-14 | 23 |
| 14-16 | 18 |
| 16-18 | 13 |
| 18-20 | 12 |
| 20-30 | 30 |
| 30-40 | 6 |
| 40-50 | 2 |
| 50+ | 9 |

Aggregate Frequencies

Aggregate Diameters .01 in.

| | <i>Total</i> |
|--------|--------------|
| 0-1 | 1014 |
| 1-2 | 294 |
| 2-3 | 73 |
| 3-4 | 36 |
| 4-5 | 26 |
| 5-10 | 35 |
| 10-20 | 39 |
| 20-40 | 36 |
| 40-60 | 19 |
| 60-80 | 11 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|----------------|--------|-----------|-------|---------|---------|-----------|
| <i>Overall</i> | 0.0073 | 546.3556 | 8.258 | 61.1794 | 30.5626 | 27.0199 |
| <i>Stddev</i> | 0.0016 | 0.0016 | 2.571 | 8.6349 | 6.681 | 6.8082 |

Tester Richard
Bridge: I40/440 Interchange
Date: 6/29/95
Job #: 60184
Comment: Good

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 106 |
| 2-4 | 354 |
| 4-6 | 287 |
| 6-8 | 125 |
| 8-10 | 95 |
| 10-12 | 57 |
| 12-14 | 31 |
| 14-16 | 21 |
| 16-18 | 11 |
| 18-20 | 15 |
| 20-30 | 29 |
| 30-40 | 8 |
| 40-50 | 5 |
| 50+ | 11 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|-----|
| 0-1 | 772 |
| 1-2 | 453 |
| 2-3 | 119 |
| 3-4 | 43 |
| 4-5 | 32 |
| 5-10 | 72 |
| 10-20 | 75 |
| 20-40 | 45 |
| 40-60 | 16 |
| 60-80 | 1 |
| 80-100 | 2 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|--------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0075 | 533.6864 | 9.0175 | 58.697 | 32.2855 | 27.9304 |
| <i>Stddev</i> | 0.0024 | 0.0024 | 2.8255 | 6.7869 | 6.0303 | 11.7267 |

Tester Richard
Bridge: I40 Interchange at Alma
Date: 7/7/95
Job #: R40032
Comment: Bad

Air Void Frequencies

Air Void Diameters .001 in.

| | <i>Total</i> |
|-------|--------------|
| 0-2 | 130 |
| 2-4 | 315 |
| 4-6 | 183 |
| 6-8 | 115 |
| 8-10 | 36 |
| 10-12 | 24 |
| 12-14 | 12 |
| 14-16 | 12 |
| 16-18 | 7 |
| 18-20 | 7 |
| 20-30 | 18 |
| 30-40 | 7 |
| 40-50 | 5 |
| 50+ | 13 |

Aggregate Frequencies

Aggregate Diameters .01 in.

| | <i>Total</i> |
|--------|--------------|
| 0-1 | 885 |
| 1-2 | 541 |
| 2-3 | 156 |
| 3-4 | 67 |
| 4-5 | 25 |
| 5-10 | 63 |
| 10-20 | 37 |
| 20-40 | 50 |
| 40-60 | 22 |
| 60-80 | 7 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|----------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.0068 | 591.1931 | 6.2142 | 61.1763 | 32.6095 | 19.0563 |
| <i>Stddev</i> | 0.0025 | 0.0025 | 2.2984 | 7.585 | 6.5161 | 6.8836 |

Tester Jennifer
 Bridge: I-430 R.BB.
 Date: 7/5/95
 Job #: 6852
 Comment:

Air Void Frequencies

| <i>Air Void Diameters .001 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-2 | 331 |
| 2-4 | 348 |
| 4-6 | 91 |
| 6-8 | 60 |
| 8-10 | 21 |
| 10-12 | 11 |
| 12-14 | 11 |
| 14-16 | 7 |
| 16-18 | 8 |
| 18-20 | 7 |
| 20-30 | 6 |
| 30-40 | 4 |
| 40-50 | 4 |
| 50+ | 9 |

