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Concrete Overlays and Inlays, Effects of High Temperature on Concrete, and Statistical Techniques in Construction

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Continuously Reinforced Concrete Overlay of Existing Continuously Reinforced Concrete Pavement

ALFRED B. CRAWLEY AND JOE P. SHEFFIELD

The design and construction of a 6-in. unbonded continuously reinforced concrete (CRC) overlay of a 20-yr-old continuously reinforced concrete pavement (CRCP) are described. This is the first time a CRC overlay has been placed over an existing CRCP. The existing CRCP was an experimental project when built and had several features that were being tried for the first time in Mississippi. One of these features, smooth wire fabric reinforcement, led to the need for the overlay. The CRC overlay project had several items new to Mississippi, including (a) a new, statistically oriented quality assurance specification for rigid pavement; (b) the closing of one side of an Interstate highway to traffic; and (c) plain concrete shoulders paved monolithically with CRC mainline overlay. The distress in the 20-yr-old pavement, design, and construction procedures, contract award provisions, traffic control features, and postconstruction evaluation are discussed, and some interim recommendations are presented.

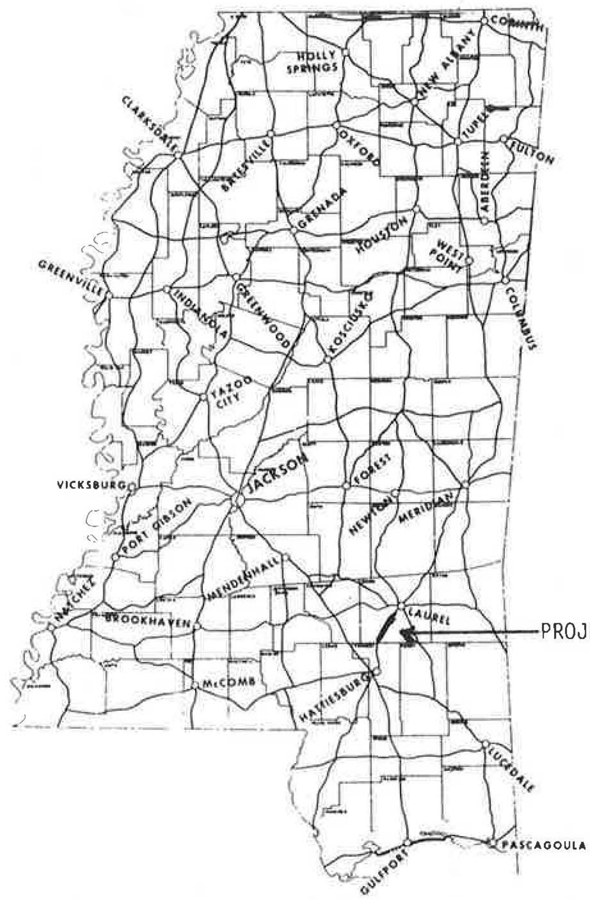
Since the late 1950s, more than 13,000 equivalent two-lane miles of continuously reinforced concrete pavement (CRCP) has been constructed. Even though various problems have arisen with this type of pavement, most of this mileage is giving acceptable performance.

Continuously reinforced concrete (CRC) has also been used as an overlay to rehabilitate existing roadways. The equivalent of 350 two-lane miles of CRC overlay has been constructed since its introduction in 1959; most of the overlays have been in service for less than 10 yr. Both flexible and rigid pavements have been given CRC overlays, but until recently no CRC overlay had been placed over an existing CRCP (1,2).

In January 1981, the Mississippi State Highway Department (MSHD) constructed a CRC overlay over an existing CRCP on I-59 in Jones County in southeast Mississippi. The pavement that was overlaid is located in the northbound lanes of I-59 in Jones County between Moselle and Ellisville. This is a divided highway with a depressed median. The location is shown in Figure 1, and a typical section is shown in Figure 2.

