IDENTIFYING COMBINATIONS OF MATERIALS THAT LEAD TO PREMATURE DETERIORATION IN CONCRETE PAVEMENTS

Rachel J. Detwiler, Mohamad A. Nagi, and Sankar Bhattacharja
Construction Technology Laboratories, Inc.
5420 Old Orchard Rd.
Skokie, Illinois 60077
USA

ABSTRACT

Premature deterioration of concrete pavements can result from incompatible combinations of materials that were individually acceptable. The likelihood of problems due to incompatibility increases with the complexity of the mix design: the more ingredients, the greater the chance of an undesirable interaction. On some jobs it may be necessary to substitute one ingredient for another during the course of the job, often on short notice. Standard specifications for cementitious materials are broad enough to allow for significant differences between two sources. Thus materials should not be substituted for one another without appropriate testing. This paper provides a review of the literature on incompatibility of concrete materials, supplemented by a survey of selected highway officials and paving contractors. Particular emphasis is placed on early stiffening of fresh concrete, early cracking of hardened concrete, and inadequate air-void systems.

INTRODUCTION

Paving concrete is normally an "abuser friendly" material, performing satisfactorily even when the conditions under which it is batched, mixed, transported, placed, consolidated, finished, and cured (or not cured) are less than ideal. However, the introduction of new chemical admixtures and supplementary cementing materials has increased the possibility of incompatibility and made concrete more sensitive to marginal concreting practices. Such incompatibility increases the likelihood of premature deterioration.

In practice, the causes of premature distress are not always identified. The *Pooled Fund Study* conducted by Jennings et al. (1997) consisted of a historical and statistical survey of premature deterioration in a total of 54 concrete pavements in six states. The information collected was entered into a data base and statistically analyzed using two different models. Because the researchers had to rely on historical data rather than direct observation, they could not establish causal relationships between the factors cited and the distress observed. Rather, they determined statistically significant associations between them.

Often the cause of distress in any given pavement is not known, or is the subject of controversy. One reason for this is that a systematic forensic investigation of any failure is a complex process. Such an investigation would normally include a historical review of the site and the environment, condition surveys, and tests of the engineering properties of the concrete, supplemented as necessary by the laboratory methods of microscopy and chemistry. This type of investigation requires adequate funding, equipment, facilities, and personnel with appropriate training. Few departments of transportation have all of these resources at their disposal. Second, even a good forensic investigation may not be definitive. An examination of distressed concrete may reveal several potential causes without showing which ones actually contributed to the distress. The ideal method of sorting out causal relationships is to recreate the process of progressive failure in the laboratory, and then compare the laboratory specimens with specimens taken from the field. However, this process is so laborious and time-consuming as to be prohibitive in almost all cases. An additional difficulty is that specimens from a distressed pavement are not always available after the fact. Thus it is not surprising that in many cases the cause is never identified, or that several independent experts examining the same pavement disagree as to the cause of the failure.

In effect, highway officials and materials suppliers often do essentially the same thing as was formalized by the *Pooled Fund Study*. They observe that some problem seems to occur more frequently with the use of particular material, or primarily under certain job conditions. But in most cases they are unable to establish definitive causal relationships. Under pressure to solve a problem of early distress as quickly as possible, they must use engineering judgment to make changes in their selection of materials, mix proportions, and procedures to minimize or eliminate the problem. They may never know whether a particular change actually solved the problem, or whether some other factor beyond their control was responsible.

EARLY STIFFENING

Stiffening may be caused by excessive calcium sulfate in the form of hemihydrate (plaster) in the cement in the plastic state (often called "false set") or the uncontrolled early hydration reactions of the C_3A (early stiffening). False set may be overcome by continued mixing of the concrete, while early stiffening is not reversible. Early stiffening in the cement paste leads to loss of workability, with consequent difficulties in concrete placement and consolidation. When concrete is hard to place, additional water may be added, reducing both strength and durability and increasing the potential for shrinkage and cracking. The addition of admixtures (such as high range water reducers) improves workability without these negative effects, but adds considerably to cost of the concrete. High range water reducers may retard setting, depending on the amount used. The tendency to early stiffening is more often attributed to interactions among the various cementitious materials and chemical admixtures than to the individual cementitious materials.