Aggregate Frequencies

| <i>Aggregate Diameters .01 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-1 | 2727 |
| 1-2 | 527 |
| 2-3 | 164 |
| 3-4 | 78 |
| 4-5 | 51 |
| 5-10 | 92 |
| 10-20 | 45 |
| 20-40 | 52 |
| 40-60 | 24 |
| 60-80 | 6 |
| 80-100 | 1 |
| 100+ | 0 |

| <i>Overall Analysis</i> | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|-------------------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.0049 | 824.2258 | 4.5001 | 72.2308 | 23.2691 | 19.3393 |
| <i>Stddev</i> | 0.0019 | 0.0019 | 1.7117 | 4.5042 | 3.7893 | 7.6745 |

Tester Jennifer
Bridge: Little Missouri River
Date: 6/27/95
Job #: 1616
Comment: Good sample

Air Void Frequencies

| <i>Air Void Diameters .001 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-2 | 389 |
| 2-4 | 337 |
| 4-6 | 138 |
| 6-8 | 55 |
| 8-10 | 27 |
| 10-12 | 32 |
| 12-14 | 22 |
| 14-16 | 14 |
| 16-18 | 6 |
| 18-20 | 10 |
| 20-30 | 22 |
| 30-40 | 6 |
| 40-50 | 7 |
| 50+ | 5 |

Aggregate Frequencies

| <i>Aggregate Diameters .01 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-1 | 3455 |
| 1-2 | 550 |
| 2-3 | 122 |
| 3-4 | 70 |
| 4-5 | 48 |
| 5-10 | 151 |
| 10-20 | 90 |
| 20-40 | 49 |
| 40-60 | 6 |
| 60-80 | 0 |
| 80-100 | 0 |
| 100+ | 0 |

| <i>Overall Analysis</i> | <i>L</i> | <i>spec surf</i> | <i>A(%)</i> | <i>Agg(%)</i> | <i>past(%)</i> | <i>A/past(%)</i> |
|-------------------------|----------|------------------|-------------|---------------|----------------|------------------|
| <i>Overall</i> | 0.0049 | 810.392 | 5.8197 | 73.976 | 20.2043 | 28.8044 |
| <i>Stddev</i> | 0.002 | 0.002 | 1.6447 | 4.7559 | 4.3039 | 10.0732 |

Tester Jennifer
Bridge: Little Missouri River
Date: 6/27/95
Job #: 1616LMR
Comment: Good Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 389 |
| 2-4 | 337 |
| 4-6 | 138 |
| 6-8 | 55 |
| 8-10 | 27 |
| 10-12 | 32 |
| 12-14 | 22 |
| 14-16 | 14 |
| 16-18 | 6 |
| 18-20 | 10 |
| 20-30 | 22 |
| 30-40 | 6 |
| 40-50 | 7 |
| 50+ | 5 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 3455 |
| 1-2 | 550 |
| 2-3 | 122 |
| 3-4 | 70 |
| 4-5 | 48 |
| 5-10 | 151 |
| 10-20 | 90 |
| 20-40 | 49 |
| 40-60 | 6 |
| 60-80 | 0 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|--------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0049 | 810.392 | 5.8197 | 73.976 | 20.2043 | 28.8044 |
| <i>Stddev</i> | 0.002 | 0.002 | 1.6447 | 4.7559 | 4.3039 | 10.0732 |

Tester Jennifer
Bridge: Moro Bay Bridge
Date: 6/26/95
Job #: gr70070
Comment: Good Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 799 |
| 2-4 | 291 |
| 4-6 | 82 |
| 6-8 | 38 |
| 8-10 | 25 |
| 10-12 | 27 |
| 12-14 | 15 |
| 14-16 | 8 |
| 16-18 | 9 |
| 18-20 | 6 |
| 20-30 | 25 |
| 30-40 | 10 |
| 40-50 | 6 |
| 50+ | 22 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 3277 |
| 1-2 | 435 |
| 2-3 | 110 |
| 3-4 | 31 |
| 4-5 | 24 |
| 5-10 | 77 |
| 10-20 | 66 |
| 20-40 | 63 |
| 40-60 | 20 |
| 60-80 | 4 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0054 | 747.3925 | 7.8018 | 72.3343 | 19.8639 | 39.2764 |
| <i>Stddev</i> | 0.0034 | 0.0034 | 4.1665 | 4.7818 | 3.118 | 26.4253 |

