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Roller-Compacted Concrete Pavements and Concrete Construction

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Contents

Mix Design, Thickness Design, and Construction of Roller-Compacted Concrete Pavement

THOMAS D. WHITE

ABSTRACT

Roller-compacted concrete pavement (RCCP) is a technology that involves the use of conventional materials and construction equipment in an unconventional application. The result is a savings in time of concrete pavement construction, which ultimately translates into a significant cost savings. Some interesting cause-and-effect relationships based on observations, limited research, and construction of RCCP are presented. In particular these relationships help to identify some of the constraints as well as the guidelines for successful mix design, thickness design, and construction.

In the evaluation of new concepts or technology, judgment may frequently be inhibited by the assumption that unknown phenomena and factors play an important role in function and performance. This is true in many areas of science and engineering and it is true specifically in pavement technology. However, it is not always necessary to invoke new postulates; generally it is a question of recognizing or understanding how prevailing, known phenomena evolve or how the existing factors interact to cause success or failure. The purpose of this paper is to discuss roller-compacted concrete pavement (RCCP) as a new pavement technology by examining its components and the various stages through which the mixture must go to become an effective RCCP, which involves in large part what is already known of a broad area of materials and construction.

In general terms, conventional concrete used in pavements is proportioned to satisfy a design flexural strength and maintain workability within a reasonable water-cement ratio. Other important factors considered in the mix design are durability and economy. Soil-cement is proportioned in large part for durability, with a water content to obtain maximum density as well as overall economy. RCCP mixtures cannot be designed totally as a conventional concrete mixture or as a soil-cement mixture but must be designed on the basis of key features of both these applications of cement in pavement construction. In addition, construction involves the use of asphalt equipment and techniques, which must be taken into consideration.

The expected in situ properties of RCCP are significant in their performance and as a result also to thickness design because of a high flexural strength relative to that of conventional concrete pavements. Increased flexural strength may affect thickness design in one of two ways: first, the increased flexural strength may be included in the selection of a design thickness, and second, existing acceptable thicknesses may be retained by using a factor of safety to reduce the higher RCCP flexural strengths to a value numerically comparable with that of conventional concrete pavement and therefore accommodate the increased strength through an increased confidence level.

Several RCCP projects (1,2) have been built in widely dispersed parts of North America. The geographical spread and varying conditions of the sites may be a fortunate occurrence because the differences provide the basis for some conclusions about the design, construction, and performance of RCCP.

The following projects are used as sources of information:

1. Fort Stewart, Georgia;
2. Fort Hood, Texas; and
3. Vancouver and Caycuse Camp, Cowichan, British Columbia, Canada.

The range in climate at these three locations is important. Fort Stewart is in a climatic region that is wet and hot with high humidity. The climate at Fort Hood is dry and hot with low humidity. In contrast, the British Columbia sites are wet and cold with high humidity. Certainly from a climatic point of view the design of the experiment is not satisfactory, but questions of cause and effect related to some major factors can be addressed because variation in climate is well represented.

In addition to the major climatic differences of the referenced RCCP sites, the following items of interest can be associated with the individual sites and are partly addressed in the balance of the paper.

1. Fort Stewart:
   a. Dowels
   b. Moist-sand cure
   c. Sawing
   d. Compaction
   e. Flexural strength
   f. Transportation
   g. Durability
2. Fort Hood:
   a. Burlap cure
   b. Spray cure
   c. Compound cure
   d. Sawing
   e. Unsound concrete
   f. Uncontrolled cracking
   g. Coarse aggregate
   h. Field compaction
   i. Laboratory compaction
   j. Quality control
   k. Transportation
   l. Laboratory durability

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3. British Columbia:
    a. Spray cure
    b. Spacing of uncontrolled cracks
    c. Thickness comparison and heavy loads
    d. Field durability

The foregoing list of factors is not complete but presents several points that form the basis for meaningful discussion.

MIX DESIGN AND PROPERTIES

As previously mentioned, roller-compacted concrete (RCC) must pass through several transitional stages to become an effective pavement. These stages include features of conventional concrete, soil-cement, and asphalt.

The basic mix design procedure is analogous to that for conventional concrete and is outlined in American Concrete Institute (ACI) Standard 207.5R-80 (Roller Compacted Concrete, 1983). However, there are several points that need to be emphasized. First, quality of the portland cement concrete (PCC) will be determined by the quality of its components, for example, cement, coarse aggregate, and fine aggregate. In general, because a PCC mix is used for RCCP does not bring any new and unknown reactions or phenomena into consideration. The differences in application of a conventional concrete and an RCC will come in the form of a lower water-cement ratio and higher density of the RCC. Both of these properties would generally be expected to have a beneficial effect on concrete properties. Careful note should be made of the word "generally," because the use of a minimum water-cement ratio in a hot, dry, and windy environment may prove inadequate for early hydration. The observations and consequences of this problem will be addressed in the following discussion.

Durability

Early laboratory freeze-thaw tests at the U.S. Army Engineer Waterways Experiment Station (WES) of field RCCP samples from Fort Stewart indicated inadequate durability; however, these laboratory results did not agree with the satisfactory durability demonstrated by RCCP in British Columbia over a several-year period. This difference in laboratory and field results was interesting and suggested the need for a close look at the type and amount of air existing in a concrete mixture that is compacted externally to a high density. In addition, cognizance should be taken of the development of the laboratory freeze-thaw test for a supersaturated condition and the potential for mitigation of a supersaturated condition first in a PCC with high density and second in a pavement structure. Current laboratory tests at WES in which more attention was given to achieving durability indicated satisfactory results. Also, prototype accelerated freeze-thaw tests being completed at the U.S. Army Engineer Cold Regions Research and Engineering Laboratory indicate satisfactory durability.

Gradation

The maximum aggregate size influences several aspects of RCCP. Larger maximum aggregate will result in greater segregation, which could result in honeycombed areas of the pavement. In addition, because construction utilizes asphalt laydown and handwork techniques, the larger maximum aggregate sizes will make handwork details difficult.

An impervious surface with a nominal macrotexture similar to that of an asphalt concrete is desired for RCCP. This type of surface can be achieved on asphalt concrete by both reducing the maximum size aggregate and requiring a dense- (well-) graded gradation. A 3/4-in. maximum aggregate size has provided an acceptable macrotexture for asphalt pavements. Using a denser gradation in RCCP will compensate somewhat for lack of a fluid paste and at the same time will contribute to increased density. The smaller maximum aggregate sizes may increase costs; however, material costs are only marginally significant in the economics associated with RCCP.

Laboratory Consolidation and Compaction

Laboratory compaction techniques affect determination of consistency, amount of water for compaction, and target density for field compaction. Consistency for conventional concrete can be evaluated with the Vebe apparatus, and with some modification this device can be used to evaluate RCCP mix consistency (ACI Standard 207.5R-80). Optimum moisture content and density for soil-cement can be determined with AASHTO Standards T 99 and T 134. For a dense-graded aggregate mixture such as that proposed here for RCCP, AASHTO T 180 is recommended in lieu of T 99 and T 134. Subsequent required field density can be set as a percentage of the laboratory standard density.

Once the cement or pozzolan proportions or both have been selected, water content can be adjusted within a reasonable range of water-cement ratio to achieve consolidation with the modified consolidation technique for the Vebe apparatus. This process is used to arrive at a consistency within the cement paste for efficient compaction. The cement paste is analagous to the soil binder in base, subbase, or other granular construction materials and the asphalt mastic in asphalt mixtures.

Density of PCC achieved through consolidation is desired for quality, durability, and strength. PCC consolidation is a one-time process, after which the PCC physical characteristics (density and thickness) are expected to remain constant. Soil-cement is compacted for generally the same reasons, except that effective utilization of soil-cement for stabilization and strength is related to achieving an optimum density for a given soil-cement combination. The optimum density is achieved by varying the soil-cement binder consistency by adding water. Asphalt-concrete strength and binder content are also tied to the compaction. However, unlike PCC and soil-cement, asphalt concrete will continue to densify under the compactive effect of traffic.

The laboratory compactive effort adopted for soil-cement is related to the constructability or density that can be achieved with certain types of compaction equipment (3,p.30) (e.g., a tamping roller). On the other hand, laboratory compaction for asphalt concrete was initially selected on the basis of density achieved in test sections trafficked with prototype traffic, although the percentage of the laboratory (standard) density was adopted on the basis of ability of field equipment available at the time (the 1940s) to achieve compaction (4-6). The factors related to selecting a target density for RCC are similar to those for soil-cement because once the compacted concrete has been cured, traffic will not induce additional changes. Therefore, selection of a laboratory compactive effort should be related to constructability and achieving a uniform density. However, the density achieved by a vibratory roller used to compact RCC should be evaluated in light of the soil-cement standard compactive effort based on a tamping roller and an expected difference in compactability of the dense-graded aggregate mixture of RCCP.

PLANT PROCESS

Desired long-term performance of pavements, including RCCP, will be achieved first by use of quality materials and construction. Second, consistency in materials and the production process must be maintained. The reason for addressing this second point is generally obvious, but if the recommendations by Pittman and White (1) discussed earlier in this paper are accepted, denser aggregate gradation and a smaller maximum aggregate size than that normally found in concrete mixtures will be used. Some components of an asphalt plant process can aid in achieving efficiency of mixing and the necessary volume of RCC production to support paving operations.

Prior recommendations on gradation were made based on experience with asphalt mixtures; however, in production of asphalt mixtures even a 3/4-in. maximum size aggregate tends to segregate unless care is taken in stockpiling, feeding, and handling. Effective control of gradation for a 3/4-in. dense-graded asphalt mixture generally involves use of three cold bins for the coarse aggregate, fine aggregate, and sand. Mineral filler, if required, is fed from a separate, enclosed hopper. It is anticipated that with multiple stockpiles, a greater portion of which will be fine aggregate as compared with that used in the usual concrete mixture, potential exists for high water-content variations. Stockpile water contents must be monitored because the water content of RCC is lower than that of the usual cement, so any variations in water will be accentuated.

Once the mixing process has started in normal PCC mixtures and a paste has been developed, segregation of the aggregate is inhibited. A paste, or binder of asphalt or mineral filler or both in asphalt mixture, also functions to inhibit segregation. However, the low water content in RCC restricts development of a paste and consequently segregation is a potential problem. Care must be exercised in the transportation, laydown, and handwork associated with constructing an RCCP to maintain homogeneity in the finished product.

CONSTRUCTION

The approach to construction of RCCP must begin with consideration of the use to which the pavement will be put and the expectation for performance. In addition, some limitations exist because of the equipment used for hauling, placement, and compaction.

Transporting

If RCC material is mixed and transported in ready-mix trucks, the drums of those trucks should be in good condition and clean. Even when these conditions are met, unloading is slow, but not meeting these conditions will result in even greater difficulty. Large paving projects are best served by using dump trucks (or the equivalent) to haul the paving mixture rather than using ready-mix operations. In fact, at this point the construction process can be compared with that in an asphalt paving project; the only problem, as mentioned earlier, is segregation, because of a lack of paste. Truck bed covers may be required to retain moisture under extreme conditions of temperature and humidity in the same way that a cover is required to maintain temperature of an asphalt mix in cool weather.

Laydown

The laydown, finish, and compaction processes are the key to the economy of RCCP because only a fraction of time is required for them as compared with those in conventional concrete pavement. For example, in lieu of forming or slipforming individual lanes of conventional concrete pavement, which require setup and preparation time for each paving lane, an RCCP is paved with a highly mobile asphalt laydown machine that generally is only constrained by mix production and number of trucks available for hauling. The time for construction of an RCCP is in the range of 30 percent of that required for a conventional concrete project.

Handwork associated with the laydown operation utilizes most of the same tools used for asphalt, including lutes, rakes, and shovels. However, the work requires more effort because the mix is not as plastic as either conventional concrete or asphalt and is more analogous to working with a soil binder and gravel mixture that has been partially compacted.

Joints

Two types of joints will naturally occur as a result of the construction: longitudinal and transverse. Transverse joints will occur as a result of long delays or at the end of a day's paving. These joints can be handled like transverse joints in asphalt paving, the only difference being that the RCC pavement thickness is significantly greater. Longitudinal joints fall into two categories: fresh and cold. If multiple pavers or short paving lanes are utilized, longitudinal joints are not a great problem. In these cases the joints can be ameliorated. However, delays in construction that would allow hydration to proceed beyond the point when adequate bond occurs will result in a cold joint with poor bond.

An important consideration for RCC pavements is the approach to be adopted for sawing joints. Concrete pavement sawing is normally expected to take place after there has been sufficient hydration to prevent the concrete from raveling and spalling during the sawing action. Because the water content of RCCP is marginal for hydration, external climatic conditions such as high temperature, low humidity, and wind can reduce the already low moisture content, resulting in retardation of surface hydration, which delays a strength gain adequate to accommodate sawing. The delay can result in uncontrolled cracking, which precludes the benefit of sawing a regular joint pattern. Additional moisture may be lost in the region of the crack in addition to the amount required for hydration, resulting in unsound concrete around the crack. The critical nature of adequate moisture for hydration in RCC pavement mixtures was demonstrated through experience at Fort Hood, where conditions of high temperature, low humidity, and wind existed during construction. At this RCCP project there was initially some sawing, but because a slow gain in strength, raveling and spalling, and uncontrolled cracking occurred in spite of the sawing, a decision was made to discontinue the practice. Before the pavement was opened for operations, a steel bar was used to sound the concrete on either side of both the uncontrolled cracks and the sawn joints. Results indicated that unsound concrete existed on both sides of all uncontrolled cracks but none was found at sawn joints. The conclusion that can be drawn is that water from the sawing operation completed the hydration process around the sawn joints. It is possible that raveling from sawing would be less of a problem at the uncontrolled cracks than future spalling of unsound concrete. The decision to stop sawing at Fort Hood because of raveling and appearance of random cracks appears to have been made without consideration of the basic remedies.

When uncontrolled cracking occurs in concrete pavement in spite of sawing, it is common to balance options of sawing sooner or making a deeper saw cut. Because RCC gain in strength is slow, sawing sooner would not be a remedy; however, sawing deeper would be appropriate to ensure a weakened plane to control cracking. In retrospect the initial saw cut at Fort Hood was a minimum T/6 and the subsequent cracking adjacent to saw cuts should have come as no surprise. Other logical reasons for sawing deeper are that the strength gain would be faster and lower in the pavement because of a higher moisture content and that sawing deeper would have ensured an adequate weakened plane.

Although laydown with an asphalt paving machine can be string or wire controlled for elevation and alignment, subsequent compaction leaves a free edge with a variable alignment and unless multiple laydown machines are used or very short lanes are paved, the resulting juncture of two paving lanes will be a discontinuity that will result in an irregular crack. Except as noted, sawing will not eliminate formation of this longitudinal crack.

Concern with cracks versus use of joints is predicated on accelerated deterioration and maintenance that are associated with such cracks in concrete pavements. In two-lane pavements some control can be exercised through use of two paving machines and sawing. However, in broad areas of pavement such as parking areas or loading docks, longitudinal cracks will be a problem.

Curing

The effects of various types of curing have been significant in helping to understand the observations of RCC pavement construction and performance. Curing types include membrane, burlap, water spray, and wet sand. A membrane cure does not appear to supply the extra moisture for early surface hydration, particularly at cracks. Some coordination problems have been encountered when burlap is used. Water spray is the most common type, apparently because of the ease of use. The water spray cure does supply water for hydration but care must be used to keep from eroding the surface with excess water or the force of the spray. Excess surface water may also saturate adjacent construction areas. A wet-sand cure, used at Fort Stewart as an expedient because of the lack of curing compound, apparently supplies adequate moisture for early hydration as indicated by successful sawing of joints. The wet-sand cure may be an overlooked technique.

Smoothness and Texture

Smoothness and texture of RCCP have been partly addressed. Smoothness, although not given a great deal of attention, has not been a particular problem, and driving at 45 mph diagonally across the parking area paving lanes at Fort Hood could be described as no rougher than driving across some recently constructed concrete highway pavements. The Fort Hood project was paved without using a stringline for grade control of the asphalt paver.

The larger the maximum aggregate size, the more segregation will occur in the mix and the more pronounced will be the surface texture. An aggregate gradation, already discussed, closer to a dense asphalt gradation can reduce segregation and reduce the coarseness of the surface texture. A rubber-tired roller has also been used to knead together surface cracking caused by a vibratory steel-wheeled roller. The resulting surface texture resembles that of an asphalt pavement.

DESIGN

Pavement thickness design criteria are generally based on an envelope of failures recorded in pavement test sections or performance observations of prototype pavements. In some cases, design criteria have been modified as a result of laboratory testing. However, incorporation of laboratory data into design criteria has been infrequent because of an inability to directly compare field performance and laboratory test results; for example, there is a shift function that is not generally apparent. Consequently, when new pavement technology or materials are considered without field performance, a great deal of uncertainty develops in its subsequent application. Such a situation exists with RCCP.

In pavement design the flexural strength is a significant design factor (1). Laboratory flexural strength tests of pavement samples indicate that RCCP can develop a 25 percent higher flexural strength than a conventional concrete pavement. In large part, this higher flexural strength will come from a higher density achieved from the compaction applied during construction. Taking advantage of this higher strength hinges on being able to achieve the necessary density uniformly from construction practices. Secondary consideration must be given to the question of whether the higher-density, higher-modulus concrete mixture will have a different fatigue relationship from that of conventional concrete. A related, secondary consideration is the minimum thickness required for the RCCP to maintain its integrity.

Adequate mix design, uniformity of materials, and moisture and density control will ensure a desired flexural strength for design. It is doubtful that significant differences in fatigue properties will exist for RCCP, because too many other factors affecting the quality and performance of the concrete will predominate. Performance of RCCP related to loading has been very satisfactory. In British Columbia a conventional 24-in. concrete pavement has shown severe structural distress, whereas a 17-in.-thick RCCP subjected to the same loads for the same length of time has not (2). The minimum thickness required to ensure that RCCP will maintain its integrety has only been addressed recently through construction of thinner test sections (1).

One measure of concrete pavement performance is the amount of cracking. Cracking of conventional concrete pavements is minimized through proper spacing of sawn joints. Unless the same approach is adopted for RCCP, uncontrolled cracking will occur with discontinuities that would lead to further undesired cracking. This would be a natural and not unexpected phenomenon, and increased thickness may be required to compensate for such occurrences.

Foundation strength is less significant in concrete pavement thickness design than flexural strength, but in the case of RCCP two considerations highlight the need for foundation strength and quality. Functionally, a strong foundation (subgrade modulus greater than 200 psi) will result in less working of cracks because of better pavement support. The effect will be less spalling and additional cracking. From a practical consideration, compaction on a weak resilient foundation may result in low density. Also vibratory compaction directly on a weak wet foundation could result in the foundation material's pushing into and contaminating the

TABLE 1 Thickness Using AASHTO Interim Guide Method for Highways (7)

	M_R (psi)	Working Stress (psi)	E (psi)	Traffic (vehicles/day)	k (pci)	Thickness (in.)
Conventional concrete	600	457	4.2×10^6	8.4×10^6	300	9.5
RCC	750	564	4.2×10^6	8.4×10^6	300	8.5

TABLE 2 Thickness Using COE/AF Method for Storage Areas (8)

	M_R (psi)	Traffic (vehicles/day)	Traffic Category	Design Index	k (pci)	Thickness (in.)
Conventional concrete	600	100	VII	10	300	9.7
RCC	750	100	VII	10	300	8.3

TABLE 3 Thickness Using PCA Method for Industrial Floors on Grade (9)

	M_R (psi)	Factor of Safety	Working Stress (psi)	Stress per 1,000 lb	Thickness (in.)
Conventional concrete	600	2.0	300	7.5	9.0
RCC	750	2.0	375	9.4	7.8

RCC mix. A layered foundation with an adequately compacted subgrade and granular base course similar to an asphalt pavement would provide an adequate foundation for compaction as well as a high-strength pavement foundation. Characteristics of the base course should also mitigate erosion and pumping.

The question of design philosophy as pointed out in the introduction revolves around whether to apply the higher flexural strengths directly into existing design procedures or to apply a factor to reduce the flexural strength to a value associated with conventional concrete before the design procedures are involved.

The effect on thickness of using a high flexural strength for RCCP is examined in a specific example by using three thickness design sources--those of AASHTO (7), U.S. Army Corps of Engineers and Air Force (COE/AF) (8), and the Portland Cement Association (PCA) (9,10). The following assumptions are made:

Single-axle forklift load = 40,000 lb,
Wheel spacing = 50 in.,
Contact area = 200 in.^{2},
Total forklift operations = 300,000,
Modulus of subgrade reaction (k) = 300 pci,
Modulus of rupture (M_R), conventional concrete (CC) = 600 psi, and
Modulus of rupture (M_R), roller-compacted concrete (RCC) = 750 psi.

Although not directly applicable, the AASHTO Interim Guide (7) was used to estimate thickness requirements by using a factor of 1.33 to compute working stress, a concrete modulus (E) of 4.2×10^6 psi, and a slab thickness (D) of 10 in., which results in an equivalent load factor of 27.91 with a terminal serviceability (p_t) of 2.5. The resulting thicknesses are shown in Table 1.

A more direct thickness design method for storage areas is available in TM 5-822-6; AFM 88-7 (8,Ch.1). The specifications used in the design include the 40,000-lb single-axle forklift, 64 vehicles per day, traffic category VII, and a design index of 10. The results are shown in Table 2.

The PCA thickness design procedure for industrial concrete floors was applied by using the axle load of 40,000 lb, single-wheel spacing of 50 in., a 200-in.^{2} single-wheel contact area, and a factor of safety (FS) of 2.0 (10). These thicknesses are shown in Table 3.

As a point of interest the new PCA thickness design procedure for highways was invoked and the allowable load repetitions were checked for both fatigue and erosion assuming a condition of shoulders, aggregate interlock joints, load safety factor of 1.0, and thickness of 9.5 in. and 8.5 in. for conventional concrete and RCC, respectively. The results of this analysis are shown in Table 4.

In general, the 25 percent higher flexural strength is translated into a 1-in. reduction in thickness. It was not the purpose of the example to compare the different thickness design methods and for the assumptions there is little difference between the methods. The COE/AF design procedure may be classified as more conservative than the PCA method as a result of the combination of several factors related to traffic and loading. The PCA method is less conservative because the procedure is more specific as to design input factors and the required design thickness can be more narrowly focused. Perhaps the greatest difference underscored is the effect of considering erosion in the new PCA highway design procedure. The allowable repetitions,

TABLE 4 Thickness Using PCA Method for Highways (10)

	M_R (psi)	Thickness (in.)	Stress Ratio	Allowable Repetitions	Erosion Factor	Allowable Repetitions
Conventional concrete	600	9.5	0.245	340,000	2.36	270,000
RCC	750	8.5	0.227	1,700,000	2.48	120,000

considering erosion, are less than the assumed traffic for the design used to arrive at the 9.5-in. and 8.5-in. thicknesses. However, the allowable repetitions are adequate if dowels are considered.

CONCLUSIONS

During mix design and construction, RCCP must be treated as a transitional material, which requires consideration of portland cement concrete, soil-cement, and asphalt-concrete technologies. It is perhaps unusual to draw from such diverse technologies but also interesting that when basic questions of materials, construction, and performance are arrayed against existing knowledge, cause and effect are easily explained. In the future, with wider application, RCCP will not appear to be an unknown technology but will be applied without hesitation.

Confusion about RCCP durability because of laboratory test results should no longer exist. On the other hand, density from the standpoint of a laboratory standard or field constructability has not been fully defined. The crux is what density can be achieved with existing compaction equipment and what the relation of this field density is to a laboratory standard.

Hydration in concrete is assumed to take place, but under certain climatic conditions, RCCP is characterized by marginal water content, which may be critical for hydration. Experience with different curing techniques has determined that use of a curing compound is not adequate because discontinuities in the membrane can allow loss of the marginal moisture. On the other hand, water spray and wet sand are adequate because a positive moisture condition is maintained at the RCCP surface for hydration. Wetted burlap will accomplish the same purpose but coordination has yet to be worked out for application, maintenance, and conflicts with paving operations.

Allowing uncontrolled cracking is an expediency that may result in decreased performance and increased maintenance. Sawing can be accomplished for control of crack patterns in two-lane pavements and may help control transverse cracking in broader pavement areas. However, longitudinal cracking in broad paved areas will be difficult to control.

Increased flexural strength of RCCP translates into approximately a 1-in. reduction in design thickness. A minimum RCCP thickness for the pavement to maintain its integrity has not been defined. Defining this minimum thickness will involve evaluation of climatic stresses, load stresses, and effects of naturally occurring and constructed discontinuities that would lead to an abbreviated pavement life.

Smoothness of RCCP has been acceptable and may be expected to be even better when the full capability of asphalt laydown equipment is utilized.

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Roller-Compacted Concrete for Heavy-Duty Pavements: Past Performance, Recent Projects, and Recommended Construction Methods

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ABSTRACT

Roller-compacted concrete (RCC) for pavements combines the technologies of cement-treated aggregate base (soil-cement) and portland cement concrete to produce a rigid slab of moderately high strength capable of carrying heavy wheel loads. Aggregate should be well-graded gravel or crushed rock, 100 percent passing the 7/8-in. (22-mm) sieve. Fine aggregate up to 14 percent passing the No. 200 (75-μm) sieve is acceptable. RCC for heavy-duty pavement has been used in British Columbia since 1976. The first installation was a 4-acre (1.6-hectare) log-sorting yard on Vancouver Island. Since that time 10 other RCC heavy-duty pavements have been built. In 1983 a coal storage area using RCC was the first project in a severe winter climate. In 1985 RCC was used for container storage areas in Houston, Texas, and Tacoma, Washington. The U.S. Army Corps of Engineers built heavy-duty RCC pavements to carry military vehicles at Fort Hood, Texas, and Fort Lewis, Washington. An aircraft parking area was built at Portland, Oregon--the first use at an airport. Some of the most significant projects that have been built since 1976 are reviewed and the construction process is described.

Roller-compacted concrete (RCC) has been used over the last 10 to 12 years in several parts of the world, primarily in water control structures. During that same period, a soil-cement type mixture with 12 to 14 percent portland cement content was being used for heavy-duty pavements in the forest industry of British Columbia, Canada. Because the materials and mixing process for both the dams and pavements are similar, the term "roller-compacted concrete" has been chosen to describe the heavy-duty pavement construction process.

It is important for the reader to have a clear understanding of what is meant by roller-compacted concrete in reference to heavy-duty pavements. RCC for paving differs from its application in water control structures in the following respects:

- The surface of RCC pavement is subjected to abrasion from traffic, log stackers, container carriers, and military vehicles;
- In winter climates, RCC must withstand the action of freezing and thawing cycles and the possible application of deicing salt; and
- RCC heavy-duty pavement has a much higher portland cement content than that used in dams. Fly ash is sometimes used in the range of 15 to 20 percent by weight of total cementitious materials.

The following definition of RCC heavy-duty pavement is suggested(1): the mixture of

1. Aggregate (natural gravel or crushed rock), which is graded from 100 percent passing the 7/8-in. (22-mm) sieve to a maximum of 14 percent passing the No. 200 (75-μm) sieve (note: in two-layer construction, the maximum size of coarse aggregate may be increased to 1 1/2 in. (40 mm) in the base layer);
2. Type I (Type 10 in Canada) portland cement (may include other cementitious additives such as fly ash);
3. Water in the quantity required for maximum density; and
4. Other additives, such as accelerators, retarders, and water reducers.

The mixture is blended into a heterogeneous mass of zero slump. Placement is by asphalt paving machine and compaction is achieved with steel-wheel and rubber-tired rollers.

RCC heavy-duty pavements may be single or multiple layers depending on slab thickness and the capacity of the placing equipment. The pavements contain a high portland cement content (12 to 14 percent by weight of aggregate) in the top layer for freeze-thaw durability and wear resistance. To assure a smooth, dense surface, the maximum coarse aggregate size in the surface layer should not exceed 7/8 in.

In multiple-layer construction, the portland cement content of the bottom layer may be reduced to 6 to 8 percent by weight, similar to ordinary cement-treated aggregate, because wear resistance is not a factor.

EARLY RCC PAVEMENT PROJECTS

The first roller-compacted concrete heavy-duty pavements in Canada were built in British Columbia in the mid-1970s by the forest industry. In response to pressure from the provincial government to reduce the amount of wood debris entering the water from coastal log-sorting operations, the forest industry expanded its program of construction of dry-land log sorts. The goal was total elimination of log sorting in the water. A number of dry-land log sorts were already in operation that had asphalt or gravel surfacing.

The asphalt did not perform well when subjected to hot summer temperatures, hydraulic fluid and fuel spillage, and damage from equipment forks. On the

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unpaved surfaces gravel was always being lost when wood debris was cleaned off the surface. The wood debris could not be burned when contaminated with gravel. The forest industry was ready for an alternative paving system that could solve these problems.

RCC appeared to offer the solution to most of them. RCC is not affected by summer temperature; petroleum products do not attack it; equipment forks cannot gouge it; and debris cleaned off the surface is not contaminated and thus is completely burnable.

RCC has another built-in benefit: it gains strength with age. New and higher-capacity equipment is constantly being introduced into the logging industry, which means higher axle loads. The RCC property of strength gain with age offers a factor of safety to the user at no additional cost.

British Columbia Forest Products, one of Canada's largest integrated forest products companies, took all these factors into account in 1976 when planning the 4-acre (1.6-hectare) dry-land log sort at Caycuse, Cowichan Lake, Vancouver Island. It was decided to build the entire pavement structure in cement-stabilized aggregate. The slab thickness of 14 in. (355 mm) required placement in two lifts. The top lift had a cement content of 8 percent by weight. At that time 13 percent was an arbitrary figure based on engineering judgment and some laboratory testing of beams and cylinders.

The pavement was placed into service in the fall of 1976. The paved area was doubled in 1979, again using RCC. The Caycuse dry-land log sort stands out as an excellent example of RCC performance. Surface maintenance over 9 years of constant use has been limited to replacement of a few areas of delamination between layers and patching of some raveled joint areas where compaction of the surface during placement was inadequate. British Columbia Forest Products reports that the Caycuse dry-land log sort is one of their most cost-efficient log-sorting operations.

MacMillan Bloedel Ltd., another of Canada's large integrated forest products companies, investigated RCC for some of their logging operations in the Queen Charlotte Islands off the north coast of British Columbia. The ability to use substandard aggregate, available locally at low cost, along with fewer handling problems associated with portland cement compared with liquid asphalt were perceived as advantages. By the end of the 1970s MacMillan Bloedel had four dry-land log sorts in service in the Queen Charlotte Islands and another at Port McNeil on northern Vancouver Island.

All of these projects used a slab thickness similar to the Caycuse pavement--14 in. constructed in two layers with a portland cement content of 13 percent by weight of aggregate.

The last of what could be considered the first RCC projects was built by Crown Forest Industries (formerly Crown Zellerbach Ltd.) at their sawmill and plywood plant at Coquitlam, British Columbia (near Vancouver), in 1979. Here raw logs are lifted from the Fraser River and stored until used by the mill. The loaded log stackers place the timber in storage fingers off a main traveled runway. Satisfaction has been such that two expansions of the storage area using RCC have been built since 1979. Although the RCC was slightly higher in first cost than an asphalt alternative, less expensive two-wheel drive log stackers operating on a smooth, hard surface could be used in place of four-wheel drive machines. The saving in equipment cost justified the higher first cost of the RCC pavement.

THICKNESS DESIGN

Slab thickness design for the early RCC pavements in British Columbia relied on engineering judgment, which was based on experience from heavy-duty cement stabilized base and portland cement concrete paving projects. This "super soil-cement," with cement content of 13 percent, was undoubtedly stronger than the normal cement-stabilized base mixture but little other engineering information about the material was available. Typical slab thickness was in the range of 12 to 14 in. (300 to 350 mm).

The thickness design method currently being used for heavy-duty RCC pavements in Canada is the Portland Cement Association (PCA) airport thickness design procedure (2).

It is assumed that the RCC exhibits flexural strength properties similar to those of normal slump concrete. However, this may attribute a higher degree of consistency to the RCC mix than should be expected. The whole process of aggregate selection, cement and water addition, and mixing of the materials in a pugmill is less precise than that for regular concrete. Therefore, to account for potential mix deficiencies, the flexural stress value selected in the design procedure should be lower than that for a typical slump concrete. This could be applied as a reduction coefficient to the stress value. A reduction of the order of 10 to 15 percent is suggested.

One of the variables considered in rigid pavement design is the magnitude and frequency of wheel loadings. Unlike a highway where this information can be obtained from traffic and loadometer surveys, the traffic pattern on a dry-land log sort, for instance, is difficult to predict. Initially the owners will have a layout for the various operations that will be performed; however, this layout may be changed in later years. Therefore, with the possible exception of some of the remote parts of the yard, it must be assumed that the maximum wheel load and unlimited applications of load will be needed over most of the surface.

A typical log-stacker load is 220,000 lb (100 000 kg). Container sidelift equipment has a loaded-axle value of 250,000 lb. Table 1 shows a summary of core samples from several British Columbia projects. The values obtained support the view that RCC has strength properties similar to those of regular concrete and that therefore concrete pavement thickness design methods may be valid for RCC as well.

MIX DESIGN

The design of an RCC pavement has its origin in cement-treated aggregate base technology. The first RCC pavement at the Caycuse dry-land log sort was modification of a cement-treated base specification.

Through experience it is now recommended that the maximum size of coarse aggregate should not exceed 7/8 in. to provide a dense and smooth operating sur-

TABLE 1 Typical RCC Core Strengths from Some British Columbia Projects

Project	Year Built	Date of Sample	Avg Compressive Strength (psi)
Caycuse log sort, Vancouver Island	1976	1980	4,210
Caycuse log sort, Vancouver Island	1976	1984	5,880
Fraser Mills log yard, Vancouver	1982	1983	4,700
Bullmoose Coal Mine	1983	1983	2,220[a]
Fraser Surrey Dock, Vancouver	1984	1984	4,570[b]

Note: 1 psi = 6.89 MPa.

[a] Values are average 56-day core tests. The cementitious portion of the mix contained 50 percent natural pozzolan.

[b] Values are for 28-day field-cured samples (Nov. 1984).

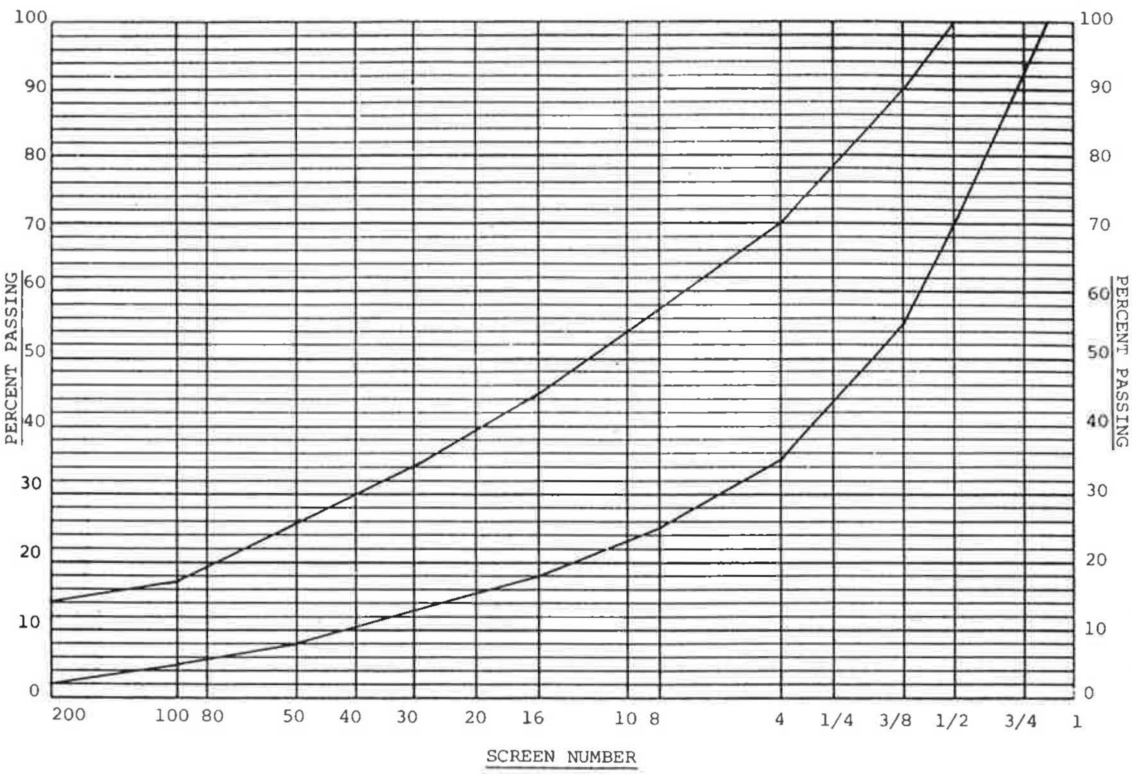

FIGURE 1 Typical aggregate gradation envelope for single-lift mix.

face. The fraction passing the No. 200 sieve can be as high as 14 percent. In two-lift construction the maximum coarse aggregate size in the base layer can be increased to 1 1/2 in. (40 mm) because a smooth operating surface is not required. Figure 1 shows a typical aggregate gradation envelope for a single-lift mix.

Water content for the mix is selected on the basis of optimum moisture content for maximum density and not on the water-cement ratio. Hydration of the portland cement is a secondary effect of the presence of the water. Compaction at the correct moisture content is vital during the placing operation to ensure a high-density surface finish and close surface-level tolerance.

Although the cement content on the early projects was selected rather arbitrarily at what was considered to be a safe level, materials engineers now approach selection of cement content on the basis of flexural strength of laboratory cast specimens. Typical 28-day flexural values of 600 to 700 psi (4.1 to 4.8 MPa) are attainable and frequently specified. Most samples are made on a vibrating table or using procedures for making and testing zero-slump concrete specimens (3). Whether this procedure truly represents field conditions for RCC in place is unknown; however, the error is probably on the side of safety.

The question of other additives to the RCC mix is often raised. Several of the projects in place have used fly ash (the Bullmoose Coal Mine loadout used a natural pozzolan at 50 percent of total cementitious content). Most have been in the range of 15 to 20 percent. It is suggested that pozzolanic additions be limited to 20 percent for pavements until further evaluation of field samples is made.

The need for an air-entraining agent in the RCC mix has often been discussed. It was included in some early projects; however, there is no conclusive evidence that freeze-thaw durability is improved. Other projects have used no air-entraining agent and are performing well in freeze-thaw environments.

CONSTRUCTION

The construction procedures involved in building an RCC pavement are not complicated. Common roadbuilding equipment is used, and although there are parts of the process that require particular attention, no specially skilled tradesmen are needed (4).

Because RCC heavy-duty pavement is a rigid slab, all of the load is carried in the slab itself. There is no need for a granular base, although a 6-in. (150-mm) lift of crushed gravel is often used as a working surface for fine grading and accurate thickness control of the RCC slab.

Subgrade preparation is no different than for other roadbuilding methods--uniformity of the subgrade should be ensured, soft subgrade areas should be removed, the water table should be kept out of the freezing zone where possible, and any fine-grained, frost-susceptible materials should be removed.

RCC is usually produced in a continuous-flow pugmill mixer. A premix-type concrete batching plant can be used, although the rate of production is slower. The mix-in-place method of blending portland cement with aggregate is not suitable for RCC construction and should not be used. It does not provide the necessary quality control or uniformity of blending of the portland cement and aggregate.

Care must be taken to minimize segregation while handling the RCC mix. Crushed aggregate is helpful in this respect. Discharging the mix into a holding hopper off the end of the pugmill belt and periodically emptying the mix into waiting trucks also reduces segregation.

Dump trucks are normally used to haul the mix to the paving site. A maximum elapsed time of 60 min after mixing is the usual specification controlling haul and placing. At the pavement site the RCC mix is placed by an asphalt paving machine, preferably the type with the oscillating-tamping screed rather than the vibrating-pan screed. Although some pavers are capable of placing material up to 24 ft (7.3 m) wide, it has been found that limiting the placing width to 12 to 14 ft (3.6 to 4.3 m) helps to reduce segregation. The augurs tend to segregate the mix at the greater placing width.

It must be stressed that on the basis of experience to date, the only placing equipment that is suitable for obtaining a quality RCC pavement surface is an asphalt paving machine. Graders, bulldozers, or other types of spreaders tear the surface during final finishing. Graders have been used successfully, however, to spread base-layer material.

The depth of lift that can be placed with the conventional asphalt paving machine is limited to about 10-in. (250-mm) uncompacted thickness. It should be noted that the thickness of the lift and the amount of compaction needed to bring it to the specified density have a significant bearing on the surface tolerance of the completed pavement. A typical 10-in. uncompacted lift will compress by about 1 to 1 1/2 in. (25 to 40 mm) under vibratory compaction. Some sections of the surface may become more compressed than others during rolling, which leads to unevenness in the final surface. It is reported that new heavy-duty pavers are capable of compaction over 90 percent of Modified Proctor density, leaving much less compaction required from subsequent rolling. They are also reported to be capable of much thicker lift placement, which in some cases will eliminate the need for two-lift construction.

In a discussion of surface tolerance it is important for owners and designers to have a clear concept of the type of equipment that will use the pavement and at what speed it will travel. A pavement that is perfectly satisfactory for the slow-speed operation of a container carrier may not be smooth enough for short-wheelbase trucks hauling the containers to the transfer area.

At the present time, RCC pavements are not recommended for moderate or high-speed traffic. Until the construction industry can consistently build RCC pavements to meet a surface tolerance of 1/4 in. (6.35 mm) in 10 ft (3 m) or less, RCC should only be used for heavily loaded industrial pavements with traffic at low operating speeds.

Compaction of the freshly placed RCC mix should follow immediately behind the placing operation without delay. The rolling sequence starts with vibratory steel-wheel equipment followed by a rubber-tired compactor. A nonvibrating steel roller may be used for final finish. In-place density is measured by conventional field methods, usually the nuclear densitometer.

When the design thickness of the slab is such that two-lift construction is required, the top layer must be placed on the same day as the base layer so that maximum bond between layers is obtained. No special procedures are used to bond the two layers together, but where temperature, wind, and humidity may dry out the surface of the base layer, it should be kept moist.

It has been found that density of the free edge can be improved by delaying compaction of the outside 18 in. (450 mm) for about 20 to 30 min after placement.

JOINTS

RCC exhibits normal concrete shrinkage properties. Spacing of shrinkage cracks is somewhat greater, however, generally 40 to 60 ft (12 to 18 m).

No attempt is made to form or sawcut control joints in an RCC pavement. On the basis of 10 years of observation of forest industry pavements, the joints show no serious distress and require minimum maintenance. On a dry-land log sorting yard, wood debris soon fills the joint space, thus acting like a joint filler.

Cold joints in an RCC pavement require special attention. These occur when construction stops at the end of a working shift and a vertical face remains along the final pass of the paver. Once compacted, this face should be cut back with a grader blade into compacted material and left for the night.

When construction starts the following day, the cut-back, hardened face should be sprayed with water immediately ahead of the first pass of the paver. On some projects a cement-water grout has been brushed onto the hardened face. Special care should be taken to ensure adequate compaction along this cold joint. Observation of existing projects in service shows that this area of the pavement is the most prone to deterioration.

The vertical construction joint between passes of the paver as the surface is placed does not require any special bonding treatment. Water spray should be available, however, in the event of severe drying conditions. Delay of edge compaction until the next strip of pavement is placed ensures good density at the longitudinal joint.

Manholes and catchbasins are usually boxed out and later set to grade with regular concrete.

Some engineers have considered the value of offsetting the vertical joint of the top lift by 3 to 4 ft (1 to 1.3 m) when two-layer construction is used. The value of good bond between lifts, so that the total slab acts as a single layer, cannot be overstressed. To leave a strip of base exposed for a day or more to gain the advantage of offset vertical joints in the base and top layers is unwise. It is recommended that (a) all base and surface layers be completed in the same working day and (b) the vertical joint be continuous through both layers.