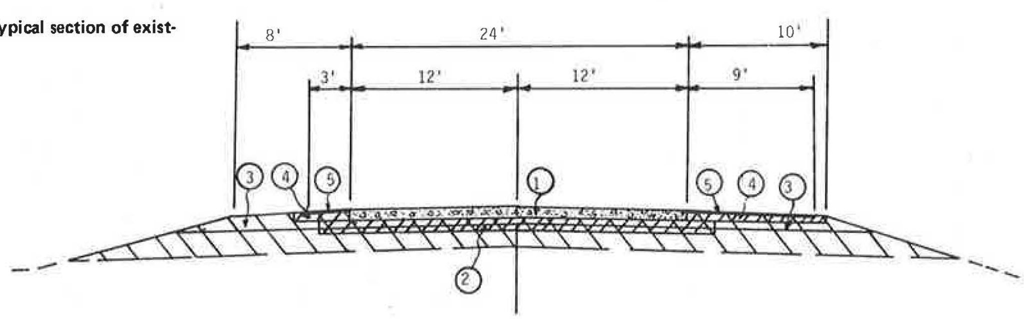
Between January and April 1962, the northbound and southbound lanes of this section of highway were constructed on a soil-cement base as an experimental project. Because this was only the second CRCP to be constructed in Mississippi, different schemes of reinforcement were incorporated into the project. In

Figure 1. Location of CRC overlay.



the southbound lanes, reinforcement was provided by using deformed steel bars with longitudinal steel percentages of 0.5, 0.6, and 0.7 in the southern, central, and northern thirds of the roadway, respec-

Figure 2. Typical section of existing CRCP.



Notes:

1. 8" Continuously reinforced cement concrete pavement (3/16"/' slope)
2. 6" Cement-treated base, 28' wide
3. Roadbed topping (sand-clay)
4. 6" Cement-treated shoulder (1/2"/' slope)
5. Double bituminous surface treatment

tively. In the northbound lanes, smooth welded wire mesh was used for reinforcement with the same longitudinal steel percentages as used in the southbound lanes. The welded wire mesh was fabricated in mats 11.5 ft wide and 24 ft long. Centerline tie bars were placed on 30-in. centers. The mats were overlapped 13 in. lengthwise and tied together by wire. It should be noted that smooth wire mesh is not typical of the reinforcement used in CRCPs constructed in the past 20 yr.

The southbound lanes have performed satisfactorily since construction. The northbound lanes, however, have developed some problems.

Over a period of approximately 3 months after placement, 10 abnormally wide cracks were noticed. Two additional cracks were found during the first year of pavement life. All of these wide cracks occurred at the laps of the welded wire mats. The typical repair performed by the contractor can be summarized as follows:

1. Make two saw cuts, each approximately 1 in. deep, from 30 to 40 in. apart, with the crack approximately centered between them;
2. Using a jackhammer, remove the concrete to the level of the reinforcement;
3. Weld 0.5- by 24-in. reinforcing bars to the adjacent mats at the ends;
4. Coat the entire zone, both concrete and steel, with epoxy glue; and
5. Place fresh concrete, incorporating high-early-strength cement to restore the section, and cure with wet burlap.

Almost all of these contractor-repaired joints were still performing well at the time of the overlay.

After the roadway was opened to traffic, many other wide cracks developed, again all at the laps of the welded wire fabric mats. These have been repaired by MSHD maintenance forces by using a variety of techniques. In some areas, the concrete was removed to the level of the reinforcement and replaced with a bituminous mix. In other areas, the full depth of concrete and reinforcement was removed and replaced with a bituminous mix, sometimes by using expansion material to create an expansion joint. Other techniques, involving both portland cement concrete (PCC) and bituminous concrete, were also used. Almost all of these repairs failed to correct the problem, and the problem areas became progressively worse with the spalling of the concrete.

There was also evidence of some pumping at the pavement edge due to roof water entering the edge joints. Little success had been achieved in sealing these edge joints, and most of them were sufficiently open to permit surface water to enter. By mid-1980, the condition of the pavement had deteriorated to the point where corrective action was essential.