Tester Jennifer
Bridge: Moro Bay Bridge
Date: 2/26/95
Job #: R70070
Comment: Bad sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 779 |
| 2-4 | 291 |
| 4-6 | 82 |
| 6-8 | 38 |
| 8-10 | 25 |
| 10-12 | 27 |
| 12-14 | 15 |
| 14-16 | 8 |
| 16-18 | 9 |
| 18-20 | 6 |
| 20-30 | 25 |
| 30-40 | 10 |
| 40-50 | 6 |
| 50+ | 22 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 3277 |
| 1-2 | 435 |
| 2-3 | 110 |
| 3-4 | 31 |
| 4-5 | 24 |
| 5-10 | 77 |
| 10-20 | 66 |
| 20-40 | 63 |
| 40-60 | 20 |
| 60-80 | 4 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0054 | 747.3925 | 7.8018 | 72.3343 | 19.8639 | 39.2764 |
| <i>Stddev</i> | 0.0034 | 0.0034 | 4.1665 | 4.7818 | 3.118 | 26.4253 |

Tester Richard
Bridge: Pine Bluff
Date: 6/23/95
Job #: R20063
Comment: Bad sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 231 |
| 2-4 | 407 |
| 4-6 | 171 |
| 6-8 | 65 |
| 8-10 | 27 |
| 10-12 | 20 |
| 12-14 | 9 |
| 14-16 | 9 |
| 16-18 | 9 |
| 18-20 | 5 |
| 20-30 | 15 |
| 30-40 | 5 |
| 40-50 | 4 |
| 50+ | 13 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|-----|
| 0-1 | 916 |
| 1-2 | 475 |
| 2-3 | 179 |
| 3-4 | 68 |
| 4-5 | 43 |
| 5-10 | 57 |
| 10-20 | 46 |
| 20-40 | 53 |
| 40-60 | 10 |
| 60-80 | 6 |
| 80-100 | 2 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0056 | 716.1885 | 5.6565 | 61.7931 | 32.5504 | 17.3778 |
| <i>Stddev</i> | 0.0021 | 0.0021 | 2.188 | 8.0769 | 7.1049 | 6.4351 |

Tester Margaret
Bridge: Rickey Branch Brdge
Date: 7/5/95
Job #: R60044b
Comment: Bad

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 695 |
| 2-4 | 478 |
| 4-6 | 178 |
| 6-8 | 105 |
| 8-10 | 46 |
| 10-12 | 31 |
| 12-14 | 14 |
| 14-16 | 16 |
| 16-18 | 12 |
| 18-20 | 7 |
| 20-30 | 28 |
| 30-40 | 7 |
| 40-50 | 6 |
| 50+ | 7 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 1262 |
| 1-2 | 640 |
| 2-3 | 209 |
| 3-4 | 67 |
| 4-5 | 31 |
| 5-10 | 81 |
| 10-20 | 41 |
| 20-40 | 43 |
| 40-60 | 17 |
| 60-80 | 9 |
| 80-100 | 1 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| | 0.0045 | 890.7128 | 7.9565 | 66.6904 | 25.3531 | 31.3828 |
| <i>Stddev</i> | 0.001 | 0.001 | 2.0914 | 4.5715 | 3.4932 | 8.1001 |

Tester Margaret
 Bridge: Rock Creek
 Date: 6/6/95
 Job #: b60350ml
 Comment: Bad Sample

Air Void Frequencies

| <i>Air Void Diameters</i> | <i>Total</i> |
|----------------------------------|---------------------|
| 0-2 | 945 |
| 2-4 | 583 |
| 4-6 | 184 |
| 6-8 | 76 |
| 8-10 | 39 |
| 10-12 | 45 |
| 12-14 | 20 |
| 14-16 | 16 |
| 16-18 | 11 |
| 18-20 | 9 |
| 20-30 | 27 |
| 30-40 | 10 |
| 40-50 | 2 |
| 50+ | 11 |

Aggregate Frequencies

| <i>Aggregate Diameters .01 in.</i> | <i>Total</i> |
|---|---------------------|
| 0-1 | 1332 |
| 1-2 | 607 |
| 2-3 | 191 |
| 3-4 | 76 |
| 4-5 | 35 |
| 5-10 | 59 |
| 10-20 | 56 |
| 20-40 | 32 |
| 40-60 | 17 |
| 60-80 | 4 |
| 80-100 | 2 |
| 100+ | 0 |