When RCC is placed over large areas, the direction of paving should be parallel to the shortest dimension. Although this may appear contrary to the natural inclination of the contractor, it does ensure that cold joints left overnight are in the shortest direction.

CURING

RCC requires curing in the same way as regular concrete. Because the water content in the mix is established on the basis of optimum moisture content for maximum density and not the water/cement ratio, there is practically no free water available as a reserve for curing. Therefore water from an external source is vital to strength gain in the first days after placement.

Most projects built to date have used aluminum pipe irrigation systems with sprinkler heads to distribute the water. Although the initial cost may be high, once in place the system can be set up to operate automatically at minimum cost. On a dry-land log sort, water is usually available from a nearby stream or lake and can be obtained by using a small pump. A public water source could supply curing water for a container terminal or bulk storage pavement.

Time of year, daily maximum temperature, wind, and humidity all have a bearing on curing conditions. Cool fall weather with frequent rainfall could reduce the amount of curing needed, whereas hot, dry summer conditions may dictate constant sprinkler operation.

A water truck with spray bar is also an acceptable curing method, provided that the operator is well aware of the need to keep the pavement surface continuously wet.

Membrane-forming concrete curing compound can be used if the cost can be justified. Because of the more open nature of an RCC surface, the curing compound should be applied at the rate of 100 to 150 ft^2/gal (2.0 to 3.0 m^2/L).

The length of cure time is important. Once again the same factors affecting regular concrete apply. A minimum of 5 days' moist curing is recommended under normal summer temperatures. This could be reduced to 4 days in hot weather with full-time wet cure and should be extended to 6 days or longer in cool spring and fall weather conditions. In some cases, when the owner requires access as soon as possible, the cement content of the RCC mix can be increased by 3 to 4 percent to provide early strength gain in the slab. In other cases, maximum loads should be restricted for the first week or two while the slab gains strength.

COST

The aggregate for RCC can have a very wide gradation curve (Figure 1). Many sources can be considered for RCC raw material that would otherwise be unsuitable for regular gravel base course. It has been found that aggregates with as much as 14 percent passing the No. 200 (75-μm) mesh sieve produce compressive strengths of 4,000 psi (27.6 MPa) or more at 28 days.

The acceptability of previously rejected gravel pits also means shorter haul distances, a further cost saving. The reduced need to open new aggregate pits might eliminate the need for environmental impact studies, surely a significant cost benefit.

The total thickness of an RCC pavement is much less than that for a flexible pavement of the same load-carrying capacity. In cut situations this means a saving in excavation cost. In fill locations it means that lower-cost fill material can be placed to within a few inches of the final RCC base grade, thus saving the cost of expensive gravel.

Figure 2 shows a typical cost estimate format that an engineer might follow. The figures are relevant to coastal British Columbia in 1985.

COST ESTIMATE FORMAT

Cost Information:

	$/Ton
Portland cement, Type I	86.00
Aggregate, purchase, haul, crush, stockpile at plant	4.00
Process through central mix plant	2.30
Set up and dismantle cental mix plant (Lump Sum)	20,000.00
Place RCC mix through paver, including compaction	1.50
Haul mix on site	1.00
Cure	0.50

Engineering Information:

Area of the project 8,000 sq. yd.
Traffic and loading data (Supplied by the designer)
Subgrade information (Specific to the project)
RCC slab thickness (assumed values) T = 12 in.
T_1 = 4 in.
T_2 = 8 in.
Portland cement content (by weight of aggregate)
T_1 = 12%
T_2 = 8%
Aggregate unit weight 2 Tons/cu. yd.

* * * * * * * * *

Cost Estimate		$/Ton of Mix
1. Supply aggregate into stockpile at mixing plant site		4.00
2. Portland cement content:		
Top layer: .12(4/12 x 2000) x ($.043) = 3.44		
Base layer: .08(8/12 x 2000) x ($.043) = 4.59		
Total:		8.03
3. Process through mixing plant	2.30	
4. Place, compact, and finish	1.50	
5. Cure pavement (water)	0.50	
Total: (process, place & compact, cure)		4.30
6. Haul mix on site		1.00
7. Set up and dismantle central mix plant: $20,000/(8000 x 12/36) x 2) =		3.75
Subtotal:		21.08
Add: Engineering (5%), Profit (15%)		4.37
Total Cost per Ton		$25.45
Cost per sq. yd.		$16.97

FIGURE 2 Typical cost-estimate format.

RECENT DEVELOPMENTS

RCC Overlay: Honeymoon Bay, British Columbia

In the summer of 1985, 4 acres (1.6 ha) of an existing asphalt dry-land log-sorting yard were overlaid with 11.5 in. (292 mm) of RCC. This was the first use of RCC as an overlay to an existing pavement in Canada.

The original yard was built in 1974 and is located within a few miles of the first RCC log sort at Caycuse, as noted elsewhere. The flexible pavement structure was 4 in. (100 mm) of asphalt on 18 in. (450 mm) of granular base. Plate-bearing tests in the spring of 1985 indicated k-values from 200 pci to less than 100 pci. Numerous failures in the original pavement had been patched with both asphalt and concrete.

The equipment using the yard was the Wagner L90 log stacker.

The gravel aggregate was 3/4 in. (20 mm) crushed material with 8 percent passing the No. 200 mesh sieve. Cementitious content was 12 percent by weight, composed of 10 percent portland cement and 2 percent fly ash. Laboratory testing before the job yielded flexural strengths in the range of 650 psi (4.5 MPa) at 28 days.

The mixing-plant capacity was 500 tons/hr. Because the RCC slab was designed for a thickness of 11.5 in. (292 mm), the contractor planned for two-lift construction. The existing failed asphalt surface was first sprayed with water. The first lift of RCC mix, approximately 6 in. (150 mm) thick, was end dumped onto the old surface and spread with a motor grader. A vibratory steel roller then compacted this base layer. Within a few minutes, the asphalt paver placed the surface layer to final grade and a steel vibratory roller completed the work.

Freeze-Thaw Testing

During 1984 and 1985 the Cold Regions Research and Engineering Laboratory of the U.S. Army Corps of Engineers carried out a program studying the freeze-thaw durability of RCC at their facility in Hanover, New Hampshire. A full report will soon be published by the Corps of Engineers. Until now, severe cold weather experience has been limited to one project in northern British Columbia--the Bullmoose Coal Mine at Tumbler Ridge. The design depth for frost in this area is 8 ft (2.4 m). Most other RCC log yards are in coastal areas where numerous freeze-thaw cycles occur but where winter temperatures are not severe.

Portland International Airport

Early in 1985 the Port of Portland, operators of the Portland, Oregon, International Airport, awarded a contract for construction of 9 acres (3.6 hectares) of parking area for commercial jet aircraft at the Portland Airport. The general contractor was Intertec Contracting Company of Portland, Oregon, and the subcontract for placing the RCC was awarded to Jack Cewe Limited of Vancouver. The Cewe Company has built most of the RCC projects in Canada.

The Portland Airport project was the first use of RCC heavy-duty paving at an airport facility in North America. Thickness of the slab was 14 in. (355 mm). The aggregate was 3/4 in. crushed gravel and the cementitious content was 14 percent, consisting of 10 percent portland cement, Type I, and 4 percent fly ash.

One of the outstanding features of this project was the substantial cost saving (approximately 30 percent) over the conventional flexible pavement that was tendered as an alternative at the time of bidding. The flexible alternative included 6 in. (150 mm) of asphalt pavement on 24 in. (600 mm) of granular base. (See paper by Abrams et al. in this Record.)

Fraser River Harbour Commission

The Fraser River empties into the Strait of Georgia near Vancouver. The course of the river through the interior of the province is such that it continuously picks up thousands of tons of sand and silt, which settle out as the river slows and discharges into the ocean. The river is navigable by ocean vessels for several miles upstream from the Strait of Georgia to the city of New Westminster, a few miles south of Vancouver.

In 1984 the Fraser River Harbour Commission decided to build an experimental entrance road into one of the dock areas. The aggregate was dredged sand, which is continuously removed from the river to keep shipping channels open. If the waste sand could be used successfully as RCC raw material, an extremely economical heavy-duty pavement could be produced.

The test project was on a very small scale, 2,500 tons, which resulted in some problems in consistency of mixing and placing. Furthermore, the work was carried out in November when nighttime temperatures dropped to freezing. Although part of the work incurred some frost damage and required a subsequent asphalt overlay, about half of the project was highly successful and remains in service as an exposed RCC pavement.

Compressive strength of the RCC at 28 days was 4,570 psi (31.5 MPa). Typical shrinkage crack spacing of approximately 50 ft (15 m) occurred. The 10-in. (250-mm) slab was placed with an asphalt paving machine in two layers. The aggregate was a blend of 50 percent natural gravel, 100 percent passing the 1-in. (25-mm) sieve, and 50 percent poorly graded river sand, predominantly in the No. 40 to No. 80 sieve sizes. There was very little minus No. 200 material.

In spite of the poor gradation of the mixture, good compaction was possible when 12 percent portland cement was added to the mix. In appearance the high sand percentage resulted in a very dense sandy surface. Although the pavement has only been exposed to traffic for 12 months, early observations indicate that it is resistant to abrasion and freezing and thawing.

CONCLUSION

Use of RCC for heavy-duty pavement grew from cement-treated aggregate base technology. Increasing the portland cement content of the mixture caused the surface of the pavement to gain abrasion and freeze-thaw resistance and eliminated the need for a protective surface. This "super soil-cement," which is now called roller-compacted concrete, is serving the needs of the forest industry in British Columbia by providing an economical, low-maintenance pavement that reduces equipment breakdowns, increases operating speeds, and generally improves the economics of harvesting trees.

In the last 3 years RCC use has expanded to military installations, container yards, airports, and heavy equipment storage. By the end of 1985 it was estimated that the equivalent of over 300 acres (120 hectares) was in place in the United States and Canada, with approximately 70 percent of that area having being built in the last 18 months of that period. Projects totaling over 100 acres are now in various stages of design and construction.

Enthusiasm must be tempered with caution. RCC is still an industrial pavement to be used for slow-speed traffic. The surface texture of RCC is more open than the dense, closed surface that most engineers associate with normal portland cement concrete. What some may consider to be a pavement with inferior appearance is frequently less expensive than other alternatives.

Three areas need further study and development:

1. A simple and reliable field method is needed for determining the quality of RCC in place. It may be that some of the present nondestructive methods of evaluating normal concrete can be applied to RCC. Work is in progress on a modified rebound hammer technique that may be applicable to RCC testing.
2. A method of sampling the fresh RCC mixture for laboratory testing is needed. At this time the method for sampling and testing zero-slump concrete is frequently used. Other engineers use field-molded beam specimens. A program to compare field-molded samples by various methods with beams or cores obtained from RCC pavements in place could provide valuable data on field control.
3. A placing method is needed that will ensure attainment of the specified density when RCC is placed in layers thicker than 10 in. There is lack of agreement among engineers about the ability to adequately compact RCC in single-layer construction of 14 to 16 in. (355 to 400 mm). New placing equipment now being introduced may provide the solution. Further studies to answer the question of density versus layer thickness are needed.

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Construction of Roller-Compacted Concrete Pavements

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ABSTRACT

Roller-compacted concrete pavement (RCCP) is the product of a relatively new concrete paving technology in which a zero-slump portland cement concrete mixture is spread with modified asphalt pavers and compacted with vibratory and rubber-tired rollers. Because of the ease and simplicity of this construction method, savings of one-third or more of the cost of conventional concrete pavement construction are possible. RCCP construction procedures are described, from subgrade and base-course preparation to curing procedures. Equipment needed for mixing, transporting, placing, compacting, and curing RCCP is detailed. Procedures for fresh-joint, cold-joint, and horizontal-joint construction are recommended, and various curing procedures and their results are discussed. Quality control and quality assurance practices, including checking the in situ density of RCCP with a nuclear density gauge and the smoothness with a 10-ft straightedge, are outlined. Applications of RCCP and potential construction problems are described. Advantages and disadvantages of using RCCP instead of conventional concrete pavement are discussed.

Roller-compacted concrete pavement (RCCP) is the product of a new concrete paving technology in which a zero-slump portland cement concrete mixture is spread with a modified asphalt paver and compacted with a vibratory roller. This construction method, which is very similar to asphaltic concrete paving, has the potential for savings of one-third or more of the cost of conventional (slip-form or fixed-form) concrete paving construction, therefore combining the more attractive features of concrete and asphalt paving.

The idea of placing and compacting a zero-slump concrete mixture in lifts originated in the early 1970s for use in mass concrete structures. Almost 0.5 million yd^3 of roller-compacted concrete (RCC) was placed in both the Tarbela Dam, Pakistan, in 1975 and in the Willow Creek Dam in Hepner, Oregon, in 1982 (1). The Willow Creek Dam was constructed at about one-third the cost of a conventional concrete dam (2), illustrating significantly the economic advantages of using RCC.

The mass concrete construction techniques of the early 1970s evolved quite naturally to include pavement structures. Two test sections of RCCP were built at the U.S. Army Engineer Waterways Experiment Station (WES) in Vicksburg, Mississippi, in the mid-1970s--one to study the compaction of zero-slump concrete with various vibratory rollers (3) and the other to study the use of marginal material for RCCP aggregates (4). At about the same time, the Canadians were experimenting with RCCP in the timber industry as a low-cost alternative for constructing log-sorting and storage yards, which frequently handle axle loads in excess of 100 tons (5). These pavements have performed exceptionally well under harsh loading and environmental conditions, and by 1983 the Canadians had built over 28 acres of storage yards and an 11-mi haul road with RCCP.

The first major use of RCCP by the U.S. Army Corps of Engineers was at Fort Hood, Texas, in July 1984. A 20,000-yd^2 parking area for M1 tanks and armored personnel carriers was built in 11 days at a savings of 15 percent over the cost of reinforced-concrete pavement. In November 1984 a 700-ft-long road was built at Fort Lewis, Washington, for tracked and rubber-tired military vehicles. Railroad shipping terminals in Houston, Texas, and Tacoma, Washington, were built recently to thicknesses up to 18 in. to accommodate heavy loadings applied by container-handling vehicles used to unload freight from rail cars.

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SUBGRADE AND BASE-COURSE PREPARATION

The subgrade and base-course requirements are basically the same for RCCP as they are for conventional concrete pavements. In areas where the pavement or base course might be subjected to seasonal frost action, the base course should adequately drain any water that infiltrates through the pavement or subgrade. The base course should also provide sufficient support so that the RCCP is fully consolidated through its entire thickness upon compaction.

TEST SECTION

Construction of a test section allows the contractor to demonstrate his ability to mix, haul, place, compact, and cure RCCP before the major construction takes place. The test section is usually constructed at least 1 month before the start of the major construction, so that samples for strength testing may be taken directly from the test section.

The test section also serves several other important functions. The rolling pattern should be established, so that optimum density is achieved with a minimal number of passes. Nuclear density gauge readings should be correlated with densities of cores taken from the test section and measured in the laboratory. The nuclear density gauge operated in the direct transmission mode has been reported to be as accurate as the sand cone test and core testing in determining the density of cement-bound materials (6). However, calibration of the nuclear gauge during test section construction ensures that an accurate correlation exists during the major construction.

The test section should contain both longitudinal and transverse cold joints and fresh joints, because these are the most critical areas in obtaining surface smoothness and adequate density. A suggested minimum size is three paving lanes wide, each 150 ft long, with one and one-half lanes placed the first day and the rest placed the next day (Figure 1).

Cores and beams should be taken from the test section after 28 days to determine the correlation between the flexural strength and splitting tensile strength of the RCCP. This eliminates the need for the time-consuming and expensive practice of sawing beams during construction to check the design flexural strength.

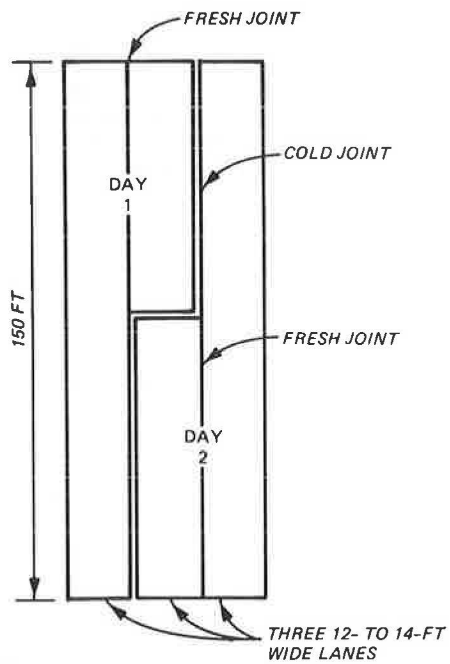

FIGURE 1 Test section.

BATCHING, MIXING, AND TRANSPORTING

RCC needs a vigorous mixing action to disperse the relatively small amount of water evenly throughout the matrix. This action has been best achieved by using a twin-shaft pugmill mixer, commonly used in asphaltic-concrete mixing. Batching of the concrete may be accomplished successfully in either a continuous-mixing or a weigh-batch plant. The continuous-mixing plant is more advantageous because it is easier to transport to the site, takes less time to set up, and has a greater output capacity than the weigh-batch plant. The weigh-batch plant allows more accurate control over the proportions of material in each batch, but generally does not have enough output capacity for large paving jobs. The output of the plant should be such that the smooth, continuous operation of the paver or pavers is not interrupted, and for all but the smallest jobs (1,000 yd^2 or smaller), the capacity of the plant should ideally be no less than 250 tons/hr. The output (or production) of the plant should not be greater than the laydown capacity of the pavers nor greater than the rolling capacity of the rollers. The plant should be located as close as possible to the paving site to minimize the haul time between the batch plant and the pavers.

RCC is typically hauled from the mixer to the pavers in dump trucks. These trucks should be equipped with protective covers to guard against adverse environmental effects on the RCC such as rain or extreme cold or heat. The truck should dump the concrete directly into the paver hopper.

PLACING

For most pavement applications, RCCP is placed with an asphalt paver or similar equipment. Automatic grade-control devices, such as a traveling ski or electronic string line, allow the RCCP to be spread at an accurate grade and acceptable smoothness. A paver having a vibratory screed or one equipped with a tamping bar will provide a satisfactory surface texture and some initial compaction when the RCCP is placed. Necessary adjustments on the paver to handle the RCC include enlarging the feeding gates between the feed hopper and screed to accommodate the large volume of stiff material moving through the paver and adjusting the spreading screws in front of the screed to ensure that the RCC is spread uniformly across the width of the paving lane.

The fresh concrete should be placed and compacted within 45 min after water has been added to the batch. When adjacent lanes are paved, the new concrete should be placed within 90 min of placing the first lane (forming a fresh joint), unless procedures for cold-joint construction are followed (see section entitled Joints). When rectangular sections are paved, paving in the short direction will minimize the length and number of cold longitudinal and transverse joints. Two or more pavers operating in echelon may reduce the number of cold joints by one-half or greater, and are especially suitable in road construction where the entire width of the road can be placed at one time.

COMPACTION

RCCP is best compacted with a dual-drum (10-ton) vibratory roller making four or more passes over the surface to achieve the design density (one back-and-forth motion is two passes). To achieve a higher-quality pavement, the primary compaction should be followed with two or more passes of a 20-ton rubber-tired roller (90 psi minimum tire pressure) to close up any surface voids or cracks. A dual-drum static (nonvibratory) roller may be required to remove any roller marks left by the vibratory or rubber-tired roller.

Ideally, the consistency of the RCCP when placed should be such that it may be compacted immediately after placement without undue displacement of the RCCP. However, no more than 10 min should pass between placing and the beginning of the rolling procedure. The rolling should be completed within 45 min of the addition of water at the mixing plant. A good indication that the RCCP is ready for compaction is found by making one or two static passes on the RCCP within 1 ft of the edge of the lane before vibrating begins and observing the material during the two passes to ensure that undue displacement does not occur. If the RCCP is too wet or too dry for compaction upon placing, the water content should be adjusted at the plant to correct this. Only minor changes in water content from the design mix should be made; otherwise, a new mix design may be needed. With practice, the roller operator should be able to tell whether the consistency of the RCCP is satisfactory for compaction.

ROLLING PATTERN

The rolling pattern should be established so that the specified density is achieved with a minimum number of roller passes. After making the static passes for a consistency check, the vibratory roller should make four vibratory passes on the RCCP using the following pattern, which has yielded satisfactory results in earlier projects: Two passes (one back-and-forth motion with the roller) should be made on the exterior edge of the first paving lane (the perimeter of the parking area or the edge of a road) so that the rolling wheel extends over the edge of the pavement 1 to 2 in. This is done to confine the RCCP and prevent excessive lateral displacement of the concrete upon further rolling. The roller should then shift to within 12 to 18 in. of the interior edge and make two passes. This will leave an uncompacted edge to set the height of the screed for an adjacent lane and allows both lanes of the fresh joint to be compacted simultaneously. Any remaining uncompacted material in the center of the lane should be compacted with two passes of the roller. The pattern should be repeated once to make a total of four passes on the lane (or more if the specified density is not achieved) (see Figure 2). If the interior edge will be used to form a cold joint, it should be rolled exactly as the exterior edge was rolled, taking care to maintain a level surface at the joint and not round the edge.

When the adjacent lane is placed, two passes should be made about 12 to 18 in. from the outer edge of the lane (again to confine the concrete), followed by two passes on the fresh joint. The first two passes should extend 1 to 2 in. over the outer edge of this adjacent lane if the lane will form an outer edge of the completed pavement. Any remaining uncompacted material in the lane should be rolled with two passes of the roller. This pattern should be repeated to make a total of four passes over the RCCP. Additional passes may be necessary along the fresh joint to ensure smoothness and density across the joint (see Figure 3).

When the end of a lane is reached, the roller should roll off the end of the lane, rounding off the end in the process. This rounded end should be trimmed to form a vertical face through the entire depth of the pavement. An alternative method involves confining the uncompacted end of the lane with a crosstie or beam anchored to the base course, thereby forming a vertical face at the end of the lane after compaction.

During the course of the vibratory compaction, the roller should never stop on the pavement in the vibratory mode. Instead, the vibrator should be turned on only after the roller is in motion and should be turned off several feet before the roller stops moving. The stopping point of successive rolling passes should be staggered to avoid forming a depression in the RCCP surface. The roller should be operated at the speed, amplitude, and frequency to achieve optimum compaction, which will probably occur at a high amplitude and low frequency (because of the thick lifts) and at a speed not exceeding 2 mph.

The vibratory compaction should be followed immediately with two or more passes of the rubber-tired roller so that the surface voids and fissures close to form a tight surface texture. This rolling may be followed by a light dual-drum roller to remove any roller marks on the surface, but this may not be necessary. All exposed surfaces of the RCCP must be kept moist with a light water spray after the rolling

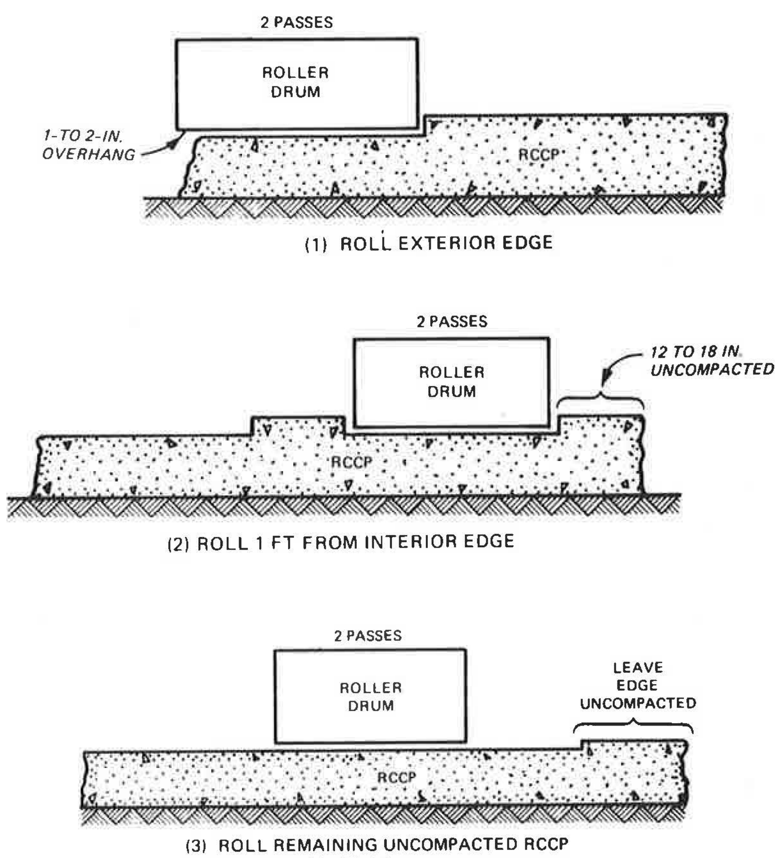

FIGURE 2 Compaction of first paving lane.

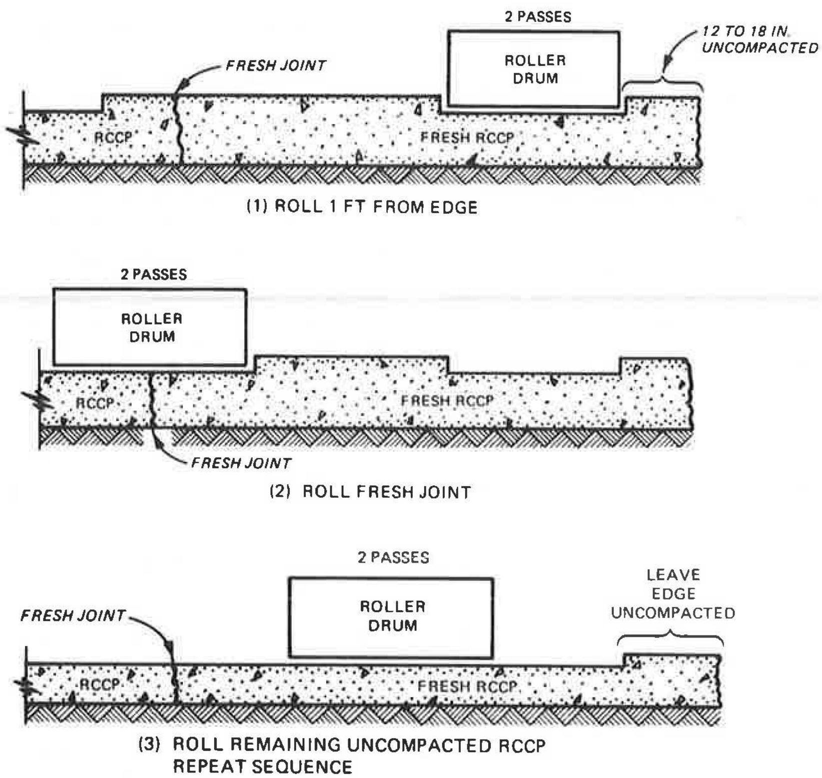

FIGURE 3 Construction of fresh joint.

process until the curing procedure has been completed.

JOINTS

A cold joint is formed between two adjacent lanes of RCCP when the first lane has hardened to such an extent that the uncompacted edge cannot be consolidated with the fresher second lane. This happens when there is a delay between placement of adjacent lanes, such as at the end of the day's construction. This hardening may take from one to several hours, depending on properties of the concrete and environmental conditions. Nevertheless, the adjacent lane should be placed against the first lane within 90 min or be considered a cold joint.

Before fresh concrete is placed against hardened in-place pavement to form a longitudinal cold joint, the edge of the in-place pavement must be trimmed back to sound concrete to form a vertical face along the edge. This vertical face should be dampened before placement of the fresh lane begins. The height of the screed should be set to an elevation approximately 25 percent higher than the desired thickness of the compacted concrete. The screed should overlap the edge of the hardened concrete lane 2 or 3 in. The excess fresh concrete should be pushed back to the edge of the fresh concrete lane with rakes or lutes and rounded off so that a minimal amount of fresh material is left on the surface of the hardened concrete. The loose material should not be broadcast over the area to be compacted because this may leave a rough surface texture after rolling. The edge of the fresh lane adjacent to the hardened concrete should be rolled first in the static mode with about 1 ft of the roller on the fresh concrete to form a smooth longitudinal joint (see Figure 4).

Transverse cold joints are constructed in a similar manner. After the rounded-off edge has been cut back and the vertical face has been wetted, the paver is backed into place and the screed set to the proper elevation by using shims sitting on top of the hardened concrete. The excess material should be pushed back, as mentioned before, and a static pass made in the transverse direction across the first foot of the freshly placed lane. Care should be exercised in rolling the joint to ensure a smooth surface across the joint.

The sawing of contraction joints in RCCP has proven to be unnecessary in past projects. Cracks were allowed to form naturally in all of the Canadian-built RCCP, and virtually no distress has been observed at the cracks. These pavements have endured over 7 years of very heavy loads and numerous freeze-thaw cycles. Attempts to saw joints at Fort Hood and Fort Lewis produced a ragged edge along the saw cut where pieces of cement paste and aggregate were knocked out by the saw blade.

The stiff consistency of RCCP does not lend itself to the application of load-transfer devices, such as dowels or keyed joints, although dowels were used in cold-joint construction at Fort Stewart. There the dowels were driven into the RCCP before the final set, and the dowels in the adjacent fresh lane were carefully worked around by hand. Until an efficient method is developed to insert and align dowels properly in RCCP, the use of dowels should be limited.

In two-lift construction, the cold transverse and longitudinal joints should be carefully aligned in the upper and lower lifts to form a uniform, vertical face through the depth of the pavement along the joint. If the edge of the upper lift is not even with the edge of the lower lift, the lower lift should be cut back even with the edge of the upper lift (see Figure 5).

CURING

Because of the lower water content used in an RCCP mixture, a combination of moist curing and membrane

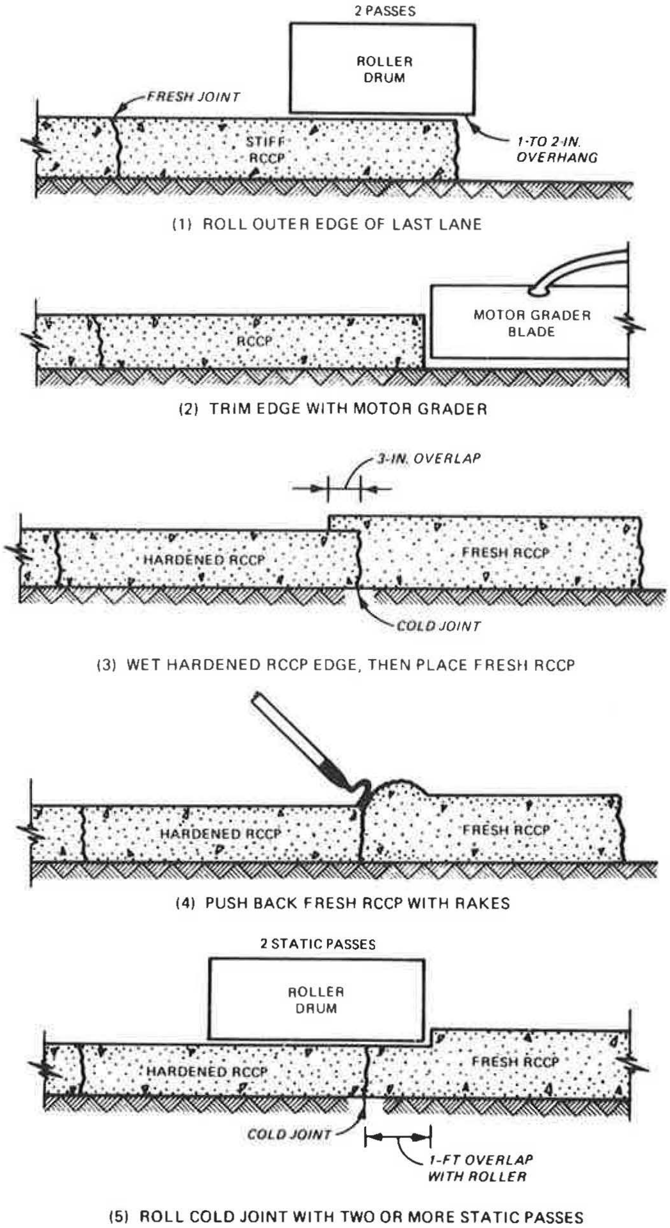

FIGURE 4 Construction of cold joint.

curing is recommended to prevent drying and scaling of the RCCP surface. The pavement surface should be kept continuously moist after final rolling for at least 24 hr by means of a water spray truck, sprinkler (fog spray) system, or wet burlap or cotton mat covering. If burlap or mats are used, they should be thoroughly wetted, placed on the RCCP so that the entire surface and exposed edges are covered, and kept continuously wet. After the initial moist curing period, the RCCP should be cured until it is at least 7 days old by one of the following methods: water-spray curing, burlap or cotton mat covering, or membrane-forming curing material. The curing material may be a white-pigmented membrane curing compound or an asphalt emulsion. The curing compound or emulsion must form a continuous void-free membrane and should be maintained in that condition throughout the curing period. An irrigation sprinkler system has been used to cure RCCP in Canada and at Fort Lewis, but caution should be exercised so that the fines in the surface of the RCCP are not washed away by excessive spraying.

Continuous moist curing of the RCCP for at least 7 days should be considered if frost resistance is of concern. Preliminary results of laboratory freezing and thawing tests conducted at WES and the North Pacific Division Laboratory indicate that RCCP with a sufficiently low water-cement ratio that has been moist-cured for an extended period tends to be more frost-resistant.

All vehicular traffic should be kept off the RCCP for at least 14 days. If absolutely necessary, a water-spraying truck and a membrane-spraying truck may be driven onto the RCCP before that age, but this practice should be kept to a minimum.

QUALITY CONTROL AND ASSURANCE

Quality control and quality assurance consist of testing materials going into the concrete, checking the plant calibration regularly, measuring the in-place density of the RCCP with a nuclear density gauge, fabricating cylinders and beams to model in-

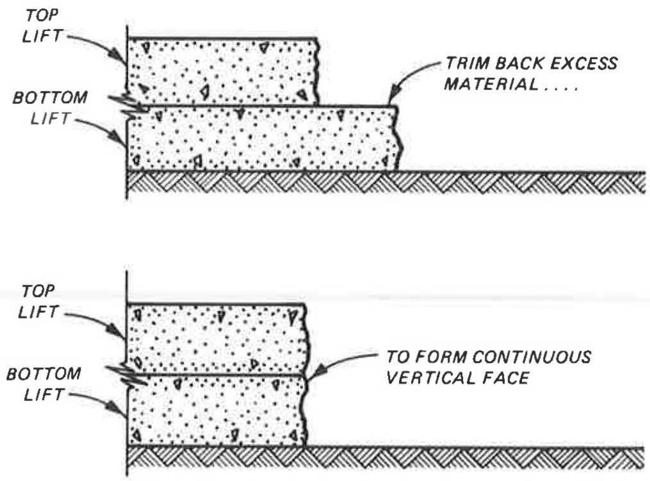

FIGURE 5 Two-lift cold joint.

place density and strength, checking the smoothness of the finished RCCP with a straightedge, and taking core samples from the RCCP for measurement of density, tensile splitting strength, and thickness.

Moisture content of the fine and coarse aggregates should be determined daily as necessary and appropriate changes made in the amount of mixing water. The calibration of the plant should be checked daily before production begins. The samples used for calibration should be taken from the conveyor belt between the cement and fly-ash silos and the pugmill.

Washed gradation tests should normally be performed on the combined aggregates three times a day: in the morning, at midday, and in the afternoon. The samples should be taken from the conveyor before the cement or fly ash is added to the combined aggregates. The amount of material passing the No. 100 sieve should be determined during this analysis. After each gradation test, a washout test (ASTM C 685) may be performed on the combined dry ingredients on samples taken from the conveyor belt between the cement and fly-ash hoppers and the pugmill. By washing the dry ingredients over the No. 4 and No. 100 sieves and weighing the material in each size category, the approximate proportions of coarse aggregate, fine aggregate, and cement and fly ash combined may be determined and checked against predetermined limits.

Field density tests should be performed on the RCCP with a nuclear density gauge operated in the direct transmission mode according to ASTM D 2922. At least one field reading should be taken for every 100 ft of each paving lane. The readings should be taken as closely behind the rolling operation as possible. The readings should be adjusted by using the correlation determined in the test section construction and checked against a specified density. Areas that indicate a deficient density should be rolled until the specified density has been achieved.

Cylinders and beams should be fabricated by filling cylinder molds in two layers and beam molds in a single layer and consolidating each layer of concrete in the field on a vibrating table. Although there is no standardized method for fabricating RCC cylinders or beams, the method described in ASTM C 192 has been used successfully, with and without the use of a surcharge weight to aid in consolidation of the mixture. Whatever method is used should be performed consistently throughout the project. Four beams (one group) should be fabricated during each shift of construction, two to be tested at 14 days and two at 28 days. The beams should be tested for flexural strength according to ASTM C 78. Six cylinders (one group) should be fabricated for every 300 yd^3 of RCC placed, with one group coming from the same batch of RCC as that used in the beams. Two cylinders should be tested each at 7, 14, and 28 days. The cylinders should be tested for splitting tensile strength according to ASTM C 496. Cylinders of RCC used with high fly-ash contents or for airfield pavements may be tested at 90 days.

Cores should be taken from the RCCP when the pavement is 7 days old. One core should be taken at every fifth nuclear gauge density test site within 2 to 5 ft of the test hole. The density and thickness of the core should be measured, and the core should be field cured under conditions similar to the RCCP curing conditions. The cores should be tested for splitting tensile strength (ASTM C 496) when they are 28 days old.

The finished surface of the RCCP should not vary more than 3/8 in. from the testing edge of a 10-ft straightedge. Smoothness should be checked as closely behind the finish roller as possible, and any excessive variations in the surface should be corrected with the finish roller. Particular attention should be paid to smoothness across fresh and cold joints, because this is usually a critical area for surface variations. A skilled vibratory roller operator is essential in minimizing smoothness problems. The final surface texture of the RCCP should resemble that of an asphaltic-concrete pavement surface.

Inspections are vital in the quality control operations. At least one inspector should be stationed at the mixing plant and one at the job site to ensure that a quality pavement is being built.

At the mixing plant, the inspector should check mixing times occasionally and spot-check the consistency and appearance of the mix coming out of the plant. He should also coordinate the aggregate moisture content tests, the gradation tests, calibration of the plant, and washout tests to see that they are performed properly and with the right frequency.

At the job site, the inspector should make sure that the base course and cold joints are moistened before the RCC is placed against them and that the RCC is placed and compacted within the proper time limitations. The paver operation should be checked to ensure that proper grade control is continuously maintained and that no gaps or discontinuities are left in the pavement before rolling. The inspector should make sure that the roller begins compaction at the proper time and that the proper rolling pattern and number of passes are used. Adequate smoothness across joints should be assured as well as the tightness of the surface texture after final rolling. He should spot-check the final compacted thickness of the RCCP on occasion and make corrections if appropriate. He should make sure that the curing procedures are implemented as specified. The inspector should make sure that all exposed surfaces of the RCCP are kept moist at all times and that the curing compound, if used, is applied properly and in a continuous fashion. He should also coordinate the nuclear gauge density test, the coring and sample fabrication procedures, and the surface smoothness test to see that they are performed properly and at the required frequency.

ADVANTAGES AND DISADVANTAGES OF USING RCCP

The primary advantage of using RCCP versus conventional concrete pavement is the relatively low construction cost; savings of 15 and 30 percent have been realized at Fort Hood and Tacoma, respectively. As contractors become more confident in their ability to place RCCP efficiently, the total cost of RCCP construction should drop even more. No elaborate paving trains or forms are needed to place RCCP, and only a small construction crew is used. This may open areas for concrete pavement applications that may have been deemed impractical or uneconomical in the past. Some savings in material costs may be realized should fly ash be substituted in RCCP for a portion of the cement (usually the costliest ingredient) as it has been in previous jobs. Some savings in aggregate processing may be achieved if a greater percentage of nondeleterious fines is allowed in RCCP to enhance workability or finishability.

As could be expected of any pavement whose construction technique is in its infancy, RCCP has its drawbacks. The degree of surface smoothness achieved to date has been less than that for conventional concrete pavement, limiting its present applications to parking areas, tank trails, or secondary roads. Also, some raveling of surface material may occur along cold joints and occasional surface areas as a result of improper construction or curing techniques. The freeze-thaw durability of RCCP has not been consistently satisfactory in laboratory tests conducted at WES. Fortunately, these problems can be controlled and corrected if proper mixing, construction, and curing techniques are known and implemented by all of the engineers, contractors, and quality control inspectors involved.

CONCLUSIONS

RCCP combines relatively simple asphaltic pavement construction techniques with the strength and durability of portland-cement concrete to provide an economical alternative to conventional concrete pavements. Because the quality of RCCP depends largely on the timeliness and proper execution of the placing, rolling, and quality-control operations, adequate education of all contractors, engineers, and quality-control personnel is essential in producing a satisfactory pavement. Though a relatively new paving technique, RCCP is rapidly gaining a place in the concrete paving market.

ACKNOWLEDGMENT

Information for this paper is taken largely from A Guide for the Design and Construction of Roller-Compacted Concrete Pavement, by David W. Pittman and Steven A. Ragan, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi. The paper will be available as an Engineering Technical Letter supplement in the near future.

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Roller-Compacted Concrete Pavement at Portland International Airport

JORY M. ABRAMS, JERRY L. JACKSHA, ROBERT L. NORTON, and DAVID J. IRVINE

ABSTRACT

Presented are the owner's and the consultant's perspectives on the development, design, and evaluation of key issues involved with construction of a roller-compacted concrete (RCC) pavement at Portland International Airport, Portland, Oregon. RCC has been used in the past for pavements that support heavy wheel loads, but this parking apron is the first use of it as a surface pavement for commercial jet aircraft in the United States. The loading conditions and other factors that led to the selection of RCC as an alternative to conventional asphalt-concrete (AC) pavement are discussed. Federal Aviation Administration methodology was used to develop pavement sections for both RCC and AC pavements. Specifications were developed to improve surface tolerances, smoothness, and joint control, and a method was devised to test field density. Bids were accepted for both alternatives and the six low bidders offered the RCC option at a lower cost than the AC option. The lowest RCC bid was 32 percent under the lowest AC bid.

The Port of Portland, Oregon, retained CH2M HILL to design an 8-acre aircraft parking apron at Portland International airport (PDX) for passenger jets. Portland cement concrete (PCC), roller-compacted concrete (RCC), and asphalt concrete (AC) were studied as pavement alternatives. Plans and specifications were prepared and alternative bids received for RCC and AC pavements. Port of Portland officials selected the RCC alternative and awarded the contract in May 1985; paving began in August 1985. In this paper the evaluation process, the design of the RCC and AC pavements, bid results, mix design, and key construction issues considered during the design are outlined.

RCC is a material with a low water/cement ratio. It has been used as pavement for log-sorting yards and shipping facilities and in dam construction. When RCC is used as a pavement, the design and the curing process are similar to those of conventional PCC, but the mixing and placing procedures are similar to those of a cement-treated base (CTB). Conventional asphalt paving equipment with steel drum vibratory rollers for compaction has been most often used for pavement applications. The trend, however, is toward pavers with tamping bars for precompaction of the RCC. These pavers may eliminate the need for compaction rolling and also solve much of the shoving problem that occurs when rollers make their initial pass on a thick lift of uncompacted RCC. The maximum size aggregate for RCC pavements should be 3/4 in., with typical cement content values of 12 percent and a water/cement ratio in the range of 0.3 to 0.4.

PORT OF PORTLAND'S REQUIREMENTS

To support a noise abatement program at Portland International Airport, a navigational aid system was recently installed by the Port of Portland. This new installation created a need to relocate the itinerant-aircraft parking area away from the new antenna system. Personnel at the Port were aware of several high-load pavements (log-sorting yards and port shipping facilities) that had been built in British Columbia using RCC and they became interested in the possible use of this material for the new parking area. It was decided that further investigation of RCC during the preliminary design phase was warranted. After that investigation, it was decided to proceed with RCC as an alternative to AC pavement.