Stiffening due to portland cement alone was summarized by Hansen (1960, 1961). Early stiffening depends on several factors, including C_3A content and reactivity; alkali content; and the form, content, and distribution of sulfates in the cement. Sulfates (usually in the form of gypsum) are added to cement to control the hydration of the C_3A. By itself, C_3A hydrates rapidly, causing stiffening. C_3A hydrating in the presence of sulfate ions forms ettringite on its surface. The ettringite acts as a barrier, limiting further reactivity. If supplementary cementing materials, particularly Class C fly ash, contain aluminate phases such as C_3A or Klein's compound, and the sulfate is not well distributed in the cement paste, the concrete may experience early stiffening.

A delicate ionic balance is necessary in plastic concrete to prevent early stiffening. Chemical admixtures, particularly water reducers, may disturb this balance. Cements in which setting is marginally controlled may exhibit abnormal stiffening when used with these admixtures. Meyer and Perenchio (1980) addressed the problem of cement/admixture incompatibility.

Early stiffening can be deleterious to pavement performance. Loss of workability necessitates increased consolidation effort. If the concrete cannot be thoroughly consolidated, loss of strength and durability may result (Whiting et al., 1987). Low strengths can lead to early cracking and pavement failures. If extraordinary efforts are used to consolidate the concrete, the entrained air void system may be altered, leading to decreased freeze-thaw durability (Stark, 1986; Tymkowicz and Steffes, 1999).

Frequently there is more than one factor contributing to the unsatisfactory performance of concrete. In some cases, the problems encountered could have been prevented or mitigated by the use of information that was available at the time of specification. While incompatibility between the cement and the chemical admixture caused some early stiffening problems which contributed to poor consolidation on a section of Interstate 75 in Kentucky in 1998, the primary cause was inappropriate proportioning of the concrete mixture. The mix proportions as specified using the aggregate selected produced a harsh concrete that would have been extremely difficult to consolidate in any case.

EARLY-AGE CRACKING

In addition to early stiffening effects, cracking in concrete can be caused by a number of other factors. Shrinkage can occur in the fresh (plastic) or hardened concrete. The major cause of plastic shrinkage cracking is thought to be the tensile stress developed as water evaporates from the surface of the concrete, leaving the capillaries partially filled and creating surface tension. As the concrete has not yet hardened its tensile strength is low, and cracks can develop, especially at the surface where drying is greater. The risk may be higher in concretes that exhibit early stiffening, as the mix does not remain fluid long enough to allow a layer of bleed water to remain on the surface. While bleeding is generally thought of as detrimental to concrete because of its contribution to settlement cracking and weak surfaces, it is of some benefit with relation to the potential for plastic cracking: evaporation from the concrete surface cannot occur if it is covered with bleed water. In the absence of bleeding, loss of water from the concrete surface can occur rapidly if precautions are not taken. These precautions include: (1) strict adherence to specifications regarding evaporation rates and cessation of concrete placement if relative humidities are low and temperatures and wind speeds are high, (2) use of fog sprays, (3) use of evaporation retarding agents during and immediately after finishing, and (4) initiation of wet curing as soon as possible after finishing. Time may not permit utilization of such precautions in many large pavement jobs, so careful testing and selection of materials which are least susceptible to plastic shrinkage cracking is important.

Cracking can also occur somewhat later in the life of the pavement due to autogenous shrinkage, drying shrinkage, thermal effects and external loads. Cracking occurs when and where the maximum principal tensile stress exceeds the tensile strength of the concrete. The stress can result from any combination of imposed loads and restraint of volume changes due to thermal effects, drying, or autogenous shrinkage.