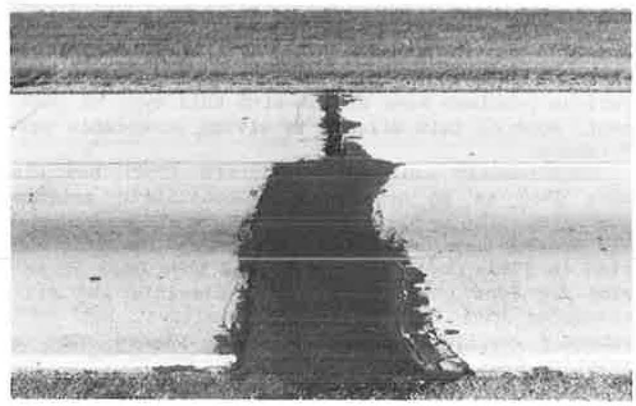
Reasons for Selecting CRC Overlay

The rideability and present serviceability index (PSI) of the northbound roadway diminished due to the unsuccessful repair operations previously discussed. This, coupled with the evidence that pumping at the pavement edge was beginning, indicated that effective rehabilitative work was needed to protect the investment made in this segment of the Interstate system and to restore the PSI to an acceptable level. An overlay would accomplish these objectives.

Mississippi had had success with an unbonded CRC overlay constructed in 1971 on I-20 near Vicksburg, which had given MSHD personnel valuable experience and expertise in this type of construction. In comparing the alternative of a CRC overlay with a bituminous overlay, attention should be directed to the roadway system involved and pavement life. Because the subject roadway is a segment of a busy Interstate highway and the rehabilitation disrupts the normal traffic flow and increases the likelihood of accidents, it is important that such rehabilitations be as infrequent as possible. Based on experience, a CRC overlay would provide a much longer pavement life than a bituminous overlay.

Figure 3. Extra-wide crack in existing CRCP.

Figure 4. Bituminous patch in existing CRCP.



An unbonded overlay was chosen primarily to eliminate or reduce the possibility of reflective cracking. To achieve a good bonded or partially bonded overlay would require reestablishing the continuity of the existing reinforcement in the numerous areas of distress. An unbonded overlay would allow much simpler patching of the 62 locations where the existing reinforcement was no longer continuous. A secondary reason for choosing an unbonded overlay was the need to correct some minor grade irregularities for which a bituminous leveling course would be ideal. Based on this reasoning, an unbonded overlay was selected.

Because this is the first CRC overlay of an existing CRCP (to our knowledge) to be constructed, the project should provide much valuable information that will advance the work of pavement rehabilitation. This information should be especially helpful in light of the fact that much of the Interstate system is approaching the end of its design life.

EVALUATION OF EXISTING CONDITION

Visual and Pictorial Evaluation

All patches and areas of distress were photographed just before construction of the overlay. Figures 3 and 4 show typical areas of distress. Minor to moderate spalling of the random cracks was evident in some areas, and the spalling was usually much worse in the passing lane than in the right lane. This type of spalling was also noticed in the southbound lanes. The spalling appears to be stabilized and is not considered to be a problem at this time.

Deflection, Roughness, and Skid Resistance

Before the overlay was constructed, deflection measurements were made with a Dynaflect. These measurements showed an average maximum deflection of 0.651 mil and a standard deviation of 0.235 mil. The measurements indicated that the pavement and subgrade were strong.

Mays Ride Meter testing was performed to determine pavement serviceability. The PSI, based entirely on the results of the Mays Ride Meter, was 2.6.

Skid resistance was determined by using the MSHD ASTM skid trailer. The average skid number for the pavement was 46.

Traffic Volume

The current average daily traffic (ADT) for the northbound roadway was 4,960, and the equivalent annual 18-kip single-axle loading was 317,000. The accumulated equivalent 18-kip single-axle loading was 4.5 million. The ADT for the design year (2000) is forecast to be 8,465.

Cracking

Two 1,000-ft sections were mapped for cracks before construction of the overlay. Typical cracking is shown in Figure 5. The average crack spacing was about 7 ft.

Shoulders

The 10-ft outside shoulder and the 4-ft inside shoulder appeared to be in good structural condition after a visual inspection. Some minor faulting was noticed in isolated areas. The longitudinal joint between pavement and shoulder was open in most areas. Minor staining was noticed at two locations near distressed areas in the CRCP, probably due to pumping of fines from under the pavement. Distressed areas were almost always confined to the CRCP, and the shoulders were maintained in good condition.