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RCC EVALUATION PROCESS

The initial RCC evaluation consisted of determining the feasibility of using RCC as a surface pavement material and developing the most up-to-date criteria for its use. This initial evaluation was based on research of previous projects, review of existing literature, discussions of RCC applications with experienced engineers and contractors, viewing of a test section placed by the U.S. Army Corps of Engineers at Fort Lewis, Washington, and the engineering experience and judgment of the design team.

CH2M HILL's evaluation indicated that the state of technology had progressed sufficiently to merit an RCC alternative design. The advantages of RCC for this particular application were its better resistivity to chemical attack from hydraulic oils and fuels, better long-term durability and therefore lower maintenance costs, negligible rutting or creep problems under long-term heavy loading, and a recent history of competitive costs.

On previous projects, a variety of natural and processed aggregates have been used for RCC. The two primary sources of processed aggregate have been concrete aggregate, consisting of gravel and sand mixtures, and AC aggregate, consisting of crushed material. The ability to use local sources of aggregate by modifying the mix design has been one of the main advantages of using RCC.

The Portland metropolitan area has an abundant supply of processed aggregate and some sources of naturally occurring gravels. Availability of aggregates was confirmed by contacting local suppliers to discuss gradation limits, availability, production, and costs. The preliminary gradation chosen was an Oregon Department of Transportation (ODOT) 3/4-in. minus crushed aggregate.

A processed AC aggregate was selected because it is less susceptible to segregation during placement than rounded or subrounded gravel. In addition, discussions with the Corps of Engineers indicated that RCC has a higher flexural strength when angular rock is used. It was thought that AC aggregate would give a better surface finish, and there were enough local suppliers of this material to ensure that costs would not be prohibitive. Natural gravels were not considered because of the availability of the crushed processed AC aggregate and because there is the question of material uniformity in most natural sources.

An important task was to evaluate whether local contractors were available and receptive to use of RCC. Numerous northwestern contractors were contacted to determine their experience with RCC or CTB. CTB is similar to RCC in that the cement and aggregate are often mixed in a pugmill or batch plant, placed in thick lifts, and compacted with vibratory rollers. For this application, the main difference between CTB and RCC was that the RCC surface required much tighter elevation tolerances. In addition, to obtain the high flexural strengths on which the design was based, the RCC mix had to be more tightly controlled in terms of amounts of aggregate, water, cement, and pozzolan. Transverse and longitudinal joints as well as the curing process were also critical for placing RCC as a finished surface pavement.

There are several contractors in the area with CTB experience, and a few had RCC experience. It was found that many asphalt-paving contractors were interested in learning about RCC and in bidding on the project. Generally, they were receptive to use of RCC because the equipment required and the placement techniques used were similar to those of AC or CTB.

DESIGN CRITERIA

Field Investigation

A field investigation was conducted by CH2M HILL and Port of Portland personnel in December 1984. This investigation included surveying, soils sampling, and testing. Soils investigation included the excavation of test pits with a backhoe. Materials encountered were tested for density, moisture content, and California bearing ratio (CBR). In general, the test pits showed that there were about 2 1/2 in. of AC pavement above 4 to 6 in. of crushed aggregate base course. The existing subgrade consisted of clean sand known locally as Columbia River dredge sand, having a CBR value of 20 percent. This sand was located as near the surface as 0.75 ft and as deep as 7.0 ft (the maximum pit depth). Layers of silty sand with a CBR of 5 percent were also located in several of the backhoe pits. This silty sand was as near the surface as 1.0 ft and as deep as 6.0 ft. This weaker silty sand was used for the design subgrade.

Aircraft Operations

Forecasts of aircraft operations for the parking apron were prepared by the Port of Portland for a 20-year design life. The aircraft predicted to use the facility and their respective weights and operations are summarized in Table 1.

TABLE 1 Forecast of Aircraft Operations for Port of Portland North Side Remote Parking

Aircraft	Aircraft Weight (lb)	Predicted Operations per Year
B747	564,000	8
DC10-10	363,500	22
A300	281,000	4
B767	270,000	22
DC8-50	240,000	8
B757	198,000	24
B727	154,500	184
DC9-50	110,000	26
DC9-80	128,000	6
B737	103,000	64

AC Pavement Design

The AC pavement design was based on the methodology provided in Federal Aviation Administration (FAA) Advisory Circular 150/5320-6C (1). On the basis of the aircraft weights and operations data in Table 1, the design aircraft chosen for the AC pavement was a DC-10. The recommended pavement section from this analysis was 5 in. of AC, 8 in. of bituminous base, 4 in. of aggregate base, and 19 in. of subbase, for a total pavement section thickness of 36 in. (see Figure 1). The AC pavement section was verified by using a computerized multilayered systems analysis.

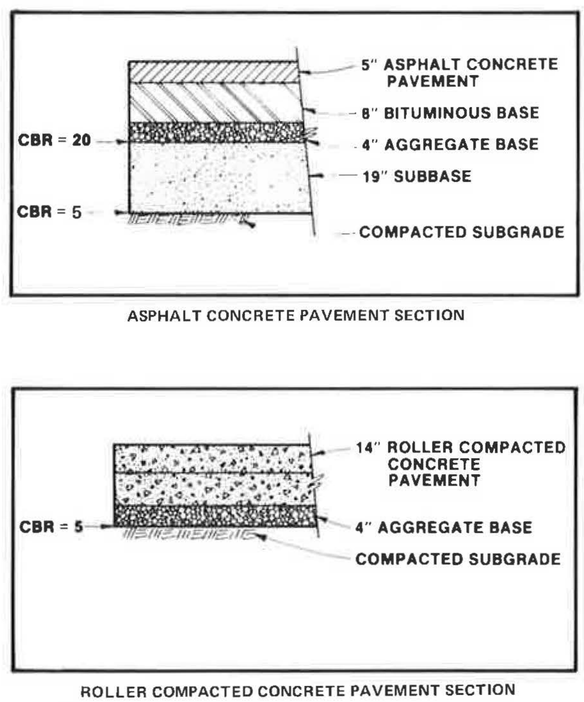

FIGURE 1 Pavement sections.

RCC Pavement Design

The design procedures used to determine thickness requirements of RCC pavement for this project were also based on FAA's Advisory Circular 150/5320-6C (1) and are similar to those normally used for PCC pavements. The design aircraft for RCC was determined to be a Boeing 727 with a total gross weight of 154,500 lb instead of the DC-10 used for AC.

The critical aircraft for a flexible pavement design is not necessarily the same as that for a rigid pavement design because of inherent differences in the performance of rigid versus flexible pavements subjected to aircraft load. The pavements designed for this project were based on the loading conditions presented in the section entitled Aircraft Operations and thus can be considered equivalent.

In accordance with the referenced advisory circular, a subgrade modulus value K was estimated for the subgrade by converting field CBR values and analyzing known K-values for soils of this type. A K-value of 100 pci was used for the silty sand subgrade. The flexural strength for RCC pavement was assumed to be 800 psi. With these criteria, it was determined that an RCC pavement section thickness of 13 in. was required to support the aircraft loading. For pavements supporting aircraft weighing over 100,000 lb, the FAA recommends the use of a stabilized subbase. For this design, 4 in. of stabilized subbase would have reduced the RCC pavement thickness to 12 in. As a cost-saving measure, it was decided to recycle the existing asphalt concrete and base rock on the site to provide a working platform and subbase for the RCC. Because this recycled material does not qualify as a stabilized subbase, the total RCC pavement section thickness was left at 13 in.

The design strength was later modified during the mix design to 700 psi, on the basis of the 28-day beam tests, causing the thickness to be increased by 1 in., to 14 in. (see Figure 1). The RCC pavement thickness was verified by using the Portland Cement Association's computerized concrete pavement design program (2).

KEY CONSTRUCTION ISSUES

To achieve FAA's recommended surface tolerances and texture (3), attention was focused on five areas:

1. Materials,
2. Mix design,
3. Construction equipment,
4. Paving techniques and paving details, and
5. Field density control.

Materials

Aggregate

For this particular pavement, the locally available State of Oregon AC aggregate was chosen. This aggregate gradation was modified slightly by adding intermediate screens and by increasing the percent passing the No. 200. Some of the advantages of using AC aggregate are as follows:

- It is less susceptible to segregation during placement as compared with rounded or subrounded gravel and coarse concrete aggregate;
- The angularity of the particles has generally resulted in higher flexural strengths than those obtained with rounded or subrounded gravel;
- It produces a surface similar to that of conventional asphalt concrete, which was desirable for this pavement application; and
- There are currently 10 to 15 local suppliers.

Portland Cement

In accordance with ASTM C150, the portland cement used for this particular mix is a Type I cement. The specifications, however, also allowed Type II and Type I-II cements.

Pozzolan

The pozzolan for the mix was an ASTM C618, Class C or Class F. Pozzolan Class F was the preferred material because of its uniform chemical composition. It is supplied from Centralia, Washington.

Mix Design

The RCC mix design is very sensitive to aggregate gradation, and it was not feasible to specify a particular aggregate source in the bid documents. Therefore, to assist contractors in preparing a bid, a preliminary mix design was included in the bid documents, as follows:

Material	Weight per Cubic Yard (lb)
Cement	360
Pozzolan	150
Water	165
Aggregate	3,510

After the low bidder designated an aggregate source, a final mix design was completed that used actual aggregate material being stockpiled for the RCC. The mix design method used was that outlined in the Corps of Engineers Manual EM 1110-2-2006 (4). The materials breakdown by weight per cubic yard is as follows:

Material	Weight per Cubic Yard (lb)
Cement (Type I)	488
Pozzolan (Centralia Class F)	119
Water	260
Aggregate	3,250

Although properties of the aggregate selected by the contractor were still within the range specified in contract documents, they differed from those used to develop the preliminary mix design. The average gradation of the aggregate is shown in Figure 2.

The final design flexural strength was 700 psi at 90 days, which would require at least 600 psi at 28 days. Given the project schedule, the flexural strength of the mix design was confirmed by using 28-day beam tests. Beams were made according to ASTM C192-81, with some modifications in the method of consolidating the RCC in the molds. An average of five beam tests at 28 days produced a flexural strength of 710 psi. Some additional strength is anticipated between 28 and 90 days. It is planned to cut beams in the pavement to confirm the flexural strength at 90 days.

Construction Equipment

Mixing Plant

The mixing plant specified is a central type having a stationary, twin-shaft, pugmill-type mixer. The plant is to be either a weigh-batch type or a continuous-mix type. The specified minimum plant output is a manufacturer-rated capacity of 600 tons/hr, which is on the high side of conventional plant output for the area. This output was specified to allow rapid placement of the RCC and thereby reduce the

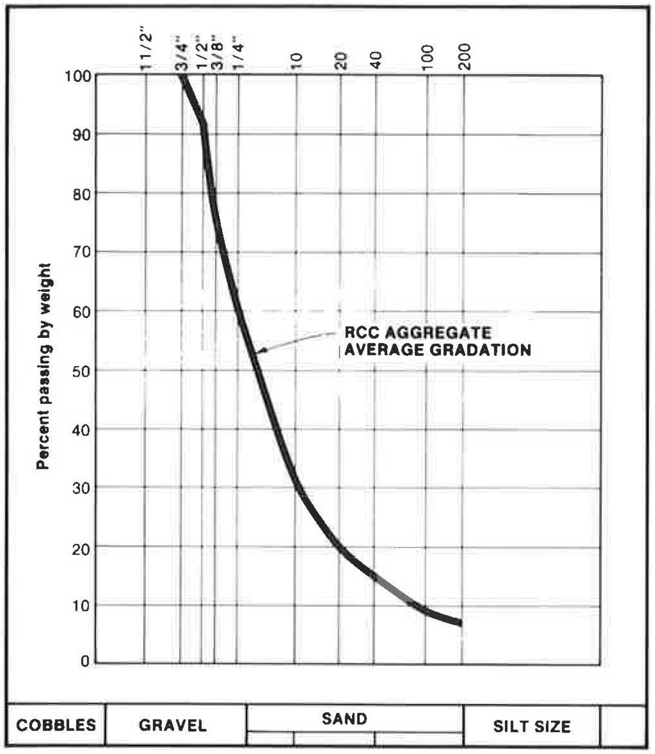

FIGURE 2 RCC aggregate gradation.

number of cold joints. Drum mixers were not allowed because of past problems with mixing uniformity and consistent output.

Paving Machine

Two RCC pavers were specified in order to eliminate the cold joint between the top and bottom lifts of RCC. Each paver was specified as a heavy-duty, track-mounted, self-propelled type, commonly used in asphalt concrete paving but modified to place RCC. Modifications to the augers are typically required to place the thicker RCC lifts without voids or segregation. Because of the surface tolerances, the pavers are to be electronically controlled for both line and grade.

As previously discussed, the current trend is toward a paver with tamping bars for precompaction. The contractor proposed a single paver of this type along with paving pattern modifications. This proposal was accepted and one of these pavers was used throughout the job.

Rollers

Vibratory rollers specified for compaction were to be self-propelled, double-drum, steel-wheel rollers having a shipping weight of at least 10 tons. Finish rolling with a rubber-tired roller was not required; however, many of the contractors believed that the surface would be improved with such treatment.

Curing Equipment

A water truck with a spreader pipe containing fog spray nozzles was used for the initial cure of the RCC. It is important to initially apply a fog spray rather than use a sprinkler system to prevent erosion or washing away of the surface cement. Subsequent curing was done with a sprinkler system. The contractor proposed this method of curing and equipment as an alternative to the specified water-and-burlap cure.

Paving Techniques and Paving Details

Paving

The RCC section of 14 in. is to be placed in two equal thicknesses, or 7 in. per lift. No more than 60 min was allowed to elapse between compaction of the first lift and placement of the second. On the basis of experience, this requirement was needed to allow a bond between the first and second lift. Misting or watering on the first lift was not allowed.

Construction Joints

Two types of construction joints occur: fresh joints where the adjacent RCC material is placed within 60 min and cold joints where adjacent material is placed after more than 60 min. For fresh vertical joints, the only requirement was to have a near-vertical face before the adjacent RCC panel was placed. Fresh horizontal joints (between lifts) required no preparation before placement of the second lift. Cold horizontal joints were not allowed.

Tolerances

The surface tolerance of the RCC pavement was specified for both elevation and localized variance. The surface-elevation tolerance allowed was ±.03 ft from the specified finished grade. The localized variance was ±1/4 in. at any point measured along a 10-ft straightedge. (The tolerance on the first lift was only that its thickness be within ±1/2 in. of the specified thickness.)

There were also tolerances on the RCC material during mixing. The variation in cement and pozzolan was ±2.0 percent by weight and for water, ±3.0 percent by weight. The particle gradation tolerance for the aggregate was between 2 and 6 percent by weight, depending of the sieve designation.

Curing

A water-cure method rather than a membrane method was used to cure the RCC. Water was to be applied in a continual fog spray 24 hr a day for 7 days. Because of anticipated warm weather during paving, it was important to apply the fog spray immediately after final rolling of the finished RCC surface.

Field Density Control

The minimum acceptable field density was 98 percent of a specified density, which was determined during the final mix design period by constructing a 24-in.-square by 15-in.-high block of RCC from the mix and measuring its unit weight. The project team, in conjunction with the Corps of Engineers North Pacific Division Materials Laboratory, decided to use a large block of RCC as representative of the maximum density. There are no ASTM standards or other guidelines for the preparation of such a block.

The RCC was placed in a steel mold in three layers and consolidated with a hand-held pneumatic tamper, and vibrations were made to the mold until no further consolidation was observed. Densities obtained in the field were compared against the unit weight of this RCC block. A nuclear densimeter was used to calculate the field density according to ASTM D2922.

The large block of RCC was considered representative of the actual field-placed RCC. Past studies show that field densities determined with the nuclear densimeter were consistent but underestimated the density when compared with actual cores taken from the pavement. Therefore, use of the RCC block allows a means to calibrate the nuclear densimeter field readings. Daily calibration of the nuclear densimeter on this block will allow reliable and consistent field density measurements during placement of the pavement.

BID TABULATION

A cost savings of approximately $220,000 was realized for the RCC alternative over the AC alternative. The six lowest bidders all offered the RCC option at a lower cost than AC, as shown in Table 2. The lowest RCC bid was about 32 percent under the lowest AC bid. If the life-cycle costs of RCC and AC were compared, an even greater savings would result. In time, it is expected that this differential will narrow as RCC contractors increase prices for greater profit margins and AC contractors become more competitive.

Of the six contractors that bid for RCC, only one had had previous RCC experience, but each of the other five had had experience with CTB construction. In awarding the contract to the low RCC bidder, Port of Portland officials considered both the construction cost savings involved and the long-term heavy load conditions. The FAA was consulted and gave its approval to proceed.

TABLE 2 Paving Bids for Port of Portland North Side Remote Parking

	Bid ($)	
Contractor	RCC Alternative	AC Alternative
A	687,370.65	914,400.15
B	739,327.00	–
C	780,517.75	917,041.00
D	796,205.00	–
E	791,893.00	1,011,142.05
F	800,585.00	956,404.00
G	–[a]	908,773.25
H	–	1,040,811.80
I	–	1,036,323.40

[a]No bid.

CONCLUSIONS

RCC used for a pavement as a final wearing surface is relatively new. Typically, RCC is designed and cured as a PCC pavement and is mixed and placed similarly to CTB. The final surface texture falls between that of AC and that of PCC.

Where RCC has been used for heavy-duty pavements, it has provided significant savings on a construction cost basis as an alternative to AC and PCC pavements and is less expensive to maintain than AC pavements. Another benefit of RCC is its resistance to chemical attack from hydraulic fluid, fuel, and other hydrocarbons.

To achieve the desired RCC surface, tolerance, texture, and field density, particular attention should be given to the following:

- Gradation, degree of angularity, and tolerances of aggregate;
- Representative sampling of aggregate for mix design;
- Determination of maximum density for field density control;
- Allowable tolerances in RCC during pugmill mixing;
- Type of paver (conventional vibratory screed versus precompaction tamping bar screed);
- Paving patterns for compatibility of paving equipment and site configuration;
- Type of rollers to achieve density and surface texture desired;
- Treatment of horizontal and vertical joints;
- Allowable tolerances in the final surface elevations, localized deviations, and overall thickness; and
- Curing of RCC (water versus impermeable membrane).

As more knowledge is gained about the details of RCC design and construction and as manufacturers continue to improve paving equipment, RCC will become an even more acceptable pavement option.

ACKNOWLEDGMENTS

The authors wish to express their thanks to the following individuals and companies for their valuable assistance in preparing this paper: U.S. Army Corps of Engineers; Portland Cement Association; Canadian Portland Cement Association; Jack Cewe, Ltd., Vancouver, British Columbia; and Intertec Construction Company, Portland, Oregon.

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Evaluation of the Frost Resistance of Roller-Compacted Concrete Pavements

STEVEN A. RAGAN

ABSTRACT

An investigation was conducted to evaluate the frost resistance of samples taken from roller-compacted concrete (RCC) pavements with laboratory testing procedures. Nine existing pavements were sampled and tested for microscopic determination of air-void content and parameters of the air-void system, resistance to rapid freezing and thawing, critical dilation, and compressive and flexural strength. The pavements ranged in age from 1 month to 8 years at the time of sampling. Results of the rapid freezing-and-thawing tests indicate that the frost resistance of the concrete is, as might be expected, a function of the bubble spacing factor ($\bar{L}$) of the air-void system. These samples having $\bar{L}$'s smaller than 0.011 in. generally had durability factors (DFE_{300}) of 60 or greater. Those samples having $\bar{L}$'s greater than 0.016 in. generally had DFE_{300}'s less than 40. The samples having $\bar{L}$'s less than 0.016 in. also contained approximately 2.0 to 4.0 percent, by volume, of air voids; in the sections that were counted, chord lengths that were less than 0.04 in. long but irregularly shaped were found. The presence of larger-than-anticipated amounts of small air voids appears to be related to cohesiveness of the mixture, pugmill mixing, and the method of compaction. Preliminary results of the dilation tests indicate that those samples judged to be frost susceptible when tested by rapid freezing and thawing may, in fact, offer some degree of frost resistance. The dilation test may be appropriate, therefore, to determine whether an RCC sample is frost resistant at the time of test or to measure the period of frost immunity.

The Commander, U.S. Army Corps of Engineers, has recommended that all field operating activities having military construction and civil works design responsibilities consider the use of roller-compacted concrete (RCC) for various horizontal concrete construction applications (1). These applications include paving for tactical equipment shops and tracked-vehicle wash facilities, tank trails, open storage areas, and marshalling areas. Significant cost savings are anticipated when RCC is used in lieu of conventional concrete, because of the labor savings associated with the production, placement, and compaction of RCC. However, the frost resistance of RCC pavements is of concern, particularly because results from earlier investigations indicate difficulty in securing entrained air in RCC (2).

The Waterways Experiment Station (WES) is currently conducting an extensive research program on RCC pavement criteria development. Included in this program is an investigation on RCC pavement frost resistance. The initial phase of this investigation, discussed in this paper, consisted of obtaining representative samples from existing RCC pavements and performing laboratory testing on them to determine their air-void system parameters and resistance to frost damage. A second phase is planned in order to determine whether materials and construction criteria can be developed that will produce relatively frost-resistant RCC pavements.

Nine RCC pavements were sampled and tested for microscopic determination of air-void content and parameters of the air-void system, resistance to rapid freezing and thawing, critical dilation, and compressive and flexural strength. Although the majority of testing was conducted at WES, some of the rapid freezing-and-thawing and strength testing was performed by the U.S. Army Corps of Engineers North Pacific Division Laboratory (NPDL) in Troutdale, Oregon, and Southwestern Division Laboratory (SWDL) in Dallas, Texas. A number of no-slump concrete beams were also fabricated by WES and NPDL during mixture proportioning studies and tested for resistance to rapid freezing and thawing and microscopic determination of air-void content and parameters of the air-void system.

RCC PAVEMENT DESCRIPTIONS

Fort Stewart, Georgia

A test section 234 ft long by 20 ft wide was constructed in July 1983. The pavement ranges in thickness from 9 to 13 in. and currently serves as an access from a tracked-vehicle parking area to a series of tank trails.

The materials used in the RCC were batched in a weigh-batch type concrete plant and mixed in revolving-drum truck mixers. The mixture contained crushed coarse aggregate conforming to ASTM C 33 size designation No. 57 and natural fine aggregate. The gradings of both aggregates are shown in Table 1. The mixture also contained approximately 611 lb/yd³ of Type I portland cement and had a water-cement ratio (W/C) of 0.33. Mixture proportions are given in Table 2.

Concrete was discharged from the truck mixers into a front-end loader bucket, transported to the prepared sand-clay base, and spread to the approxi-

U.S. Army Corps of Engineer Waterways Experiment Station, Vicksburg, Miss. 39180-0631.

TABLE 1 RCC Aggregate Gradings

Sieve Size[a]		Cumulative Percent Passing														
		Fort Stewart		Fort Hood		Fort Lewis			USACRREL				WES Beams		NPDL Beams	
						Natural Concrete				Concrete						
Standard	Alternative	Mixture A	Mixture B	Mixture A	Mixture B	Mixture A	Mixture B	Crushed Asphalt	Pit Run	Mixture A	Mixture B	Port of Tacoma	Mixture A	Mixture B	Mixture A	Mixture B
50 mm	2 in.			100												
37.5 mm	1½ in.	100		98												
25.0 mm	1 in.	95		30	100	100			100	100			100		100	
19.0 mm	¾ in.	71		8	97	83		100	99	99		100	93		95	
12.5 mm	½ in.	31		–	–	47		98	–	68		98	–		54	
9.5 mm	⅜ in.	11		1	42	25		86	77	31	100	85	25		29	100
4.75 mm	No. 4	2	99		8	100 1	100	60	69	2	99	55	4	99	2	97
2.36 mm	No. 8		96		0	90 1	84	41	55	1	87	41	1	87	1	74
1.18 mm	No. 16		86			69	64	29	43		68	31		71		56
600 μm	No. 30		57			45	40	19	33		43	22		42		35
300 μm	No. 50		20			17	15	11	24		15	15		13		15
150 μm	No. 100		4			4	4	6	14		4	10		7		3
75 μm	No. 200		1			1	2	–	8		2	5		–		–

Note: USACRREL = U.S. Army Cold Regions Research and Engineering Laboratory; WES = Waterways Experiment Station; NPDL = North Pacific Division Laboratory. Dashes indicate data not available.

[a]U.S. standard sieves.

TABLE 2 Mixture Proportions

Project	Weight, Saturated Surface–Dry (lb/yd^3)								
	Portland Cement	Fly Ash	4.75-19.0 mm	4.75-25.0 mm	4.75-37.5 mm	4.75 mm-75 μm	19.0 mm-75 μm	Water	AEA (oz/cwt)
Fort Stewart	611	–	–	1,971	–	1,406	–	202	–
Fort Hood									
Mixture A	306	162	–	–	2,294	1,285	–	153	–
Mixture B	376	186	2,165	–	–	1,366	–	180	–
Fort Lewis									
Mixture A	320	172	1,967	–	–	1,546	–	176	–
Mixture B	499	–	–	–	–	–	3,468	206	–
USACRREL									
Mixture A	564	–	–	–	–	–	3,336	209	–
Mixture B	567	–	1,745	–	–	1,640	–	199	–
Mixture C	567		1,745			1,640		199	7.0
Port of Tacoma	450	100	–	–	–	–	3,400	256	–
WES beams									
Mixture A	561	–	1,960	–	–	1,472	–	196	–
Mixture B	548	–	1,837	–	–	1,437	–	191	7.5
NPDL beams	351	180	1,936	–	–	1,560	–	169	–

Note: AEA = air-entraining admixture; USACRREL = U.S. Army Cold Regions Research and Engineering Laboratory; WES = Waterways Experiment Station; NPDL = North Pacific Division Laboratory. Dashes indicate material not used.

mate desired grade. A 19,000-lb single steel-drum vibratory roller was used to compact the mixture. A layer of sand approximately 3 in. thick was spread on the compacted pavement surface and moistened periodically for 3 days in order to cure the RCC. Samples were taken from the pavement approximately 3 months after construction.

Fort Hood, Texas

An 18,150-yd^2 RCC parking area was constructed adjacent to a tactical equipment maintenance shop in July 1984. The 10-in.-thick pavement is designed to support 120,000-lb tracked vehicles as normal traffic.

All concrete materials were mixed in a continuous-mixing pugmill. Two mixtures were used in the pavement: one having a 37.5-mm (1 1/2-in.) nominal maximum size aggregate (NMSA), and the second having a 19.0-mm (3/4-in.) NMSA. Natural fine aggregate was used in both mixtures. Aggregate gradings are shown in Table 1. The 37.5-mm (1 1/2-in.) NMSA mixture contained approximately 306 lb/yd^3 Type I portland cement and 162 lb/yd^3 Class C fly ash and had a W/C of approximately 0.31. The 19.0-mm (3/4-in.) NMSA mixture contained approximately 376 lb/yd^3 Type I portland cement and 186 lb/yd^3 Class C fly ash and had a W/C of approximately 0.30. The concrete mixture proportions for both are given in Table 2. Only the pavement containing the 37.5-mm NMSA mixture was sampled and tested in this investigation.

The no-slump concrete was placed on a 6-in.-thick lime-stabilized base with an asphalt paver and compacted with a 20,000-lb dual steel-drum vibratory roller. Damp cotton mats were used to moist-cure the RCC for the first 24 hr, at which time the mats were removed and a pigmented membrane-forming curing compound was applied. The pavement test section was sampled approximately 1 month after its construction.

Fort Lewis, Washington

A 700-ft-long by 22-ft-wide RCC pavement test section, ranging in thickness from 6 to 7 3/4 in., was constructed in October 1984. The test section currently serves as a secondary road for both rubber-tired and tracked vehicles.

Two concrete mixtures were used in the pavement. One contained 19.0-mm NMSA natural coarse aggregate conforming with the grading limits of ASTM C 33, size designation No. 67, and natural fine aggregate. This mixture also contained approximately 320 lb/yd³ Type I portland cement and 172 lb/yd³ Class F fly ash and had a W/C of approximately 0.32. The second mixture contained both coarse and fine crushed aggregate graded from a 12.5-mm (1/2-in.) nominal maximum size to the 75-µm (No. 200) sieve size. This aggregate was typical of one that might be used in an asphalt paving mixture. The concrete also contained approximately 499 lb/yd³ Type I portland cement and had an approximate W/C of 0.41. No fly ash was included in the second mixture. The aggregate gradings and concrete mixture proportions used for this project are given in Tables 1 and 2, respectively.

The concrete materials were mixed in a continuous-mixing pugmill, and the resulting concrete was placed with an asphalt paver. Compaction was achieved with a single steel-drum vibratory roller having a mass of approximately 20,000 lb. A rubber-tired roller was also used to knead and tighten the RCC surface. Following the compaction operations, the concrete was continuously moist-cured for 7 days by use of a water-spray system. Hardened pavement samples representing each mixture were obtained approximately 3 weeks after completion of construction.

U.S. Army Cold Regions Research and Engineering Laboratory (USACRREL)

These nominal 25- by 18-ft by 8-in. RCC pavement test sections were constructed in November 1984. One of the test sections contains concrete having a 19.0-mm nominal maximum size natural aggregate. The aggregate was graded in a single size range to the 75-µm (No. 200) sieve size. Approximately 564 lb/yd³ of Type II portland cement was used in the mixture. The W/C was approximately 0.37.

A second section was constructed with no-slump concrete containing crushed coarse aggregate having a 19.0-mm nominal maximum size and a natural fine aggregate. The concrete also contained approximately 567 lb/yd³ of Type II portland cement and had a W/C of approximately 0.35.

The third test section was constructed with a concrete mixture similar to that used in the second test section, except that an air-entraining admixture (AEA) consisting of an aqueous solution of neutralized Vinsol resin was added. The aggregate gradings and concrete mixture proportions used in the three RCC test sections are shown in Tables 1 and 2, respectively.

A pugmill of the batch type, normally used for asphalt production, was used to mix the concrete materials. The concrete was placed with an asphalt paver onto a crushed bank-run gravel base and compacted with a dual steel-drum vibratory roller having a mass of approximately 20,000 lb. The completed test sections were moist-cured 14 days with damp burlap strips. The pavements were sampled approximately 1 month after completion of construction.

Port of Tacoma, Intermodal Railyard Development, Washington

This nominal 90,000-yd² RCC pavement varies in thickness from 12 to 17 in. It was constructed in April 1985 and serves as a container storage facility.

The no-slump concrete mixture contained a crushed aggregate including both fine and coarse material having a nominal maximum size of 12.5 mm and graded to the 75-µm sieve size. The aggregate was typical of that used in asphalt paving. The mixture also contained 450 lb/yd³ Type I portland cement and 100 lb/yd³ Class F fly ash and had a W/C of approximately 0.43. The aggregate grading is given in Table 1 and the mixture proportions in Table 2.

The materials were mixed in a continuous-mixing pugmill, and the resulting concrete was placed in two equal layers with asphalt pavers onto a crushed gravel base. Dual steel-drum rollers, each having a mass of approximately 20,000 lb, were used to compact the concrete and a rubber-tired roller was used to tighten the pavement surface. Water-spray trucks were used to moist-cure the RCC for 7 days after completion of compaction. Samples were taken from the pavement approximately 3 weeks after construction was completed.

Caycuse, British Columbia

An RCC dry-land log-sorting yard was constructed on Vancouver Island in 1976. It has an area of approximately 22,000 yd² and a nominal thickness of 14 in. A single-size-range aggregate having a 19.0-mm nominal maximum size and graded to the 75-µm sieve size was used in the concrete. The aggregate grading and concrete mixture proportions were not available to the author when this paper was prepared. However, the contractor responsible for constructing the pavement has reported that approximately 8 percent of the aggregate, by mass, was finer than the 75-µm sieve and that two mixtures were used in the pavement. A mixture containing approximately 7 percent portland cement, by mass of the aggregate, was used in the lower 8 in., and one containing approximately 12 percent portland cement, by mass of the aggregate, was used in the upper 6 in.

Concrete materials were mixed in a continuous-mixing pugmill. The concrete was placed with an asphalt paver and compacted with single steel-drum vibratory rollers. Samples were taken from the pavement approximately 8 years after it had been constructed.

Laboratory-Fabricated Specimens

In addition to the RCC pavements previously described, specimens were also fabricated for test by WES and NPDL as part of the USACRREL and Fort Lewis mixture proportioning studies. The concrete materials used by both laboratories were mixed in small revolving-drum mixers. Specimens for rapid freezing-and-thawing, compressive strength, and flexural strength tests were fabricated on vibrating tables with the aid of surcharge weights. The aggregate gradings and mixture proportions used in the laboratory studies are given in Tables 1 and 2, respectively.

TEST PROGRAM

As was previously stated, the primary purpose of the investigation discussed here was to characterize the air-void system parameters and evaluate the frost resistance of samples obtained from RCC pavements. At least one representative sample from each pavement described here was microscopically examined in accordance with ASTM C 457, Modified Point-Count Method, in order to determine the air-void content and bubble spacing factor ($\bar{L}$). Initially, the entrained and entrapped air-void content of each sample was determined. The criteria given in the definition of air void in ASTM C 125 were used to distinguish entrained and entrapped air voids. However, the predominant irregular shape of the observed sections of the small air voids, discussed in further detail in the following paragraphs, resulted in very low entrained air-void contents in the samples. These results appeared inconsistent with what one might expect, given the relatively small $\bar{L}$'s determined in a number of the samples. Therefore, the decision was made to characterize the air voids by size only, irrespective of shape. Determinations were made for each specimen of the percentage of voids whose section chord lengths were less than 0.04 in. and the percentage of those whose section chord lengths were greater than 0.04 in.

The rapid freezing-and-thawing tests were conducted in accordance with applicable sections of ASTM C 666, Procedure A. Samples were sawed into prisms 3 1/2 by 4 1/2 by 16 in. and stored in a 73°F water bath for 14 days before the start of testing. Each specimen was continued in test until it was subjected to 300 freezing-and-thawing cycles or until its relative dynamic modulus of elasticity reached 50 percent of the initial modulus, whichever occurred first. The NPDL conducted the freezing-and-thawing tests on the Fort Lewis and Caycuse samples as well as the NPDL-fabricated specimens. The South West Division Laboratory conducted the freezing-and-thawing tests on the Fort Hood samples. WES conducted the testing on the remainder of the samples.

The dilation test (ASTM C 671) was also used as a means to evaluate the potential frost resistance of the samples. The test may also be used to determine whether a specimen becomes frost susceptible during a particular period of interest. Powers (3) stated that concrete frost resistance or durability is not a measurable property, but that the expansion that occurs during a slow-cooling cycle when the concrete or its aggregates become critically saturated is measurable and will provide an indication of potential frost resistance. That is, if a specimen shrinks normally in the freezing range, it is immune to frost action; if it dilates, it is not immune. ASTM C 671 requires that dilation be determined by measuring the vertical distance from straight-line projection of the prefreezing, length-versus-time contraction curve at constant cooling rate and the maximum deviation of the strain from it. The test is conducted by monitoring the length of a specimen as its temperature is lowered.

There were some differences in the dilation test method followed in this investigation and that prescribed by ASTM. The major one was that the 3-in.-diameter by 6-in.-long RCC cores were cooled in water-saturated kerosene from an unspecified but convenient temperature range of 35° to 55°F at a rate of approximately 5°F per hour to a minimum of -10°F. The ASTM method specifies cooling the specimens in water-saturated kerosene from 35° to 15°F at 5°F per hour. Critical dilation (D_c) in ASTM C 671 is defined as the dilation during the last cycle before the dilation begins to increase sharply by a factor of 2 or more. The method states that dilations less than 0.005 percent should not be interpreted as indicating D_c even if the criterion for D_c is met numerically.

A dilation criterion, based on the results of a single test, has also been proposed by Buck (4). He suggests the following:

1. If the dilation is 0.005 percent (= 50 millionths) or less, the specimen may be regarded as frost resistant; that is, the dilation is not critical.
2. If the dilation is 0.020 percent (= 200 millionths) or more, the specimen may be regarded as not frost resistant; that is, D_c has been exceeded.
3. If the dilation is in the range between 0.005 percent and 0.020 percent, an additional cycle or more should be run.

Ten specimens representing six of the nine pavements were tested. At the time that this paper was prepared, only five to seven freezing cycles per specimen had been completed. Test results were evaluated by using both the ASTM dilation criterion and that suggested by Buck.

Compressive and flexural strength tests provided physical data on samples representing each pavement. The tests were conducted in accordance with ASTM C 42.

DISCUSSION OF RESULTS

The air-void size distribution and $\bar{L}$ of each RCC sample are given in Table 3. As was previously noted, the air voids were categorized according to chord lengths of the counted air-void sections, irrespective of section shape. Figure 1 shows a typical RCC and a conventional air-entrained concrete polished section. The irregular shape of the voids in the RCC section is believed to result from compaction operations associated with RCC pavement construction. However, no work was conducted in this investigation to confirm this belief.

The percentage of air voids, by volume, smaller than 0.04 in. ranged from 0.1 to 9.6 in the pavement samples and from 0.6 to 2.5 in the laboratory-fabricated specimens. Large dosages of AEA were added to the mortar fraction of the concrete represented by

TABLE 3 Microscopic Air-Void Data

			Air Content (%) by Chord Length (in.)			
Project	Mixture	Specimen No.	Less Than 0.04	Larger Than 0.04	Total	$\bar{L}$ (in.)
Fort Stewart		1T1	0.2	8.0	8.2	–
		1B1	0.1	3.7	3.8	–
		2T1	1.9	3.4	5.3	0.020
		2B1	0.8	21.6	22.4	–
		3T1	0.6	2.5	3.1	–
		3B1	0.3	4.0	4.3	–
		4T1	0.8	10.0	10.8	–
		4B1	0.2	8.5	8.7	–
Fort Hood	A	1	1.3	1.6	2.9	0.012
Fort Lewis	A	5A	2.5	2.6	5.1	0.012
		9B	1.9	0.2	2.1	0.005
		10B	9.6	0.8	10.4	0.011
	B	17A	1.6	3.1	4.7	0.015
		17B	1.8	2.2	4.0	0.018
USACRREL	A	1T	–	–	5.1	–
		1B	2.3	3.2	6.1	0.008
	B	3T	–	–	5.3	–
		3B	3.6	5.0	8.6	0.010
	C	2T	2.9	2.3	5.2	0.010
		2B	–	–	4.8	–
Port of Tacoma		2A-T	5.5	1.6	7.1	0.010
		2A-B	2.5	0.7	3.2	–
		1D-T	3.0	1.5	4.5	0.013
		1D-B	4.1	0.5	4.6	–
		2H-T	3.5	2.0	5.5	–
		2H-B	6.1	4.5	10.6	0.010
Caycuse		2A	0.5	10.8	11.3	0.026
WES beams	A	1	1.1	2.0	3.1	0.030
	B	1	2.5	2.1	4.6	0.013
NPDL beams		2	0.6	1.1	1.7	0.018
		8	0.9	0.8	1.7	0.019
		9	1.1	2.8	3.9	0.022

Note: Dash indicates data not determined.

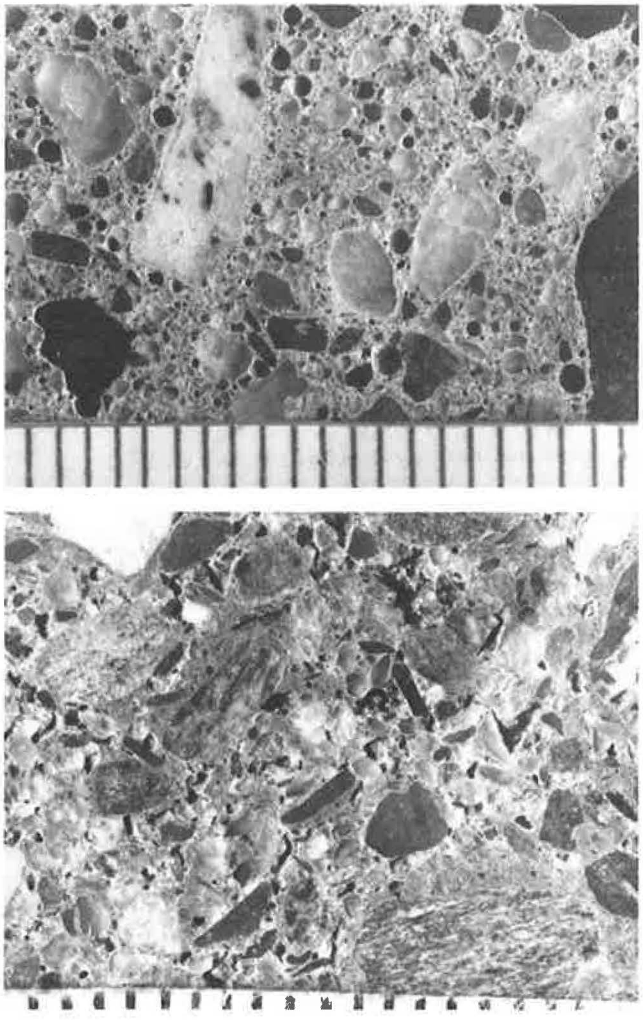

FIGURE 1 Polished sections showing air-void shapes, magnification = 4.1. ***Top:*** **conventional air-entrained concrete;** ***bottom:*** **RCC.**

USACRREL specimen C-2T and WES specimen B-1. The mortar was mixed for approximately 1 min before the addition of the coarse aggregate. The USACRREL concrete was mixed in a pugmill mixer of the batch type, and the WES concrete was mixed in a small revolving-drum mixer. The effect of the AEA on the concrete air-void system is somewhat ambiguous. Specimen C-2T has 2.9 percent, by volume, of air voids smaller than 0.04 in. and 2.3 percent of the voids larger than 0.04 in. However, USACRREL specimen B-3B, which represents similar concrete without AEA, has 3.6 percent of the air voids smaller than 0.04 in. and 5.0 percent larger. Similarly, USACRREL specimen A-1B, which also represents a non-air-entrained concrete, has 2.3 percent of the air voids smaller than 0.04 in. and 3.2 percent larger than 0.04 in. The $\bar{L}$'s of the three specimens are not significantly different.

WES specimen B-1 had 2.5 percent, by volume, of its air voids smaller than 0.04 in. and 2.1 percent of the voids larger than 0.04 in. WES specimen A-1, which represents similar concrete without AEA, had 1.1 percent of its voids smaller than 0.04 in. and 2.0 percent larger than 0.04 in. However, the $\bar{L}$ of specimen A-1 is approximately 2.5 times that of specimen B-1.

Those specimens representing concrete that was pugmill mixed generally had greater percentages of air voids smaller than 0.04 in. than those specimens representing concrete mixed in revolving-drum mixers. The specimens representing pugmill-mixed concrete also generally had smaller $\bar{L}$'s. Although the mechanisms responsible for creating these desirable air-void systems in the non-air-entrained concrete are not yet fully understood, pugmill mixing appears to be a contributing factor. The cohesiveness of the mixture may also play an important role in the maintenance of the small air voids during compaction. An RCC mixture that is highly cohesive due to a low water content and large aggregate surface area may prevent the escape of a large percentage of the small air voids during the compaction operations. Additional investigative work is needed to confirm these proposed explanations.

RAPID FREEZING-AND-THAWING TESTS

Table 4 summarizes the results of the rapid freezing-and-thawing tests. In general, these values may be interpreted (5) as follows: A DFE_{300} less than 40 means that the concrete is probably unsatisfactory with respect to frost resistance; 40 to 60 is the range for concretes with doubtful performance; and greater than 60, the concrete is probably satisfactory. With these criteria, the test results indicate that the RCC samples associated with Fort Stewart, Fort Hood, and Caycuse were frost susceptible, as were the WES- and NPDL-fabricated specimens. The results of the tests of the Fort Lewis specimens indicate doubtful to satisfactory performance, whereas the USACRREL and Port of Tacoma test results indicate satisfactory performance.