In the last few years, several pavement projects have experienced early-age cracking. In Pennsylvania, sections of Interstate 80 in PennDOT Districts #1 and #3 experienced cracking due to differential shrinkage of the slab (slab curling), instability of the subbase leading to load-induced settlement, and thermal expansion and contraction. Two recent pavement projects in Minnesota experienced transverse cracking at two- to three-foot intervals because the concrete did not gain sufficient strength to allow joints to be cut until after cracking had begun. On Interstate 35 in 1997, engineers believe the cause of the retardation was an overdose of the water reducer used to provide adequate workability during warm weather. On US Route 71 in 1998, a ternary blend containing 50% cement, 35% slag, and 15% Class C fly ash cracked within the first two days. The problem was solved by altering the mix proportions. With a blend of 60% cement, 30% slag, and 10% fly ash, the strength gain was adequate. Two pavements in Southern California, Interstate 15 in San Bernardino County in 2000 and State Route 73 in Orange County in 1999, experienced cracking due to shrinkage. Interstate 15 also exhibited cracking above the tie bars. This cracking was attributed to poor consolidation of a stiff mix and shallow placement of the tie bars. The shrinkage cracking in State Route 73 was attributed to late or improper curing, excess water in the mix, and extreme environmental conditions. In another case, a pavement using a portland cement and water reducer experienced cracking due to retardation, which prevented the concrete from gaining sufficient strength to allow the joints to be cut in time to prevent cracking. Changing to another admixture resolved the problem.

In several of these cases, incompatibility among two or more of the concrete ingredients was a contributing cause of the cracking. However, other factors such as weather, design factors, and construction practices also contributed to the cracking. It may not be appropriate to examine these factors in isolation, since a particular combination of ingredients may work well at some temperatures and not others. Also, a particular combination of materials may be more sensitive to certain construction variables such as mixing time or curing practices.

INADEQUATE AIR-VOID SYSTEM

Excessive vibration to consolidate the concrete can result in loss of air. This phenomenon can be observed in the form of vibrator trails along the length of the pavement. The visible vibrator trail results from the displacement of coarse aggregate, leaving relatively high water/cement ratio mortar. Longitudinal drying shrinkage cracks may form in the vibrator trails. Stutzman (1999) found that the air-void systems of deteriorated pavements in Iowa were worst in the visible vibrator trails. Tymkowicz and Steffes (1999) reported that cores taken from longitudinal cracks in Interstate 80 in Iowa contained 3% air in the top half and 6% in the bottom half. They suggested excessive vibration as a possible cause of the reduced air content in the upper portion of the pavement. Their investigation indicated that the radius of effective consolidation of a vibrator was smaller than is commonly believed. Cores taken between vibrator trails showed significant entrapped air within 100 mm of the vibrator location. Simon et al. (1992) found similar results.

Stark (1986) found that internal vibration resulted in air loss and increased air-void spacing factors, with higher frequencies having the most pronounced effect. Concretes with low water/cement ratios lost less air than those with higher water/cement ratios. The resistance to frost damage was related to both spacing factor (thus vibration) and water/cement ratio. Simon et al. (1992) also found that near the vibrator, the total volume of air was reduced by removal of the entrapped air and a portion of the entrained air. They did not find that the loss of air was necessarily detrimental to frost resistance.

Cross et al. (2000) investigated a series of projects in which the concrete compressive strengths were much lower than specified and did not increase significantly with time. They found that an interaction between synthetic air-entraining admixtures and low-alkali cements resulted in clustering of air voids at the interface between the aggregate particles and the cement paste. This interaction was exacerbated by high summer temperatures.

MATERIALS ASSOCIATED WITH PREMATURE DISTRESS

Class C fly ash

Jennings et al. (1997) cited the use of Class C fly ash as one of the factors most strongly associated with pavement deterioration. Numerous respondents to the survey also implicated Class C fly ash as problematic. One state highway department no longer allows the use of fly ash in highway pavements because of negative experiences associated with it. (Class C fly ash is the only fly ash available in that state.) Gress (1997) pointed out that the exact causes of problems associated with some combinations of portland cement and Class C fly ash are unknown, making it impossible to specify limits on the chemical composition. He recommended suspending the use of combinations of Class C fly ash and portland cement which have been found to undergo early distress unless additional testing shows them to be acceptable. There are several possible reasons for the perceived problems associated with Class C fly ash:

1. *Available alkalis*. Available alkalis accelerate the hydration of the cementitious materials. Several survey respondents associated the combination of Class C fly ash and hot weather with problems in pavements. Later on, the alkalis may react with alkali-susceptible aggregates.