DESIGN

Selection of Rigid Overlay

In addition to the reasons listed earlier for selecting a rigid instead of a flexible overlay, another factor should be mentioned. Sixty-two distressed areas, as typified by those shown in Figures 3 and 4, needed repair before the overlay was placed. The repair method chosen was to remove the concrete, steel, and asphaltic concrete currently in the distressed area and replace them with hot-plant-mix asphaltic concrete. The resulting joints between the CRCP in place and the new asphalt concrete patch would be active due to longitudinal movement of the CRCP. If a bituminous overlay were placed, these joints would probably propagate through the overlay in a short period of time and would need regular maintenance to keep an effective sealant in place. It was thought that an unbonded rigid overlay would bridge the patched area and preclude the need for regular joint maintenance.

Basis of Overlay Design

The design of the CRC overlay was based more on experience than on any set design procedure, predominantly due to the rarity of this type of construction. The experience the MSHD gained in 1971 from the CRC overlay of a 9-in jointed reinforced concrete pavement on I-20 near Vicksburg was utilized in the design. The experiences of other states in constructing CRC overlays were also investigated and provided helpful information that was used as a guideline in the design (3).

The AASHTO Interim Guide for the Design of Pavement Structures (4) includes a procedure for designing PCC overlays for existing PCC pavements. The procedure was developed by the U.S. Army Corps of Engineers primarily for use in the design of runways and taxiways at airports. The design equation developed for an unbonded overlay was

$$h_o = (h_d^2 - ch^2)^{0.5} \tag{1}$$

where

- h_o = thickness of overlay slab (in.),
- h_d = thickness of new pavement from regular PCC pavement design analysis (in.),
- h = thickness of existing pavement (in.), and
- c = coefficient depending on the condition of the existing slab, ranging from 0.35 for badly cracked slabs to 1.0 for slabs in excellent condition.

Because this design equation would result in a slab thinner than the minimum being used by most states in placing unbonded CRC overlays (5-6 in.), it was not used in design.

Design of Pavement Overlay

The overlay design called for a 6-in. CRC overlay on the two 12-ft traffic lanes. Figure 6 shows a typical section of the overlay. Longitudinal steel consisted of No. 5 bars at 7.25-in. spacing, which provided a longitudinal steel percentage of 0.71. The first and last bars were spaced 2.62 in. from the outside edges of the traffic lanes. Transverse steel consisted of 30-in.-long No. 4 bars spaced 36 in. center to center and supported on chairs. There was 2.5 in. of concrete cover over the reinforcement. The contract provided that, if the contractor proposed a suitable method for placing the longitudinal steel through the paver, the transverse steel could be eliminated and replaced with tie bars. The longitudinal center joint was to be formed by inserting a strip of plastic material along the centerline of the pavement to induce longitudinal cracking along a controlled plane.

The shoulders were to be constructed of 6-in. plain PCC and were to be the first such shoulders constructed in Mississippi. The outside shoulder was to be 10 ft wide and the inside shoulder 4 ft wide. Transverse contraction joints with dimensions of 0.25x0.5 in. were specified at approximate 20-ft intervals. The shoulders were to be tied to the pavement with 30-in.-long No. 5 tie bars on 30-in. centers. Corrugated rumble strips were to be formed in the fresh concrete at approximately 60-ft intervals.

To provide end restraint for the CRC overlay, each end of the overlay was to be tied to the existing lug anchors, which were constructed monolithically with the original pavement. To do this, the existing pavement was to be removed down to the top of the lug anchors for a distance necessary to form a transition by which the grade would match the grade of the overlay at a rate of 2 in./100 ft (approximate transition distance of 300 ft). The concrete pavement in the transition was to be of varying thickness.

Surface Texture of Overlay

The two traffic lanes were to be given a transverse groove finish. After a burlap drag had been used, the final surface texture was to be produced by a metal tine finishing device. The texturing device was to produce uniform, transverse parallel grooves 0.5 in. on centers and 3/16 in. deep.

Repair of Failed Areas

The only repair work performed on the existing pave-

Figure 5. Section 1 mapped before CRC overlay.