The rapid freezing-and-thawing test results and specimen $\bar{L}$'s are paired in Table 5. This summarization indicates that the frost resistance of RCC is, as expected, a function of the $\bar{L}$. Spacing factors less than 0.008 in. are typically associated with concrete having good resistance to freezing and

TABLE 4 Results of Rapid Freezing-and-Thawing Tests

Project	Mixture	Specimen No.	Avg DFE_{300}
Fort Stewart		1T1-1T6	8
		1B1-1B5	6
		2T1-2T2	9
		2B1-2B3	5
		3T1-3T4	8
		3B1-3B4	4
		4T1-4T4	6
		4B1-4B4	6
Fort Hood	A	5-7	10
		8-10	8
Fort Lewis	A	5A,B; 9A,B; 10A,B	47
	B	17A-17D	59
USACRREL	A	1T-3T	88
		1B-3B	89
	B	1T-3T	39
		1B-3B	69
	C	1T-3T	68
		1B-3B	91
Port of Tacoma		1A-T,B; 2A-T,B	82
		1D-T,B; 2D-T,B	79
		1H-T,B; 2H-T,B	78
		3H-T,B; 4H-T,B	
Caycuse	12 percent cement	1A-1C	6
		2A-2C	3
WES beams	A	1-3	10
	B	1-3	48
NPDL beams		1-4	10
		5-9	11

TABLE 5 Specimen DFE_{300}'s and $\bar{L}$'s

Project	Mixture	Specimen No.	DFE_{300}	$\bar{L}$ (in.)
Fort Stewart		2T1	8	0.020
Fort Hood		5	10	0.012
Fort Lewis	A	5A	44	0.012
		9B	71	0.005
		10B	20	0.011
	B	17A	59	0.015
		17B	44	0.018
USACRREL	A	1B	89	0.008
	B	3B	75	0.010
	C	2T	81	0.010
Port of Tacoma		2A-T	84	0.010
		1DT	75	0.013
		2HB	82	0.010
Caycuse		2A	3	0.026
WES beams	A	1	10	0.030
	B	1	48	0.013
NPDL beams		2	10	0.018
		8	–	0.019
		9	11	0.022

thawing. In the case of the tests of RCC, it was found that $\bar{L}$'s smaller than 0.011 in. generally resulted in DFE_{300}'s of 60 or greater; $\bar{L}$'s of 0.011 to 0.016 in. resulted in DFE_{300}'s of 40 to 60; and $\bar{L}$'s greater than 0.16 resulted in DFE_{300}'s less than 40. The relatively small $\bar{L}$ coupled with the low DFE_{300} of the Fort Hood specimen was unexpected. However, it is not known if the aggregates used in the RCC are frost susceptible. The small $\bar{L}$ and low DF_{300} of Fort Lewis specimen A-10B were unexpected and unexplainable. Figure 2 graphically shows the relationship between DFE_{300} and $\bar{L}$ of the RCC samples. The figure is subdivided into zones of performance based on the criteria noted earlier.

Dilation Tests

The rapid freezing-and-thawing test uses a higher freezing rate than is ordinarily encountered in outdoor weathering. Cooling takes place at approximately 25°F/hr in the laboratory test, whereas in practice 5°F/hr is not normally exceeded. The dilation test was selected as an alternative means of evaluating the frost resistance of some of the pavement samples because the cooling rate used in the test is comparable to natural cooling rates. Powers suggested that when dilation-testing specimens that represent concrete subject to seasonal drying, a period of immunity of approximately 16 weeks (8 freezing cycles) of continuous exposure to water should assure immunity during any winter season (3). Studies by Larson and Cady (6) suggest that the length of the soaking period exerts considerably more influence on the rate of deterioration than does the number of intermediate cooling cycles.

Table 6 shows the specimen dilation test results. It is apparent from this table that a variation of 100 to 200 min between cycles for a specimen is not indicative of damage. Each of the specimens appears to be immune to frost damage, as defined by ASTM C 671, for the first five to seven cycles of test. However, USACRREL specimen A-4, Fort Stewart specimen FS-4, and Fort Hood specimen 20-F each experienced large dilations after six, five, and three cycles,

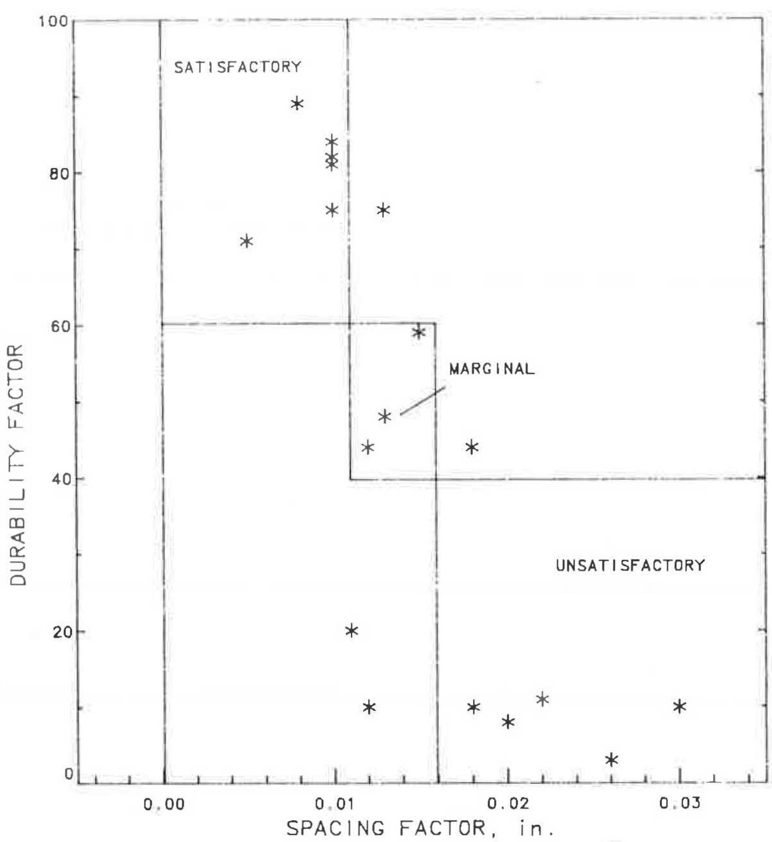

FIGURE 2 Durability factor (DFE_{300}) versus spacing factor ($\bar{L}$) based on data from Table 5.

TABLE 6 Dilation Test Results

			Test Cycle													
			1		2		3		4		5		6		7	
Project	Mixture	Specimen No.	Micro-inches	Millionths	Micro-inches	Millionths	Micro-inches	Millionths	Micro-inches	Millionths	Micro-inches	Millionths	Micro-inches	Millionths	Micro-inches	Millionths
Fort Stewart		FS-4	375	75	425	85	130	26	250	50	220	50	430	86	325	65
Fort Hood	A	20-F	200	40	230	46	230	46	450	90	500	100	575	115	575	115
USACRREL	A	4	200	40	340	68	300	60	400	80	300	60	300	60	550	110
	B	4	175	35	125	25	125	25	250	50	175	35	250	50	175	35
	C	4	250	50	330	66	330	60	450	90	450	90	–	–	550	110
Port of Tacoma		3A-T	110	22	–	–	190	38	160	32	210	42	–	–	–	–
		3D-T	130	26	175	35	252	55	250	50	225	45	–	–	–	–
		3D-B	130	26	100	20	210	42	200	40	330	66	–	–	–	–
		5H-T	140	28	275	55	200	40	300	60	260	52	–	–	–	–
		5H-B	100	20	100	20	275	55	200	40	290	58				

Note: Dashes indicate malfunction.

respectively, and additional testing may result in dilation in excess of D_c.

The single-test dilation criterion suggested by Buck (4) indicates that, in general, all of the specimens fall in the marginal zone between frost resistant and frost susceptible. That is, the dilation value obtained during the last freezing cycle of each is generally within the range of 50 to 200 millionths. This suggests that additional cycles should be run to determine when the D_c of 200 millionths is exceeded. Regardless of the dilation criterion used to evaluate the results, it is apparent that each of the specimens tested has some resistance to frost damage down to -10°F for at least 5 to 7 cycles (10 to 14 weeks of continuous exposure to water). In the case of those samples having large $\bar{L}$'s, the low W/C ratios were apparently adequate to provide a measure of protection.

Compressive and Flexural Strength Tests

A summary of the sample compressive and flexural strength test results is shown in Table 7. The average compressive strengths range from 2,930 to 8,920 psi, and the average flexural strengths range from 510 to 1,010 psi.

TABLE 7 Compressive and Flexural Strength Test Results

Project	Mixture	Approximate Age (days)	Avg Compressive Strength (psi)	Avg Flexural Strength (psi)
Fort Stewart		90	5,220	1,010
Fort Hood	A	28	4,780	830
Fort Lewis	A	90	5,790	690
	B	90	8,920	960
USACRREL	A	40	2,930	510
	B	40	6,500	860
	C	40	4,370	600
		40	6,500	860
Port of Tacoma		35	5,220	705
Caycuse		8 years	5,880	540
WES beams	A	28	6,250	730
	B	28	5,740	680
NPDL beams		28	6,900	650

CONCLUSIONS

1. The microscopic examinations of representative sections of the RCC pavement samples indicate that air-void systems normally associated with at least partially frost-resistant concrete may be created without the use of AEA. The creation of these air-void systems appears to be related to pugmill mixing, the cohesiveness of the mixture, and the method of compaction. The inclusion of AEA in the mortar fraction of a no-slump concrete mixture that was pugmill mixed did not significantly increase the percentage of air voids smaller than 0.04 in.

2. The shapes of the examined air-void sections were irregular. These irregular shapes are believed to result from the compaction operations associated with RCC pavement construction.

3. The frost resistance of RCC, as evaluated by the DFE_{300}, is a function of $\bar{L}$. Durability factors of 60 or more were associated with those specimens having $\bar{L}$'s smaller than 0.011 in. Those specimens having $\bar{L}$'s of 0.011 to 0.016 in. resulted in DFE_{300}'s of 40 to 60, and those having $\bar{L}$'s larger than 0.016 in. resulted in DFE_{300}'s of less than 40.

4. The dilation test appears to provide an effective measure of the frost resistance of RCC. Use of the test would appear appropriate to determine whether a sample of RCC is frost resistant at the time of test or to measure the period of frost immunity. The latter use might be particularly important in the case of pavements, which are typically subject to seasonal drying.

5. The dilation data indicate some degree of frost resistance for each specimen tested. The frost resistance of specimens having large $\bar{L}$'s may be attributed to the low W/C of the RCC mixtures. Such concrete has little freezable water in the paste, and also has a low permeability. Therefore, it is more difficult to critically saturate.

6. Concrete must meet three requirements before it may be considered immune to frost action. It must be made with non-frost-susceptible aggregates and a proper air-void system and must be cured to an appropriate degree of maturity so as to reduce the fractional volume of freezable water on saturation to limits that can be accommodated by elastic volume change and by the air-void system. RCC pavements can be constructed with non-frost-susceptible aggregates and can be appropriately cured. The air-void systems observed in many of the sampled RCC pavements should be sufficient to protect them against frost damage in all but the most severe environments. Additional investigative work is needed to determine whether an entrained air-void system can be effectively produced in RCC that would cause it to be immune to frost damage in all exposures.

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Construction Techniques for Roller-Compacted Concrete

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ABSTRACT

Roller-compacted concrete (RCC) is a new material that uses a construction technique that applies the material property theories of both soil mechanics and concrete. The resulting product is a construction material with the strength characteristic of conventional concrete. Because RCC is a new material, the construction process for its placement is in a state of trial and adjustment; however, a unique feature of RCC is the use of standard earthmoving equipment for transportation, placement, and compaction. Earthmoving equipment reduces the labor-intensive process necessary for conventional concrete placement. The RCC mix is placed in a solid (no vertical joints) horizontal layer across the total placement area. No cure time is necessary between layers.

In 1970 Raphael presented the idea of a new construction material that would exist between conventional concrete and earth fill (1,pp.221-247). With the equipment and techniques of an earth embankment project, a structure would be constructed with this new material in a continuous-cycle-type operation. The completed structure would, however, develop the strength and material characteristics of conventional concrete. The material would be an intermedium substance having the advantages of both earth fill and conventional concrete; therefore, a more economical structure or project could be achieved through reductions in materials, labor, and time (1).

These ideas become realities with the introduction of zero-slump concrete. The new construction material is known as roller-compacted concrete (RCC), roll-crete, or rolled concrete. As actual project experience is gained, the construction process for RCC is being developed, modified, and adjusted. Projects to date have been approached by applying new ideas to previous earthwork or concrete construction techniques. Engineers around the world have gained experience on a first generation of RCC projects and are beginning to report the results of their field experimentation with RCC placement techniques on those projects (2-5). This paper is a state-of-the-art review of the construction processes used for RCC construction.

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MATERIAL FUNDAMENTALS

Materials play an important part in dictating construction processes. RCC construction techniques for mixing, transporting, and placement are controlled primarily by material characteristics in relation to moisture retention, segregation, and compaction.

In the wet stage, RCC exhibits the material handling properties of a moist, granular soil; standard earthmoving equipment can be used for transportation, placement, and compaction. The term "wet" is used here instead of "plastic," as would be expected with concrete construction, because the no-slump concrete (RCC) does not react during placement operations as a plastic mix. After the RCC hardens, it is a concrete with the physical properties of conventional concrete and a finished appearance similar to that of conventional concrete (6).

Strength Characteristics

The density of RCC is developed through vibratory compaction. The resulting final strength is based on the percent composition of materials, the type and gradation of the aggregates, and the percent of optimum density obtained. The required RCC strength is specified for each individual project. The design strength is based on the structural use of the RCC: as a massive material with relatively low strength in a gravity structure where the compressive strength of the material is not a controlling factor; as a high-strength material with low mass in paving applications for high loadings; or somewhere in between, as a material with an average mass and strength. Higher-strength RCC requires a greater degree of control during the construction process and limits the selection of materials and equipment.

Gary N. Reeves of Freese and Nichols, Inc., and Lewis B. Yates of Trinity Engineering Testing Corporation approach RCC strength and mix quality using standard soil mechanics methods and theory; that is, as unit dry density increases, the material's strength will also increase and permeability will decrease (4). They develop a Proctor-type curve of dry density versus moisture using the project's proposed aggregates. This project- and aggregate-specific curve then provides the information necessary for controlling field placement operations. After placement, the RCC is cured and treated as conventional concrete.

Both the U.S. Army Corps of Engineers (7-9) and the U.S. Bureau of Reclamation (4-10) view RCC more as a concrete material in terms of design and field construction control. However, these two federal agencies each use slightly different approaches and techniques (11).

At Upper Stillwater Dam, the Bureau plans to use processed aggregates and high cementitious materials to provide bond strength without extensive joint cleanup. In planning Elm Creek Dam, the Corps is looking at increasing the amount of mortar to aid in attaining bond. This will be accomplished by the addition of portland cement, pozzolan, and more fines in the aggregate (12).

Aggregate Requirements

Generally, the specification requirements for the aggregates in RCC are not as strict as those for aggregates used in conventional concrete. Because heavy rollers, common to earth or rock construction, are utilized to densify RCC, aggregate gradation is not as important to achieving desired density as is slumpable concrete production (11). If mass, not strength, is the controlling characteristic, then a wide gradation range for aggregate selection exists. Many aggregates can be utilized directly from their bank or pit-run condition without special processing, thereby lowering project cost. Washing of aggregates to remove fines is not always necessary. The controlling factor as to the amount of acceptable fines in a given situation is their plasticity. Some studies have shown that fines in RCC can improve both strength and compactibility (8,12). Material excavated during site preparation, which would otherwise be wasted, can often be used as RCC aggregate (Figure 1).

FIGURE 1 Three-hopper layout used to blend one site-mined sand and two imported aggregates for RCC pugmill production.

The larger rock-fill type of construction equipment allows the use of large-size aggregates. A rule of thumb for aggregate size selection is that the maximum size should not exceed one-third of the thickness of the layer of RCC being placed. Segregation problems, associated with loading, transporting, and unloading the RCC, increase with larger-size aggregate. Segregation can be reduced by partial remixing during spreading operations. This remixing is accomplished when the material is "knocked down" and contoured to the correct lift thickness (3,13,14). Rock ladders (4,15) and drop pipes (16) have been used to avoid segregation.

CONSTRUCTION

The construction process for RCC is based on a three-step cycle that results in continuous placement of the RCC mix. The cycle consists of production, transportation to the site, and spreading and compaction. Continuous placement means that one RCC layer is placed immediately on the preceding layer.

Production

Because continuous placement eliminates the delay due to curing time between layers or segments or both, as required by conventional concrete, a greater demand is placed on mixing-plant production capability, availability, and efficiency. Most concrete plants cannot produce a sufficient volume of RCC. This problem can be solved by enlarging the plant, utilizing more than one plant, or a combination of enlarging and multiple plants. The use of more than one plant provides a safety factor. If an individual plant breaks down, RCC can still be produced, ensuring that placement operations are not interrupted. With the use of holding hoppers for temporary storage, both continuous and noncontinuous batch-type plants can provide a sufficient volume of material for continuous placement.

Pugmill-type plants have been the project norm (Figure 2). Standard conventional concrete central plants have been utilized only on a limited basis because of the performance problems associated with the lower consistency characteristic of zero-slump concrete. Contractors are beginning to investigate other plants in order to be competitive by utilizing equipment already in their possession.

FIGURE 2 Continuous-mix pugmill plant mixing RCC and loading into dump trucks.

Some mixing-plant experimentation is being performed by Tyger Construction Company, Inc., on their Upper Stillwater Project. Tyger has a Noble 600 batch plant with two 8-yd³ tilt mixers and a Noble 600 modified to accept two 4-yd³ high-intensity spiral-flow mixers on the project (4).

Mix-in-place techniques are not applicable because RCC is a controlled-mix product.

Transportation

The wet RCC can be transported from the plant by conveyor system (4); large buckets, the 8-yd size that is common in conventional concrete (1); or standard earthmoving haul units (4,14,17). Buckets are hauled on flatbed trucks or railway cars to the construction site, where they are swung into position by use of a crane. The haul route can be fixed for the duration of the project because the crane provides mobility at the job site (13). On small construction projects where space is severely limited, the use of buckets or a conveyor system is advantageous (15). The Shimajigawa Dam project in Japan used 12-yd³ skip cars running down tracks laid on the right abutment (18). Even on projects of limited extent, the use of haul units may still be required because of other factors that dictate a distant batch-plant location.

On many of the projects completed to date, various earthmoving haul units have been utilized: scrapers (17), rear dump trucks (17,18), and wheel loaders (16). The number of haul units will be controlled by the batch-plant output and haul-unit cycle time. Because of the height of fall during placement, segregation sometimes occurs when rear dump trucks are used (Figure 3) (14,19). Because of the zero-slump characteristics of RCC, vibrators can be added

FIGURE 3 Euclid R-22 end dump used to transport RCC.

to rear dump trucks to smooth material flow and reduce buildup in the bed.

The utilization of scrapers provides better control of layer thickness during placement. This reduces the time necessary for the knock-down equipment to finish preparing the layer for compaction. Scrapers having positive displacement systems cause the RCC to flow from the haul unit with no substantial buildup occurring in the unit.

No impurities can be tracked onto the RCC. This requires that provisions be made for keeping haul-unit tires clean or for cleaning the tires before a machine enters the RCC area. Special haul-road surfacings, such as washed gravel or clean sand that will not adhere to tires, can be used to control surface contamination. Special wheel-cleaning spray systems have been used on two projects (20,21). Another solution is to keep the haul units on the RCC surface during loading. The Japanese did this at Shimajigawa by using skip cars to deliver the mix to the dump trucks on the dam (18). At Middle Fork Dam, a wheel loader stayed on the dam and was fed by a rock ladder initially. As the dam gained height above the pugmill elevation, feeding of the loader was handled by a radial stacker (15). A good maintenance program is also necessary in order to prevent equipment oil leaks or fuel spills from contaminating the RCC.

Travel in the RCC area should be in one direction, that is, a single entrance into and a separate exit from the area. One-directional travel across the RCC area prevents the haul units from interfering with the placement equipment. This helps to provide a continuous flow of the mix and steady production.

Spreading

Like earth embankment fills, RCC is placed at a specified layer thickness. The most common specified thickness in the United States has been 1 ft. Each layer is a continuous sheet over the total RCC area of the structure. Because the layers are placed over the total area, there are no vertical joints. In Japan, a built-up lift technique has been used in which a 1.6- to 2.3-ft lift is created by spreading several thin (6- to 8-in.) layers and then compacting the thick lift. This procedure is reported to have helped in reducing segregation (22).

It is advantageous to limit internal structural elements. One method of handling interior structural elements is to place sand in the required space during the RCC placement. Such areas can be excavated after placement of the RCC is complete. This allows interior structural elements to be built following RCC activities.

Either a wheeled or a track bulldozer or a grader or both are used to knock down and spread the RCC. Early studies recommended the use of rubber-tired equipment for three reasons (13):

1. It was thought that the RCC would build up to a greater extent on tracks than on wheels;
2. There was concern about tracks scarring the RCC surface; and
3. Wheeled tractors are generally heavier, thereby providing more initial compaction.

Recently constructed projects have utilized track-type tractors (Figure 4) for the following reasons:

1. Experience has shown that buildup on tracks does not occur;
2. Wheeled tractors can spin, causing indentions in the RCC layer and increased segregation; and
3. The benefit of initial compaction does not speed the placement operation because the spreading operation is generally the controlling laydown cycle element (23).

FIGURE 4 John Deere 450 track-type tractor used to spread dump-truck-delivered RCC.

A custom-built spreader box mounted on a heavy shovel was used to control the lift placement on the Galesville Dam project in Oregon. Dump trucks hauled the RCC and dumped the mix into the box for spreading (20).

Water trucks or hose spray systems are often employed to maintain a wet surface until the placement of another layer. A wet surface must be maintained in order to achieve bonding between layers (23).

Compaction

Vibrator compaction rollers (Figure 5) are used to achieve mix density. Because most towed vibratory rollers are designed to turn in one direction only, operation on the RCC in reverse direction will tear up the semicompacted layer and material will stick to the roller. For maneuverability, the use of a self-propelled vibratory roller allows both forward and reverse travel without adverse effects to the RCC layer (13).

FIGURE 5 Vibrator roller used to compact RCC.

CONSTRUCTION CONTROL

Control over the mixing operation of RCC is maintained by obtaining samples at set intervals from the batch plant and testing for water and cement content. At Willow Creek Dam, a set of three samples was run during each shift of operation (24).

One of the best controls is the experience of the operator as he learns to judge the consistency of the material by sight (13). In most concrete operations, complaints concern the question of adding more water, but with RCC the roller operators complain when the mix is too wet. A wet mix results in difficulty in rolling. A seemingly crude but accurate test for proper moisture content is the reaction of the vibratory roller to the RCC. An indicator of the optimum moisture content is when a wave just begins to show in front of the vibratory roller (23).

Control of compaction is maintained by in-place density tests using a nuclear density meter or by specifications that require a certain number of passes for a specified roller weight. If a nuclear device is used for density testing, the reading must be taken before the cement hydration and initial set can affect the results. The time frame for consistent results is from 30 to 40 min after mix batching (4).

One roller can usually compact for several knockdown machines. On projects completed to date, compaction time has not been critical. Control on these past projects was by a specified number of passes of a roller with a specified weight. When compaction time becomes a critical factor, nuclear density meters can be used for individual layer density control. This permits almost instant feedback as to roller effectiveness (19). Excessive rolling will add to construction cost and result in decreased dry density and strength (4).

Cores and samples can be collected for compressive strength and other tests. However, construction of the RCC will often be completed or have progressed considerably before the results of such tests are available. The RCC construction process is fast; therefore the use of cores and samples is inadequate for field monitoring. However, they do provide the historical data for evaluating the final product.

FACING FINISHES

The vertical face of the RCC can be finished in several ways. Facing systems that have been used are (a) natural, uncured slope; (b) high-strength-concrete slipformed curb; and (c) concrete panel.

Natural Facing

RCC can be compacted to within several inches of the vertical edge by use of light rollers. The top RCC layer is cured by a conventional concrete method in order to achieve maximum strength. During continuous operations, curing between layers is not necessary because each successive layer serves to retain the moisture in the previous layer; maximum strength is thereby obtained and a curing operation is eliminated. If the outside vertical face is left with no protection, a foot or so of the exposed side will lose moisture to the atmosphere. This exposed area will have a reduced strength, but the dried portion protects the interior mass of the RCC from further loss of moisture, and the strength lost at the edges does not affect the overall strength of the structure.

Curb

Low-slump, high-strength concrete can be used in slipform pavers to form curbs. The exact makeup of the curb concrete will depend on its structural purpose. If the curb's purpose is to provide both a form for the RCC mix and freeze-thaw protection, conventional concrete is appropriate (10).

The RCC can be compacted against the completed curbs. Vertical expansion joints must be included because the curbs are designed like conventional concrete. The slipforming can be a continuous operation. The slipforming machine can be guided by a wire or by laser control systems. The curbs are raised 2 to 3 ft at a time in a continuous process along the vertical edge of the RCC layers. Construction of the curb precedes the placing of the RCC layers by 1 or 2 ft vertically. The curb is usually "green" (still in the curing process) when a layer of RCC is compacted against it. This creates some bonding of the two materials and helps to form a solid structure. Curing for the curb facing and the last layer of the RCC must be provided (19).

Panel

Precast concrete panels have been used to form a vertical face. A modified version of the system patented by Reinforced Earth was used on one project. The RCC layers must gain sufficient strength to retain the preset bolts to which the framing is attached before additional panels are added. To prevent buildup of pressure behind them, the panels are not sealed (8).

COLD JOINTS

The concept of continuous placement of the RCC layers exists in theory only. Shutdowns of the RCC operation will occur, but these should be minimized. Shutdowns or delays will result in the forming of a cold joint, which occurs when there is a time lapse between the placement of layers and the layer just placed is allowed to dry out to a point where bonding with a new layer would not occur. The creation of bonding between such layers often requires a special treatment.

The treatment of cold joints between RCC layers is dependent on the condition (clean or dirty and percent water content) of the old or cold RCC surface layer. The common approach is to sandblast the old surface clean, if necessary, and then to wet the surface and apply a 3-in.-thick rich RCC layer. The rich mix has a higher water and cement content and a smaller maximum aggregate size. The 3-in. rich layer plus the next 12-in. layer of RCC are compacted as one mass. Other cold surface treatments consist of water only, dry cement only, water-cement slurry, and water-cement-sand mixtures (Figure 6). The normal continuous cycle can be resumed once this cold surface bonding is effected.

FIGURE 6 Laborer wetting a sand-cement cold-joint mixture (note limited RCC buildup in upper corner of truck bed).

CONCLUSION

Continuity and therefore efficiency are dependent on the design of the RCC construction process to produce a succession of operations. The three parts of the total cycle--production, transportation, and spreading and compaction--must complement one another and be synchronized. Project conditions will dictate specific equipment selection. Selection of the size and number of haul units is based on haul-cycle time and the volume of RCC being utilized per unit of time in the placement area. The mixing plant must be capable of producing a sufficient volume of RCC so that the haul units are not delayed in supplying the placement operation. A delay in any part of the total cycle produces a delay in all the other parts of the cycle (6). The adaption of standard construction equipment and processes to RCC construction has proved that RCC is a construction material that offers savings in both money and project duration.

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The Effects of Superplasticizers on the Engineering Properties of Plain Concrete

SEUNG BUM PARK and MANG TIA

ABSTRACT

The effects of superplasticizers on fresh and hardened concrete were investigated. The experimental program included tests on the slump and slump loss, bleeding, setting time, air content, compacting factor, Vee Bee, compressive strength, tensile and flexural strength, permeability, shrinkage, and freeze-thaw durability. Properties of superplasticized concrete were compared with those of conventional (high-slump) and base (low-slump) concretes. Superplasticizers were observed to have an appreciable fluidifying action in fresh concrete. They permitted a significant water reduction while maintaining the same workability. Bleeding of superplasticized concrete was much lower than that of conventional concrete of the same consistency. The compacting factor and Vee Bee value of superplasticized concrete were not significantly different from those of conventional concrete of the same consistency. This indicates that the use of superplasticizers did not affect the tendency of segregation of fresh concrete. The compressive, tensile, and flexural strengths of superplasticized concrete were significantly higher than those of conventional concrete. The permeability and drying shrinkage of superplasticized concrete were significantly less than those of conventional concrete, but there were no significant differences between base and superplasticized concrete. Compared with base concrete, non-air-entrained superplasticized concrete had slightly higher freeze-thaw durability, and superplasticized concrete with an appropriate amount of entrained air gave even better resistance to freezing and thawing.

Superplasticizers, which include sulfonated melamine condensates developed in West Germany in 1968, naphthalene condensates developed in Japan in the latter half of the 1960s, and modified lignosulfonated condensates, have been used to produce high-strength concrete and water-reduced concrete (1). The term "superplasticized concrete" refers to a concrete with high fluidity and favorable workability made by adding superplasticizer to normal low-consistency concrete that has a low water content without producing any detrimental effects on the concrete (2). Superplasticizers not only have higher water-reducing effects as compared with those of the conventional water-reducing admixtures (ASTM Type A), but also do not cause retardation of setting and excessive air entrainment in spite of the large amount of superplasticizer added (3).

Superplasticized concrete was developed in West Germany around 1971 for the purpose of improving concrete workability, and guidelines for production and placement were established in 1974 (4). A report on this subject was published in England by the Cement and Concrete Association and the Cement Admixture Association in 1976 (1) and proposed guidelines for design and control of superplasticizer concrete were published by the Japan Society of Civil Engineers and the Japan Society of Architecture in Japan in 1980 (5). Symposia on superplasticized concrete were held in Canada in May 1978 and in June 1981, and the proceedings were published by the American Concrete Institute (6,7). Proposed guidelines for design and control of superplasticized concrete were also published in Canada in 1981. There has been increased interest in the use of superplasticized concrete for improved quality and workability due to the wide use of the concrete pump. The use of lower-quality aggregates also requires a higher-quality cement paste to produce concrete of suitable strength. Hence, further investigation in the production and practice of superplasticized concretes is needed for their effective application. The effects of four standard types of superplasticizers and an air-entraining (AE) water-reducing admixture on the flow properties of fresh concrete and on the strength, permeability, drying shrinkage, and durability of hardened concrete are presented. The properties of superplasticized concrete are compared with those of conventional high-slump concrete and base (low-slump) concrete.

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MATERIALS AND TEST METHODS

Materials

Cement

Normal Type I portland cement was used in this study. The properties of the cement are shown in Table 1.

Aggregates

River gravel of 25-mm (1-in.) maximum size was used as coarse aggregate and river sand was used as fine aggregate. The aggregates consisted mainly of granite. Both were used in saturated-surface dry condition. The gradation of these aggregates is shown in

TABLE 1 Properties of Type I Portland Cement

Property	Amount
Specific gravity	3.15
Specific surface area (cm^2/g)	3,250
Residue of 88 μ (%)	1.8
Soundness autoclave expansion (%)	0.17
Time of setting (hr:min)	
Initial	3:50
Final	6:30
Compressive strength (kg/cm^2)	
At 3 days	178
At 7 days	245
At 28 days	331
Composition (%)	
CaO	61.1
SiO	21.2
Al_2O_3	5.5
Fe_2O_3	3.2
MgO	3.1
SO_3	2.3
Insoluble	0.1
Ignition	2.1

Note: 1 kg/cm^2 = 14.23 psi.

Figure 1 and their physical properties are shown in Table 2.

Admixtures

Four superplasticizers were used in this study. They included the sulfonated naphthalene condensates (NP-10), the sulfonated melamine condensates (NP-20), and the combined sulfonated naphthalene and sulfonated lignin condensates (Sanflo FB and Sanflo FBF). An ASTM Type-D water-reducing AE admixture (Sanflo K) that contains mainly sulfonated naphthalene and modified lignin condensates was also used for comparison purposes. The properties of these admixtures are given in Table 3.

Concrete Mixes

A total of 13 mixes were used in this study. They included a conventional high-slump concrete, a base (low-slump) concrete, eight non-air-entrained superplasticized concretes (using the four superplasticizers and two levels of dosage), two air-entrained superplasticized concretes (using the superplasticizer Sanflo FBF and the AE water-reducing admixture Sanflo K, and the two levels of dosage), and an air-entrained concrete (using the AE water-reducing admixture, Sanflo K). The proportions of these 13 mixes are shown in Table 4. A fixed cement content of 350 kg/m^3 (590.6 lb/yd^3) and a fixed sand-aggregate ratio of 43 percent were used for all the mixes. The base concrete was made to have a target slump of 12 cm (4.7 in.) and the rest of the mixes were made to have a target slump of 18 cm (7.1 in.). The two levels of dosage of superplasticizer used were 0.4 and 1.0 percent by weight of cement. For the air-entrained concretes, 0.25 percent (by weight of cement) of the AE water-reducing admixture, Sanflo K, was used.

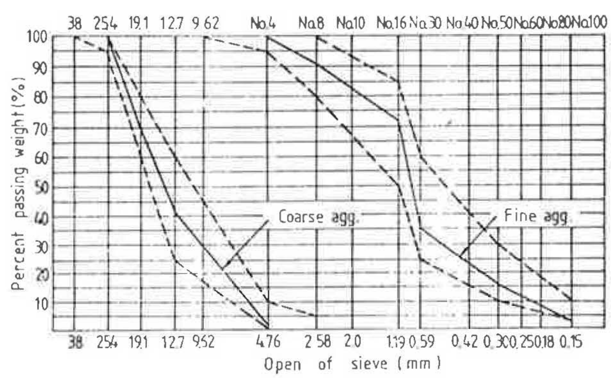


FIGURE 1 Gradation curves of fine and coarse aggregate.

Testing Methods

Concrete Mixing

The water requirements for the concrete mixes were determined by trial mixes to obtain the target slump of 12 cm for the base concrete and 18 cm for the other 12 mixes. A tilting mixer of 54-L (1.9-ft^3) capacity was used. The base concrete was made by a 3-min mixing. In accordance with the recommendations of the manufacturers, superplasticizers of 0.4 or 1.0 percent (by weight of cement) were added to the base concretes and mixed again for 1 min to make the non-air-entrained superplasticized concretes. The superplasticizer Sanflo FBF (0.4 or 1.0 percent by weight of cement) and the ASTM Type-D AE water-reducing admixture Sanflo K (0.25 percent by weight of cement) were mixed and added at the same time to make the air-entrained superplasticized concretes. The AE concrete (Mix 13) was made by adding 0.25 percent (by weight of cement) of Sanflo K to a high-consistency concrete. Concretes were tested for slump and air content immediately after mixing. The temperature of the laboratory and concretes was in the range of 20° to 25°C (68° to 77°F).

Slump Test

Slump in conventional high-consistency concrete, base concrete, and superplasticized concrete was measured every 30 min up to 120 min after mixing to study the change of slump with time.

Air-Content Test

The air content of freshly mixed concrete was measured by the pressure method according to ASTM C231-82.

TABLE 2 Physical Properties of Aggregates

Type of Aggregate	Specific Gravity	Absorption (%)	Fineness Modulus	Unit Weight (kg/m^3)	Degradation Loss (%)	Sodium Sulfate Soundness Loss (%)	Organic Purity
Fine	2.57	1.01	2.82	1630	–	0.14	Good
Coarse	2.63	1.39	6.61	1680	14.2	0.20	Good

Note: 1 kg/m^3 = 0.0625 lb/ft^3.

TABLE 3 Physical Properties of Admixtures

Admixture	Appearance	Specific Gravity	pH	Viscosity[a] (cp)	Total Solids[b] (%)
NP-10	Dark brown liquid	1.18	8	20	37-43
NP-20	Dark brown liquid	1.13	7	10	38-42
Sanflo FB	Brown liquid	1.20	9	25	37-40
Sanflo FBF	Light brown liquid	1.14	9	–	33-40
Sanflo K	Brown liquid	1.21	9	–	36-41

Note: Testing temperature = 20°C (68°F).

[a]Approximate.

[b]ASTM C 494.

Setting-Time Test

Setting time was measured by means of the Proctor penetration-resistance test according to ASTM C803-82. Times of initial set and final set are defined as the times at which the penetration resistances are 35 kg/cm² (500 psi) and 280 kg/cm² (4,000 psi), respectively.

Bleeding, Compacting Factor (CF), and Vee Bee Consistency (VB) Tests

The bleeding test was done with the cylindrical container having an inside diameter of 25 cm (9.84 in.) and an inside height of 28 cm (11.02 in.) and made of metal in accordance with ASTM C232-71, Method A. The amount of bleeding water was expressed as a percentage of the net mixing water contained within the test specimen. Compacting factor and Vee Bee tests were done according to British Standard 1881, Part 2 (9).

Compressive, Tensile, and Flexural Strength Tests

Compressive, tensile, and flexural strength tests were conducted according to ASTM C39-83, C496-71, and C293-79 test methods. For flexural strength tests, specimens of 15 x 15 x 55 cm (5.9 x 5.9 x 22 in.) were molded and tested at ages of 7 and 28 days. Three replicate samples were tested for each mix combination.

Permeability Test

The permeability test was done with an output-pressure type of tester. Cylindrical specimens of 15 cm (5.9 in.) in diameter and 30 cm (11.8 in.) in height having a center hole of 2 cm diameter were used. The coefficient of permeability was calculated by using the following equation:

$$K = [\rho \log(r_o/r_i)/2\pi h] \cdot [Q/(P_o - P_i)]$$

where

K = coefficient of permeability,
Q = quantity of water flow,
P_o, P_i = external and internal water pressure of specimen,
r_o = radius of specimen, and
r_i = radius of center hole.

Drying Shrinkage Test

According to the ASTM C157 testing method, specimens of 10 x 10 x 40 cm (3.94 x 3.94 x 15.75 in.) were made and the change in length was measured at an accuracy of 1/1,000 mm (4 x 10^{-5} in.) by the use of a comparator with a microscopic readout at ages of 7, 28, 60, 91, and 180 days. A small deviation from the ASTM method was that shrinkage was presented in decimals rather than in percentages.

Freezing-and-Thawing Test

In accordance with ASTM C666-84, Procedure A, Test Method for Resistance of Concrete to Rapid Freezing and Thawing, specimens of 7.62 x 7.62 x 35.56 cm (3 x 3 x 14 in.) were made and the relative dynamic modulus of elasticity was measured at 20-cycle intervals. The relative dynamic modulus is defined as the ratio of the retained dynamic modulus to the initial dynamic modulus, expressed as a percentage. One cycle of freezing and thawing takes 3.5 to 4.0 hr and the range of temperature was -18° to 4°C (0° to 39°F). The test was continued until the relative dynamic modulus of elasticity reached 60 percent.

RESULTS AND DISCUSSION

Physical Properties of Fresh Concrete

Slump and Air Content

Slump and air content of fresh concrete after mixing are shown in Table 4. It may be noted that the air

TABLE 4 Mix Proportions of Concrete

Mix No.	Type of Concrete	Dosage of Superplasticizer[a] (%)	AE Water-Reducing Agent (%)	S/A[b] (%)	W/C[c] (%)	Unit Weight (kg/m³) Water	Cement	Sand	Gravel	Slump (cm)	Air (%)
1	Conventional	–	–	43	53.7	188	350	775	1051	18.4	1.1
2	Base	–	–	43	50.3	176	350	788	1069	12.1	1.3
3	Sanflo FBF	0.4	–	43	49.7	174	350	790	1072	19.5	1.2
4	Sanflo FBF	1.0	–	43	47.4	166	350	799	1084	18.6	1.4
5	Sanflo FB	0.4	–	43	50.0	175	350	789	1070	19.9	1.0
6	Sanflo FB	1.0	–	43	47.7	167	350	798	1082	18.1	1.3
7	NP-10	0.4	–	43	49.1	172	350	792	1075	17.7	1.1
8	NP-10	1.0	–	43	48.3	169	350	796	1079	17.4	1.2
9	NP-20	0.4	–	43	49.7	174	350	790	1072	18.5	1.0
10	NP-20	1.0	–	43	48.0	168	350	797	1081	19.1	1.4
11	Sanflo FBF and	0.4	0.25	43	48.6	170	350	750	1018	18.5	4.3
12	Sanflo K	1.0	0.25	43	46.0	161	350	760	1031	19.1	4.7
13	Sanflo K	–	0.25	43	50.9	178	350	749	1004	18.7	4.1

Note: 1 kg/m³ = 0.0625 lb/ft³; 1 cm = 0.3937 in.

[a]Amount of admixture solution expressed as a percentage by weight of cement.

[b]Sand-aggregate ratio.

[c]Water-cement ratio.

content of all the non-air-entrained concretes is around 1 percent, whereas that of the air-entrained concretes is around 4.5 percent. The added superplasticizers did not have any significant effects on the air contents of the concretes. The initial slumps were close to the target slumps for all the 13 mixes.

The water content of all 13 mixes is also shown in Table 4. It may be seen that when the dosage of superplasticizer was 0.4 and 1.0 percent, the average water reduction was 8 and 12 percent, respectively. The water reduction was within the expected range for the types of superplasticizers used. When the AE water-reducing admixture was used at the dosage of 0.25 percent (for Mix 13), water reduction was only about 6 percent. Therefore, the superplasticizers were noted to have excellent water reduction efficiency.

Change of slump with respect to elapsed time for dosages of 0.4 and 1.0 percent of superplasticizers is shown in Figures 2 and 3, respectively. The rate of slump loss for the concretes with a dosage of 1.0 percent is significantly higher than that for the concretes with a dosage of 0.4 percent. The results are similar to the test results of Mailvaganam (10) and Murray and Lynn (11), which indicated that slump loss was more rapid when the dosage of superplasticizer was higher.

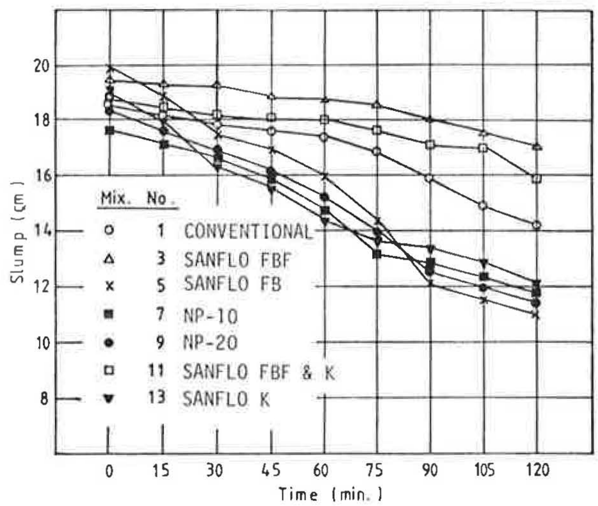


FIGURE 2 Change in slump by time elapsed at 0.4 percent dosage of superplasticizers.

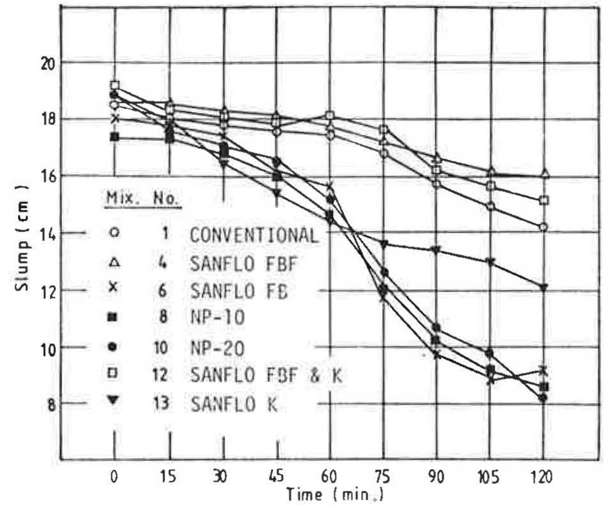


FIGURE 3 Change in slump by time elapsed at 1.0 percent dosage of superplasticizers.