2. *Calcium*. Calcium can also affect both the rate of hydration and the tendency of alkali-susceptible aggregates to react. Since Class C fly ash has cementitious properties of its own, it hydrates along with the cement in fresh concrete, contributing to the heat of hydration. When alkali-susceptible aggregates are used, Class C fly ash may be worse than useless in terms of controlling the expansions. Bhatty and Greening (1978) found that cement hydration products having low CaO/SiO_2 ratios are more effective in binding alkalis. For this reason, supplementary cementing materials with low calcium contents limit the availability of alkalis that could react with the aggregate. Bhatty (1985) concluded that the amount of SiO_2 needed to prevent deleterious expansions due to alkali-silica reaction should be such that the CaO/SiO_2 ratio does not exceed about 1.5. The calcium content of Class C fly ash may not be low enough to provide good control of expansions except at relatively

high dosages. Several of the survey respondents mentioned that Class C fly ash was used at 15% replacement of the cement, which is too low to be of any benefit, and in some cases may be close to the "pessimum" dosage.

3. *Free lime*. Free lime, if present in the fly ash, would increase the water demand of the concrete, further contributing to the loss of workability.

4. *Sulfates*. Jennings et al. (1997) cited the sulfate content of the cementitious materials, and one of the survey respondents mentioned sulfates in the fly ash specifically. This may or may not be a separate factor, as fly ashes are often treated with alkali sulfates during the collection process.

5. *Alumina*. Class C fly ashes usually contain alumina in some quantity. The likelihood that some of the alumina is in the form of C_3A and C_4A_3ŋ is greater in Class C fly ash than in Class F fly ash because of the higher calcium content. If the hydration of C_3A is not controlled by gypsum, it can cause flash set. Portland and blended cements contain sulfates, usually in the form of gypsum, in order to control the setting of the C_3A. Normally the cement is formulated to contain sufficient gypsum to balance the C_3A it contains, but not the additional C_3A in fly ash added at the batch plant.

<u>Aggregates</u>

Alkali-silica reactivity. Aggregates susceptible to alkali-silica reactivity may cause deleterious expansion and cracking unless suitable control measures are taken. The Portland Cement Association (1998) developed a protocol for determining whether aggregates are likely to be susceptible and for evaluating the effectiveness of control measures. Table 1 summarizes the protocol for determining the potential for alkali reactivity for an aggregate. The aggregate is examined petrographically according to ASTM C 295. If it contains more than the specified limit of any of the reactive constituents listed in Table 1, it is to be considered potentially reactive. The aggregate is also tested according to ASTM C 1260. If the expansion at 14 days exceeds 0.10%, it is to be considered potentially reactive. An aggregate found to be potentially reactive by either of these two criteria may be further evaluated by ASTM C 1293. If the expansion at one year exceeds 0.04%, it is to be considered potentially reactive. If the expansion at one year does not exceed 0.04% and there is no prior evidence of reactivity in the field, it is considered nonreactive.

If an aggregate is found to be potentially reactive by this protocol, mitigation measures such as the use of blended cement, fly ash, or slag can be tested according to ASTM C 1260. A combination of cementitious materials that limits the expansion at 14 days to no more than 0.10% is considered acceptable for use with that aggregate.

Alkali-carbonate reactivity. A similar expansive reaction results from the combination of alkali with carbonates in some dolomitic limestones and calcitic dolomites. Much less is known about this reaction, as it occurs only with a few aggregates in isolated locations. ASTM C 586 is designed to test for potentially deleterious expansions. One such case occurred on two sections of I-20 in Louisiana. Dubberke and Marks (1987) found a dolomitic aggregate associated with rapid deterioration to contain ankerite. A similar aggregate from Kingston, Ontario, is subject to alkali-carbonate reaction. However, the authors did not test their aggregate for susceptibility to alkali-carbonate reaction.