Overall Analysis

| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|-----------------------|----------|------------------|-------------|---------------|----------------|------------------|
| <i>Overall</i> | 0.0043 | 931.2805 | 9.0864 | 66.0549 | 24.8587 | 36.5524 |
| <i>Stddev</i> | 0.0016 | 0.0016 | 3.4386 | 7.7192 | 5.314 | 11.3677 |

Tester Margaret
Bridge: Rock Creek
Date: 6/5/95
Job #: g60350
Comment: Good Sample

Air Void Frequencies

| <i>Air Void Diameters</i> | <i>Total</i> |
|---------------------------|--------------|
| 0-2 | 408 |
| 2-4 | 571 |
| 4-6 | 282 |
| 6-8 | 114 |
| 8-10 | 65 |
| 10-12 | 50 |
| 12-14 | 18 |
| 14-16 | 28 |
| 16-18 | 9 |
| 18-20 | 7 |
| 20-30 | 19 |
| 30-40 | 8 |
| 40-50 | 6 |
| 50+ | 7 |

Aggregate Frequencies

| <i>Aggregate Diameters .01 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-1 | 1091 |
| 1-2 | 608 |
| 2-3 | 196 |
| 3-4 | 58 |
| 4-5 | 30 |
| 5-10 | 58 |
| 10-20 | 55 |
| 20-40 | 44 |
| 40-60 | 19 |
| 60-80 | 4 |
| 80-100 | 2 |
| 100+ | 0 |

| <i>Overall Analysis</i> | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|-------------------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.0052 | 763.8317 | 8.8221 | 65.8529 | 25.325 | 34.8357 |
| <i>Stddev</i> | 0.0017 | 0.0017 | 2.925 | 5.8467 | 4.7002 | 14.0494 |

Tester Margaret
Bridge: Rickey Branch Bridge
Date: 6/30/95
Job #: G6044ml
Comment: Good Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 69 |
| 2-4 | 600 |
| 4-6 | 185 |
| 6-8 | 107 |
| 8-10 | 60 |
| 10-12 | 41 |
| 12-14 | 20 |
| 14-16 | 15 |
| 16-18 | 11 |
| 18-20 | 5 |
| 20-30 | 24 |
| 30-40 | 7 |
| 40-50 | 5 |
| 50+ | 10 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|-----|
| 0-1 | 853 |
| 1-2 | 571 |
| 2-3 | 160 |
| 3-4 | 50 |
| 4-5 | 23 |
| 5-10 | 81 |
| 10-20 | 50 |
| 20-40 | 65 |
| 40-60 | 16 |
| 60-80 | 8 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0063 | 635.7316 | 7.4602 | 64.6283 | 27.9115 | 26.7282 |
| <i>Stddev</i> | ..0016 | 0.0016 | 1.8955 | 6.4042 | 5.4178 | 7.6829 |

Tester Richard
Bridge: Red River
Date: 6/30/95
Job #: 1566
Comment: Bad

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 42 |
| 2-4 | 248 |
| 4-6 | 150 |
| 6-8 | 109 |
| 8-10 | 69 |
| 10-12 | 30 |
| 12-14 | 28 |
| 14-16 | 19 |
| 16-18 | 15 |
| 18-20 | 6 |
| 20-30 | 13 |
| 30-40 | 7 |
| 40-50 | 5 |
| 50+ | 12 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|-----|
| 0-1 | 735 |
| 1-2 | 324 |
| 2-3 | 84 |
| 3-4 | 44 |
| 4-5 | 34 |
| 5-10 | 75 |
| 10-20 | 47 |
| 20-40 | 67 |
| 40-60 | 17 |
| 60-80 | 8 |
| 80-100 | 1 |
| 100+ | 0 |

Overall Analysis

| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|---------|--------|-----------|--------|---------|---------|-----------|
| Overall | 0.009 | 446.8087 | 7.0587 | 62.8879 | 30.0533 | 23.4874 |
| Stddev | 0.0046 | 0.0046 | 4.0544 | 11.5168 | 10.3866 | 19.7521 |