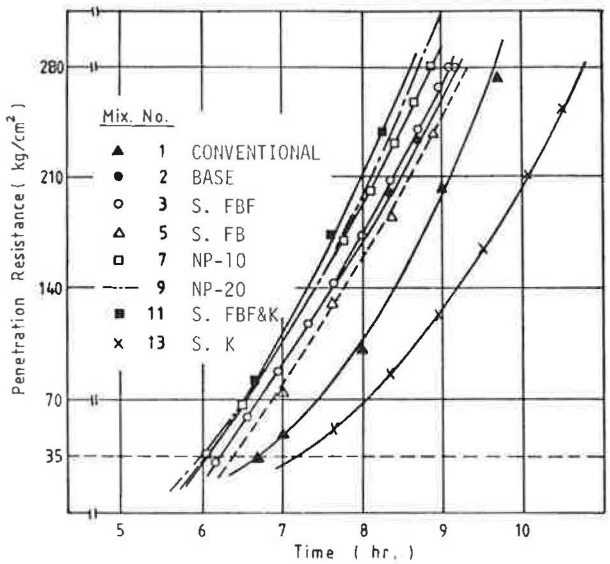


FIGURE 4 Comparison of setting time at 0.4 percent dosage of superplasticizers.

Setting Time

Figures 4 and 5 show test results for setting time by Proctor penetration-resistance tester for concretes at dosages of 0.4 and 1.0 percent of superplasticizers, respectively. No noticeable delay of set with increased dosage of superplasticizer was found, although noticeable delay of set was found in the concrete containing AE water-reducing admixture (Mix 13). Therefore, when hydration delay and strength decrease caused by delay of cement hydration are considered, concrete with superplasticizers will be more desirable than concrete with water-reducing admixtures.

Bleeding

Test results for bleeding of concretes at dosages of 0.4 and 1.0 percent of superplasticizers are shown in Figures 6 and 7, respectively. It can be seen that bleeding of the superplasticized concretes was significantly lower than that of conventional high-consistency concrete of the same slump and that bleeding decreased as the dosage of superplasticizer increased. It can also be noted from results of Mixes 11 and 12 that the addition of AE water-reducing admixture further reduced the bleeding of fresh concrete.

A lower bleeding value generally indicates a lower amount of settlement or subsiding of concrete after placement. Thus, the bleeding results indicate that superplasticizers may be used to reduce subsiding of concrete (Table 5).

CF- and VB-Values

Table 6 gives the CF- and VB-values of all 13 concrete mixes. It can be noted that the CF-values of the superplasticized concretes are not significantly different from those of the conventional high-consistency concrete (Mix 1) and the concrete with AE water-reducing admixture (Mix 13) but are much higher

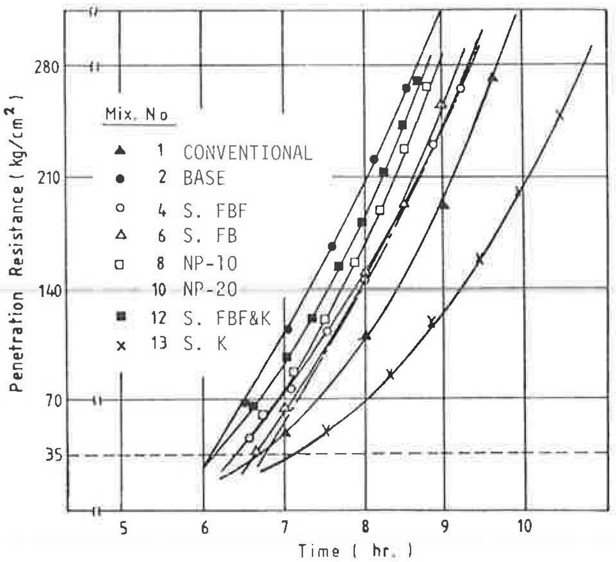


FIGURE 5 Comparison of setting time at 1.0 percent dosage of superplasticizers.

than that of the base concrete (Mix 2). The VB-values of the superplasticized concretes are not significantly different from those of the conventional high-consistency concrete and the concrete with AE water-reducing admixture but are much lower than that of the base concrete. A higher CF-value generally indicates a higher tendency for segregation, whereas a lower VB-value generally indicates a higher tendency for segregation. However, no apparent

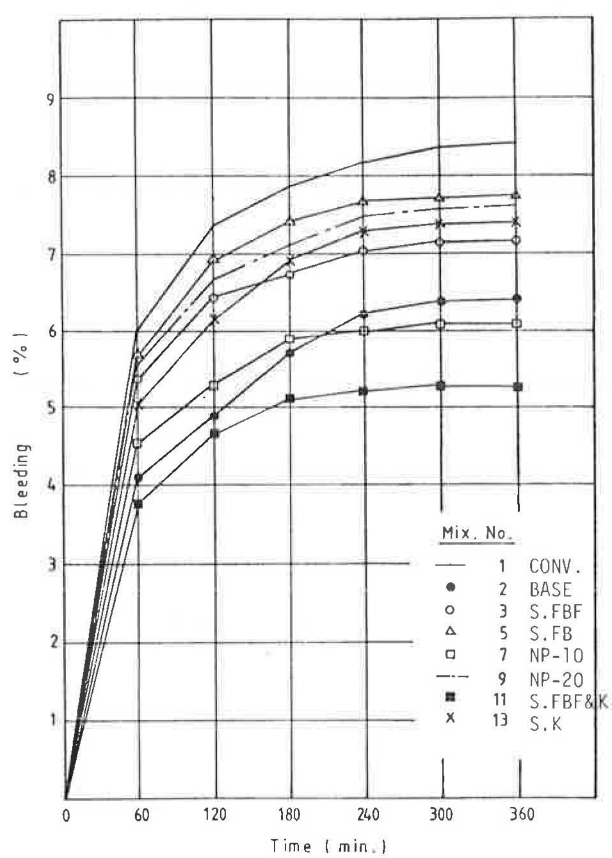


FIGURE 6 Comparison of bleeding ratio at 0.4 percent dosage of superplasticizers.

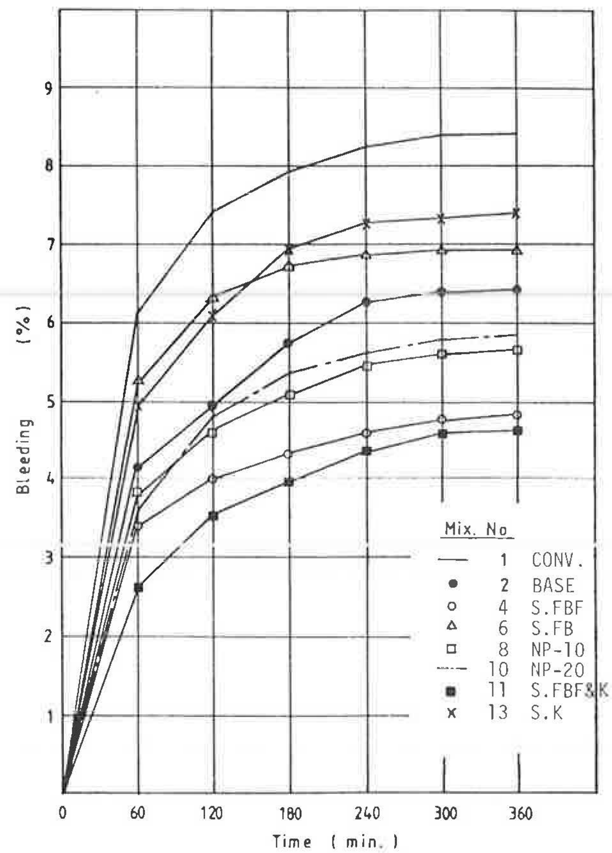


FIGURE 7 Comparison of bleeding ratio at 1.0 percent dosage of superplasticizers.

segregation of fresh concrete was observed for any of the mixes in this study. As the quantity of superplasticizer increased from 0.4 to 1.0 percent, the CF-value showed a slight reduction whereas the VB-value showed a slight increase. This indicated that an increase in dosage of superplasticizer did not affect the tendency of segregation. However, results by Moon (12) indicate that an excessive dosage of superplasticizer causes segregation of fresh concrete. Therefore, the dosage of superplasticizer should be within the range where segregation would not be created.

Physical Properties of Hardened Superplasticized Concrete

Compressive Strength

Table 6 shows the test results of compressive strength in standard curing conditions at 7 and 28 days. The compressive strengths are compared and displayed in Figures 8 and 9. The compressive strength of the superplasticized concretes was significantly higher than that of the conventional concrete. At 7 days, the compressive strength of superplasticized concrete was nearly the same or somewhat higher than that of the base concrete. At 28 days, the compressive strength of superplasticized concretes was about 4 percent higher than that of the base concrete. This result is similar to the test results of Roberts, which indicated that the use of superplasticizer can significantly increase the strength of concrete (13). The difference in compressive strength between superplasticized concrete and base concrete was increased by an increased dosage of superplasticizer. The superplasticized

TABLE 5 Test Results for Concrete-Bleeding Tests

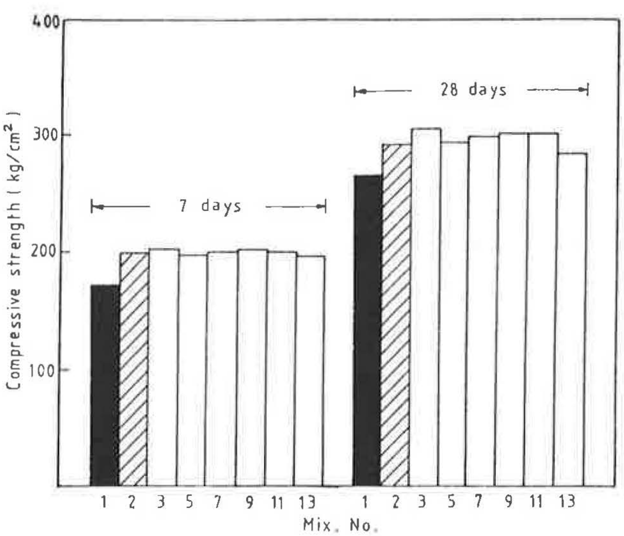
Mix No.	Bleeding (%) by Time (min) 10	20	30	40	50	60	120	180	240	300	360
1	3.15	3.90	4.46	5.07	5.62	6.05	7.43	7.89	8.21	8.39	8.41
2	2.17	2.46	3.13	3.59	3.87	4.13	4.92	5.78	6.34	6.35	6.39
3	2.46	3.19	3.84	4.66	5.05	5.48	6.42	6.77	7.08	7.12	7.14
4	0.94	1.49	2.13	2.67	3.01	3.36	4.05	4.11	4.60	4.78	4.81
5	2.59	3.32	4.06	4.79	5.27	5.75	6.89	7.42	7.59	7.65	7.69
6	1.16	1.72	2.26	2.86	3.12	5.19	6.32	6.74	6.82	6.87	6.91
7	2.08	2.50	3.37	3.78	4.15	4.67	5.36	5.82	6.05	6.07	6.11
8	1.23	1.87	2.44	2.92	3.24	3.86	4.58	5.04	5.48	5.52	5.68
9	2.47	3.26	4.02	4.55	5.13	5.57	6.64	7.09	7.47	7.55	7.59
10	1.29	1.92	2.49	3.07	3.31	3.63	4.76	5.32	5.61	5.79	5.83
11	1.94	2.31	2.78	3.18	3.45	3.81	4.67	5.06	5.16	5.23	5.28
12	0.89	1.07	1.45	2.05	2.31	2.64	3.51	3.98	4.35	4.60	4.63
13	2.16	2.89	3.52	4.37	4.85	5.08	6.14	6.96	7.28	7.31	7.37

TABLE 6 Test Results for Concrete: Other Tests

Mix No.	CF	VB (sec)	Setting Time (hr:min) Initial	Final	Compressive Strength (kg/cm²) 7 Days	28 Days	Tensile Strength (kg/cm²) 7 Days	28 Days	Flexural Strength (kg/cm²) 7 Days	28 Days	K[a]
1	0.963	2.4	6:40	9:40	172	268	16	24	32	44	32.70
2	0.930	4.5	6:05	8:40	201	295	19	29	40	50	14.62
3	0.980	2.1	6:15	9:05	203	309	20	31	41	52	12.15
4	0.970	2.5	6:25	9:20	205	318	23	33	42	54	11.65
5	0.976	2.0	6:20	9:15	200	296	19	28	39	49	13.08
6	0.968	2.7	6:40	9:10	203	307	20	30	40	50	12.84
7	0.965	1.9	6:05	8:50	201	302	20	29	39	51	12.46
8	0.950	2.4	6:35	8:55	204	309	22	30	43	53	11.23
9	0.967	1.8	6:15	9:05	202	306	19	30	42	51	12.30
10	0.960	2.5	6:45	9:20	206	310	22	33	43	54	11.57
11	0.981	1.9	6:05	8:35	201	305	21	20	40	53	13.05
12	0.970	2.1	6:15	8:50	207	316	23	32	42	55	12.06
13	0.985	2.0	7:10	10:45	198	287	18	26	37	48	12.80

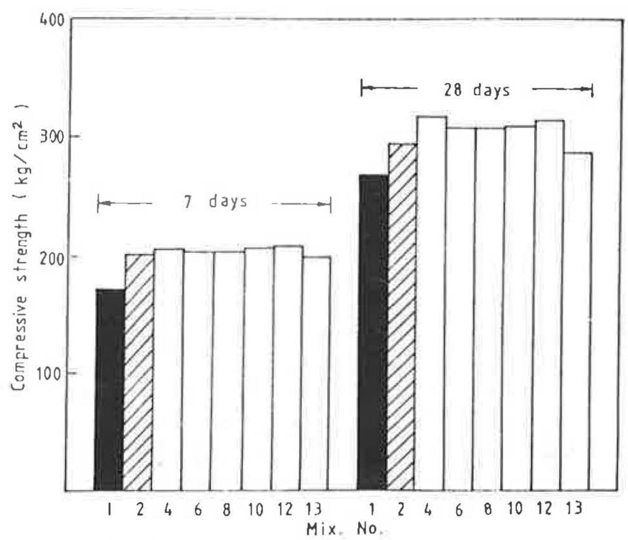
Note: 1 kg/cm^2 = 14.23 psi.

[a]K = coefficient of permeability (unit x 10^{-3} cm/sec).



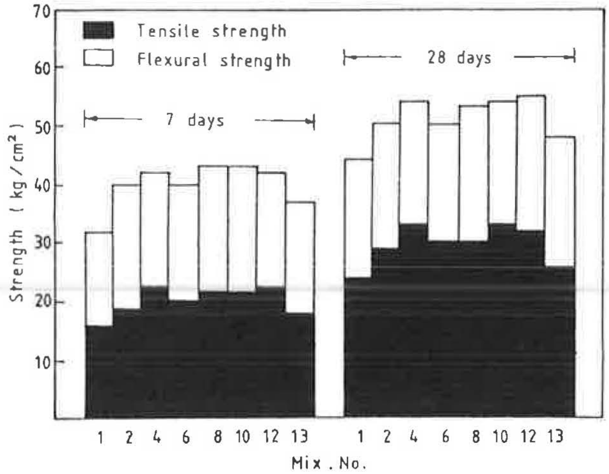
MIX NO.	TYPE OF CONCRETE
1	CONVENTIONAL
2	BASE
3	SANFLO FBF
5	SANFLO FB
7	NP-10
9	NP-20
11	SANFLO FBF & SANFLO K
13	SANFLO K

FIGURE 8 Comparison of compressive strength at 0.4 percent dosage of superplasticizers.



MIX NO.	TYPE OF CONCRETE
1	CONVENTIONAL
2	BASE
3	SANFLO FBF
5	SANFLO FB
7	NP-10
9	NP-20
11	SANFLO FBF & SANFLO K
13	SANFLO K

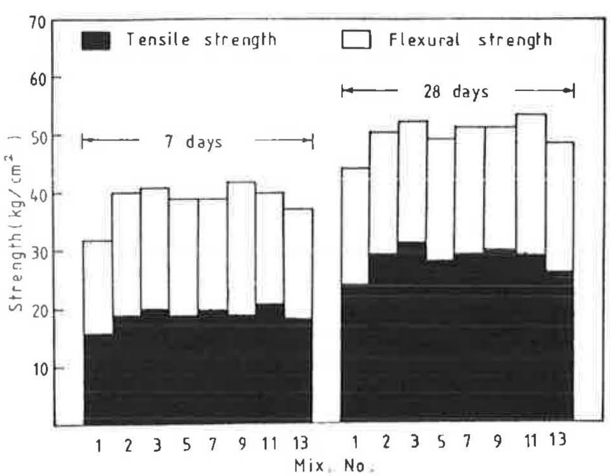
FIGURE 9 Comparison of compressive strength at 1.0 percent of superplasticizers.



MIX NO.	TYPE OF CONCRETE
1	CONVENTIONAL
2	BASE
4	SANFLO FBF
6	SANFLO FB
8	NP-10
10	NP-20
12	SANFLO FBF & SANFLO K
13	SANFLO K

FIGURE 10 Comparison of tensile and flexural strength at 0.4 percent dosage of superplasticizers.

concretes also showed a higher strength as compared with that of the concrete with water-reducing AE admixture (Mix 13). Therefore, it can be concluded that using superplasticizers can increase the workability by improving the flow properties without reducing the strength of the base concrete.



MIX NO.	TYPE OF CONCRETE
1	CONVENTIONAL
2	BASE
3	SANFLO FBF
5	SANFLO FB
7	NP-10
9	NP-20
11	SANFLO FBF & SANFLO K
13	SANFLO K

FIGURE 11 Comparison of tensile and flexural strength at 1.0 percent dosage of superplasticizers.

Tensile and Flexural Strength

Tensile and flexural strength at 7 and 28 days are shown in Table 6 and Figures 10 and 11. The superplasticized concretes had significantly higher tensile and compressive strengths than those of the conventional concrete. No significant difference in tensile and flexural strength between the base concrete and the superplasticized concretes was noted at 7 days. However, the 28-day strength of superplasticized concretes was about 5 percent higher than that of the base concrete.

Drying Shrinkage and Permeability

Figures 12 and 13 show the drying shrinkage of the concretes at dosages of 0.4 and 1.0 percent of superplasticizers, respectively. It may be noted that the drying shrinkage of the superplasticized concretes decreased slightly as the dosage of superplasticizer increased from 0.4 to 1.0 percent. The drying shrinkage of the superplasticized concretes was slightly lower than that of the conventional high-consistency concrete but was about the same as that of the base concrete and that of the concrete

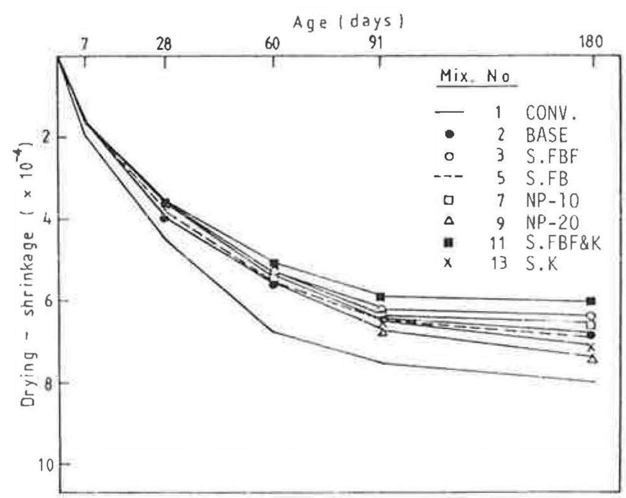


FIGURE 12 Comparison of drying shrinkage at 0.4 percent dosage of superplasticizers.

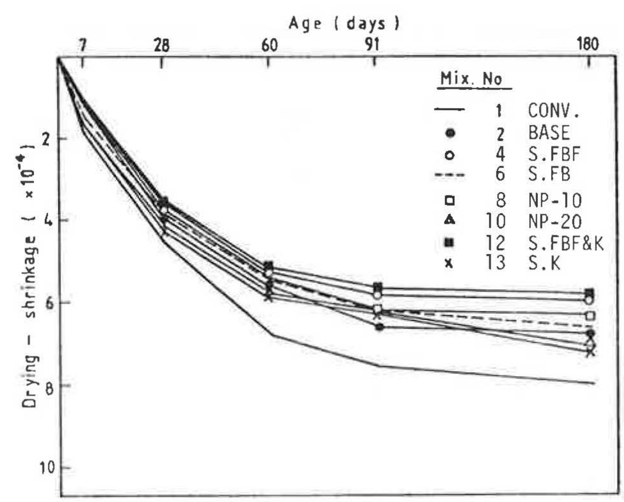


FIGURE 13 Comparison of drying shrinkage at 1.0 percent dosage of superplasticizers.

TABLE 7 Test Results of Drying Shrinkage

	Length Change (10^{-4} cm/cm)				
Mix No.	7 Days	28 Days	60 Days	91 Days	180 Days
1	1.98	4.54	6.86	7.50	7.91
2	1.53	3.92	5.54	6.31	6.81
3	1.42	3.85	5.36	6.07	6.48
4	1.31	3.61	5.12	5.83	5.95
5	1.57	3.97	5.68	6.49	7.01
6	1.45	3.85	5.47	6.23	6.54
7	1.51	3.89	5.49	6.24	6.56
8	1.36	3.74	5.36	6.12	6.27
9	1.57	4.13	5.80	6.58	7.42
10	1.40	4.02	5.53	6.31	6.90
11	1.31	3.76	5.28	5.99	6.02
12	1.28	3.57	5.06	5.76	5.81
13	1.59	4.04	5.57	6.28	7.04

with AE water-reducing admixture. By a close examination of the mix proportions of these concretes (shown in Table 4), it may be noted that drying shrinkage generally increased as the water content increased (Table 7). When two concretes have about the same water content, their drying shrinkage would be about the same, regardless of the dosage or type of superplasticizer used.

As shown in Table 6, the permeability coefficient of superplasticized concretes was about 10 percent less than that of the conventional high-consistency concrete and decreased with increased dosage of plasticizer. The permeability coefficient of superplasticized concretes was not significantly different from that of the base concrete or that of the concrete with AE water-reducing admixture.

Freezing-and-Thawing Resistance

Table 8 shows the freezing-and-thawing-test results of the 13 concrete mixes used in this study. Figures 14 and 15 present the relative dynamic elastic moduli as functions of freezing-and-thawing cycles for concretes with 0.4 and 1.0 percent dosage of superplasticizers, respectively. The relative dynamic modulus of elasticity of the superplasticized concretes increased as the dosage of superplasticizer increased. As compared with the non-air-entrained concretes, the air-entrained concretes showed higher relative dynamic modulus at the same number of freezing-and-thawing cycles. The results of the freezing-and-

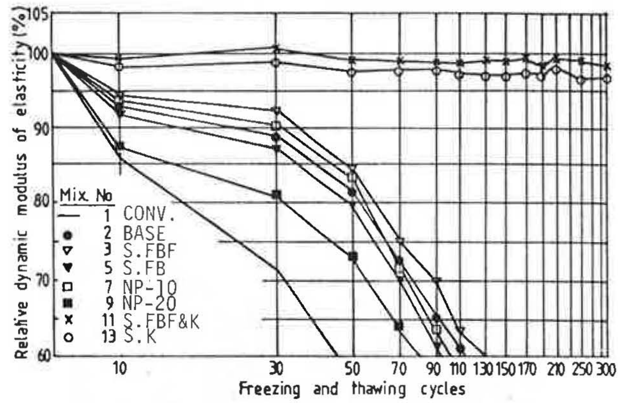


FIGURE 14 Resistance of concrete to rapid freezing and thawing at 0.4 percent dosage of superplasticizers.

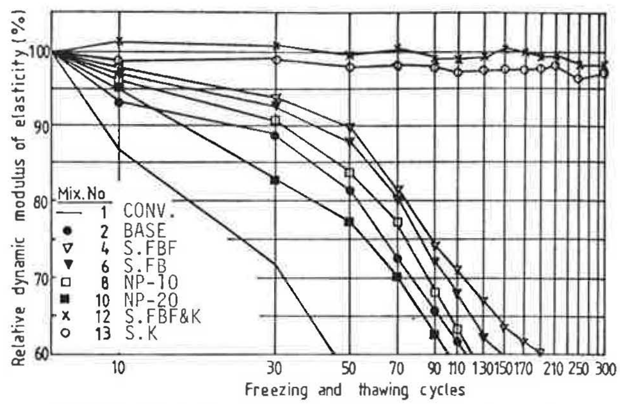


FIGURE 15 Resistance of concrete to rapid freezing and thawing at 1.0 percent dosage of superplasticizers.

thawing tests indicated that superplasticizers can improve the durability of concrete, whereas an AE admixture along with superplasticizers can further improve the durability of concrete.

CONCLUSION

This study investigated the fundamental engineering properties of superplasticized concrete, including

TABLE 8 Test Results on the Resistance of Concrete to Rapid Freezing and Thawing

Mix No.	Dynamic Modulus[a] of Elasticity (kg/cm²)	Final Cycle No.	Relative Dynamic Modulus[b] of Elasticity (%)	DF (%)	Weight[a] (kg)	Decreased Ratio of Weight (%)
1	3.561×10^5	45	60	9	4.817	−8.7
2	3.718×10^5	117	60	23	4.858	−5.7
3	4.162×10^5	132	60	26	4.865	−4.9
4	4.369×10^5	189	60	38	4.891	−2.4
5	3.605×10^5	93	60	19	4.860	−5.0
6	4.184×10^5	147	60	29	4.867	−3.1
7	3.798×10^5	101	60	20	4.871	−5.8
8	4.076×10^5	123	60	25	4.881	−5.2
9	3.621×10^5	77	60	15	4.865	−7.4
10	3.983×10^5	98	60	20	4.885	−6.3
11	4.158×10^5	300	99.6	100	4.662	−1.3
12	4.217×10^5	300	99.8	100	4.691	−0.8
13	3.605×10^5	300	98.7	99	4.629	−1.5

Note: 1 kg/cm² = 14.23 psi; 1 kg = 2.2 lb.

[a]Measured before test.

[b]Measured at end of cycle.

flow; compressive, tensile, and flexural strength; drying shrinkage; permeability; and freezing and thawing resistance.

The results obtained in this study are summarized as follows:

1. The use of superplasticizer can significantly reduce the water requirement for workability in concrete. The amount of water reduction increases with the dosage of superplasticizer. However, slump loss with elapsed time is more rapid when the dosage of superplasticizer is higher.

2. An increased dosage of superplasticizer has no effect on setting time. However, an increase in dosage of a lignin-base AE water-reducing admixture can cause significant retardation of setting time. Therefore, use of superplasticizer rather than AE water-reducing admixture will be more desirable in terms of setting and hardening of concrete.

3. Bleeding of superplasticized concrete is significantly lower than that of conventional high-consistency concrete of the same slump. This indicates that superplasticized concrete has a lower tendency for settlement.

4. CF- and VB-values of the superplasticized concrete are not significantly different from those of conventional concrete of the same consistency. This indicates that the tendency for segregation of fresh concrete is not affected by the superplasticizers.

5. Compressive, tensile, and flexural strengths of superplasticized concrete are much higher than those of conventional concrete of the same consistency.

6. Drying shrinkage of superplasticized concrete is less than that of conventional high-consistency concrete and similar to that of base concrete and concrete using a lignin-base AE water-reducing admixture.

7. The permeability coefficient of superplasticized concrete is about 10 percent less than that of base concrete and much less than that of conventional high-consistency concrete. Therefore, it should be desirable to use superplasticizer in the production of watertight concrete.

8. Freezing-and-thawing resistance of superplasticized concrete increases slightly with increase in dosage of superplasticizer. The use of an AE admixture in superplasticized concrete can greatly increase the freezing-and-thawing resistance. Therefore, using superplasticizer together with an AE admixture will produce superplasticized concrete with sufficient flowability and good durability.

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Dowel Placement Tolerances for Concrete Pavements

SHIRAZ D. TAYABJI

ABSTRACT

The results of an investigation conducted to develop placement tolerances for dowels at concrete pavement joints are presented. A theoretical analysis of dowel misalignment was attempted. The purpose of the analysis was to compute restraint stresses induced in the concrete pavement for different levels of dowel misalignment. However, because of the complexity of correctly incorporating the three-dimensional nature of dowel misalignment, the theoretical analysis was not completed. The effect of dowel misalignment was then investigated in the laboratory by conducting pullout tests on sections of concrete slabs incorporating a joint and dowels with different levels of misalignment. Test results are presented that indicate that pullout loads were relatively low for dowel misalignment levels of less than 1 in. per 18-in. length of dowel bar and a maximum joint opening of 0.25 in.

Joints are provided in concrete pavements to control transverse and longitudinal cracking that results from restrained deformations caused by moisture and temperature variations in the slab. Because joints create a discontinuity in the pavement, use of joints may reduce load-carrying capacity of the pavement at the joint. To ensure adequate load transfer, load-transfer devices are used at joints by many highway agencies.

Current practice for load-transfer devices at joints has evolved over a period of time. Some of the systems used have included the I-beam, Starlug, two-component devices, and round steel dowel bars. Today, round steel dowel bars are the most widely used. Current recommended practice for doweled joints is for dowel diameters to be one-eighth of slab thickness, dowel spacing to be 12 in., and dowel length to be 18 in.

Dowel bars require care in placement to minimize detrimental effects of misalignment. It is generally specified that dowels be placed as much as is practical parallel to the longitudinal axis and the horizontal plane of the pavement. Generally, limits on permissible tolerances are specified individually by state highway agencies. The different categories of dowel misalignment and their possible effects on pavement behavior are illustrated in Figure 1.

Before December 1980, FHWA specified limits on dowel placement (1). However, the current FHWA Technical Advisory T5140.18 of December 15, 1980, on rigid pavement joints does not specify limits on misalignment but cautions that "close tolerances for dowel placement are extremely important for proper functioning of the slab and for long-term performance" (2). This advisory also states that "care must be exercised in both specifying dowel placement tolerance and in evaluating the adequacy of construction placement" (2).

In the past, an alignment error of 1/4 in. per 18-in. length of dowel has been considered acceptable. However, many state highway agencies specify different permissible levels of misalignment. For example, the Illinois Department of Transportation specifies in their Standard Specifications for Road and Bridge Construction, dated October 1979, that any deviation from correct alignment greater than 1/8 in. in 12 in. should be corrected before any concrete is placed. The Georgia Department of Transportation specifies an allowable tolerance of 3/8 in. per foot in both the horizontal and vertical directions.

No clear consensus exists as to the level of practical limits on dowel placement tolerances. When limits are specified, contractors often state that they are neither practical nor realistic. In addition, it is a slow process to determine levels of misalignment once the pavement has been constructed. Attempts have been made to measure levels of misalignment by using radar devices or by using a pachometer and taking partial-depth or full-depth cores near the ends of the dowel.

The primary reason for placing limits on dowel placement tolerance is to minimize problems associated with locked joints. Pavement slabs should be free to expand and contract with changes in slab temperature and moisture. Resistance to movement is provided by subbase friction and locked joints. For slabs up to 40 ft, resistance due to subbase friction is not as significant.

The magnitude of restraint due to locked joints depends on the degree of dowel misalignment, number of misaligned dowels, and dowel corrosion. Locked joints may result in transverse cracking, corner breaks, and spalling at the concrete face around the dowel. Once a spall occurs around a dowel, load-transfer effectiveness of the dowel may decrease.

STUDY OBJECTIVES

The investigation reported in this paper was undertaken to study the effects of dowel misalignment on pavement performance. Specific objectives were as follows:

1. To use analytical methods to perform stress analysis of the joint system incorporating dowels with different levels of misalignment,
2. To conduct laboratory tests to determine the effect of dowel misalignment, and
3. To select placement tolerance criteria based on study results.

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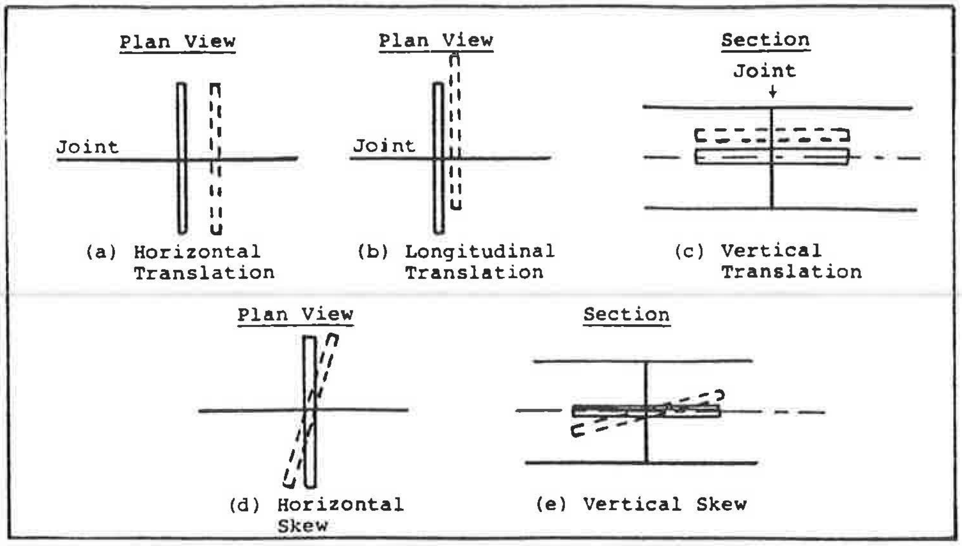

Type of Misalignment	Effect on		
	Spalling	Cracking	Load Transfer
a	-	-	(Yes)*
b	-	-	(Yes)*
c	Yes	-	(Yes)*
d	Yes	Yes	Yes
e	Yes	Yes	Yes

*Effect will depend on amount of translation

FIGURE 1 Effects of dowel misalignment.

FACTORS AFFECTING DOWEL MISALIGNMENT

The following factors affect level of dowel misalignment when basket assemblies are used:

1. Basket rigidity,
2. Quality control during basket fabrication,
3. Care during basket transportation and placement,
4. Fastening of basket to subbase,
5. Location of saw cut over basket,
6. Paving operation (the large roll of concrete ahead of the paver may displace individual dowels or the basket assembly), and
7. Field inspection during construction.

The following factors affect level of dowel misalignment when dowels are implanted:

1. Implanting machine operation,
2. Strike-off after dowel placement,
3. Consolidation (vibration) after dowel placement,
4. Location of saw cut over implanted dowels, and
5. Field inspection during construction.

For basket assemblies, basket rigidity and proper fastening of the basket assembly to the subbase are critical. Even a small movement or rotation of the basket assembly during the paving operation is sufficient to cause noncompliance of dowel placement.

For implanted dowels, different paving sequences have been used to achieve proper placement of dowel bars. Some paving sequences used strike-off and concrete consolidation (internal vibration) operations following dowel placement. In other paving operations, concrete consolidation after dowel placement was not used (3). However, degree of compliance with allowable dowel placement tolerances has been reported as unsatisfactory for all these procedures (3).

The amount of misalignment that can be tolerated greatly depends on joint spacing and climate. Greater misalignment can be tolerated if the need for joint movement (opening) is not large. The magnitude of restraint due to locked joints depends on the degree of load-transfer device misalignment as well as dowel corrosion. As indicated in Figure 1, excessive restraint to slab movement may result in transverse and corner cracking and spalling at the concrete face around the dowel. Sample calculations of restraint that needs to be developed to cause midslab cracking are presented in Table 1.

BACKGROUND

Only a few investigations have been conducted to study levels and effects of dowel misalignment. The number of field investigations has been limited because of lack of practical methods for evaluating alignment of dowels in place.

TABLE 1 Calculated Restraint to Cause Midslab Cracking in 10-in.-Thick Slab

Age (days)	Tensile Strength (psi)	Compressive Strength (psi)	Concrete Modulus (psi 000,000s)	Allowable Strain (millionths)	Restraint to Cause Cracking (lb/ft width)
1	87	700	1.5	58	10,400
3	184	1,800	2.3	80	22,100
7	258	2,750	2.9	89	31,000
28	333	3,800	3.4	97	40,000
365	425	5,250	4.2	102	51,000

Note: (a) Before midslab cracking occurs, spalling may take place around load-transfer device. (b) Tensile stress in the form of curling restraint stress and load stress also exists. Therefore, even a reduced level of restraint can contribute significantly to crack formation. (c) Age, strength, and modulus relationships are general and are used for illustration only.

An early field study conducted in Indiana by Smith and Benham found a large number of misaligned dowels (4). As a supplement to the field work, laboratory tests were conducted using small slab sections incorporating a joint and dowels spaced at 12-in. centers. In these tests, 3/4-in.-diameter dowels were placed at different levels of misalignment and loading was applied at 28 days to open the joint. Results indicated that for a 6-in.-thick slab section, an alignment error in excess of 1 in. caused spalling when joints were opened 3/4 in. For a 5-in.-thick slab section, an alignment error of 1/4 in. caused slight spalling. Test results also showed that if the joint was not opened more than 1/2 in., alignment errors up to 1 1/2 in. could be tolerated without spalling. Generally, the load required to open a contraction joint 1/2 in. did not exceed 3,000 lb per dowel.

In another study, conducted by Segner and Cobb at the University of Alabama, slab sections 5 ft wide, 5 1/2 ft long, and 10 in. thick were used (5). Dowels used were 1 1/4 in. in diameter and 16 in. long. Testing was done at 2 and 7 days. Load required to open a joint 1/2 in. for a 1-in. vertical misalignment of a dowel was about 4,000 lb, and for a 1-in. horizontal misalignment of a dowel the load was about 2,000 lb. Spalling was produced for a vertical misalignment of 1 in. or for a horizontal misalignment of 3 in. at a joint opening of about 0.9 in.

Theoretical effects of misalignment have been studied by Friberg (6) and Weaver and Clark (7). Friberg assumed that in a misaligned dowel, the dowel deflection must equal the transverse component of the movement in a parallel displacement of the slab. The relationship between the deflection of the dowel and dowel misalignment was then determined by Friberg as follows (6):

$$\alpha i = \{(P/2EI)\ [(1 + Ba)^2/B^3] + (a^3/6)\} \qquad (1)$$

where

- α = misalignment of the dowel in the direction of slab movement (rad),
- i = total slab end movement,
- a = total joint width,
- P = dowel shear developed due to misalignment,
- E = modulus of elasticity of dowel steel,
- I = moment of inertia of dowel section,
- B = relative stiffness of dowel and concrete = $(GD/4EI)^{1/4}$,
- G = modulus of dowel concrete reaction, and
- D = dowel diameter.

With this equation, dowel shear developed due to misalignment can be calculated. The calculated shear values can then be used to compute concrete bearing stresses under dowels. Shear loads calculated by using the equation are given in Table 2 for a 1 percent dowel misalignment and different levels of slab end movements. However, this analysis considers only dowel bearing effects and not the effects of dowel slippage or the resistance to dowel movement of the concrete surrounding the dowel. The analysis does not provide information on development of tensile stress in the pavement slab as a result of dowel misalignment.

TABLE 2 Dowel Shear Induced due to Misalignment of 1 Percent (6)

Dowel Diameter (in.)	Final Joint Width (a_i) (in.)	Dowel Shear Induced (lb) i = 0.25 in.	i = 0.50 in.
1.00	0.25	815	1,630
	0.50	695	1,390
1.25	0.25	1,235	2,465
	0.50	1,090	2,175

Note: i = change in joint width due to slab end movement.

Recent investigations have concentrated on comparing misalignment levels and performance of joints having machine-implanted dowels and preset basket assemblies (8-10). These studies have been conducted because of concern about dowel placement accuracy when machine implanters are used. In the Pennsylvania study (8), horizontal, vertical, and longitudinal misalignments were measured at implanted and conventionally placed dowel bar joints. Two bars each from five joints were chosen for each placement type. A pachometer was used to locate the dowels and 4-in.-diameter cores were drilled to the top of the bars at each end of the bar. The average values of misalignment are given in Table 3. Sixty percent of the implanted dowels and 40 percent of conventionally placed dowels were outside specified limits of tolerance. The Pennsylvania Department of Transportation specifies an allowable tolerance of 1/4 in. per 18-in. length of dowel bar in both the horizontal and vertical directions.

In an investigation conducted for the American Concrete Pavement Association, visual surveys and misalignment determinations using a metal detector

TABLE 3 Levels of Misalignment Measured in the Field (8)

Placement Method	Project	Vertical Skew (in.)	Horizontal Skew (in.)	Vertical Translation (in.)	Horizontal Translation (in.)
Basket	2E	5/8	1/4	0	1-3/8
	2E	1/8	0	3/8	1
	6E	7/16	0	1/32	1
	6E	1/16	0	5/16	1-1/4
	9E	1/16	0	7/16	1
	9E	1/16	1/8	7/16	15-16
	15E	3/8	1/4	1/16	5/8
	15E	1/16	1/4	1-1/4	3/8
	17E	1/8	3/8	11/16	11/16
	17E	0	0	11/16	1/8
Implanted	1	1/16	1/4	11/16	7/8
	1	1/16	1/4	3/4	7/8
	2	1/16	3/16	7/16	5/16
	2	3/16	0	3/4	1/4
	9	3/16	3/8	5/8	5/16
	9	1/8	3/8	5/8	7/16
	19	1/16	3/4	15/16	3/8
	19	1/16	1/4	3/4	3/8
	28	1/16	1/4	1-3/16	1/8
	28	1/8	0	1	0

Note: Specified tolerances for these projects were a skew of 1/4 in. per 18 in. length of dowel bar in both the vertical and horizontal directions and a vertical or horizontal translation of ±1 in.

were made at several sites in Alabama to compare joints with mechanically implanted and conventionally placed dowels (9). Projects studied were constructed between 1958 and 1969. A statistical analysis was conducted to identify trends. It was found that there was no significant difference between implanted and preset dowel joints with respect to joint-related distress. However, no statistically valid conclusions could be drawn from the misalignment data.

In a Tennessee investigation (10), misalignment levels were determined at several sites by uncovering dowels in freshly placed plastic concrete and by core drilling in hardened concrete. On the basis of the findings, it was recommended that horizontal and vertical skew tolerances be 1/2 in., vertical tolerance be ±1 in., and longitudinal tolerance be ±1 1/2 in.

In a study conducted during 1982 by the Georgia Department of Transportation, dowel bar placement was investigated at five highway projects (3). Three projects had implanted dowels and two projects had used dowel basket assemblies. Project details are given in Table 4. Dowel placement was determined by coring and use of a metal detector. In addition, distress at joint locations was observed. A total of 261 joints were evaluated in detail and another 400 to 500 joints were examined for signs of distress.

A summary of Georgia's field evaluation is given in Table 5. It is clear from Table 5 that there is substantial noncompliance with the specification requirement for the projects with the implanted dowels. However, no dowel-related distress was found in any of the joints that were examined (3). In addition, it was reported that during construction of the five projects, all joints had started "working" within a few days of construction. However, because of the noncompliance problem with implanted dowels, the study recommended that implanting of dowels not be allowed. The study also recommended that improvements be made in methods and equipment for implanting dowels and that studies be conducted to determine permissible levels of dowel misalignment.

ANALYTICAL MODELING

Analytical modeling was used to perform stress analysis of joint systems incorporating dowels with different levels of misalignment. The following items were considered in the analysis:

1. Slippage between dowel and concrete,
2. Simulation of temperature drop in the concrete slab, and
3. Dowel misalignment levels.

An analysis was conducted to simulate slab end movement due to temperature change within the slab. Restraint to slab end movement would be induced by the misaligned dowels. One of the difficulties in an analysis of a doweled system is the complexity of modeling the slip between the dowel and the concrete. Recently, the finite-element method has been used to model slippage at joints in rock masses. However, this type of modeling is still under development.

Initial modeling of a misaligned dowel was conducted by using computer program SAP4 (11). Program SAP4 is a general-purpose finite-element computer program developed at the University of California at Berkeley. Program SAP4 cannot model slip behavior directly. Therefore, slip behavior was modeled by using "soft" elements at the interface between the dowel and the concrete. After work started with the SAP4 program, another finite-element computer program was made available, denoted BMINES, which was developed by Agbabian Associates for the U.S. Bureau of Mines (12). Program BMINES is a static, two- or three-dimensional, nonlinear, finite-element computer program for analysis of structural and geological systems. It has the capability to consider slippage at cracks and joints.