TABLE 1. POTENTIAL FOR ALKALI-SILICA REACTIVITY (PCA, 1998)

Test Method	Aggregate is potentially reactive if …
ASTM C 295, "Standard Guide for Petrographic Examination of Aggregates for Concrete"	It contains *any* of the following minerals in quantities exceeding the limits shown: • 5.0% optically strained, microfractured, or microcrystalline quartz • 3.0% chert or chalcedony • 1.0% tridymite or cristobalite • 0.5% opal • 3.0% natural volcanic glass
ASTM C 1260, "Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)"	The expansion at 14 days is greater than 0.10%.
ASTM C 1293, "Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction"	The expansion at one year is greater than 0.04%.

Spalling. Three of the survey respondents cited spalling as a form of premature deterioration. One of the respondents observed that it seemed to be associated with the use of siliceous river gravels in the concrete. In another instance the aggregate was limestone. Spalling is typically considered to occur over a period of time as a result of compressive stress due to incompressibles in the joint or crack. However, minor spalling at joints has been known to occur at early ages due to microcracking that results from the "chattering" action resulting from improper joint sawing. Under traffic loading, the microcracking can lead to spalling at early ages. Fortunately, such occurrences are rare and are easily taken care of by instituting proper maintenance of joint sawing equipment and modifying joint sawing operations. Long-term joint and crack spalling are minimized by regular maintenance consisting of joint and crack re-sealing.

D-cracking. D-cracking is an aggregate-related freeze-thaw deterioration resulting in closely spaced parallel cracks along joints and random cracks (Schwartz, 1987). Stark (1976) has shown that essentially all aggregates that contribute to D-cracking are of sedimentary origin and range from pure limestone to dolomite in composition. A durability factor exceeding 60 by AASHTO T161 (NRC, 1999), a concrete freeze-thaw test, has been the most commonly accepted criterion for the exclusion of susceptible aggregate even though the test duration is at least two months. A rapid, simple test to identify susceptible aggregate is the Iowa Pore Index Test (Marks and Dubberke, 1982).

To be resistant to the effects of freezing and thawing, the concrete must have both sound aggregate and a proper air-void system, specifically 9±1% air in the paste (Klieger, 1956) and a spacing factor less than 0.2 mm (NRC, 1999). Schlorholtz (2000) identified poor construction practices that contributed to premature freeze-thaw deterioration due to a poor air-void system. Frost-resistant concrete is critical in producing durable pavements.

Aggregate grading. In a well-graded aggregate, the smaller particles efficiently fill up the spaces between the larger particles so that the void space remaining is minimized. Gress (1997) emphasized the importance of using the largest maximum aggregate size compatible with a continuous uniform aggregate grading. This requires a minimum of cement paste to fill the space between aggregate particles. The importance of minimizing the content of cement paste includes:

- minimizing the cost of the concrete, since cement is the most expensive component
- minimizing the heat of hydration, which is a function of the type and quantities of the various cementing materials
- minimizing the volume of entrained air required to achieve an acceptable air-void system, since the volume of cement paste to be protected is reduced
- increasing the volume stability of the hardened concrete, since aggregates generally restrain the shrinkage of the cement paste.

With a poorly graded aggregate, the concrete will be difficult to mix, place, consolidate, and finish without increasing the quantity of cement paste. Also, aggregates with insufficient fines may result in excessive bleeding. Sometimes only poorly-graded aggregates are available, or the larger aggregate particles must be removed or crushed to a smaller size because they are susceptible to D-cracking. In that case, the mix design must be adjusted to compensate. These adjustments may include increasing the quantity of cement, using supplementary cementing materials to minimize the heat of hydration, and increasing the volume of entrained air.

<u>Cement</u>

As discussed above, the presence of alkalis in fresh concrete accelerates the hydration of the cementitious materials and the setting of the concrete. Particularly on a hot day, it may be difficult to place and consolidate the concrete as a result. In addition, the loss of slump is associated with a loss of entrained air. Jennings et al. (1997) correlated incidents of premature deterioration with alkali in the cementitious materials. Where alkali-reactive aggregates were used, the alkali content of the cement would contribute to expansive reactions. Gress (1997) discussed the appropriateness of limiting only the cement alkalis, and recommended the approach used in Canada and Europe of limiting the total quantity of available alkalis in the concrete, regardless of source. Cross et al. (2000) found that synthetic air-entraining admixtures developed an unfavorable interaction with low-alkali cements.