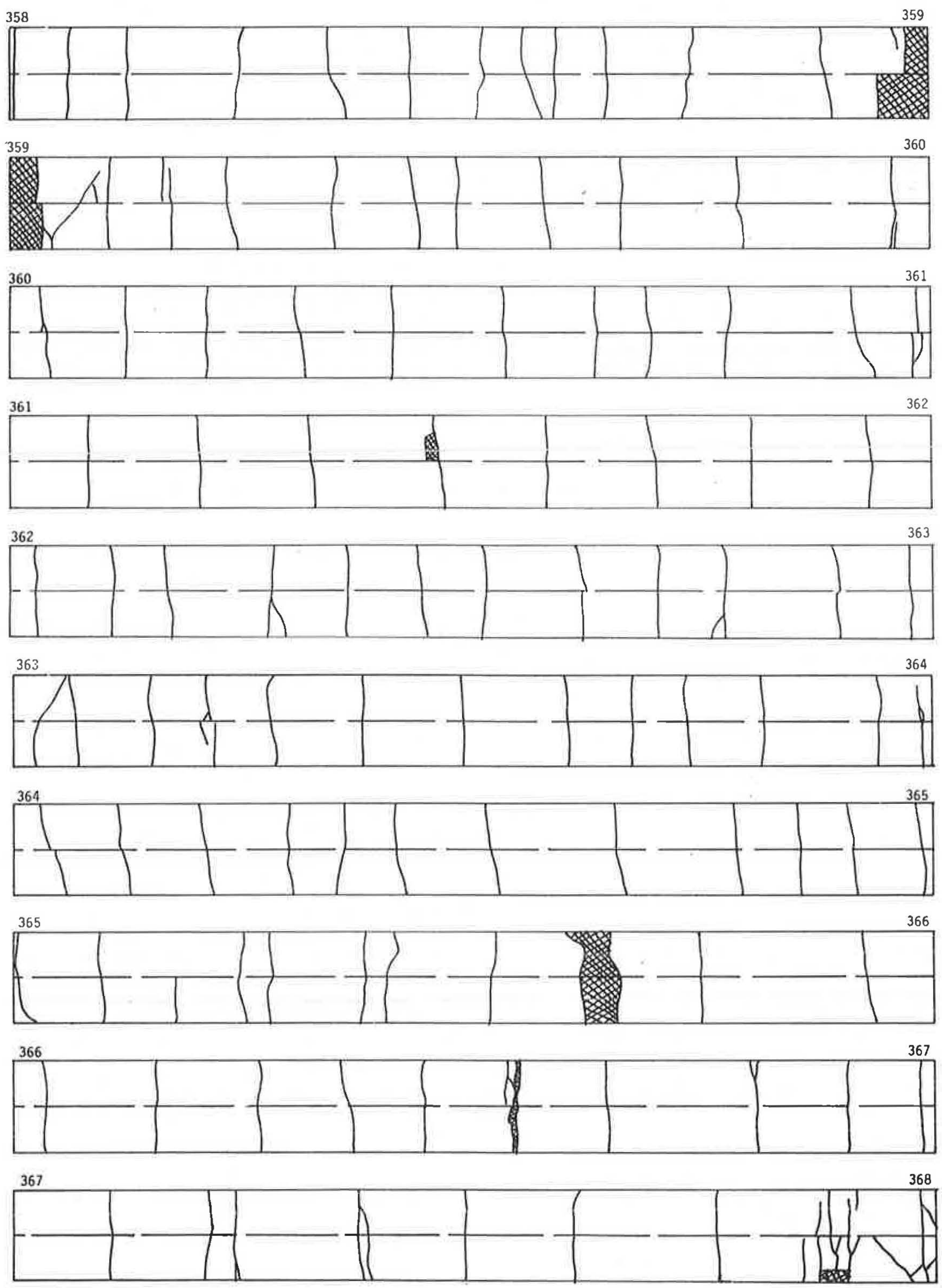
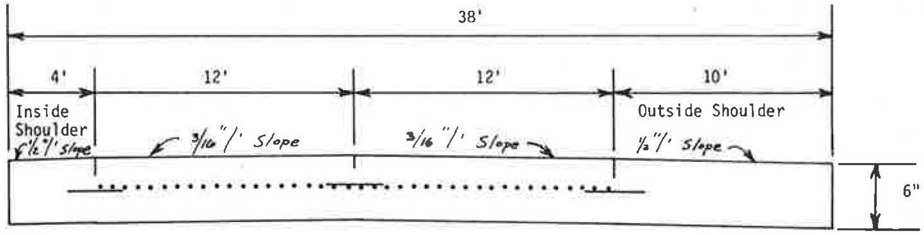


Figure 6. Typical section of CRC overlay.