Analysis was conducted only for the case of a single dowel with skew misalignment. Analysis of a full-width joint incorporating several misaligned dowels is not practical because of the difficulty in modeling the three-dimensional nature of the problem.

On the basis of attempts to theoretically model dowel misalignment, it was concluded that it is not currently feasible to conduct a rational analysis of misaligned dowel bars. The modeling of slippage between the dowel and the concrete and the simulation of the three-dimensional dowel misalignment are considered too complex to be correctly incorporated in currently available analysis techniques.

The effect of dowel misalignment was then in-

TABLE 4 Georgia Department of Transportation Project Description (3)

Project	Project No.	Location	Age (years)	Dowel Placement Method
A	I-16-1(38)115 Ct 3, Bulloch County	SR-73 to SR-67, 10.285 mi	5	Implanter
B	I-20-1(23)00 Ct 4, Carroll-Haralson	Alabama Line to US-27, 11.585 mi	3	Implanter
C	I-85-1(33)12 Ct 3, Troup County	SR 219 to Hines Road, 8.538 mi	3	Implanter
D	I-20-1(27)11 Ct 4, Carroll County	US-27 to SR-61, 11.874 mi	3	Baskets
E	GS 7-ACS-13-1(42), GS 9-ACF-13-1(44), Hall County (SR 365)	SR-23 to SR-52, 8.111 mi	1/4	Baskets

TABLE 5 Percent of Dowels out of Specification Tolerance (3)

Project	Dowel Installation	Depth[a] (in.)	Vertical[a] Rotation (in.)	Horizontal Rotation (in.)		Longitudinal Alignment (in.)	
				Core Measurements	Metal Detector Measurements	Core Measurements	Metal Detector Measurements
A	Implant	24	20	9	10	65	68
B	Implant	72	17	25	15	75	66
C	Implant	83	28	20	22	63	62
D	Basket	0	5	0	4	57	54
E	Basket	0	0	5	10	21	22

Note: The following tolerance levels were specified by Georgia Department of Transportation during construction of the listed projects: vertical tolerance ± 1 in., horizontal tolerance ± 1 in., rotation (horizontal plane) 1 1/8 in. per 18-in. length, rotation (vertical plane) 9/16 in. per 18-in. length.

[a]Core measurements.

vestigated in the laboratory. The laboratory testing program and test results are presented in the next section.

LABORATORY TEST PROGRAM

A laboratory test program was conducted to study the effect of dowel misalignment. Testing consisted of a pullout test of slab specimens incorporating a joint and dowels with different levels of misalignment. Initial tests were conducted with a single misaligned dowel per test specimen and use of rollers along the sides of the specimen to ensure that the pullout direction remained perpendicular to the joint during the test. Pullout loads measured during these tests were relatively low. Because of a concern that the low measured loads could be due to improper testing procedures, the test procedure was modified. In the modified test procedure, a pair of misaligned dowels was used. The two dowels were misaligned in opposite directions to cancel out side forces and thus eliminate any tendency for the slab sections to tilt while being pulled apart.

Test Parameters

The following test parameters were considered:

- Slab section dimensions: 3 ft wide by 7 ft long;
- Slab thickness: 8 and 10 in.;
- Misalignment levels (per 18-in. length): 0, 1/4, 1/2, 1, 2, and 4 in.;
- Misalignment category: horizontal and vertical;
- Test age: 1, 3, 7, and 28 days;
- Maximum joint opening: 0.25 in.

Test Procedure

As discussed previously, two different test procedures were used. In one procedure, a single misaligned dowel was used. In the other, a pair of misaligned dowels was used.

Test with a Single Misaligned Dowel

The test setup is shown in Figure 2. The test frame was constructed by using channel-shaped steel members. One section of the test specimen was held firmly to the rigid frame. The other section was pulled by using a hydraulic jack. Dowel misalignment was controlled by welding one end of the dowel to a chair with a base plate and nailing the base plate onto the form. A 1/8-in.-thick steel plate was used to form the joint. A form ready for casting is shown in Figure 3. Concrete was placed carefully around the dowel to ensure that the dowel misalignment remained true. Each specimen was cast over two layers of polyethylene sheets.

Two pairs of rollers were used along the sides of the test specimen to ensure that the movement of the pulled slab section was perpendicular to the joint. The bearing force on the two pairs of rollers along the pulled section was monitored by using load cells installed between the rollers and the test frame. During the test, joint opening was monitored with a pair of displacement sensors mounted on the slab surface, as shown in Figure 4. A data-acquisition system was used to record joint-opening and load-cell data.

Pullout load was applied gradually and uniformly to obtain a joint opening of 0.25 in. in about 1 min.

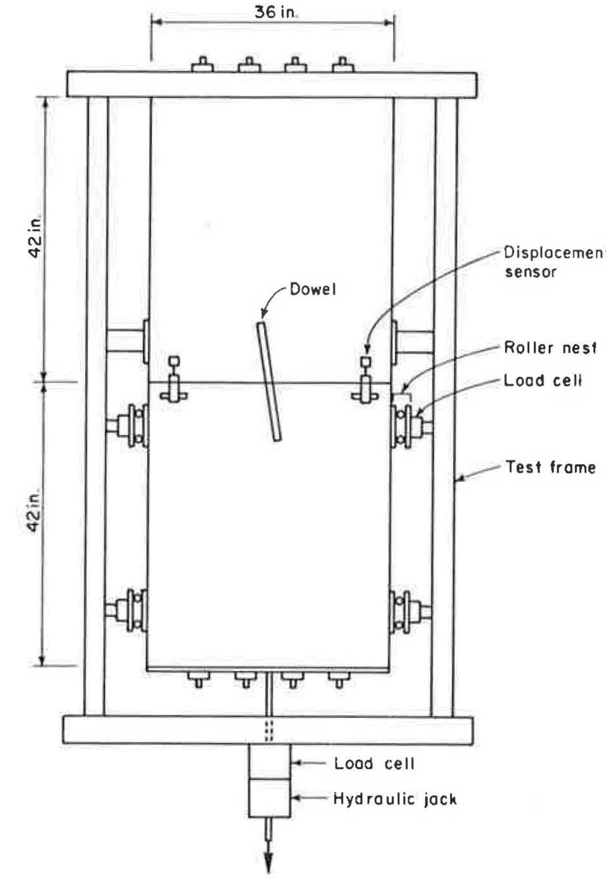

FIGURE 2 Setup for single-dowel test.

A total of 16 specimens were tested with the test procedure described. Specimen details and test results are given in Table 6. Concrete compressive strengths were as follows:

Test Series	Age (days)	Compressive Strength (psi)
A,C	1	1,960
	3	2,990
	7	3,810
	28	5,100
B,D	1	1,490
	3	2,970
	7	4,040
	28	5,420

Typical relationships between the pullout load and joint opening are shown in Figure 5. It is seen that a large portion of the pullout load is required to open the joint 0.01 in. After the joint has opened about 0.05 in., there is no further increase in the pullout load.

For each test, the pullout test was performed three times. After each, the pulled slab was pushed back to close the joint and the pullout test repeated. The maximum pullout load was always obtained under the first test. For the second and third tests, the maximum pullout load obtained was less than half that obtained for the first test.

Test results do not show significant differences in the pullout load for the different levels of misalignment. There was concern that this behavior may be due to the use of a single misaligned dowel and the possible pulling of the slab in a direction parallel to the misaligned dowel even though rollers were used along the slab sides.

A new test procedure was then developed for the

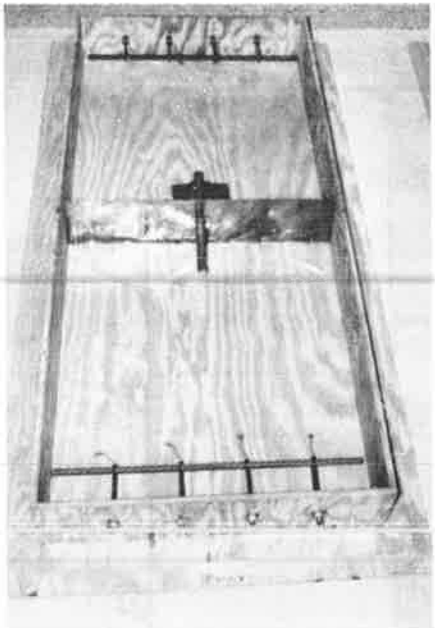

FIGURE 3 Single-dowel test: form ready for casting.

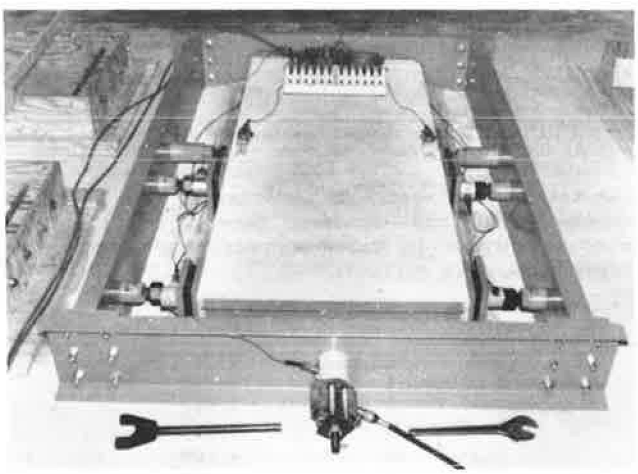

FIGURE 4 Single-dowel test: displacement sensors on slab surface.

pullout test. This procedure, using a pair of misaligned dowels, is discussed next.

Test with a Pair of Misaligned Dowels

The test setup for a pair of misaligned dowels is shown in Figure 6. The test frame was the same as that used for the single-dowel tests. However, use of the rollers along the sides of the specimen was discontinued and the specimen length was shortened to 4 ft. For this procedure also, one slab section was held firmly to the test frame while the other slab section was pulled.

For each test, each of the two dowels had the same level of misalignment. However, the dowels were misaligned in opposite directions to cancel out any tendency of the pulled-slab section to tilt horizontally or vertically. Dowel misalignment was controlled by use of chairs. A 1/8-in.-thick steel plate

TABLE 6 Test Details and Results

Test Series	Misalignment (in.)	Maximum Pullout Load (lb) by Test 1 Day	3 Days	7 Days	28 Days
A	0 horizontal	1,030	840	1,020	1,640
B	1/4 horizontal	890	670	980	2,000
C	1/2 horizontal	1,160	1,270	1,410	1,890
D	1 horizontal	1,460	1,280	1,020	NA

Note: NA = not available. Slab thickness = 8 in. Maximum joint opening = 0.25 in. Maximum aggregate size = 1 in.

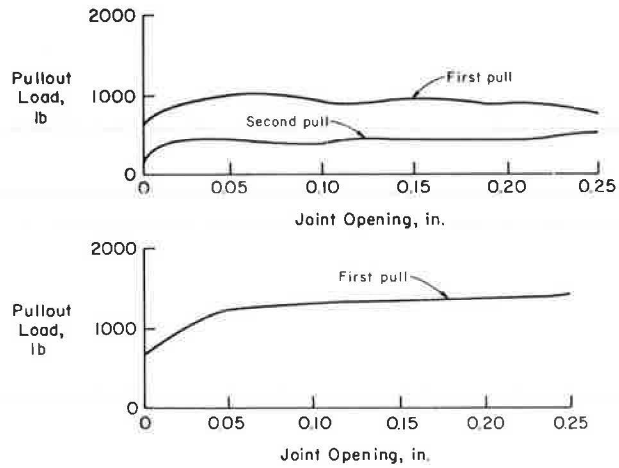

FIGURE 5 Relationships between pullout load and joint opening.

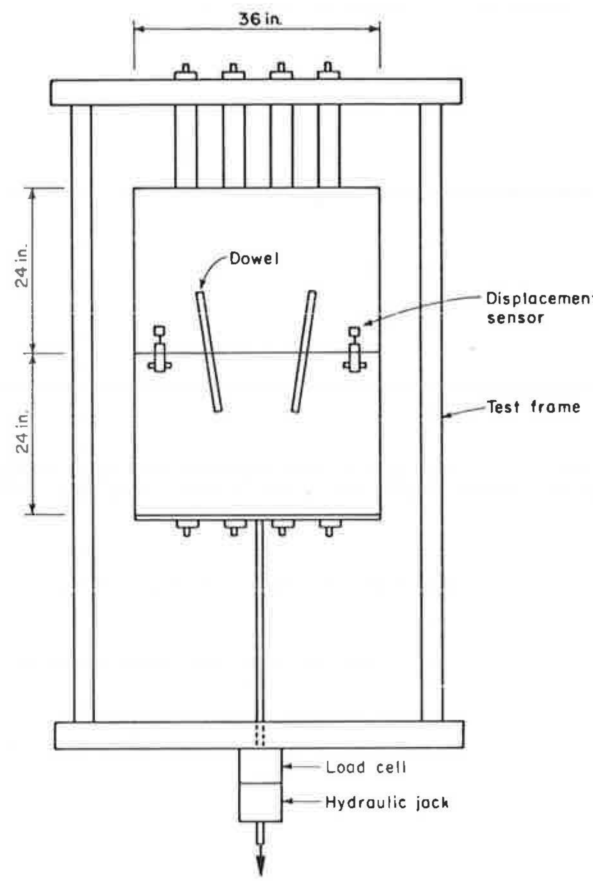

FIGURE 6 Setup for two-dowel test.

FIGURE 7 Two-dowel test: form ready for casting.

was used to form the joint. A form ready for casting is shown in Figure 7. Concrete was placed carefully around the dowels to ensure that the dowel misalignment remained true. Each specimen was cast over two layers of polyethylene sheets.

Joint opening was monitored with a displacement sensor mounted on the slab surface. An X-Y plotter was used to record the pullout load measured by a load cell and the joint opening measured by the displacement sensor. Pullout load was gradually and uniformly applied to obtain a joint opening of 0.25 in. in about 1 min.

A total of 33 specimens were tested, 24 of which had a slab thickness of 8 in. and the remaining 9 had a slab thickness of 10 in. Specimen details and test results are given in Table 7 for the 8-in.-thick specimens and in Table 8 for the 10-in.-thick specimens. Concrete compressive strengths were as follows:

Specimen Thickness and Test Series	Age (days)	Compressive Strength (psi)
8 in.		
E,F,G,H	1	1,640
	3	2,640
	7	3,530
	28	4,940
I,J,K,L	1	1,460
	3	2,640
	7	3,740
	28	5,140
10 in.		
M-U	8	4,070
	28	5,050

It may be seen from Table 7 that although there is an increase in the pullout load with increased level of dowel misalignment, the absolute magnitudes of the pullout load are still relatively low for dowel misalignment levels below 1 in. As seen from Table 8, the magnitude of the pullout load increases greatly when the dowel misalignment exceeds 1 in. It should be noted that maximum joint opening did not exceed 0.25 in. and that the pullout load would have been greater for larger joint openings than that shown in Table 8 for dowel misalignment levels exceeding 1 in. It should also be noted that no spalling was seen around dowel bars at the joint face for specimens having dowels with misalignment levels of less than 1 in.

Results of tests conducted using two misaligned dowels per specimen verify the general reliability of the test results obtained with a single misaligned dowel per specimen. The similarities in the results

TABLE 7 Test Details and Results for 8-in.-Thick Slab

Test Series	Misalignment[a] (in.)	Maximum Pullout Load per Dowel (lb) by Test 1 Day	3 Days	7 Days
E	0 horizontal	600	700	850
F	1/4 horizontal	650	750	NA
G	1/2 horizontal	800	900	1,100
H	1 horizontal	900	1,250	1,250
I	0 vertical	600	800	850
J	1/4 vertical	750	1,250	1,350
K	1/2 vertical	1,150	1,300	1,750
L	1 vertical	1,400	1,600	1,750

Note: Two dowels were used per specimen. Maximum joint opening = 0.25 in.

[a]Total per dowel.

TABLE 8 Test Details and Results for 10-in.-Thick Slab

Test Series	Misalignment[a] (in.)	Maximum 7-Day Pullout Load per Dowel (lb)
M	0	750
N	1/2 horizontal	2,250
O	1 horizontal	2,000
P	2 horizontal	4,000
Q	4 horizontal	5,500
R	1/2 vertical	1,666
S	1 vertical	2,000
T	2 vertical	R
U	4 vertical	R

Note: Two dowels were used per specimen. Maximum joint opening = 0.25 in. except for Test Series Q, for which maximum joint opening was 0.20 in. R = rejected due to excessive twisting of the test panels.

[a]Total per dowel.

of tests using the two different procedures confirm the reliability of the low levels of the pullout loads measured for dowel misalignment of 1 in. or less.

Discussion of Test Results

Laboratory test results indicate that pullout loads are relatively low for dowel misalignment levels of less than 1 in. per 18-in. length of dowel bar and a maximum joint opening of 0.25 in. A maximum joint opening of 0.25 in. was selected for the laboratory tests because joint openings in the field do not exceed this value. Joint openings in the field due to daily and seasonal volume changes generally range from about 0.05 in. to about 0.20 in. for slab lengths ranging from about 15 ft to about 40 ft.

Test results agree generally with observations reported by Smith and Benham (4) and by Segner and Cobb (5) that were discussed previously in the section entitled Background. Smith and Benham's laboratory test indicated that if a joint was not opened more than 1/2 in., alignment errors up to 1 1/2 in. per 24-in. length of the bar could be tolerated without spalling and that pullout load required to open a joint 1/2 in. did not exceed 3,000 lb per misaligned dowel. Segner and Cobb's laboratory work indicated that a pullout load of about 2,000 lb was needed to open a joint 1/2 in. when a dowel had a horizontal misalignment of 1 in. per 16-in. length of dowel and that pullout load was about 4,000 lb for a vertical dowel misalignment of 1 in.

Vertical and horizontal misalignment levels of 1/4 in. per 18-in. length of dowel bar have been considered acceptable in the past by many state highway agencies. There was relatively little difference in measured pullout loads between specimens incorporating a 1/4-in. misalignment and specimens incorporating a 1/2-in. misalignment.

Because of the limited number of tests that were conducted during the present study and because these tests did not consider the effects of multiple misaligned dowels at a joint, no recommendations are made to change levels of permissible misalignment currently specified by state highway agencies. However, data developed to date from various studies indicate that misalignment levels greater than those currently specified may be acceptable.

To ensure that a realistic specification is developed in the future, it is necessary that a practical, reliable, and cost-effective nondestructive test method be available to measure dowel misalignment in the field. Factors to be considered in developing an effective program of field evaluation of dowel-bar misalignment are given by Tayabji (13). These factors include consideration of the allowable number of misaligned dowels per joint and selection of a strategy to resolve the misalignment problem once it is identified.

SUMMARY

An investigation was conducted to develop limits on allowable levels of tolerances for dowel placement at concrete pavement joints. Theoretical analyses of the effect of dowel misalignment were conducted by using finite-element computer programs SAP4 and BMINES. Because of the complexity of modeling slippage between the dowel and the concrete and of simulating the three-dimensional nature of dowel misalignment, the theoretical analysis was not completed.

The effect of dowel misalignment was studied in the laboratory. Test results indicate that pullout loads for dowels with misalignment levels of 1 in. or less are relatively low.

However, no revisions to the currently accepted levels of dowel misalignment are recommended at this time because of the limited amount of laboratory test data and lack of sufficient data on field performance of jointed concrete pavements with misaligned dowels.

It is recommended that a concerted effort be made to document dowel misalignment in the field and to relate the levels of misalignment to performance at the joints. An adequate data base on field performance of jointed concrete pavements incorporating dowels with different levels of misalignment as well as data developed from laboratory tests can then be used to make revisions to the currently used specifications for dowel misalignment.

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Evaluation of Chace Air Indicator

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ABSTRACT

An evaluation of the Chace Air Indicator (CAI) was made for use in portland cement concrete construction. The CAI indicated higher values than the pressure method at low air content and lower values at high air content. The CAI readings corrected for mortar content and Chace factors produced values approximately 15 percent higher than the pressure method over all ranges of air content. A regression analysis procedure was used to determine a curve correction to account for the difference between the Chace factor- and mortar-corrected CAI readings and those of the pressure meter. An indication of the reliability of the results was represented by confidence intervals. The CAI does not have sufficient accuracy to measure the air content of concrete for job control purposes.

An investigation of the use of the Chace Air Indicator (CAI) in determining the amount of entrained air in structural portland cement concrete (PCC) is described. The objectives of the investigation are the following:

1. To determine the calibration and correlation requirements for the CAI,
2. To identify the limits or tolerances for the use of the CAI either for job control or as an indicator as it is now used, and
3. To determine whether the CAI can measure the amount of entrained air with sufficient accuracy for job control purposes.

For purposes of this study, job control is defined as "the measurement of the air content of PCC with equal accuracy to that measured by a pressure meter."

The study consisted of a laboratory and a field phase. The laboratory phase permitted the study of many mix design variables under controlled conditions (1). The field phase allowed for testing to establish the effect of normal variations encountered in field operations. The field phase of the study is presented in this paper.

PREVIOUS STUDIES

Bureau of Public Roads, 1957

The Bureau of Public Roads study (2) found the CAI to be useful in determining the approximate amount of entrained air in PCC in the field.

The major conclusions were as follows:

1. The CAI yielded low readings for air content above 6 percent and high readings for air content below 3 percent.
2. Because of the small amount of mortar used in a test, at least three readings should be made for each air-content determination.
3. The CAI is not considered a suitable replacement for the pressure method but is a useful supplementary test.
4. The CAI appears to be most valuable for use in determining uniformity from one batch of concrete to the next when there is no change in the mix design or materials.
5. The CAI can also be used as a rapid check to determine whether the air content is within specification limits.

Virginia Council of Highway Investigation and Research, 1960

A study by the Virginia Council of Highway Investigation and Research (3) compared CAI test results with the results of conventional pressure methods. Data from over 800 field tests were statistically analyzed and compared with results of previous laboratory research. The results of this study were in agreement with previous work and gave a field verification of the laboratory data available at that time.

The principal conclusions of this study were as follows:

1. A mortar correction based on the mortar content of the mix was recommended to account for the fact that only a mortar sample is used in the CAI test as opposed to use of a concrete sample in the pressure method.
2. A curve correction was also recommended to account for the high readings by the CAI for low air content and the low readings for high air content.
3. The CAI was found to be reasonably accurate and moderately precise for the measurement of air content in the field.
4. The accuracy of the CAI is improved with multiple readings.

Texas Highway Department, Materials and Test Division, 1970

The Texas Highway Department (4) investigated the effect of excessive temperature differentials and varying strength concentrations of isopropyl alcohol used with the CAI. This study recommended the following:

1. Seventy percent isopropyl alcohol be used in the CAI,
2. The tests should be performed with care and as rapidly as possible, and

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3. The alcohol supply should be protected from excessive temperatures to ensure that the alcohol and mortar temperatures will be relatively similar.

Virginia Highway and Transportation Research Council, 1981

The Virginia Highway and Transportation Research Council (5) found poor agreement between the pressure method and the CAI, even after the manufacturer's suggested correction factors had been applied to the CAI readings. The Virginia study revealed that CAI manufacturers do not set strict limits on the tolerances during the fabrication of the instrument; therefore, it was recommended that the Chace factor be determined for all CAIs used for air-content determination. The Chace factor is defined as the volume of one graduation on the stem expressed as a percentage of the volume of the cup. Correction factors were developed for varying Chace factors, varying mortar content, and the high CAI readings for low air content and the low readings for high air content.

The principal conclusions of this study were the following:

1. Varying mortar contents and Chace factors can be corrected for by using the following equation:

Mortar correction factor = [mortar content ($ft^3/10^3$) x Chace factor]/27.

2. Each CAI should be inscribed with its Chace factor.
3. A test result based on the average Chace-factor-based mortar-corrected and curve-corrected CAI air content for five samples provides the same confidence as is provided by one pressure-method test.
4. CAI readings should be taken as the average of a minimum of two samples.
5. The PCC investigated should be suitable for retrieving representative samples.

As a result of this study, the AASHTO Standard Method of Test for Air Content of Freshly Mixed Concrete by the Chace Indicator (T199-82) was modified to include the following recommended corrections:

1. Test results for the acceptance of concrete will be based on stem readings that have been mortar corrected, Chace-factor corrected, and curve corrected.
2. Test results for the acceptance of concrete will be based on the average of two samples. If the results differ by more than 2 percent, a third sample will be taken and the test results will be based on the average air content of the three samples.
3. Concrete that is determined to be unacceptable by the CAI will not be rejected unless a pressure-method test confirms that the concrete is unacceptable.
4. The pressure-method test will be used to determine whether concrete used in bridge decks meets specifications.

Center for Transportation Research, 1984

The Center for Transportation Research studies (1) represent the laboratory phase of this project. The variables were (a) range of slump, (b) range of air content, (c) range of temperature, (d) type of aggregate, (e) type of cement, and (f) type of admixture. The principal conclusions of these studies were the following:

1. Operator and instrument variabilities were negligible.
2. Two types of correction factor should be applied: a Chace-factor and mortar correction and a curve correction.
3. A curve correction of the form PM = $0.85X_{mc}$ was produced (the y-intercept of the best-fit line being close to zero) where PM is the pressure-meter reading and X_{mc} is the mortar-corrected CAI reading.
4. The correction to be applied was identical if one or more readings per sample were performed on the same batch. The difference was in the confidence interval indicating the reliability of the results. The 95 percent confidence interval decreased from 3.2 to 1.8 percent as the number of readings increased from 1 to 3.
5. It was observed that addition of high range water reducer at high air content resulted in decreasing air content with time as measured by both the CAI and the pressure meter. Air content measured with either device cannot be considered accurate under these circumstances.
6. Comparison of results with previously established corrections indicated a notable improvement. The confidence intervals were reduced and the best-fit line of data became almost identical to the line of equality between the CAI and the pressure meter.

FIELD TEST PROGRAM

Field Test Variables

The descriptions of the variables under investigation in the field phase of the project are presented in the following paragraphs. A summary of the numerical values obtained in the field is given elsewhere (6).

Variations Within Ready-Mix Trucks Loaded to Different Levels

Samples were taken from ready-mix trucks

1. Loaded to capacity (trucks were considered loaded to capacity if they contained more than 6 yd^3 of concrete) and
2. Loaded to half capacity (if a truck contained less than 6 yd^3 of concrete, it was considered loaded to half capacity).

Samples were taken for each condition when the truck began discharging concrete, when it had discharged half the load, and when it was nearly empty.

Variations Between Ready-Mix Trucks

Samples were taken from different ready-mix trucks during large placements. This allowed the variation in CAI readings from truck to truck to be determined.

Day-to-Day Variations

Samples were taken from 10 trucks per day for 3 days at the same job site to enable the variation in CAI readings from day to day to be determined.

Transit Time

For all samples taken in the field the transit time was recorded. Transit time is defined as the interval between the mix truck loading time and the time that the sample was taken. Analyses were performed to determine the effect of the following delivery times: less than 15 min, greater than 15 min and less than 30 min, and greater than 30 min.

Concrete Mix Temperature

The mix temperature was recorded for all samples taken in the field. The variation between CAI readings and pressure-meter readings was determined for the following categories of mix temperatures: less than 60°F, greater than 60°F and less than 80°F, and greater than 80°F.

Ambient Temperature

The ambient temperature at the time of testing was recorded for all samples. The variations between CAI readings and pressure-meter readings were determined for the following categories of ambient temperatures: less than 60°F, greater than 60°F and less than 80°F, and greater than 80°F.

Slump

A slump test was performed on each sample. The variations between CAI readings and pressure-meter readings were determined for the following categories of slump: less than 3 in., equal to or greater than 3 in. and less than 6 in., and equal to or greater than 6 in.

Variability Between CAI Units

Four different CAI units were used in the field testing program to enable the variation between CAI units to be determined.

Variability Between Operators

Two operators did all the field testing and the variation between operators was determined.

Variation in Mortar Content

The mortar content for all samples was determined by using the concrete mix design sheets furnished by the batch plants and district personnel. The variation between the CAI readings and the pressure-meter readings was determined for variable mortar content.

Air Content

The variations between CAI readings and pressure-meter readings were determined for different ranges of air contents. The actual air content of the sample was assumed to be the pressure-meter reading. The categories of air content investigated were less than 4 percent, between 4 and 6 percent, and greater than 6 percent.

Field Test Procedures

The following procedure was performed on each concrete sample taken in the field:

1. A wheelbarrow was used to take the concrete samples from the ready-mix trucks from the beginning, middle, or end of the discharge. Each sample was taken after mixing and water additions were completed. The truck number was recorded.
2. Slump and pressure-meter tests were performed after a thorough mixing of the concrete sample. Concrete temperature and ambient temperature were recorded at this time.
3. Each of the two operators performed three CAI tests on every concrete sample. The samples of mortar were obtained in the following manner: (a) the surface of the concrete in the wheelbarrow was flattened with a trowel; (b) the flattened surface was then vibrated with the trowel to settle the aggregates, and leave the mortar at the surface; and (c) samples were taken from this mortar-rich surface.
4. The times were recorded for truck arrival, sampling of the truck, pressure-meter reading, and each CAI reading.
5. After all sampling was completed at a job site, the concrete batch ticket [supplied by the Texas State Department of Highways and Public Transportation (SDHPT)] was copied for each truck sampled.

Thirty-seven field visits were made, and 232 batches of concrete were sampled. Six CAI readings and one pressure-meter reading were taken on each sample. A total of 1,392 CAI readings and 232 pressure-meter readings were recorded.

DATA ANALYSIS

Statistical Procedures

Determination of the Variation of Field Conditions

The variations between the average of three mortar-corrected CAI readings and the pressure-meter readings for each of the variables outlined earlier were determined by using statistical analysis. The mean, standard deviation, and coefficient of variation (C_v) of the difference between the average of three mortar-corrected CAI readings and the pressure-meter reading were calculated for each variable. The coefficient of variation is defined as the ratio of the standard deviation to the mean and is expressed as a percentage. It is important to note that the coefficient of variation does not represent a percentage of air content but rather a percentage variability, which gives an indication of the variables that affect the accuracy of the CAI readings.

Regression Analysis

The regression analysis procedure used in this study was presented in a companion study (1). A brief outline of the regression procedure follows.

Data Points

Three CAI tests and one pressure-method test were performed on every sample taken in the field. The mortar-corrected CAI readings or average of readings (X_{mc}) and the pressure-meter reading (PM) of a sample represent the data point for that sample.

The regression procedure was performed on each of the following sets of data points:

1. (X_{mc}, PM), where X_{mc} is the first mortar-corrected reading;
2. (X_{mc}, PM), where X_{mc} is the average of the first two CAI readings; and
3. (X_{mc}, PM), where X_{mc} is the average of the three CAI readings.

Best-Fit Straight Line of Field Data

The best-fit straight line of the field data was found by applying a regression analysis to the points (X_{mc}, PM). This best-fit line is

$$Y1 = (a1)\ X_{mc} + b1 \quad (1)$$

where a1 and b1 are parameters of the line.

Accuracy of Best-Fit Equation

The difference between Y1, as determined by Equation 1, and the pressure-meter readings (PM - Y1) represents the accuracy of Equation 1. A regression was performed on the set of points [Y1, (PMR - Y1)] to determine the value (d) to be added to Y1 to obtain PM. The linear equation evolving from this regression is

$$d = (a2)\ Y1 + b2 \quad (2)$$

where a2 and b2 are parameters of the line.

Accuracy of the Sum (Y1 + d)

Because the field data were not perfectly linear, it was necessary to determine the accuracy of the sum (Y1 + d) as a representation of PM. A regression was performed on the set of points [(Y1 + d), PM]. The result of this regression is

$$Y = (A)\ (Y1 + d) + B \quad (3)$$

where A and B are parameters of the line.

Air Content Equation

The purpose of this analysis was to find an equation for air content (Y) in terms of the mortar-corrected Chace readings (X_{mc}). This is accomplished by combining Equations 1, 2, and 3. This combination gives the final equation for Y:

$$Y = [A\ (1 + a2)\ a1]\ X_{mc} + [A\ (1 + a2)\ b1 + AB2 + B] \quad (4)$$

Equation 4 can be expressed in simpler terms as

$$Y = S(X_{mc}) + 1 \quad (5)$$

where S is A (1 + a2) a1 and I is A (1 + a2) b1 + Ab2 + B.

Confidence Interval

A confidence interval of 95 percent was determined for Equation 5. This confidence interval is denoted by 2k, where k is expressed as a percent of air content and is represented by

$$k = 1.96\ (SD)/(n)^{1/2} \quad (6)$$

where n is the number of Chace readings used in determining X_{mc} and SD is the standard deviation derived from all (PMR - Y1) values.

Results

Variations in Field Conditions

The values of the coefficients of variation (C_v) between the pressure meter and the average of three mortar-corrected CAI readings for the variables outlined in the preceding section are as follows:

1. Ready-mix trucks loaded to different levels:
   a. Trucks loaded to capacity: C_v = 10.4 percent,
   b. Trucks loaded to half capacity: C_v = 12.5 percent,
   c. Sample from beginning of discharge: C_v = 9.7 percent,
   d. Sample from middle of discharge: C_v = 11.2 percent, and
   e. Sample from end of discharge: C_v = 11.5 percent;
2. Variation between ready-mix trucks: the average coefficient of variation between trucks at the same job site: C_v = 8.7 percent;
3. Day-to-day variations: the average coefficient of variation from day to day at the same job site: C_v = 10.3 percent;
4. Transit time:
   a. Less than 15 min: C_v = 11.5 percent,
   b. Between 15 and 30 min: C_v = 15.3 percent, and
   c. Greater than 30 min: C_v = 16.2 percent;
5. Concrete mix temperature:
   a. Less than 60°F: C_v = 27.2 percent,
   b. Between 60°F and 80°F: C_v = 15.5 percent, and
   c. Greater than 80°F: C_v = 16.9 percent;
6. Ambient temperature:
   a. Less than 60°F: C_v = 23.3 percent,
   b. Between 60°F and 80°F: C_v = 14.2 percent, and
   c. Greater than 80°F: C_v = 14.9 percent;
7. Slump:
   a. Less than 3 in.: C_v = 17.2 percent,
   b. Between 3 and 6 in.: C_v = 12.5 percent, and
   c. Greater than 6 in.: C_v = 13.7 percent;
8. Variability between CAI units:
   a. CAI 2: C_v = 5.2 percent,
   b. CAI 3: C_v = 3.1 percent,
   c. CAI 4: C_v = 4.8 percent, and
   d. CAI 6: C_v = 3.2 percent;
9. Variability between operators:
   a. Henley: C_v = 3.9 percent, and
   b. Malkemus: C_v = 2.5 percent;
10. Variation in mortar content:
   a. Less than 13.0: C_v = 11.5 percent, and
   b. Greater than 13.0: C_v = 15.1 percent;
11. Air content:
   a. Less than 4 percent: C_v = 28.9 percent,
   b. Between 4 and 6 percent: C_v = 20.6 percent, and
   c. Greater than 6 percent: C_v = 18.7 percent.

Regression Analysis Results

Field Study

The results of the regression analysis performed on the field data points are presented in this section. The curve-correction equation for the first-stage regression (using one mortar-corrected CAI reading for each data point) is

$$Y = 0.681X_{mc} + 1.02 \quad (7)$$

with a 95 percent confidence interval equal to 5.6 percent of air content. Figure 1 is a graphic representation of this equation.

The curve-correction equation for the second-stage

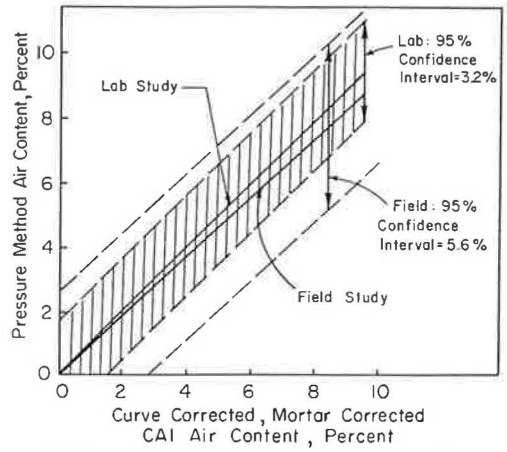

FIGURE 1 Comparison of field curve correction (Equation 7) with laboratory curve correction (Equation 10) (one CAI reading per data point).

regression (using the average of two mortar-corrected CAI readings for each data point) is

$$Y = 0.705X_{mc} + 0.897 \quad (8)$$

with a 95 percent confidence interval equal to 4.0 percent of air content. Figure 2 is a graphic representation of this equation.

The curve-correction equation for the third-stage regression (using the average of three mortar-corrected CAI readings for each data point) is

$$Y = 0.721X_{mc} + 0.829 \quad (9)$$

with a 95 percent confidence interval equal to 3.2 percent of air content. Figure 3 shows the results.

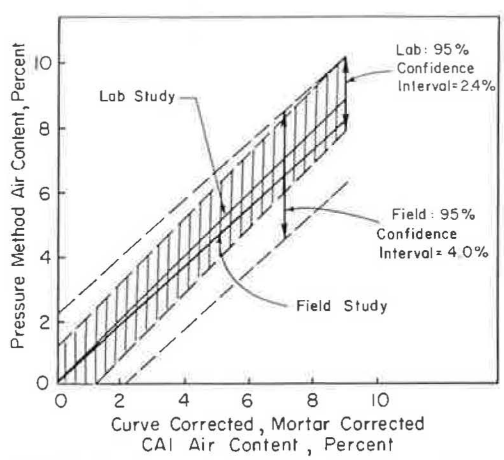

FIGURE 2 Comparison of field curve correction (Equation 8) with laboratory curve correction (Equation 11) (two CAI readings per data point).

Laboratory Study

The results of the regression analysis performed in the laboratory phase (1) are summarized in the following paragraphs.

The curve-correction equation for the first-stage regression is

$$Y = 0.840X_{mc} + 0.068 \quad (10)$$

with a 95 percent confidence interval equal to 3.2 percent of air content. Figure 1 presents the results of this equation as well as a comparison with Equation 7.

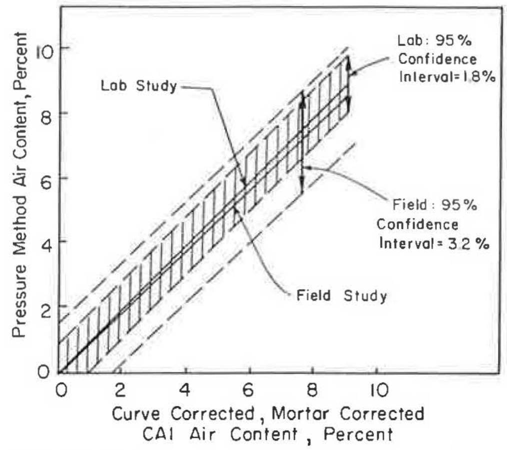

FIGURE 3 Comparison of field curve correction (Equation 9) with laboratory curve correction (Equation 12) (three CAI readings per data point).

The curve-correction equation for the second-stage regression is

$$Y = 0.843X_{mc} + 0.060 \quad (11)$$

with a 95 percent confidence interval equal to 2.4 percent of air content. Figure 2 is a graphic representation of this equation as well as a comparison with Equation 8.

The curve-correction equation for the third-stage regression is

$$Y = 0.844X_{mc} + 0.064 \quad (12)$$

with a 95 percent confidence interval equal to 1.8 percent of air content. Figure 3 shows this equation as well as a comparison with Equation 9.

Combined Field and Laboratory Analysis

The regression analysis procedure was applied to the combined laboratory and field data. This analysis was performed because the controlled environment in the laboratory was not a true representation of field conditions and the uncontrolled field environment did not allow for the testing of certain variables, for example, air content greater than 10 percent or ambient temperature less than 40°F.

The curve-correction equation for the first-stage regression is

$$Y = 0.729X_{mc} + 0.534 \quad (13)$$

with a 95 percent confidence interval equal to 4.8 percent of air content. Figure 4 represents Equation 13 and its corresponding 95 percent confidence interval.

The curve-correction equation for the second-stage regression is

$$Y = 0.780X_{mc} + 0.475 \quad (14)$$

with a 95 percent confidence interval equal to 3.2 percent of air content. Figure 5 shows Equation 14 and its corresponding 95 percent confidence interval.

The curve-correction equation for the third-stage regression is

$$Y = 0.784X_{mc} + 0.445 \quad (15)$$

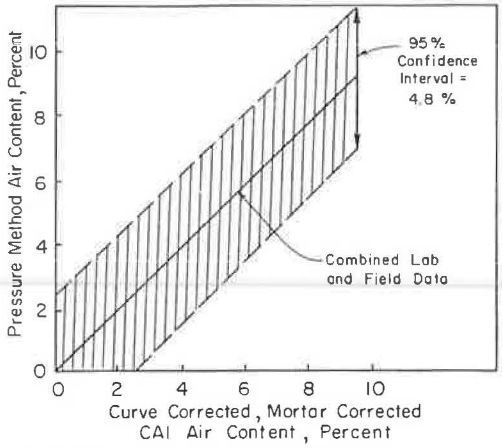

FIGURE 4 Curve correction for combined laboratory and field data (Equation 13) (one CAI reading per data point).

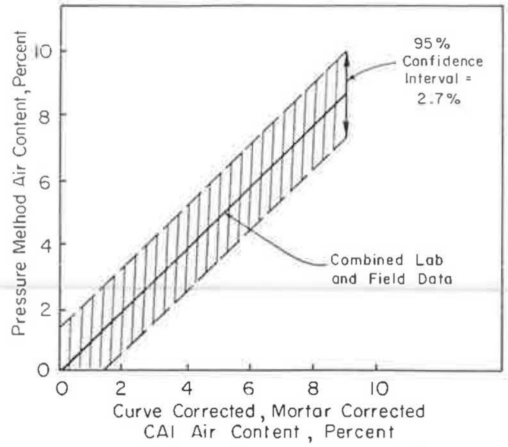

FIGURE 6 Curve correction for combined laboratory and field data (Equation 15) (three CAI readings per data point).

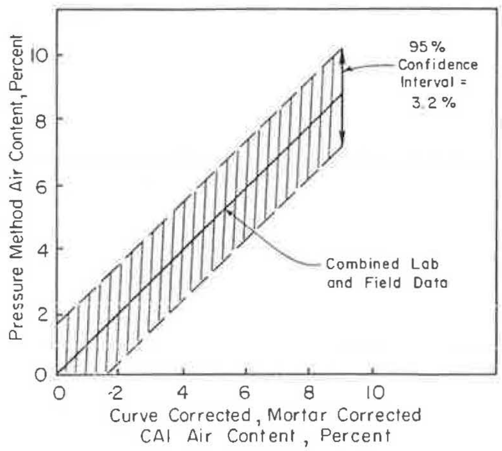

FIGURE 5 Curve correction for combined laboratory and field data (Equation 14) (two CAI readings per data point).

with a 95 percent confidence interval equal to 2.7 percent of air content. Figure 6 represents Equation 15 and its corresponding 95 percent confidence interval.