Sulfate in the form of gypsum is added to the cement to control the hydration of the C_3A in order to prevent early stiffening. According to one survey respondent, the department of transportation in his state limits the sulfate content of the cement to a maximum of 3.0%, regardless of the C_3A content. The actual sulfate content, of course, will be less than this to ensure that the limit is not exceeded. For comparison, ASTM C 150 sets maximum limits of 3.0% for Type I cements with C_3A contents up to 8% and 3.5% for C_3A contents exceeding 8%, and allows higher sulfate contents provided that the mortar-bar expansion remains below acceptable limits. The lower limit set by the department of transportation results in cements that are undersulfated, that is, the sulfate content is below the optimum for strength gain. There may also be problems in controlling flash set under certain conditions. For example, elevated temperatures both reduce the solubility of gypsum (and thus the availability of sulfate) and increase the rate of hydration of the C_3A. Additional aluminates from supplementary cementing materials would only compound the problem, increasing the possibility of early stiffening.

CONSTRUCTION PRACTICES CONTRIBUTING TO PREMATURE DISTRESS

Hot weather placements

Hot weather and/or high placement temperatures were cited by several respondents to the survey, as well as by Jennings et al. (1997). Since the construction season spans the summer months, hot weather placements are unavoidable. Elevated temperatures contribute to several problems:

- The hydration of the cementitious materials, like many chemical reactions, is accelerated by elevated temperatures, increasing the likelihood of early stiffening. Thus mixing, placement, consolidation, and finishing are all more difficult to accomplish.
- Loss of slump is associated with loss of entrained air.
- The temperature of the concrete affects the amount of air-entraining admixture required to achieve a given air content. Whiting and Nagi (1998) provided some general guidelines for adjusting the dosage. If a given dosage was needed at 21 to 24°C to achieve the desired air content, that dosage should be reduced by approximately 30% for temperatures of 4 to 10°C, and increased approximately 30% for temperatures of 38 to 43°C.
- Rapid hydration of the cement will increase the rate of heat evolution. Excessive thermal gradients within the concrete may cause early cracking.
- Hot weather, particularly on windy days, may result in rapid drying of the surface, contributing to plastic shrinkage cracking.

Some survey respondents mentioned paving at night as a means of avoiding problems due to hot weather. Other methods for reducing the temperature of the concrete as placed include keeping the aggregate stockpiles covered, sprinkling them with water to cool them, and using ice as part of the mixing water. The use of supplementary cementing materials and set retarders may also be appropriate. Most supplementary cementing materials evolve less heat than cement and hydrate later, so that the heat they do evolve does not coincide with that evolved by the cement.

A rapid loss of slump may tempt the crew to add water to the concrete. While this practice makes the concrete easier to place, it can be detrimental to its durability. Retempering with a plasticizing admixture may be acceptable. The manufacturer’s guidelines should be followed. Nagi and Whiting (1994) developed a simple test for determining the water/cement ratio of concrete as delivered. This test uses a microwave oven to evaporate the water from a sample of fresh concrete. The water content is calculated from the weights before and after drying. For improved accuracy, the aggregate may be sieved from the sample and weighed separately.

Mixing

Several survey respondents mentioned very short mixing times, some as low as 30 seconds, as a potential cause of early pavement distress. It is difficult to imagine how any mixer, no matter how powerful, could thoroughly mix concrete, let alone entrain air, in 30 seconds. The low slumps (typically 1 in. or less) of today's pavement concretes make them difficult to mix thoroughly. Air-entraining admixtures need sufficient mixing time to develop the numerous minute air bubbles required to protect the concrete from frost damage. Since most pavement concretes are transported to the site in dump trucks rather than agitator trucks, the only mixing they receive is at the central batch plant. Whiting and Nagi (1998) found greater variations in air content with short mixing times, and recommended that concrete be mixed at least 75 seconds in order to obtain an adequate air-void system.

In describing the results of an investigation of I 80 and US 20 in Iowa, Gress (1997) observed, "…uniform mixing was not achieved in the short mix cycle, and proper air entrainment was never obtained." These findings are consistent with those of Stutzman (1999), who attributed the deterioration to marginal to substandard air-void systems.