Tester Margaret
Bridge: Spring River
Date: 5/26/95
Job #: 5831ml
Comment: Bad Sample

Air Void Frequencies

| <i>Air Void Diameters</i> | <i>Total</i> |
|---------------------------|--------------|
| 0-2 | 674 |
| 2-4 | 774 |
| 4-6 | 294 |
| 6-8 | 115 |
| 8-10 | 50 |
| 10-12 | 27 |
| 12-14 | 18 |
| 14-16 | 11 |
| 16-18 | 8 |
| 18-20 | 3 |
| 20-30 | 11 |
| 30-40 | 4 |
| 40-50 | 4 |
| 50+ | 13 |

Aggregate Frequencies

| <i>Aggregate Diameters .01 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-1 | 1257 |
| 1-2 | 427 |
| 2-3 | 110 |
| 3-4 | 51 |
| 4-5 | 45 |
| 5-10 | 96 |
| 10-20 | 49 |
| 20-40 | 57 |
| 40-60 | 17 |
| 60-80 | 6 |
| 80-100 | 4 |
| 100+ | 2 |

| <i>Overall Analysis</i> | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|-------------------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.0041 | 969.215 | 8.3204 | 65.5515 | 26.1281 | 31.8447 |
| <i>Stddev</i> | 0.001 | 0.001 | 2.3787 | 6.8733 | 4.9368 | 6.2053 |

Tester Margaret
Bridge: Spring River
Date: 3/29/95
Job #: 5831
Comment: Good Sample

Air Void Frequencies

| <i>Air Void Diameters</i> | <i>Total</i> |
|---------------------------|--------------|
| 0-2 | 23 |
| 2-4 | 123 |
| 4-6 | 113 |
| 6-8 | 81 |
| 8-10 | 49 |
| 10-12 | 22 |
| 12-14 | 19 |
| 14-16 | 20 |
| 16-18 | 6 |
| 18-20 | 6 |
| 20-30 | 23 |
| 30-40 | 8 |
| 40-50 | 6 |
| 50+ | 15 |

Aggregate Frequencies

| <i>Aggregate Diameters .01 in.</i> | <i>Total</i> |
|------------------------------------|--------------|
| 0-1 | 813 |
| 1-2 | 418 |
| 2-3 | 97 |
| 3-4 | 60 |
| 4-5 | 24 |
| 5-10 | 82 |
| 10-20 | 36 |
| 20-40 | 35 |
| 40-60 | 24 |
| 60-80 | 9 |
| 80-100 | 5 |
| 100+ | 0 |

| <i>Overall Analysis</i> | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|-------------------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.011 | 362.5134 | 6.0658 | 63.2064 | 30.7278 | 19.7404 |
| <i>Stddev</i> | 0.0038 | 0.0038 | 2.7351 | 8.2767 | 8.6113 | 11.2173 |

Tester Margaret
Bridge: Taylor Loop Bridge
Date: 7/11/95
Job #: b60030ml
Comment: Bad Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 596 |
| 2-4 | 651 |
| 4-6 | 169 |
| 6-8 | 72 |
| 8-10 | 59 |
| 10-12 | 52 |
| 12-14 | 25 |
| 14-16 | 17 |
| 16-18 | 18 |
| 18-20 | 12 |
| 20-30 | 35 |
| 30-40 | 16 |
| 40-50 | 3 |
| 50+ | 21 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 1517 |
| 1-2 | 580 |
| 2-3 | 175 |
| 3-4 | 49 |
| 4-5 | 36 |
| 5-10 | 68 |
| 10-20 | 50 |
| 20-40 | 50 |
| 40-60 | 11 |
| 60-80 | 9 |
| 80-100 | 6 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0055 | 723.3453 | 9.7527 | 67.0175 | 23.2299 | 41.9833 |
| <i>Stddev</i> | 0.0015 | 0.0015 | 2.9017 | 4.8224 | 2.6173 | 10.558 |