RECOMMENDED PROCEDURES

Determination of Chace Factor

Manufacturers do not set strict limits on the tolerances during the fabrication of CAIs; therefore, it is necessary to determine the Chace factor for all CAIs to be used in the field. The Chace factor is defined as the volume of one graduation on the stem expressed as a percentage of the volume of the cup. The procedure for determining the Chace factor is as follows:

1. Mercury or a mixture of 50 percent isopropyl alcohol, 50 percent water, and a few drops of liquid detergent should be used.
2. Determine the volume of one graduation on the stem:
   a. Fill the glass indicator with the alcohol mixture about 1/2 in. below the reference line. Insert the rubber stopper and cup into the tube. Invert the CAI and check for air bubbles. Slowly rotate the CAI at approximately a 45-degree angle to release any air bubbles trapped between the cup and stopper or between the glass cylinder and the cup or stopper.
   b. Place the CAI on a level surface.
   c. Fill the stem with the alcohol mixture so that the bottom of the meniscus coincides with the lower mark on the stem.
   d. Using a pipette or syringe graduated to 0.01 ml, measure the volume of alcohol mixture that is required to raise the bottom of the meniscus to the upper mark on the stem.
   e. Divide this volume by the number of graduations on the stem to determine the volume of one graduation (vl).
3. Determine the volume of the brass cup:
   a. Remove the stopper and cup from the tube and dry the brass cup. Make sure the brass cup is clean.
   b. Place the stopper on a level surface. Using a pipette or syringe graduated to 0.01 ml, add the alcohol mixture. Fill the cup until the meniscus levels into a flat plane coinciding with the top edge of the cup. This measurement is the volume of alcohol required to fill the cup (V).
4. Calculate the Chace factor (CF) using the following equation:

$$CF = (vl/V)\ (100) \qquad (16)$$

If the CAI is kept clean, the CF-value will not change as the apparatus ages. A CAI used for over 1,000 readings in this study maintained a constant CF through both the laboratory and field phases. However, if the CAI becomes encrusted with mortar, it should be cleaned or replaced and a new CF calculated.

Every existing and new CAI should be calibrated. The Chace factor should be marked permanently on each instrument.

Determination of Mortar-Corrected CAI Reading

The mortar-corrected CAI reading (X_{mc}) is determined by using the following equation:

$$X_{mc} = [CF\ (MC)/27]\ X_{uav} \qquad (17)$$

where

CF = Chace factor,
MC = mortar content of concrete being tested (ft^3/yd^3), and
X_{uav} = average of one or more uncorrected CAI readings.

To simplify the determination of X_{mc}, it is recommended that the mortar content (MC), as determined by the concrete mix design sheets, be included in the information on the concrete batch ticket delivered by the mix-truck driver to the site inspector.

Determination of Air Content

It is recommended that Equation 15 be used in the determination of air content. This equation was chosen because it is a combination of the laboratory and field study results. Therefore, it should be a reasonable representation of the variables studied in the laboratory and the conditions encountered in the field.

The air content of a sample is determined by applying the following equation:

$$Y = 0.784X_{mc} + 0.445 \tag{18}$$

where Y is the air content (percent) and X_{mc} is the mortar-corrected CAI reading. The 95 percent confidence interval of 2.7 percent implies that there is a 95 percent probability that the value of the actual air content is between the values of (Y - 1.4) and (Y + 1.4). A 90 percent confidence interval was also computed and is equal to 2.3 percent.

A graphical determination of the air content is also possible. Equation 18 is plotted against a vertical axis representing air content (Y) and a horizontal axis representing mortar-corrected Chace readings. The graph is entered with a mortar-corrected CAI value and a line is projected vertically until the curve for Equation 18 is intersected. The line is then projected horizontally to the vertical axis and the value for air content is determined.

A sample air-content determination using a graphical procedure is given in Figures 7 and 8. Figure 7 shows the 95 percent confidence interval and Figure 8 a 90 percent confidence interval. It is assumed that a mortar-corrected CAI reading of 8.0 was computed. The line projected vertically and horizontally reveals an estimated air content of 6.7 percent. Considering a 95 percent confidence interval, the air content should be in the range from 5.3 percent to 8.1 percent. If a 90 percent confidence interval is preferred, the range of air content is 5.6 to 7.9 percent.

Air content can also be estimated by using the nomograph in Figure 9. This nomograph accounts for Chace-factor corrections, mortar corrections, and curve corrections. A sample air-content determination is shown on the nomograph. This example assumes a Chace factor of 2.5, a mortar content of 10 ft^3/yd^3, and a CAI reading of 8.5. Given these values, an air content of 6.7 percent is obtained from the nomograph.

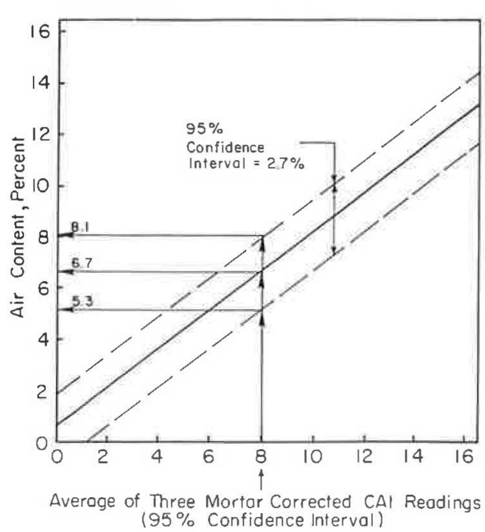

FIGURE 7 Graphical determination of air content: three CAI readings, 95 percent confidence interval.

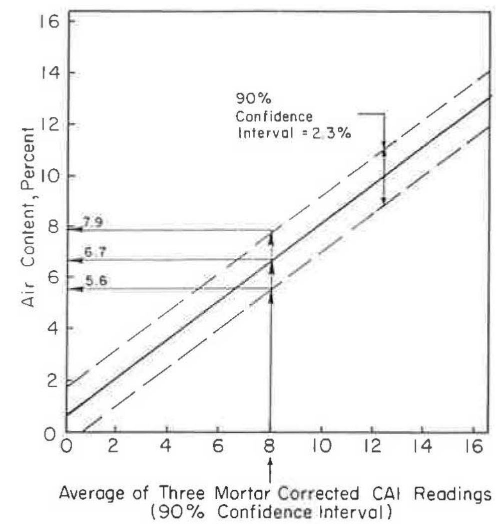

FIGURE 8 Graphical determination of air content: three CAI readings, 90 percent confidence interval.

SUMMARY AND CONCLUSIONS

Summary

The laboratory phase of the study investigated a wide range of variables, including air-content range, slump range, temperature range, cement type, admixture type, aggregate type, operator variability, and CAI variability (1).

This paper summarizes the field phase of the study (6), which allowed for testing to establish the effect of normal variations encountered in field operations.

Thirty-seven field visits were made and 232 batches of concrete were sampled. Six CAI readings and one pressure-meter reading were taken on each sample. A total of 1,392 CAI readings and 232 pressure-meter readings were recorded.

CAI readings were corrected for mortar content and Chace factor as suggested in a previous study (3). A curve correction was determined by using a regression analysis procedure. Three separate regression analyses were performed to determine curve corrections for the first of three CAI readings, an average of two CAI readings, and an average of three CAI readings.

The results of the field phase were comparable with the laboratory results. The data from the field

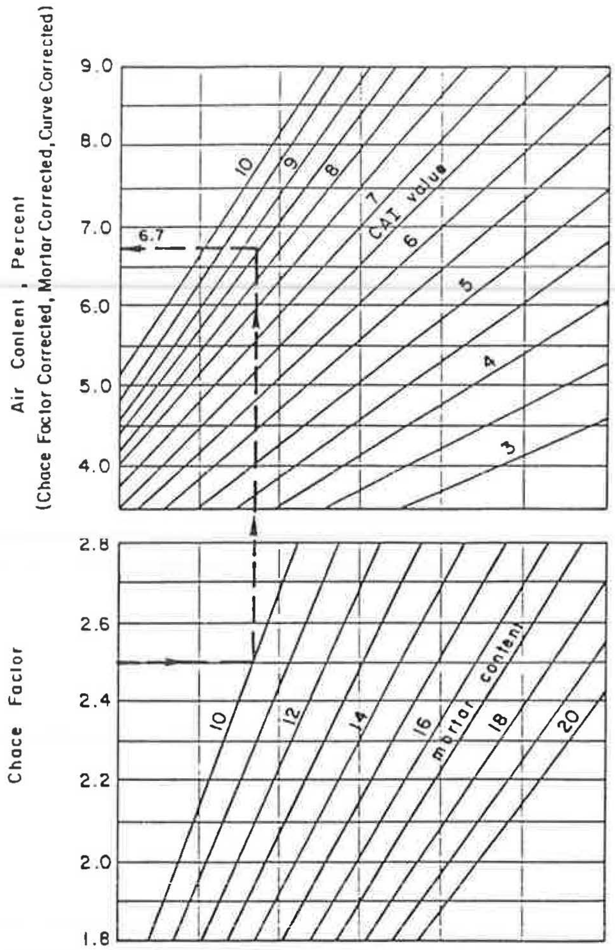

FIGURE 9 Chace-factor conversion nomograph.

and laboratory were combined, and a new curve correction was determined. This curve correction is the recommended equation for air-content determination. Recommended modifications to the CAI test procedure were developed to improve the accuracy of the instrument.

It should be noted that the SDHPT tolerance and the CAI confidence interval preclude the use of the CAI for actual air-content estimation. The SDHPT tolerance for air content of fresh concrete is ±1.5 percent. The 95 percent confidence interval for the average of three Chace-factor and mortar-corrected and curve-corrected CAI readings is 2.7 percent or ±1.4 percent. The difference of ±0.1 percent between the tolerance and confidence interval is not large enough to justify the use of the CAI for the estimation of actual air content.

Conclusions

1. Instrument and operator variability after training were not significant.
2. Recommended modifications to the test procedure improved the precision and accuracy of results.
3. If the recommended procedure for performing a CAI test is followed, the CAI can be used in the field to provide an indication of the range (high, medium, or low) of air content of fresh concrete.
4. The 95 percent confidence interval decreased from 4.8 to 2.7 percent as the number of CAI readings increased from one to three.
5. If the Chace factor of an instrument has been determined, there is no need for daily correlation with the pressure meter.
6. With the present SDHPT tolerances for air content of ±1.5 percent, the CAI is not sufficiently accurate to measure the air content of concrete for job control purposes.

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Computer-Aided Design of Portland Cement Concrete Mixes

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ABSTRACT

A prototype or first version of a software package being developed to assist in portland cement concrete mix design is described. This is not necessarily a research project but rather a research tool in which design information from various sources has been gathered into a single algorithm and organized into a usable and versatile form. The program consists of three sections: design of the first trial batch by any of the five commonly used methods, adjustment of an unsuccessful trial batch by a combination of empirical routines and those derived from American Concrete Institute methods, and a data file of various types of mix design information. This last section may be used independently or as an aide in blending or adjusting a mix. User friendliness is a key element of this algorithm. Computer-user interaction occurs throughout and is especially useful for the inexperienced designer. Designed and written initially for mainframe computers, the program features the addition of a translator for a problem-oriented language (SCAN), which leads to greater freedom from the structured input environment. Currently the software is being downloaded to a microcomputer (MS-DOS) system. Program output includes batch weights of the usual concrete ingredients as well as weights per cubic yard. Additional parameters provided include batch size, percent air, cement factor, unit weight, and water-cement ratio. A plot of unit weight versus air content is also offered. Like much software, this package is continually being modified and developed; however, it has already demonstrated its usefulness as a computer-aided design system.

The concrete mix design procedure, throughout its development, has been approached from various directions. Methods have been based, for example, on satisfying volume requirements, meeting specified unit weights, or matching particular weight or volume proportional relationships. Although the underlying precepts differ in each case, the goal of the designer is the same, that is, to create a blend of cement, water, sand, aggregate, and air that is of a certain strength, workability, and durability.

The designing of a successful mix can be a time-consuming process. Aggregate gradations as well as sizes and shapes differ, as do the qualities of various waters, cements, and admixtures. Therefore many material combinations must be blended and tested. In addition, a particular mix must reach a certain age (usually 7 or 28 days) before its design strength may be evaluated.

A wealth of information has been gathered by various organizations and individuals to assist the designer in shortening the time required to find a workable blend. To combine information from many sources and to enable the user to quickly retrieve and apply this information, an algorithm has been developed that attempts to codify and thus organize the portland cement concrete (PCC) mix design process. The program assembles information that has been compiled by organizations including the Portland Cement Association (PCA) (1) and the American Concrete Institute (ACI) (2,3) and from the work of Goldbeck and Gray (4), Popovics (5), and others. To assist the user in entering data and retrieving information, the SCAN subroutines developed at the University of Illinois have been employed throughout (6).

The program purpose is threefold. First, it provides a starting point for the design of a workable concrete mix by use of the various tables, mix methods, and information. Second, it provides the information and methods necessary to adjust a blend to correct certain first-batch problems, and, third, it is a repository of design information and criteria to assist in all phases of PCC design. The program should have some appeal to the entire spectrum of mix designers. It will be especially useful to those who may not have immediate access to the wide range of information necessary for successful concrete mix design. In this algorithm, the information is coordinated and is made available at those points in the design process where it is most likely to be required by the user.

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CAPABILITIES AND APPLICATIONS

The batch-design algorithm performs three main functions:

1. Designs a first trial batch by a user-designated method,
2. Adjusts batch quantities by changing particular parameters while holding others constant, and
3. Provides information files that can be used separately for general information or in conjunction with batching and adjusting.

In addition, the design algorithm provides printed output and line plots of certain data.

Design of First Trial Batch

First trial batches may be designed by using one of five methods: ACI volumetric, proportional by weight or volume, ACI based on consistency description, cement strength, and weight gravimetric. The consistency-description method is used for no-slump designs, whereas both the cement-strength and the weight-gravimetric methods are used to design lightweight concrete mixes. Normal-weight mixes may be designed by all of the foregoing, although the cement-strength method would not be recommended. The method used may be one of choice or of regulation, but in every case an effort has been made to provide useful assistance or information through each step of the design procedure.

Although mix designers' goals are similar in most cases, each method of obtaining a first trial batch has its own unique characteristics. A brief description of each follows.

ACI Volumetric (ACI 211.1-81)

The ACI volumetric method is used for normal-weight concrete mixes. The key to this method is to blend the correct amount of cement, sand, water, air, and coarse aggregate to achieve a solid volume of 27 ft^3 that exhibits the proper consistency and workability. The cornerstone of the method is the work of Goldbeck and Gray (4), in which a table was developed giving the necessary solid volume of coarse aggregate on the basis of maximum aggregate size and sand gradation (fineness modulus).

Proportional

The proportional method is the oldest in the program. In it the mix designer bases the material blend on a preselected proportional ratio of cement to fine aggregate to coarse aggregate (C:FA:CA). The ratio may be either by weight or by volume. Used for normal-weight mixes, the method probably comes from the practice of mixing a shovel of one material with two shovels of another.

ACI Consistency Description (ACI 211.3-75)

Similar to the ACI volumetric method, the consistency description differs in that proportional factors based on a consistency description are introduced. The goal of this method is a no-slump concrete mix.

Cement Strength (ACI 211.2-81)

Although the cement-strength method may be used for any type of concrete, it is especially useful for all types of lightweights. This is because it is not necessary to account for aggregate porosity, which in lightweight aggregates tends to be inconsistent. Mix is based on minimum cement content for a desired strength and the loose volume of sand and coarse aggregate.

Weight Gravimetric (ACI 211.2-81)

The weight-gravimetric method is useful for semi-lightweight mixes--those with normal-weight sand and lightweight coarse aggregate and for normal-weight mixes. This method requires that a preselected weight per cubic yard be established and met by the mix designer. Weights and volumes of cement, water, air, and coarse aggregate are selected and then sand weight is adjusted to achieve the desired final weight per cubic yard of concrete.

Adjustment

The adjustment routines may be applied to concrete batches either in conjunction with the first-trial-batch routines or as stand-alone programs. Fly-ash replacement of cement by weight or volume is possible as is the creation of per-yard batch volumes and parameters from the entry of material weights from a successful trial batch. Changes are possible to correct faults that may occur in the first batch design. Slump may be increased or decreased on the basis of a fixed amount of water or a change in water volume. Corrections for total aggregate moisture content may be made and the air-water proportional relationship can be changed by manipulating the water volume. The cost per cubic yard or batch may be computed given the unit costs of each item.

Following is a listing of the procedures available in the adjustment section and a brief synopsis of the capabilities of each.

Fly-Ash Replacement

Cement may be partially replaced with pozzolanic material. Options include maintenance of constant weight or constant volume of cementitious material. In addition the ratio of water to cement plus pozzolan $[w/(c + p)]$ may also be maintained equal to the water-cement ratio either by weight or by volume.

Increase or Decrease in Slump

A change in slump may be accomplished in either of two ways. Water volume may be changed and compensated for by a change in sand volume, or sand volume may be changed and then balanced by an adjustment in coarse-aggregate volume. If the volume of water is changed, the user is offered the option of maintaining a constant water-cement ratio.

Moisture Adjustment

The algorithm will, on request, adjust the batch mix to compensate for total moisture content of the aggregate. Corrected weights for water, coarse aggregate, and sand are provided.

Percent Air Adjustment

Changes in percent air are compensated for by changes in water volume. This procedure is useful if slump and workability are found to be acceptable but the air content must be changed. The water-cement ratio may or may not be changed, whereas the remaining ingredients are held proportionally constant by volume.

Batch Costs

Determination of batch costs for the individual batch items is provided. Item costs may be supplied in unit increments of pounds, gallons, or tons.

Information File

An information file of 18 sections is available for viewer use either as a separate reference or when needed during trial-batch design. Tables are available, for example, that recommend water-cement ratio, amount of water, proportional ratio, air content, slump, unit weight, and coarse-aggregate volume. Listings of state regulations, various cement types, thermal coefficients, and unit weight formulas are available as well.

Two output sections will be produced each time a trial batch is completed. The first is a listing of batch parameters, including the following: water-cement ratio, cement factor, proportional ratio by weight (C:FA:CA), slump, air content, unit weight, and batch size. Section 2 consists of a listing of quantities per cubic yard and per batch. Included are water by the pound and by the gallon, cement by the pound and by the bag, fly ash by the pound, sand and coarse aggregate by the pound bulk dry and by the pound saturated surface dry. Optionally, the amount of sand, gravel, and water to be added given the total moisture content of the aggregate or a listing of the total batch costs is also available. A linear plot of air content versus unit weight is available on request in any mix design in which entrained air is specified.

Each of the output sections appears on the screen and simultaneously is placed in an external file from which hard copy is available. Similarly, the plot data are placed into an external file that may be viewed on the Tektronix screen or generated in hard copy by using the Calcomp plotter.

ALGORITHM

The complete software package consists of five separately stored programs or groups of subroutines:

1. The main program and the input routines;
2. The information files;
3. The computation routines, including echo and output routine;
4. The SCAN subroutines (a tool for translating problem-oriented languages stored separately but used throughout); and
5. The externally stored plotting subroutines.

FORTRAN 77 was the programming language used as source code in Sections 1, 2, and 3. Source code for the SCAN subroutines (3) and for the plotter routines (7) is FORTRAN IV. The subroutines of the first four sections have been compiled and stored in two partitioned data sets, and the plotter routines, as previously noted, are stored externally.

The main program begins with a menu display from which the user may select the mix design method to be used [ACI volumetric, proportional, ACI consistency description (no slump), cement strength, or weight gravimetric], the information files, or the adjustment routines.

If a mix design method is chosen, the user is routed to a series of input routines that ask for data describing cement, water, air, sand, and coarse-aggregate requirements or parameters. The data required for each mix design type are listed in the section entitled Using the Program. Where possible, during data entry, the user will be routed to "help" sections if this avenue is requested. Input is completely interactive; that is, all data are supplied by the user in response to specific questions asked by the program (see Using the Program for examples).

Interactive input is checked for user error on a line-by-line basis as data are entered. Error messages may indicate either syntactical errors, misused data types, or unreasonable or out-of-range data values. In each case, an error message appears on the screen and the user is rerouted to the beginning of that input section where data may be reentered or the subprogram may be exited. On completion of input data, the user will be queried as to whether a listing of input data is needed. He may then continue to the computation section or may return to the beginning of data input to reenter data values.

When trial-batch computations have been completed, the user may proceed directly to the output routine or may select to pass through the adjustment subroutines.

The mix-adjustments section may be entered after completion of the trial-batch computations or directly when beginning the program. If entry is direct, it is necessary for the user to enter batch weights of cement, water, sand, coarse aggregate, and optionally fly-ash material. In this way, the unique parameters can be computed from a successful batch, as can any batch quantities that differ from those entered. Whether the user enters from the trial-batch routines or directly, the subsequent steps are the same. A menu board appears from which the user selects the particular process or procedure to be performed on the batch quantities. The user may select one of the following: slump increase or decrease, either with or without water volume change; fly-ash addition by weight or volume; adjustment of air percent or water volume; moisture adjustment; or calculation of batch costs.

The file section may be entered separately for information or accessed from the input routines when the user requests help. Information provided applies specifically to the various aspects of mix design. If the information files are entered via a call to help, only those tables that specifically address that area from which the call is made are available to the user. If the call for information is made from the opening menu selection board, the entire file is then available to the user. By limiting access in this manner, it was possible, through the use or multiple returns, to return the user directly to that point from which the main program was exited.

The output routines are accessible either on completion of the computation subroutines or after mix adjustment is completed. Output, as listed previously, is standard unless aggregate moisture corrections have been made, in which case the adjusted values are listed. If batch costs have been computed, the output will automatically include the results of the computations.

Significant assistance for data input is provided by the addition of the SCAN subroutines. SCAN is a translator for problem-oriented languages (POLs) developed at the University of Illinois; it provides a mechanism for entering free-format input to FORTRAN computer programs. Whether data input occurs interactively or in an unprompted environment, these routines allow the user to be free of the traditional FORTRAN-structured input environment.

The SCAN functions that appear most frequently in this algorithm are the logical functions MATCH, NUMI, and NUMR. The MATCH function compares user input to particular strings contained in programmed instructions. Where a match occurs, the program proceeds accordingly.

A useful feature of the MATCH function is its programmer-directed ability to read all or part of a word. For example, if the answer "yes" is expected in response to a question, the program, if instructed, will assume yes on receiving "y," "ye," or "yes."

The logical functions NUMI and NUMR allow the user to ignore data type when data are entered. A value entered through the NUMI function will be returned as an integer regardless of how it was entered by the user. Likewise, those values entered through the NUMR function will be read as real values even if the user has forgotten to enter a sign or a decimal point.

OPERATING SYSTEM

The program in its present version is available for the system on which it was developed. All main parts of the program have been written or compiled and stored on the TUCC IBM 3081 using the ANSI FORTRAN 77 languages and compiler.

Operations on and storage of the source code for the SCAN routines, the information files, and the computation and output subroutines are through the WYLBUR operating system. The input routines and the main program are accessible through the TSO operating system and it is on this system that the program is executed.

Compiled versions of each of the four program sections are stored in two partitioned data sets (PDSs) in the computer memory. It is the contents of these data sets that are executed when the program is used. As stated previously, the plotting subroutines are externally stored and are made available through the IBM Job Control Language (JCL) commands, which have been compiled and stored with the main program.

The microcomputer version can be used on any MS-DOS-compatible system. A newly developed system of input screens and plotter subroutines has been wedded to the original routines and a uniquely versatile system has been the result.

LIMITATIONS AND ACCURACY

Concrete mix design is at best a trial-and-error process. In the laboratory or in the field it is unlikely that the major concrete constituents will be weighed more accurately than to the nearest pound. For the program to provide batch weight results that are significantly more accurate than this would not be practical. Output values from any program routine, therefore, will be provided only to the nearest pound or gallon.

Input values will also vary in their accuracy. Specific gravities for concrete materials are traditionally reported to the nearest 1/100, and suggested water volume is usually provided to the nearest pound for the first trial batch. Accuracy beyond these norms would not be reasonable or necessary given that slight variations in aggregate gradation or moisture content, for example, are likely to occur.

The units system used in the program is the U.S. system (pounds). Optional unit selection will be offered to the programmer in the microcomputer version.

Limitations exist for the user when certain of the adjustment routines are implemented. Notable are the following:

1. If adjustments are made to any portion of the trial batch, changes to remaining material weights will be based on the ACI volumetric method and constant volume proportions will be maintained.
2. Adjustments to a trial batch that has been designed using the cement-strength method (lightweight aggregate structural concrete design) will be inaccurate if the user proceeds directly from trial batch design to the adjustments section. This is because of the marked difference in aggregate parameters, provided by the user, from those parameters used in the other methods. The user will be asked to provide additional information in this case.
3. Throughout the adjustments section it will be requested that the user enter values for specific factors: the amount of water required to change the slump 1 in. in a cubic yard of concrete, the amount of water required per cubic yard to provide workability equivalent to that provided by 1 percent entrained air, and the amount of sand required to reduce the slump 1 in. Suggested values for each are included in the program.

USING THE PROGRAM

Using the program is not complex; however, the user will be expected to provide aggregate information for proper program execution. The required information will differ slightly depending on the mix design method selected. A listing of the data required for each design method follows.

1. ACI volumetric and proportional methods:
   a. Sand--fineness modulus (FM), specific gravity [saturated surface dry (SSD) or bulk dry (BD)], and percent absorption; and
   b. Coarse aggregate--dry rodded unit weight, specific gravity (SSD or BD), percent absorption, and maximum size.
2. Proportional method:
   a. Sand--specific gravity, and
   b. Coarse aggregate--specific gravity (the program will not inquire whether it is BD or SSD; therefore the user needs to maintain consistency of type for accuracy).
3. Cement-strength method:
   a. Sand--dry loose unit weight, percent fine aggregate to total aggregate by weight, and percent total water; and
   b. Coarse aggregate--maximum aggregate size, dry loose unit weight, and percent total water.
4. Water method:
   a. Coarse aggregate--dry rodded unit weight, specific gravity (SSD or BD), percent absorption, and maximum aggregate size; and
   b. Sand--specific gravity (SSD or BD) and percent absorption.

When the user enters the adjustment routines, similar information will be requested. If cement replacement is to be performed, the specific gravity of the fly ash as well as whether the replacement is by volume or by weight will be required input. For moisture adjustment, the total percent moisture of the sand and the coarse aggregate will be necessary data. For the slump-adjustment and the air-adjustment routines, adjustment factors will be needed. Suggested values for all of the foregoing, except for the specific gravity of the fly ash, are available through the "help" requests.

Other information, including slump, air content, and water volume, will be requested of the user. As stated previously, help is available in tabular form to assist the user in arriving at a correct value. Selecting the "help" option will cause the program to provide the user with information necessary to supply the data.

Following are samples of the input required for prompted data entry. A few simple rules or guidelines should be followed when input is supplied:

1. When the requested data consist of more than one value, the information should always be entered on a single line, and a space should be skipped between values.
2. If the input is a string--for example, "yes," "help," "return," "proportional"--it is usually pos-

sible to enter only a portion of the string. A complete list of acceptable abbreviations is available.

3. Entering "return" or "r" anywhere within the program will cause the user to return either to the beginning of the program or to the beginning of that routine. In the latter case, it will be necessary to enter a second "return" or "r" to return to the beginning of the program.

4. Note that, on occasion, percentages will be requested as input. Values such as air content and absorption, which are typically reported as percentage values, will be entered as such. For example, 30 percent would be entered as 30 (not 0.30).

5. The unit weight of water is assumed to be 62.355 pcf and the specific gravity of portland cement is assumed to be 3.15.

6. If "Pause--(CR) to continue" appears on the terminal screen, the user must depress the carriage return to continue with program execution. Pauses have been inserted where it may be necessary for the user to examine the screen output but not necessarily to make a response.

A TYPICAL SESSION

Following is a typical interactive session. Strings, numbers, values, and words surrounded by quotes are input options. Capitalized portions of words are those letters required as input; T = terminal screen, U = user, * = comments (these do not appear on the screen), (CR) = carriage return.

```
   Follow usual log-on procedure.
T: Concrete--first trial batch--1985
   Select Procedure or Done
   "ACImix"                   *volumetric
   "PROportional"             *mix by ratio
   "CONsistency"              *no slump - volumetric
   "CEMent"                   *cement-strength
   "WEIght"
   "FILes"
   "ADJust"
   * Only those letters that are capitalized must
   be entered when user responds.
   "DONe"                     Entering "Return" at
                              any time will return
                              you to this point or to
                              calling routine.
U: ACI
T: Is prompting necessary? (Yes or No)
U: Y
T: Enter: Desired slump (inches)
          "Help" for listing of recommended slump
          for construction type
          "NC" for listing of State of North Caro-
          lina regulations
          "Return" to return to calling routine
U: H
T: Two tables are available, enter:
          "A" for slump recommended for construc-
          tion type
          "B" for consistency description at vari-
          ous slumps (used for no-slump concrete)
          "Return" to return to calling routine
U: A
T: Construction type       Max Slump      Min Slump
   Building columns        4              1
   Pavements and slabs     3              1
   Mass concrete           2              1
   (CR) to continue
U: *strike carriage return to continue execution
T: Would you like to view additional tables in this
   section? (Yes or No)
U: N
T: Enter: Desired slump (inches)
          "Help" for listing of recommended slump
          for construction type
          "NC" for listing of State of North Caro-
          lina regulations
          "Return" to return to calling routine
U: 4
T: Is concrete air entrained? (Yes or No)
U: Y
T: Enter: percent air (%)
          "Help" if air content unknown
          "Return" to return to calling routine
U: 5
T: Would you like a plot of unit weight versus air
   content? (Yes or No)
U: Y
T: Enter: water-cement ratio
          "Help" if needed
          "Return" to return to calling routine
U: 0.55
T: Enter: amount of water per cubic yard followed
          by "Gal" or "lb"
          "Help" if information necessary
          "Return" to return to calling routine
U: 35 Gal  *Skip space between
T: For coarse aggregate enter: (single line-space
   after each value)
          dry rodded unit weight (lb/cy), followed
          by maximum aggregate size (inches)
          "Return" to return to calling routine
U: 98.5  0.75
T: For coarse aggregate, enter the following: (sin-
   gle line-space after each)
          percent absorption
          "BD" or "SSD"
          "Return" to return to calling routine
U: 2.65  0.6  BD
T: For fine aggregate enter the following: (single
   line-space after each)
          Fineness modulus from 2.36 to 3.04 in-
          clusive
          Specific gravity
          Percent absorption (%)
          "BD" or "SSD"
          "Return" to return to calling routine
U: 2.8  2.55  1  BD
T: Volume of coarse aggregate per unit volume of
   concrete is 0.62. Would you like to change it?
   (Yes or No)
U: N
T: The following data has been entered
          Percent air                  5
          Slump                        4 in.
          Water-cement ratio           0.55
          Water (lb/cy)                291.8
          Water (gal/cy)               35
          Coarse aggregate data
             DRUW (PCF)                98.5
             MAX aggregate              0.75
             Bulk-dry specific gravity  2.65
             Absorption (%)             0.6
          Fine aggregate data
             Fineness modulus           2.80
             Bulk-dry specific gravity  2.55
             Absorption percent         1
T: Would you like to reenter data? (Yes or No)
U: No
T: Would you like to enter adjustment section?
   (Yes or No)
U: No  *For "yes" see the following example
T: Enter: desired batch size followed by "CF" or
          "CY" indicating unit to be used
          "Return" to return to calling routine
```

U: 3 CF
T: Batch size 3 cf
Water-cement ratio* 0.55
Cement factor (bags/cy) 5.6
Percent air (%) 5
Fresh unit weight (lb/cf)** 141.2
*with fly-ash replacement w/c - w/(C+P)
**unit weight is SSD except for mix by cement strength - or proportions
Pause
(CR) to continue
U: *hit return
T:

		Cubic Yard	Batch
Water	(lb)	292.0	32.4
	(gal)	35.0	3.9
Cement	(lb)	531.0	58.9
	(bags)	5.6	.6
Fly ash	(lb)	--	--
C. Agg	(lb-dry)	1649.0	183.2
	(lb-SSD)	1659.0	184.3
Sand	(lb-dry)	1318.0	146.4
	(lb-SSD)	1331.0	147.7

(CR) to continue
U: *hit return
T: Concrete -- First Trial Batch Design
Program - NCSU 1985
etc.
U: Done
T: Processing complete
To view output enter the following:
QED output.data
For hardcopy, enter the following:
Print output.data
For plot, enter the following:
Submit Clistlib.Clist (Plot)
Plot data may also be viewed on the Tektronix CRT (see Tektronix manual).

TYPICAL ADJUSTMENT SESSION

Options exist to enter the adjustment section either from the design routines or on initial entry into the program. On entry into the adjustment section, information is requested concerning batch weights. It is not necessary to enter this information unless the entry directly follows the log-on procedure. If entry follows mix design, the batch weight information is carried forward.

The example begins at that point in the interactive example where entry into the adjustment section is offered. The user in this case answers yes.

T: terminal U: user *: writer comments
T: Would you like to enter adjustment section? (Yes or No)
U: Yes
T: Enter batch weights? (Yes or No)
(per yard parameters will be calculated automatically)
*Only necessary to reenter batch weights if entry into adjustment section is direct from log-on or if batch to be adjusted differs from that already in the file.
U: No
T: Select letter or enter "Return" to return to calling routine
or "Done" to exit to output
"A" Fly-ash replacement of cement
"B" Increase slump
"C" Decrease slump
"D" Adjust for total moisture
"E" Adjust percent air
"F" Find batch cost
U: C
T: Enter: amount in inches by which slump is to be reduced followed by
"A" for reduction of water with increased sand
"B" for increased sand with decreased coarse aggregate
If "A" then follow with unit water multiplier (usually 0.75 to 1.00 gal/in./cy)
If "B" then follow with unit sand multiplier (approximately 5 percent/in./cy)
U: 3 A 1 *same line - skip space between
*T: Same water-cement ratio? (Yes or No)
(If "Yes," water-cement ratio is constant; if "No," cement weight is constant)
U: Yes
*At this point adjustments are made and user decides whether further adjustments are desired.
T: Select letter or enter "Return" to return to calling routine or "Done" to exit to output
"A" Fly-ash replacement of cement
"B" Increase slump
"C" Decrease slump
"D" Adjust for total moisture
"E" Adjust percent air*
"F" Find batch cost
U: A
T: WC constant by weight or volume? Enter "WC" or "Vol"
U: Vol
T: Enter specific gravity of fly ash
U: 2.20
T: Enter ratio of fly ash to total cementitious material
U: 0.35
T: Is that by weight "W" or Volume "V"?
U: W
At this point control transfers to calculation routines and then user is given option to continue or exit to output
T: Select letter or enter "Return" to return to calling routine or "Done" to exit to output
"A" Fly-ash replacement of cement
"B" Increase slump
"C" Decrease slump
"D" Adjust for total moisture
"E" Adjust percent air
"F" Find batch cost
U: Done *At this point the exit to output is made and output routines, as in interactive design example, are entered.

CONCLUSIONS

This algorithm represents an attempt to codify what is in reality an empirical process--portland cement concrete mix design. The program assembles information that has been compiled by various individuals and organizations, including the PCA, the ACI, Goldbeck and Gray, and Popovics.

The program purpose is threefold. It provides a starting point for the design of a first-batch concrete mix based on information provided by the user and the computer. It gives the user the opportunity to adjust the various mix components for workability based on rules of thumb and local conditions, and, finally, it provides the user with a large amount of general and specific mix design information available either en masse or tailored to fit a given set of circumstances.

For the aforementioned reasons, the program should appeal especially to student users, who will find the information helpful and the ability to apply the different design methods useful.

When the recommended features have been added and it is adapted for use on microcomputers, the program should become equally appealing to the mix designer in the industrial setting.

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The Effect of AC Overlays on D-Cracking in PCC Pavements

DONALD J. JANSSEN and BARRY J. DEMPSEY

ABSTRACT

Durability cracking (D-cracking) is the progressive deterioration of portland cement concrete (PCC) and is normally caused by winter freeze-thaw cycling. The PCC coarse aggregate source has been identified as causing well-designed mixes to develop D-cracking. A common rehabilitation procedure for D-cracked PCC pavements is to overlay the PCC with asphalt concrete (AC). This renews the surface, but little is known about the long-term effect of AC overlays on D-cracked pavements. The primary climatic factors responsible for D-cracking are moisture and temperature. Finite-difference transient flow computer moisture movement modeling as well as field instrumentation and laboratory measurements indicate that AC overlays have a negligible effect on the PCC pavement moisture regime. The effect of AC overlays on the PCC temperature regime was evaluated by finite-difference heat transfer computer modeling. AC overlays were found to decrease the number of freeze-thaw cycles and the rate of cooling in PCC pavements. Laboratory freeze-thaw durability tests duplicating field conditions for Interstate 70 near Vandalia, Illinois, were conducted on PCC samples with AC overlays 0 to 6 in. thick. All the PCC samples cycled to the equivalent of 5 years of winter exposure showed strength loss as determined by split tensile tests. The samples with 4-in. overlays showed the most strength loss. It was concluded that AC overlays do not prevent the progression of D-cracking in PCC; instead some overlay thicknesses accelerate it. When AC overlays are designed for D-cracked PCC pavements, the effect of decreasing strength of the deteriorating PCC should be considered.

Durability cracking (D-cracking) in portland cement concrete (PCC) is the progressive deterioration of the concrete caused by environmental factors. It can occur even when the PCC has not been subjected to physical loading and is especially common in PCC pavements that are exposed to winter freeze-thaw cycling. Figure 1 shows a severely D-cracked PCC pavement. As D-cracking progresses in a PCC pavement, the load-carrying capacity and the ability of the pavement to resist deformation and cracking under repeated traffic loading gradually decrease. With time, this deterioration can lead to total structural failure of the pavement and loss of serviceability to the user.

Studies in Illinois (1), Ohio (2), and Iowa (3), among others, have identified coarse aggregate sources that lead to D-cracking in what would normally be considered good, durable mixes. The fine aggregate source has been found to have little significance in whether a mix develops D-cracking or not (4). Extensive testing programs (1-3) to identify aggregates susceptible to D-cracking have helped to ensure that new pavements are not likely to develop D-cracking in the future. But there remain many miles of D-cracked pavement that require some form of rehabilitation in order to continue to provide adequate serviceability to the user.

A common rehabilitation procedure is to overlay existing D-cracked PCC pavement with a thin layer of asphalt concrete (AC). These AC overlays provide immediate improvement in the riding quality of D-cracked PCC pavements. Little is known about the long-term effects of AC overlays on the continued durability performance of the PCC. Can AC overlays perform well over D-cracked PCC pavements? Will the PCC continue to perform adequately? Although a layer of AC on top of the PCC will tend to decrease the number of freeze-thaw cycles that occur in the PCC, it also tends to decrease the rate of cooling. The literature differs on whether a faster cooling rate (5,6) or a slower cooling rate (7,8) is more detrimental to PCC. In addition, the AC overlay would form a barrier on top of the PCC that may prevent the evaporation of water. With evaporation reduced by an AC overlay, the PCC could be at a higher degree of saturation before winter freezing, which would lead to more severe damage (5). A thorough study to determine the effects of AC overlays on the moisture and temperature regimes in D-cracked PCC pavements is needed to predict the long-term effects of AC overlays on the progression of D-cracking in PCC pavements.

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OBJECTIVES

The general objective of this project was to evaluate the influence of various thicknesses of AC overlays on the progression of D-cracking in PCC pavements. The specific objectives were as follows:

1. Determine the changes in the moisture and temperature regimes in D-cracked PCC pavements resulting from various AC overlay thicknesses,
2. Conduct laboratory freeze-thaw durability tests on D-cracked PCC samples with various AC overlay thicknesses and attempt to correlate any

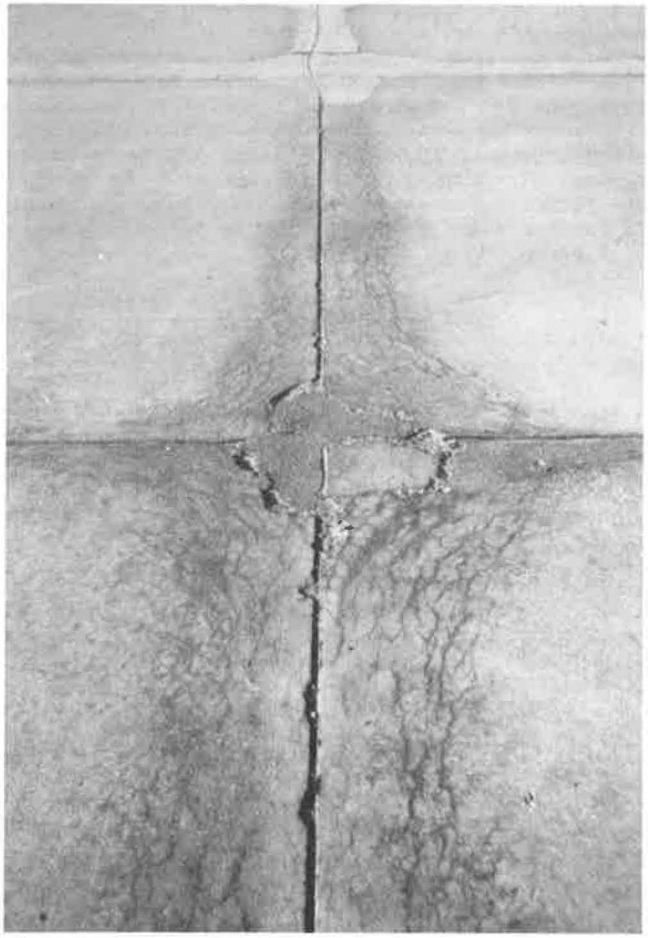

FIGURE 1 Severely D-cracked PCC pavement.

strength variations that occur with changes in sample moisture and temperature regimes, and

3. Make recommendations for design procedures that take into account the effects of the AC overlay on the continued durability performance of the PCC.

RESEARCH APPROACH

The important climatic parameters responsible for D-cracking are moisture and temperature. The PCC moisture regime was evaluated by computer modeling, laboratory moisture content measurements, and field moisture determinations. The PCC temperature regime was evaluated by computer modeling. The combined effects of changes in the temperature and moisture regimes on the progression of D-cracking were evaluated by laboratory testing that duplicated winter field conditions.

A necessary material property for moisture regime modeling is moisture characteristics, which is the relation between equilibrium moisture content and matric suction. Matric suction is the force that causes capillary rise. It is equivalent to the distance above the water table for equilibrium conditions when no flow is occurring. Detailed discussions of moisture characteristics and matric suction have been given by Janssen and Dempsey (9) and Hillel (10). Figure 2 is a moisture-characteristics curve for laboratory D-cracked PCC. It should be noted that there is very little decrease in moisture content for large increases in suction. This is due to the very fine pore structure of PCC.

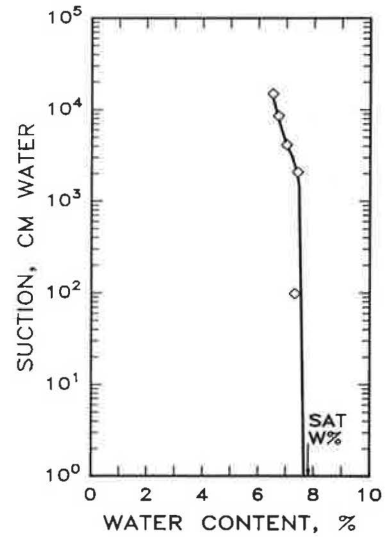

FIGURE 2 Moisture-characteristics curve for laboratory D-cracked PCC.

Figure 3 is a plot of unsaturated hydraulic conductivity versus moisture content for laboratory D-cracked PCC. This was determined from the moisture-characteristics curve and measured saturated hydraulic conductivity, or permeability (9). It should be noted that the unsaturated hydraulic conductivity decreases rapidly with a small decrease in moisture content. Details of the moisture characteristics and saturated hydraulic conductivity measurements have been given by Janssen (11).

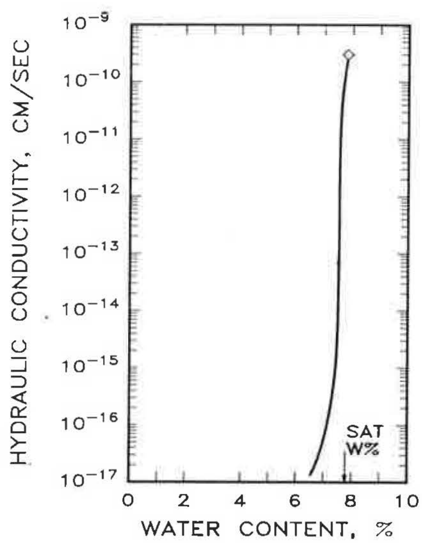

FIGURE 3 Unsaturated hydraulic conductivity curve for laboratory D-cracked PCC.