Loss of Entrained Air

Various factors contribute to the loss of entrained air. Hot weather and the consequent loss of slump have been discussed above. Long haul times or delays in placement also contribute to the loss of entrained air. Whiting and Nagi (1998) reported that the loss of air content is typically ½ to 1 percentage points per hour, but that the rate of loss is greater for air contents higher than 6%. However, when the concrete is transported to the site in non-agitating trucks, the loss of air content ranges from 1 to 4 percentage points, depending on the haul time and consistency of the concrete.

Placement techniques can also cause loss of air content. Whiting and Nagi recommend that the exact loss for given equipment and techniques be established by test so that the loss of air content can be compensated for. One of the survey respondents cited this practice as a remedy for low air contents in the pavements in his state.

Consolidation

Vibration to consolidate the concrete can also result in loss of air. This phenomenon can be observed in the form of vibrator trails along the length of the pavement. The visible vibrator trail results from the displacement of coarse aggregate, leaving relatively high water/cement ratio mortar. Longitudinal drying shrinkage cracks may form in the vibrator trails. Stutzman (1999) found that the air-void systems of deteriorated pavements in Iowa were worst in the visible vibrator trails. Tymkowicz and Steffes (1999) reported that cores taken from longitudinal cracks in Interstate 80 in Iowa contained 3% air in the top half and 6% in the bottom half. They suggested excessive vibration as a possible cause of the reduced air content in the upper portion of the pavement. Their investigation indicated that the radius of effective consolidation of a vibrator was smaller than is commonly believed. Cores taken between vibrator trails showed significant entrapped air within 100 mm of the vibrator location. Simon et al. (1992) found similar results.

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ACI Committee 309 (1996) recommended that vibrator frequency be adjustable within the range of 8,000 and 12,000 vpm, with selection of the actual frequency (as well as other vibration parameters) to be made on site. Whiting and Nagi (1998) recommended that for slipform paving, vibration frequency should be limited to 8,000 vpm for paver speeds greater than 0.9 m/min, and frequencies should be reduced for lower speeds. In a study of deteriorated pavements in Iowa, Tymkowicz and Steffes (1999) identified excessive vibrator frequency at 12,000 vpm in one project, and excessive vibration locally when the paver speed was reduced to half that of normal with a vibrator frequency of 8,000 vpm. Excessive vibration is associated with locally low air contents and premature deterioration. Two survey respondents indicated that vibration frequencies are now being continuously monitored in their states.

Control Joints

Concrete is subject to drying shrinkage and to expansions and contractions due to changes in temperature. A pavement slab is sufficiently restrained as to crack both longitudinally and transversely when such contractions occur. Properly designed control joints either prevent cracks or allow them to develop only in a controlled manner related to the location of the joint. The mechanism leading to cracking (controlled or uncontrolled) may be visualized as follows: soon after concrete is placed, drying shrinkage of the concrete begins. The rate and extent of drying shrinkage depend on the concrete mix design and quality of curing. Also, due to heat of hydration, concrete generally sets at a temperature higher than the ambient temperature. Solar radiation magnifies this effect. Subsequent cooling (e.g., at night) results in a reduction in concrete volume.

Early drying shrinkage and thermal contraction of the slab are restrained by subbase/base friction. When the tensile stresses due to restraint exceed the tensile strength, the concrete cracks. Ideally, cracking will develop at sawed joint locations. Subsequent cracking may develop due to stresses caused by restrained curling and warping and traffic loading.

Late sawing or improper depth of sawing can result in cracking away from the joints. Such cracking may develop within the first 72 hours after concrete placement. Sawing too early may result in raveling at joints and spalling at later ages. Sawing too late can lead to random cracking. Contractors typically determine the right time for joint sawing based on their own judgment and experience. Guidelines have been developed (Okamoto et al., 1991; McCullough and Rasmussen, 1999) to allow contractors to determine the optimum time for joint sawing. These guidelines include the use of the maturity technique to establish acceptable concrete strength levels. It is commonly recommended that the minimum concrete compressive strength should be about 300 psi (for softer aggregates) to about 700 psi (for harder aggregates) before joint sawing can be initiated. In addition, the computer program HIPERPAV has been developed by the Federal Highway Administration to assess the effects of specific combinations of design, construction, and environmental inputs on the early-age behavior of concrete pavements (McCullough and Rasmussen, 1999). HIPERPAV has the capability to estimate the optimum time for joint sawing under given conditions.