Tester Margaret
 Bridge: Taylor Loop Bridge
 Date: 7/11/95
 Job #: q60030ml
 Comment: Good Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 373 |
| 2-4 | 788 |
| 4-6 | 151 |
| 6-8 | 76 |
| 8-10 | 44 |
| 10-12 | 34 |
| 12-14 | 18 |
| 14-16 | 14 |
| 16-18 | 13 |
| 18-20 | 16 |
| 20-30 | 29 |
| 30-40 | 18 |
| 40-50 | 4 |
| 50+ | 14 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 1246 |
| 1-2 | 669 |
| 2-3 | 168 |
| 3-4 | 67 |
| 4-5 | 27 |
| 5-10 | 54 |
| 10-20 | 40 |
| 20-40 | 46 |
| 40-60 | 20 |
| 60-80 | 8 |
| 80-100 | 6 |
| 100+ | 0 |

Overall Analysis

| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|-----------------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.0058 | 684.2794 | 9.4001 | 65.9457 | 24.6541 | 38.128 |
| <i>Stddev</i> | 0.0023 | 0.0023 | 3.8623 | 5.858 | 3.3516 | 14.6208 |

Tester Margaret
Bridge: Tyronza Bayou
Date: 6/14/95
Job #: 100184
Comment: Bad sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 463 |
| 2-4 | 451 |
| 4-6 | 140 |
| 6-8 | 76 |
| 8-10 | 42 |
| 10-12 | 31 |
| 12-14 | 20 |
| 14-16 | 22 |
| 16-18 | 9 |
| 18-20 | 8 |
| 20-30 | 29 |
| 30-40 | 7 |
| 40-50 | 4 |
| 50+ | 10 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|-----|
| 0-1 | 845 |
| 1-2 | 418 |
| 2-3 | 98 |
| 3-4 | 41 |
| 4-5 | 21 |
| 5-10 | 61 |
| 10-20 | 60 |
| 20-40 | 64 |
| 40-60 | 24 |
| 60-80 | 10 |
| 80-100 | 5 |
| 100+ | 0 |

Overall Analysis

Overall
Stddev

| | | | | | |
|--------|-----------|--------|---------|---------|-----------|
| L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| 0.0054 | 737.3728 | 7.5314 | 73.1557 | 19.3129 | 38.9967 |
| 0.0019 | 0.0019 | 3.361 | 7.3308 | 4.6315 | 13.7678 |

Tester Margaret
Bridge: Tyronza Bayou
Date: 6/13/95
Job #: 100184
Comment: Good sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 621 |
| 2-4 | 428 |
| 4-6 | 156 |
| 6-8 | 77 |
| 8-10 | 61 |
| 10-12 | 39 |
| 12-14 | 17 |
| 14-16 | 21 |
| 16-18 | 11 |
| 18-20 | 8 |
| 20-30 | 25 |
| 30-40 | 8 |
| 40-50 | 5 |
| 50+ | 7 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 1018 |
| 1-2 | 524 |
| 2-3 | 140 |
| 3-4 | 47 |
| 4-5 | 23 |
| 5-10 | 44 |
| 10-20 | 71 |
| 20-40 | 50 |
| 40-60 | 27 |
| 60-80 | 4 |
| 80-100 | 1 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0046 | 876.9668 | 7.1627 | 66.6731 | 26.1642 | 27.3761 |
| <i>Stddev</i> | 0.0007 | 0.0007 | 1.9059 | 6.6044 | 5.0868 | 4.9963 |

Tester Margaret
Bridge: Village Creek Bridge
Date: 7/6/95
Job #: R50013G
Comment: Good Sample

Air Void Frequencies

Air Void Diameters .001 in.

| | <i>Total</i> |
|-------|--------------|
| 0-2 | 289 |
| 2-4 | 445 |
| 4-6 | 179 |
| 6-8 | 89 |
| 8-10 | 64 |
| 10-12 | 32 |
| 12-14 | 13 |
| 14-16 | 12 |
| 16-18 | 3 |
| 18-20 | 9 |
| 20-30 | 23 |
| 30-40 | 6 |
| 40-50 | 4 |
| 50+ | 11 |

Aggregate Frequencies

Aggregate Diameters .01 in.

| | <i>Total</i> |
|--------|--------------|
| 0-1 | 1004 |
| 1-2 | 527 |
| 2-3 | 134 |
| 3-4 | 40 |
| 4-5 | 34 |
| 5-10 | 62 |
| 10-20 | 40 |
| 20-40 | 45 |
| 40-60 | 24 |
| 60-80 | 10 |
| 80-100 | 3 |
| 100+ | 2 |