Moisture movement was modeled by means of a finite-difference transient flow computer program developed by Boast (12). The pavement section modeled was an 8-in.-thick PCC layer with free water at the bottom and a relative humidity of 50 percent at the top. The initial moisture content in the PCC was 5.7 percent. The results of moisture movement modeling for a period of 3 months are shown in Figure 4.

Field moisture contents were determined with psy-

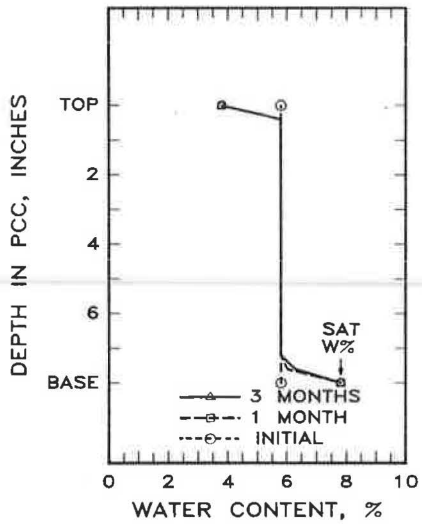

FIGURE 4 Moisture movement in PCC with 5.7 percent initial moisture content.

chrometers (13) installed in I-72 near Champaign, Illinois. Psychrometers, which have been used in the agricultural community for many years, actually measure matric suction, which is converted to moisture content with a moisture-characteristics curve. The psychrometers were installed at depths of 2 and 4 1/2 in. into the PCC for both overlaid and non-overlaid D-cracked PCC. Table 1 gives psychrometer moisture content determinations for March through November 1984. A moisture content of 7.6 percent was complete saturation.

Moisture content measurements were made on PCC samples that were freeze-thaw cycled in the laboratory. The samples were 4-in.-diameter by 8-in.-long cylinders, the bottoms of which were in contact with a moist crushed-stone base. Figure 5 gives moisture content distributions for samples with no overlay and Figure 6 for samples with 2-in. overlays. Saturation for these samples was at a moisture content of 7.8 percent.

TABLE 1 Average Moisture Contents Determined with I-72 Psychrometers

	Moisture Content (%) by Psychrometer Depth (in.)			
	No Overlay		Overlay	
Date	2	4 1/2	2	4 1/2
3/22	6.6	6.4	7.3	7.4
7/10	7.2	7.3	–	7.4
7/11	6.9	7.2	7.1	6.8
7/18	6.5	7.0	7.4	–
7/25	6.2	7.4	6.3	6.4
7/30	6.1	–	7.4	–
7/31	7.0	6.7	–	–
8/3	6.8	6.9	6.8	6.9
8/14	6.8	6.8	7.2	6.5
8/21	6.2	6.4	6.7	6.8
8/22	6.5	6.5	–	–
8/28	6.2	6.5	6.8	–
9/4	6.2	6.9	6.5	6.2
9/6	6.1	6.1	6.9	7.3
9/11	7.0	6.8	7.1	7.0
9/13	7.2	–	–	–
9/18	6.5	–	6.2	–
9/19	6.6	7.3	–	7.3
9/20	6.5	–	7.3	–
9/25	6.9	6.7	7.0	7.0
9/27	7.0	–	–	–
10/2	6.2	–	6.6	–
10/4	6.3	–	6.5	–
10/8	6.4	6.6	6.5	6.4
10/10	6.8	6.7	6.8	7.3
10/24	6.2	7.3	7.1	6.5
11/2	6.1	6.0	–	–
11/5	–	–	6.7	7.2
11/7	–	–	6.8	–
11/12	6.6	7.3	6.8	–
11/13	7.1	–	6.8	–
11/30	–	–	6.6	6.5

Note: Dashes indicate that no data were available.

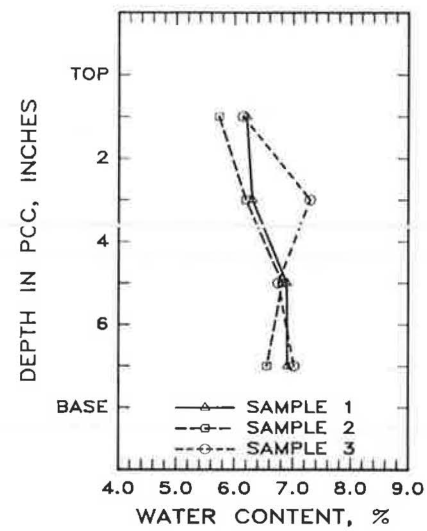

FIGURE 5 Moisture content distributions, 5-year samples, no overlay.

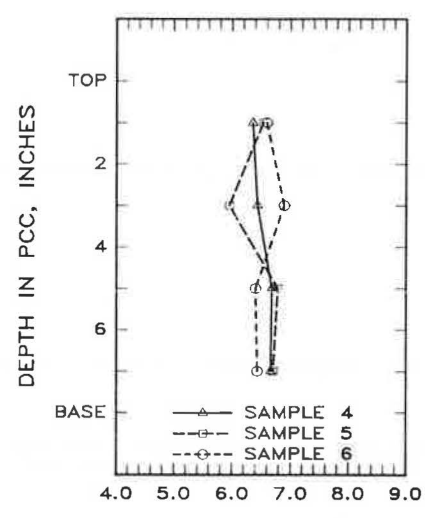

FIGURE 6 Moisture content distributions, 5-year samples, 2-in. overlay.

The temperature regime was modeled with a finite-difference heat transfer computer program developed by Dempsey (14). The modeled pavement section consisted of an 8-in. PCC surface on a 4-in. stabilized base with an AASHTO A-6 subgrade. Input consisted of climatic data including wind speed, cloud cover, daily high and low temperatures, and daily solar radiation. Climatic data from 1965 to 1980 were used to determine the average number of freeze-thaw cycles at the pavement surface for various locations

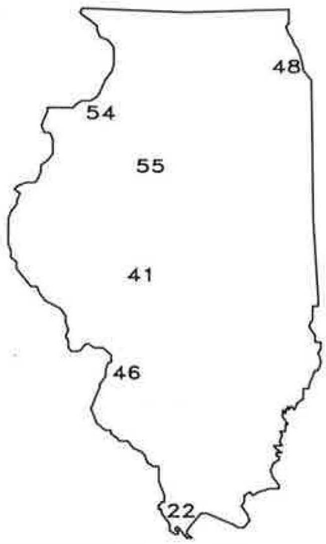

FIGURE 7 Number of freeze-thaw cycles in Illinois, top of PCC surface.

in Illinois (Figure 7). January 1971 was determined to be typical for winters in the St. Louis, Missouri, area. This time period was used to evaluate the effects of various AC overlay thicknesses on the cooling rate, the number of freeze-thaw cycles, and the minimum temperature below freezing (Table 2).

TABLE 2 Effects of AC Overlays on Freeze-Thaw Parameters, St. Louis, Missouri

	Overlay (in.)			
Parameter	None	2	4	6
No. of freeze-thaw cycles	19	7	4	2
Cooling rate, avg (F°/hr)	1.19	0.56	0.24	0.20
Minimum temperature below freezing (F°)	10	17	22	25

Freeze-thaw durability testing was performed in a programmable freeze-thaw durability testing unit developed by Dempsey (15). The specimens consisted of 4-in.-diameter by 8-in.-long PCC cylinders on dense graded crushed-stone bases (Illinois CA-6 gradation). The bottoms of the bases were in contact with water. The specimens had either no AC overlay or 2-, 4-, or 6-in.-thick overlays. The base-course thicknesses were adjusted depending on the overlay thickness in order to give a total specimen height of 18 in. (Figure 8). The sides of the specimens were wrapped with plastic, and insulation was provided between them to ensure that both moisture and heat flow were only vertical. The PCC samples were made from the same D-cracking susceptible coarse aggregate that was used in a D-cracked section of I-70 near Vandalia, Illinois.

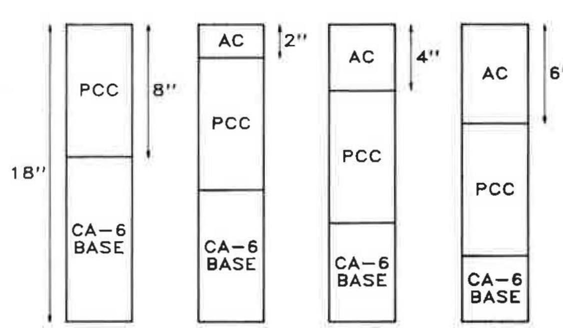

FIGURE 8 Schematic of freeze-thaw durability specimens.

The freeze-thaw cycle used in the laboratory was determined from heat transfer computer analysis of climatic data for St. Louis, Missouri, which included total daily sunshine, daily high and low temperatures, and daily average wind speed. This was the source of necessary climatic data closest to the Vandalia site being modeled. The values of pertinent freeze-thaw parameters for January 1971 for the St. Louis area, which were used to develop the laboratory freeze-thaw cycle shown in Figure 9, are given in Table 3, which also gives a summary of the freeze-thaw parameters for the laboratory freeze-thaw cycle.

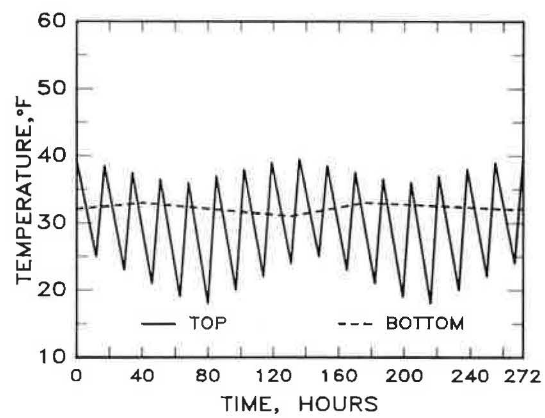

FIGURE 9 Laboratory freeze-thaw cycle: top and bottom of 18-in. specimen.

TABLE 3 St. Louis, Missouri, and Laboratory Freeze-Thaw Parameters

	St. Louis Values		Laboratory Values		
Parameter	Mean	Sample Standard Deviation	Mean	Minimum	Maximum
Cooling rate (F°/hr)	1.2	0.40	1.35	1.21	1.50
Minimum temperature below freezing (F°)	22.8	4.31	21.5	18.0	25.0
Difference between maximum and minimum temperature (F°)	17.8	6.01	16.3	14.5	18.0

Results of split-tensile tests on the PCC samples that were freeze-thaw cycled to the equivalent of approximately 2 and 5 years of winter exposure are shown in Figure 10. Figure 11 shows split tensile strength loss versus overlay thickness for the 5-year samples.

DISCUSSION OF RESULTS

The moisture movement modeling (Figure 4) indicated that moisture movement in PCC is extremely slow. After 3 months, drying of the PCC extended less than 1 in. into the PCC. This is due to the extremely low unsaturated hydraulic conductivity of the PCC. Continued drying decreases the unsaturated hydraulic conductivity even further, which tends to minimize moisture loss at depths greater than 1 in. This im-

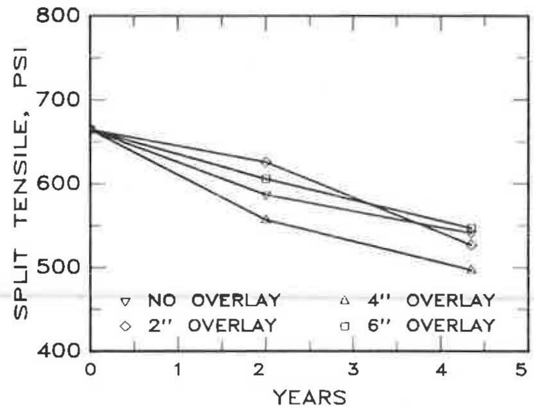

FIGURE 10 Split tensile strength versus years of freeze-thaw cycle.

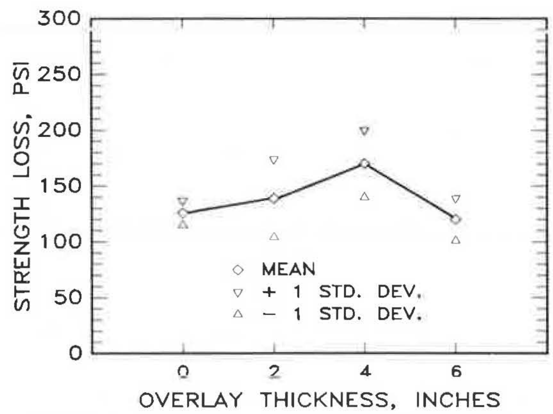

FIGURE 11 Strength loss versus overlay thickness, 5-year samples.

plies that AC overlays that prevent evaporation would have little effect on the PCC moisture regime, except near the surface where drying would be prevented.

The field moisture determinations (Table 1) show little difference between moisture contents at depths of 2 in. and depths of 4 1/2 in. There is also little difference between moisture contents of overlaid and nonoverlaid sections. This is possibly because the measurement at 2 in. is too deep to detect surface drying.

The laboratory moisture measurements (Figures 5 and 6) show drying at the tops of the nonoverlaid samples and no drying under the overlay. Actual drying at the top surface of the nonoverlaid PCC is probably of greater magnitude than is shown. Each measurement represents the average moisture content for 2 in. of sample and would not show any variation in that 2 in.

AC overlays have a definite effect on the temperature regime in PCC pavements. Table 2 indicates that an overlay decreases both the number of freeze-thaw cycles and the cooling rate at the PCC pavement surface. Although a decrease in the number of freeze-thaw cycles is beneficial, it is possible that a slower freezing rate could cause increased damage (8).

The results of the laboratory freeze-thaw durability tests indicate that none of the tested overlay thicknesses prevented D-cracking. Overlay thicknesses in the 2- and 4-in. range caused increased freeze-thaw damage to the PCC. Statistical analysis of the 5-year samples showed that there was a significant strength difference between the 4-in. and the 6-in. overlay samples at α equal to 5 percent. At α equal to 7 percent there was a significant strength difference between the 4-in. overlay and the nonoverlaid samples. Figure 11 suggests that there is some overlay thickness greater than 6 in. that would prevent freeze-thaw strength loss. This would probably be due to the prevention of freezing in the PCC.

CONCLUSIONS AND RECOMMENDATIONS

The research conducted in this investigation has led to the following conclusions:

1. The PCC pavement moisture regime is unaffected by AC overlays except within the top 1 to 2 in. of the PCC. Most of the D-cracked PCC remains at a relatively high moisture content, and evaporative drying has little effect except at the surface.
2. AC overlays cause significant changes in the pavement temperature regime. These changes include slowing the cooling rate, decreasing the number of freeze-thaw cycles, and raising the minimum pavement temperature.
3. No common AC overlay thickness stops the progression of D-cracking. Thicknesses in the 2- to 4-in. range actually accelerate the rate of freeze-thaw damage. Although AC overlays decrease the number of freeze-thaw cycles, which is beneficial, they also decrease the cooling rate. The combined effect of decreased number of freeze-thaw cycles and decreased cooling rate was detrimental for PCC samples with 2- and 4-in. overlay thicknesses.

It is recommended that when an AC overlay is an alternative for D-cracked PCC rehabilitation, the following should be considered:

1. The structural design should take into account the continued deterioration of the PCC, and
2. The economic analysis should consider a shortened design life due to the decreasing PCC strength.

Further research should be conducted in the following areas:

1. The effect of various slow cooling rates (between 0.2 and 1.5°F/hr) on the progression of D-cracking;
2. The effect of different aggregates on the rate of D-cracking progression caused by slow cooling rates; and
3. The effect of reduced PCC strength on AC overlaid PCC pavement performance.

ACKNOWLEDGMENTS

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Laboratory Investigation of Conventional and Polymer-Modified Concretes and Their Use for Repairs

STELLA L. MARUSIN

ABSTRACT

Four-inch (10-cm) concrete cubes were cast by using conventional portland cement concretes with a water-to-cement ratio of 0.35, 0.44, and 0.54 and polymer-modified concretes made with latex and epoxy modifiers. Four-inch cubes were also cut from latex-modified shotcrete repairs in the field. All cubes were immersed for 21 days in 15 percent NaCl solution and then stored in a controlled-climate room in accordance with the test procedures described in NCHRP Report 244. During 21 days of soaking and 21 days of final air drying, weight gains and losses were determined at 3, 7, 14, and 21 days during each period. Following air drying, powder samples were removed by drilling at several intervals of depth from the center of each face to the center of the cube. Then half of each cube was crushed to powder and the acid-soluble chloride ion contents of all samples were measured by a potentiometric titration procedure. At a depth of 1 1/2 to 2 in. (37 to 50 mm), the latex-modified concretes had the least amount of chloride. Laboratory tests of the influence of high temperature and wind on hand-placed and on form-cast large vertical repairs using the modified concretes showed that under arid conditions, the hand-placed repairs always cracked. Accelerated-weathering tests on small repairs made with the polymer-modified concretes left the repairs intact. On the basis of these test results, the acrylic latex modified cast-in-place concrete was successfully used to repair columns, spandrels, and balcony slabs of a high-rise housing complex and modified shotcrete acrylic latex was used to repair underground garages. Both types of repairs were in excellent condition after 5 years' service.

Deterioration of reinforced-concrete structures that contain calcium chloride (the most commonly used admixture) or that are exposed to chloride ion during their service life by exposure to ocean water or deicing salts during the winter is common and is a serious problem in many parts of the world. On the basis of research undertaken especially during the last two decades, it is now well known that the delamination and spalling of reinforced-concrete structures is caused by the electrochemical corrosion of embedded reinforcing steel within chloride-contaminated concrete. If the supply of water or oxygen can be excluded, the corrosion process will not take place.

One form of corrosion protection for concrete against the ingress of water and oxygen (and chloride ion from external sources) is the surface application of sealers, coatings, and membranes; another form can be based on the use of various chemicals added to the fresh concrete. Such specialty concretes are commercially available, and they are being used to construct new structures as well as to repair older, deteriorated ones. Such concretes have low to extremely low permeability when compared with normal concrete. The lower permeability could be attributed primarily to a reduction in the water-to-cement (w/c) ratio (1). The presence of various chemicals in concrete allows the routine production of w/c ratios of 0.26 to 0.35, values significantly lower than 0.40 to 0.55, which are commonly used and specified for concrete structures. Especially during the last decade, the polymer-modified concretes have been commonly used for vertical and horizontal patches and also for repairs by shotcrete.

The laboratory experiments described here were designed

1. To determine the saltwater weight gain and subsequent weight loss by vapor transmission, and the chloride ion content and actual chloride ion distribution profiles in conventional portland cement and polymer-modified concrete,
2. To investigate the influence of high temperature and wind on hand-placed or form-cast large vertical repairs made from polymer-modified concretes, and
3. To investigate the influence of accelerated-weathering tests on small repairs made from polymer-modified concretes.

LABORATORY INVESTIGATION

Determination of the Saltwater Weight Gain and Subsequent Weight Loss

The testing method was based on techniques developed and used by Wiss, Janney, Elstner Associates, Inc. (WJE), in a research project for the National Cooperative Highway Research Program as reported in NCHRP Report 244 (2). Six concrete mixtures were prepared, and the following materials were used in this investigation:

Wiss, Janney, Elstner Associates, Inc., 330 Pfingsten Road, Northbrook, Ill. 60062.

TABLE 1 Properties of Fresh and Hardened Concretes

Type of Concrete	SSD Quantities[a] (lb/yd^3)					Air Content (%)	Slump (in.)	w/c Ratio by Weight	28-Day Compressive Strength (psi)
	Cement	Sand	Gravel	Water	Admixtures[a]				
Conventional	522	1,226	1,857	281	–	5.4	7 1/2	0.54	4,330
Conventional	524	1,230	1,866	230	–	7.0	5	0.44	4,790
Conventional	561	1,317	1,998	196	–	6.0	2	0.35	6,640
With acrylic latex	666	1,234	1,830	192	184	2.2	10	0.29	4,200
With S-B latex	664	1,200	1,793	224	178	3.5	10	0.34	5,200
With epoxy modifier	661	1,255	1,891	171	132	4.3	9 1/4	0.26	4,850

Note: 1 psi = 6.89 kPa, 1 in. = 25 mm, SSD = saturated surface dry; S-B = styrene-butadiene.
[a]Including water contained in admixtures.

1. Cement: Type I portland cement;
2. Sand and gravel: natural river sand and river gravel [with maximum-size coarse aggregate = 3/4 in. (19.05 mm)] from the American Materials Corporation, Eau Claire, Wisconsin (both chloride-free materials);
3. Air-entraining agent: neutralized vinsol resin; and
4. Polymers: high-viscosity and low-modulus epoxy, acrylic, and styrene-butadiene latex.

The properties of the fresh and the hardened concretes are listed in Table 1.

Casting of nominal 4-in. (10-cm) concrete cubes is shown in Figure 1. After being stripped at the age of 1 day, the cubes were lightly sandblasted (Figure 2) and then, as shown in Figure 3, placed in sealed heavy-duty plastic bags for moist curing. At 21 days the cubes were removed from the plastic bags, weighed to the nearest 0.1 g, and then stored in a controlled-climate room at 73 ± 3°F (23 ± 2°C) and 50 ± 4 percent relative humidity for 14 days of air drying before being immersed in a 15 percent NaCl solution. Six cubes stored in a plastic container are shown in Figure 4. Following soaking, the cubes were stored again for 21 days in the controlled-climate room to determine vapor transmission characteristics. Cubes stored in the climate room are shown in Figure 5. During the 21 days of soaking and 21 days of final air drying the weight of each cube at 3, 6, 9, 12, 15, 18, and 21 days was determined to the nearest 0.1 g. After the final 21-day air-drying period, a 1/4-in. (6-mm) hole was drilled through the center of the face of all six sides of

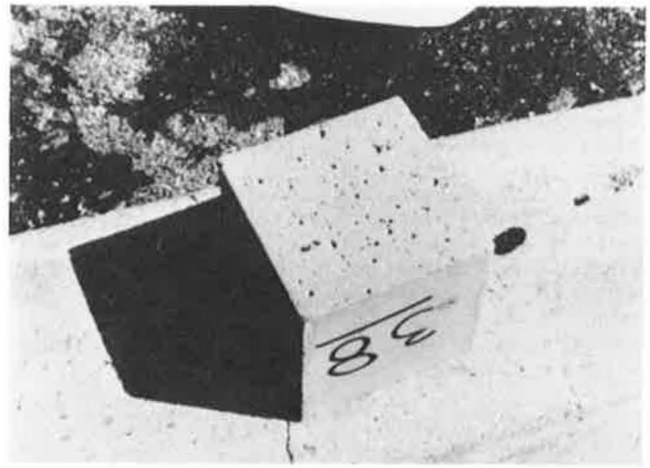

FIGURE 2 Concrete cube before (side surface) and after (top surface) light sandblasting.

the cube to obtain powder samples for chloride determination at different depth intervals. These intervals were 0 to 1/2 in. (0 to 12 mm), 1/2 to 1 in. (12 to 25 mm), 1 to 1 1/2 in. (25 to 37 mm), and 1 1/2 to 2 in. (37 to 50 mm). The chloride ion content of the drilled powder from the composite sample from the six holes was determined by using an acid-digestion, potentiometric titration procedure. Then each cube, as shown in Figure 6, was mechanically

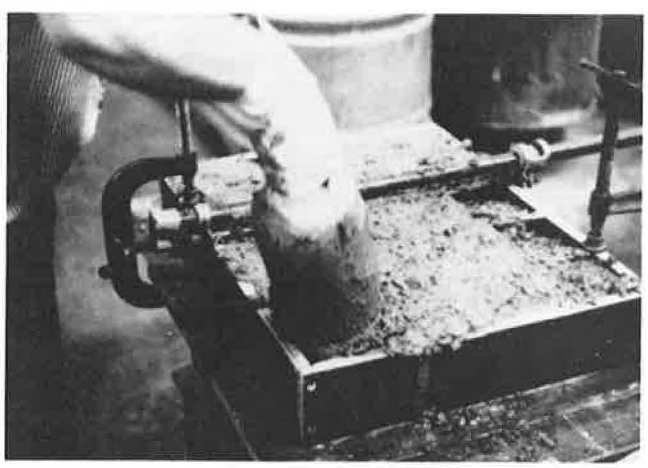

FIGURE 1 Casting the cubes.

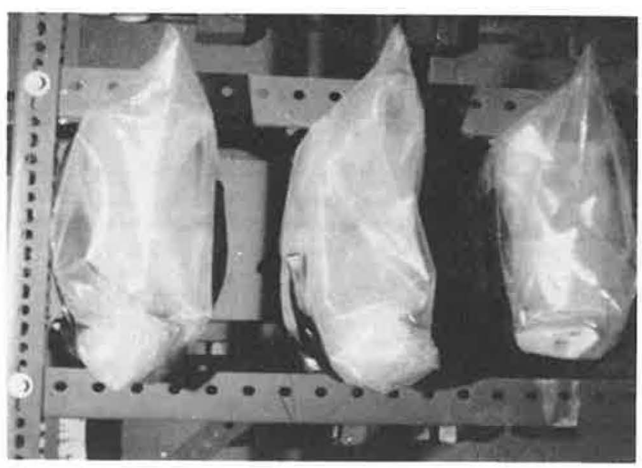

FIGURE 3 Cubes being cured in sealed heavy-duty plastic bags.

FIGURE 4 Six cubes in plastic container with 3 gal of 15 percent NaCl solution.

FIGURE 6 Cube drilled with holes to obtain samples for chloride ion determination at different depths.

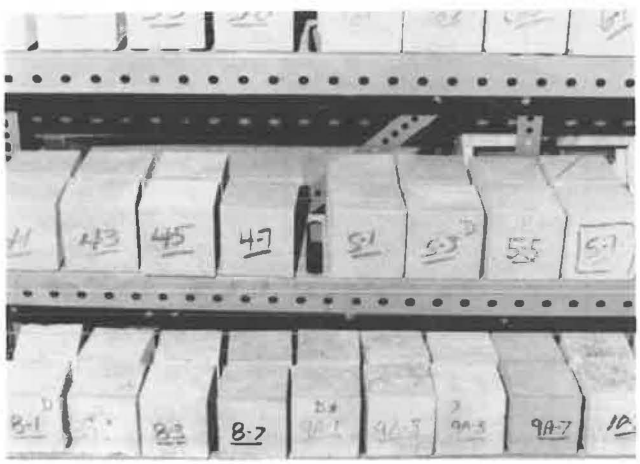

FIGURE 5 Cubes stored in a controlled-climate room at 73 ± 3°F (23 ± 2°C) and 50 percent relative humidity.

split in half. One-half was crushed and the acid-soluble chloride ion content was determined by using the method discussed earlier.

In addition the same type of testing was performed by using the 4-in. cubes that were cut during field inspection from latex- modified shotcrete.

All tests were performed on duplicate specimens and average values are listed. The test results of saltwater weight gain, weight loss by vapor transmission after the final air-drying period, and chloride ion content are listed in Table 2. The chloride ion content at different depths is given in Table 3.

The chloride ion profiles of the cubes, expressed as percentage of chloride ion at the depth interval from 0 to 1/2 in. (0 to 12 mm), are shown in Figure 7.

As shown in Table 2, the weight gain and chloride ion content in conventional concrete decreased with decrease of the w/c ratio. The three polymer-modified concretes exhibited low but widely different weight gains and chloride ion contents. The epoxy-modified concrete with the highest weight gain had the lowest w/c ratio, 0.26. The concrete containing acrylic latex (cast cubes or shotcrete) exhibited the lowest chloride ion content from all tested concretes. In terms of chloride ion content, no differences were found between the cast concrete and in-place shotcrete. All concretes were able to expel some of the absorbed water by normal vapor transmission. With the exception of acrylic-latex modified concrete (which lost 105 percent of the water), the concretes were able to expel about 65 percent of the absorbed water.

Plots of maximum weight gain and absorbed chloride ion content versus w/c ratio of half cubes are shown in Figure 8. These data show a good relationship among the cube weight gain, half-cube chloride content, and w/c ratio. Only the epoxy-modified concrete lacked this good relationship.

The data in Table 3 show that the depth interval 0 to 1/2 in. has extremely high levels of chloride, ranging from about 0.30 to 0.45 percent by weight of concrete. These values are thus about 10 to 15 times the corrosion threshold level (for acid-soluble

TABLE 2 Weight Gain and Weight Loss During Soaking and Final Air-Drying Period and Chloride Ion Content

Type of Concrete	w/c Ratio	Weight Gain after 21 Days of Soaking (%)	Weight Loss after 21 Days of Air Drying (%)	Chloride Ion Content (% by wt of concrete)
Conventional	0.54	2.50	0.82	0.259
Conventional	0.44	2.03	0.76	0.245
Conventional	0.35	1.23	0.43	0.159
With acrylic latex	0.29	0.65	−0.03	0.094
With S-B latex	0.34	1.04	0.35	0.146
With epoxy modifier	0.26	1.43	0.42	0.226
Shotcrete with latex acrylic	0.30	0.98	0.33	0.097

TABLE 3 Acid-Soluble Chloride Ion Content

Type of Concrete	w/c Ratio	Chloride Ion (% by wt of concrete) by Depth (in.) 0-1/2	1/2-1	1-1 1/2	1 1/2-2
Conventional	0.54	0.451	0.263	0.052	0.036
Conventional	0.44	0.267	0.121	0.020	0.017
Conventional	0.35	0.427	0.085	0.014	0.012
With acrylic latex	0.29	0.273	0.032	0.018	0.008
With S-B latex	0.34	0.296	0.028	0.012	0.007
With epoxy modifier	0.26	0.343	0.115	0.027	0.017

Note: Values are averages of two samples. 1 in. = 25 mm.

chloride ion content in normal-weight concrete) of about 0.03 percent. The conventional concrete with a w/c ratio of 0.54 contains chloride levels totally above the 0.03 level at all depth intervals. The conventional concretes with w/c ratios of 0.35 and 0.44 have values less than the 0.03 value at depth intervals of 1 to 1 1/2 in. and 1 1/2 to 2 in.

Concretes with acrylic and styrene-butadiene (S-B) latex show similar profiles at all depths even though their weight-gain behavior was different. The epoxy-modified concrete that had the lowest w/c ratio, 0.26, exhibited relatively poor performance when compared with the two latex-modified concretes.

The plot of chloride content within the depth intervals 0 to 1/2 in., 1/2 to 1 in., and 1 to 1 1/2 in. versus the w/c ratio in Figure 9 shows that interval 0 to 1/2 in. has extremely high chloride levels irrespective of the w/c ratio or concrete type. Thus, the chloride content in the first 1/2-in. interval does not appear to correlate well with the w/c ratio. The interval 1/2 to 1 in. shows an excellent relationship between chloride content and w/c ratio for seven of the eight concretes. Here again, the epoxy-modified concrete was inconsistent. In this interval, the chloride content was reduced by over 90 percent as the w/c ratio decreased from 0.54 down to about 0.29. A similar 90 percent decrease is not apparent in the interval 0 to 1/2 in.

The interval 1 to 1 1/2 in. shows a good relationship between chloride content and w/c ratio. In this interval, the chloride content was reduced by over 75 percent as the w/c ratio decreased from 0.54 to about 0.29 to 0.35.

It is noteworthy that five of the six concretes had less than the 0.03 percent chloride content level in the interval 1 to 1 1/2 in.

As anticipated, the poorest performance was by

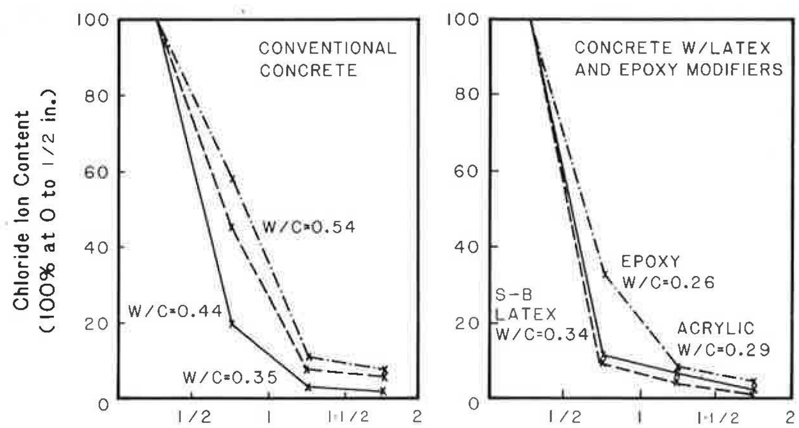

FIGURE 7 Chloride ion content at different depths.

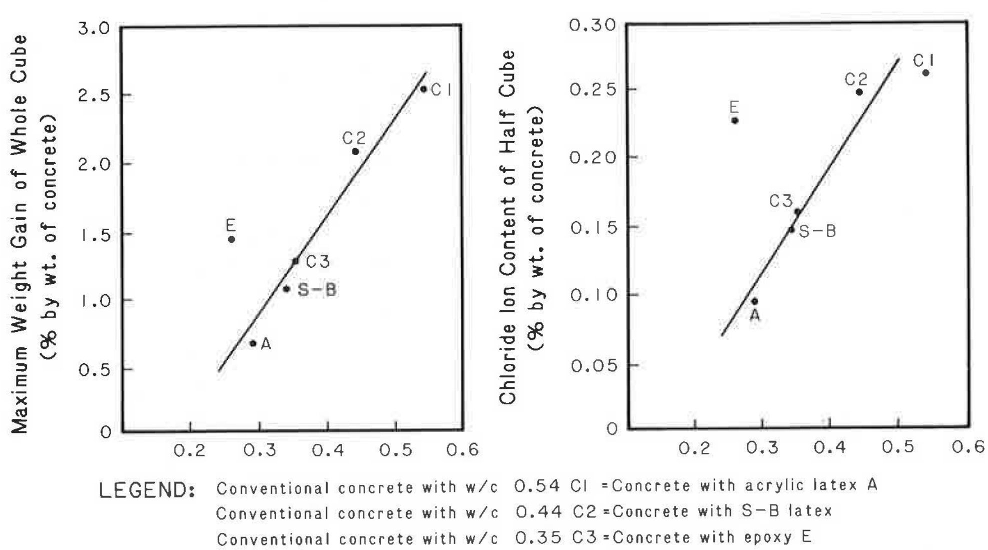

FIGURE 8 Maximum weight gain of cube versus w/c ratio *(left)*; chloride ion content of half cube versus w/c ratio *(right)*.

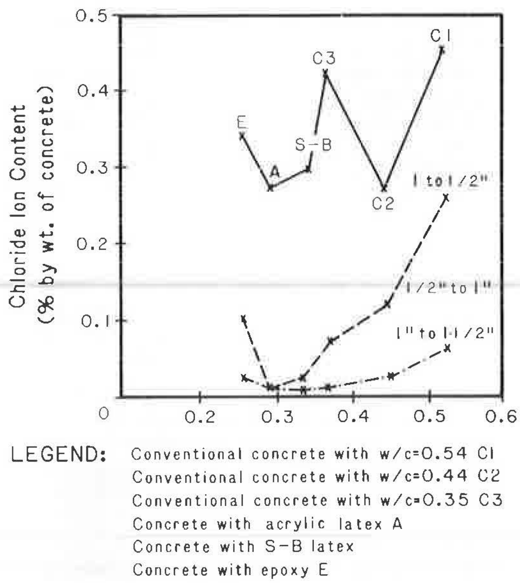

FIGURE 9 Chloride ion content within different depths versus w/c ratio.

the conventional concrete with a w/c ratio of 0.54. The best performance was by the two latex-modified concretes. Both concretes with w/c ratios of 0.29 to 0.34 show almost identical chloride ion profiles.

Figure 7 shows that the shape of the chloride ion distribution within these cubes is extremely steep within the first 1 in. In fact, from 90 to 96 percent of all the absorbed chloride is contained within the first 1 in. The shape of these distribution profiles is influenced by the w/c ratio and the concretes with the lowest w/c ratios possess the steepest gradients.

Influence of High Temperature and Wind on Hand-Placed or Form-Cast Large Vertical Repairs

The concretes with acrylic and S-B latexes and with epoxy modifier were hand placed (Figure 10) and cast into large vertical wooden forms in rooms having an air temperature of 70° and 105°F (21.1° and 40°C) with fans providing air circulation similar to that normally occurring on the side of the building during the summer. These tests with hand-placed polymer-modified concretes were totally unsuccessful because all tested specimens developed cracking problems due to cold joints and surface crazing and cracking (Figure 11). When the same polymer-modified concretes were used to make pea-gravel-sized concrete repairs in the laboratory under the same conditions with the exception that they were cast into forms, absolutely no cracking developed, even after 6 months of exposure to the aforementioned temperature conditions. The poor performance of the hand-placed repairs was similar to that observed on several actual repair projects.

Influence of Accelerated Weathering Tests on Small Repairs

As shown in Figure 12, the polymer-modified concretes were hand placed into vertically positioned lightweight and normal-weight concrete beam specimens in which repair holes had been prepared. Each beam contained three holes: two 2 in. deep and one 1 in. deep. As shown in Figure 13, dikes were added into one 2-in.-deep filled hole and the beams were then

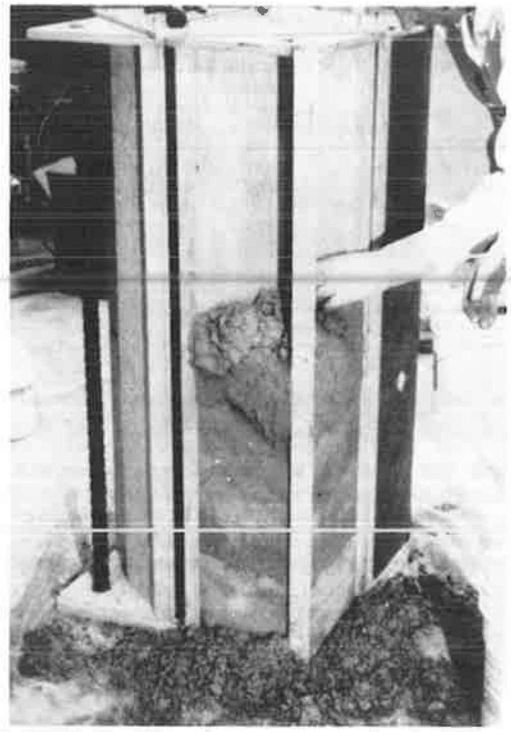

FIGURE 10 Concrete being hand placed into the large vertical wooden form.

subjected to 16 weeks of accelerated-weathering tests that included wetting, freezing temperatures to 15°F (-10°C), thawing, ultraviolet light exposure, and elevated temperatures of 130°F (54.4°C). After these tests, the specimens were examined for cracking or spalling. The concretes containing latexes and a high-viscosity and low-modulus epoxy modifier did not show any surface changes.

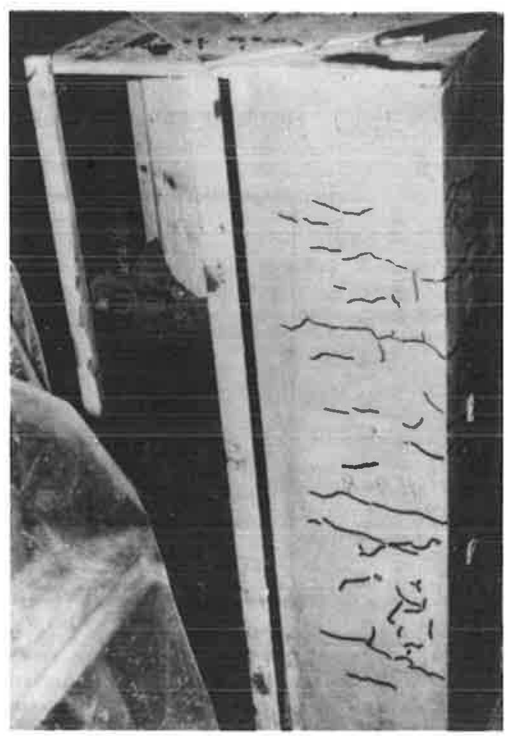

FIGURE 11 Cracking on concrete surface.

FIGURE 12 Concrete being hand placed into vertically positioned repair holes.

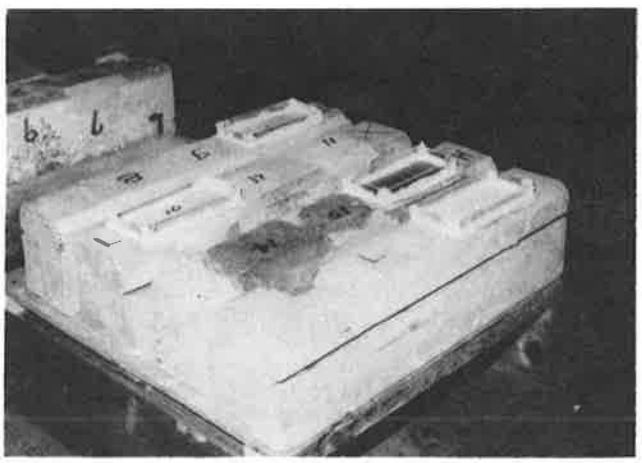

FIGURE 13 Repairs during the laboratory testing.

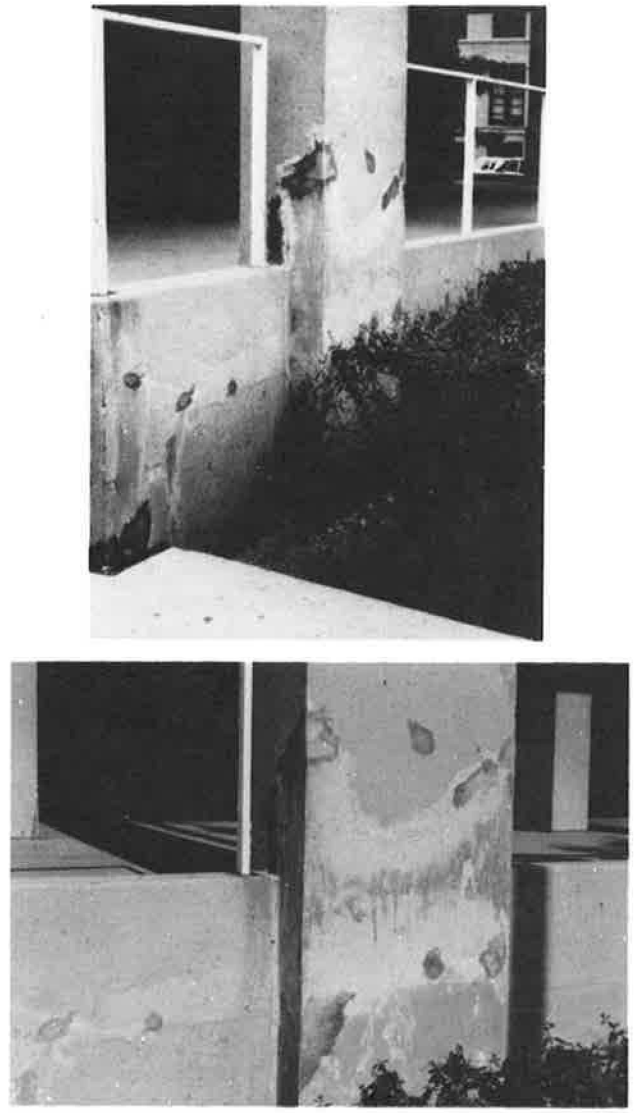

FIGURE 14 Repairs initially *(top)* and after 5 years in service *(bottom)*.

USING THE POLYMER CONCRETE FOR REPAIRS

On the basis of laboratory experiments, the concrete containing acrylic latex was recommended and used to repair columns and spandrels of a high-rise apartment housing complex by casting in place (3,pp.157-174). The original concrete had been made using calcium chloride as an admixture and severe deterioration, especially of concrete columns, began after 18 years in service. The repairs after 5 years of service are shown in Figure 14.

The concrete containing the acrylic latex was also recommended and used to repair underground garages by shotcrete. The original concrete had also been made by using calcium chloride as an admixture, and it had been exposed to deicing salt during the winter seasons. Severe deterioration occurred after 15 years in service.

Both types of repairs were in excellent condition after 5 years in service.

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