Late or improper joint sawing continues to be a matter of concern, especially late sawing in relation to the sudden approach of a cold front. The use of the maturity technique has not gained widespread acceptance and contractors continue to rely on their own judgment for determining the optimum time for joint sawing. Four of the survey respondents reported cracking due to delayed or improper cutting of control joints.

Curing

Curing provides the conditions of moisture and temperature that allow the concrete to develop strength and watertightness. Senbetta and Malchow (1987) found that proper curing can increase the abrasion resistance of concrete by 50% or more compared to air drying without the use of curing compounds or using poor quality curing compounds. When curing is not provided, or when too much time elapses between finishing and curing, the concrete is vulnerable to plastic shrinkage cracking. Effective curing methods reduce both the rate and the magnitude of early shrinkage. For paving projects, the most practical method of curing is the application of a membrane-forming curing compound. These compounds prevent the loss of water from the concrete. Kosmatka and Panarese (1988) recommended that curing compounds be applied by hand-operated or power-driven spray equipment immediately after final finishing. Normally one coat is used, but two coats may be necessary. If a second coat is applied, it should be applied at right angles to the direction of application of the first coat so as to minimize the possibility that any part of the concrete surface remains uncoated. Fugitive dyes make it easier to check for complete coverage of the surface. Senbetta and Malchow emphasized the importance of selecting the right curing compound, as their effectiveness varies a great deal.

CONCLUSION AND RECOMMENDATIONS

As Jennings et al. (1997) pointed out, "Pavement deterioration is a result of a combination of multiple factors, and should not be blamed on a single ingredient or condition." When a new material is introduced and problems develop, it is tempting to blame the new material for the problems because the contractor has "done it that way for years" and "it always worked before" — until he started to use fly ash, for example. It appears that fly ash is the problem, when it may only have been one more contributing factor. Concrete is to a large extent an "abuser friendly" material, performing remarkably well under less than ideal circumstances. However, it is not infinitely forgiving of poor or marginal practice.

Many of the individual causes of early distress in pavements have been identified, and recommendations for good practice have been developed. For example, recommendations for assessing the potential for alkali-silica reactivity (PCA, 1998) or deciding when to saw control joints (Okamoto et al., 1991; McCullough and Rasmussen, 1999) can help prevent the related distresses. The use of the computer program HIPERPAV can reduce the probability of cracking.

In addition, the commonly recommended (but often ignored) practice of making and testing trial batches could prevent many problems related to incompatibility among concrete ingredients, particularly those that are apparent in the fresh concrete. A systematic and comprehensive guide to good practice for concrete pavements has yet to be developed. Such a guide might include a protocol of tests for the individual ingredients as well as tests and observations to be performed on trial batches. The latter would be designed to screen out combinations that are either incompatible or sensitive to the conditions likely to be encountered on the job site. For example, one can expect elevated temperatures during the summer, or delays in placement when the concrete is to be trucked to the site. Certain standard test methods could prove useful in detecting potential problems. For example, the initial and final setting times of the concrete could be determined using ASTM C 403, preferably at two or three temperatures representing the conditions likely to be encountered on the job. Air-void system parameters of the concrete as placed (at elevated temperatures, consolidated by vibration after a suitable waiting period) could be determined by ASTM C 457. Visual examination of the same specimens would reveal the degree of consolidation. In addition, new test methods such as that developed by the Japan Concrete Institute (1999) to measure autogenous shrinkage from the time of casting can indicate the rate and degree of shrinkage. Studies are currently underway to provide a sound scientific and engineering basis for a protocol to include these and other test methods.

It is not always known what causes a particular ingredient to behave differently in a concrete than another of the same type. Thus a specification cannot always guarantee good performance by setting limits on the composition of the various materials. Substitution of materials during the course of a job must therefore be made with caution, and not without the use of trial batches to ensure satisfactory performance.

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