Overall Analysis

| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
|----------------|--------|-----------|--------|---------|---------|-----------|
| <i>Overall</i> | 0.0057 | 703.973 | 7.089 | 69.3987 | 23.5124 | 30.15 |
| <i>Stddev</i> | 0.0016 | 0.0016 | 2.8853 | 5.4482 | 3.9905 | 11.8578 |

Tester Margaret
Bridge: Village Creek Bridge
Date: 7/25/95
Job #: R50013B
Comment: Bad Sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 8 |
| 2-4 | 207 |
| 4-6 | 203 |
| 6-8 | 90 |
| 8-10 | 69 |
| 10-12 | 30 |
| 12-14 | 26 |
| 14-16 | 9 |
| 16-18 | 7 |
| 18-20 | 11 |
| 20-30 | 15 |
| 30-40 | 7 |
| 40-50 | 11 |
| 50+ | 9 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|-----|
| 0-1 | 716 |
| 1-2 | 388 |
| 2-3 | 110 |
| 3-4 | 44 |
| 4-5 | 24 |
| 5-10 | 49 |
| 10-20 | 46 |
| 20-40 | 42 |
| 40-60 | 17 |
| 60-80 | 14 |
| 80-100 | 5 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|--------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0084 | 476.5113 | 6.4756 | 69.974 | 23.5503 | 27.497 |
| <i>Stddev</i> | 0.0036 | 0.0036 | 2.9855 | 7.8091 | 5.8444 | 12.5638 |

Tester Richard
Bridge: White River Bridge
Date: 6/22/95
Job #: R50054
Comment: Bad sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 119 |
| 2-4 | 272 |
| 4-6 | 201 |
| 6-8 | 101 |
| 8-10 | 67 |
| 10-12 | 44 |
| 12-14 | 31 |
| 14-16 | 15 |
| 16-18 | 16 |
| 18-20 | 4 |
| 20-30 | 27 |
| 30-40 | 11 |
| 40-50 | 5 |
| 50+ | 7 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|-----|
| 0-1 | 809 |
| 1-2 | 505 |
| 2-3 | 106 |
| 3-4 | 40 |
| 4-5 | 27 |
| 5-10 | 66 |
| 10-20 | 75 |
| 20-40 | 72 |
| 40-60 | 13 |
| 60-80 | 1 |
| 80-100 | 0 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|--------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.007 | 571.6101 | 6.603 | 61.3788 | 32.0182 | 20.6227 |
| <i>Stddev</i> | 0.0019 | 0.0019 | 2.6127 | 8.3622 | 7.7049 | 8.4242 |

Tester Jennifer
Bridge: White River Bridge
Date: 6/20/95
Job #: R50054
Comment: Good sample

Air Void Frequencies

Air Void Diameters .001 in.

Total

| | |
|-------|-----|
| 0-2 | 373 |
| 2-4 | 377 |
| 4-6 | 220 |
| 6-8 | 176 |
| 8-10 | 105 |
| 10-12 | 85 |
| 12-14 | 89 |
| 14-16 | 63 |
| 16-18 | 32 |
| 18-20 | 13 |
| 20-30 | 61 |
| 30-40 | 25 |
| 40-50 | 10 |
| 50+ | 7 |

Aggregate Frequencies

Aggregate Diameters .01 in.

Total

| | |
|--------|------|
| 0-1 | 1523 |
| 1-2 | 536 |
| 2-3 | 148 |
| 3-4 | 36 |
| 4-5 | 31 |
| 5-10 | 61 |
| 10-20 | 45 |
| 20-40 | 59 |
| 40-60 | 20 |
| 60-80 | 10 |
| 80-100 | 4 |
| 100+ | 0 |

Overall Analysis

| | | | | | | |
|----------------|--------|-----------|---------|---------|---------|-----------|
| | L | spec surf | A(%) | Agg(%) | past(%) | A/past(%) |
| <i>Overall</i> | 0.0073 | 545.9277 | 12.2628 | 69.8739 | 17.8632 | 68.6485 |
| <i>Stddev</i> | 0.0011 | 0.0011 | 3.6981 | 5.3443 | 3.2417 | 20.6661 |