Notes:

Longitudinal weakened plane joints may be formed by placement of a continuous strip of polyethylene or other approved material with a minimum thickness of 20 mils and a minimum depth of D/4, where D is the thickness of the pavement.

Above reinforcement detail is applicable only if the contractor has a suitable method of placing longitudinal reinforcement by machine. Otherwise, full length transverse reinforcement supported by chairs shall be used and the centerline tie bar eliminated.

ment was replacement of all of the failed areas. The full depth of the section was to be removed and replaced with hot bituminous mix. Before removal, neat saw cuts were to be made into sound concrete on each side of the failed area to define the limits of the patch.

Bond Breaker/Leveling Course

A hot bituminous leveling course was to be applied to the existing surface to correct slight irregularities in the grade and to serve as a bond breaker. The leveling course was 1.5 in. thick. This bituminous course extended across both traffic lanes and shoulders.

Traffic-Handling Procedures

The traffic control plan called for northbound traffic to be diverted to the southbound lanes during placement of the CRC overlay and subsequent curing time, which resulted in two-way traffic in the southbound lanes. All patching and placement of the bond breaker/leveling course was to be done under traffic. Crossovers were designed for each end of the project by using precast New Jersey-type barriers and impact attenuators in the reverse curves of the crossovers. Nighttime lighting through the crossover areas, trailer-mounted electric message boards located in advance of and throughout the project, and construction signing were to be used to alert motorists to the two-way traffic zone. In the southbound lanes, double 4-in. continuous yellow temporary stripe was used as centerline striping and 36-in. snapback delineators on 100-ft centers were used to prohibit passing. The two interchanges within the project limits were to be closed to both entrance and exit traffic during the time the southbound lanes were subject to two-way traffic.

Specifications

This CRC overlay marks the first time the MSHD has used these new specifications for concrete pavement. This type of specification is sometimes called a performance-related or end-result specification. Mix design, quality control, and much of the daily record keeping were to be the responsibility of the contractor, and the MSHD was to establish general guidelines and conduct random checks. Acceptance and payment were to be based on factors such as surface smoothness, pavement thickness, and the compressive strength of cores obtained from the pavement. Because of the disruption to traffic on a section of Interstate highway, it was imperative

Table 1. Comparison of bids.

Bidder	Total of Bid Items ($)	Contract Time (days)	Total[a] ($)
Denton Construction Company	4,721,539.83	151	5,778,539.83
Bush Construction Company and T. L. James and Company	4,544,930.41	250	6,294,930.41
Cook Construction Company	5,271,196.81	212	6,775,196.81
Eisenhour Construction Company	5,215,617.24	266	7,077,617.24

[a]Includes road-user delay costs of $7,000/calendar day.

that this project be completed in the shortest time possible. It was decided that the award of contract would be based on a combination of bid prices and contract time. The MSHD established January 1, 1981, as the beginning of contract time and allowed bidders to propose their own completion dates. The bidder would then determine the number of calendar days between the beginning of contract time and his specified completion date and multiply this number by $7,000, the amount established by the MSHD as the average daily road-user cost associated with closing the northbound lanes and two interchanges to traffic. This product of calendar days multiplied by $7,000 was to be added to the total bid determined from bid prices. The sum of these two amounts was to be the amount used in the comparison of bids. Copies of special provisions describing this procedure are available from the MSHD.

CONSTRUCTION

Contract Award

Bids were opened in October 1980 after a 2-month advertisement period. Four bids were received. The lowest bid for construction items only was submitted by a joint venture of Bush Construction Company of Laurel, Mississippi, and T.L. James and Company of Ruston, Louisiana. When the amounts for road-user costs were added, however, the lowest total for comparison of bids was submitted by Denton Construction Company of Grosse Point Woods, Michigan. The contract was therefore awarded to Denton. Table 1 compares the bids submitted.

Mix Design and Quality Control

Denton Construction Company contracted with Ladner Testing Laboratories, Inc., of Jackson, Mississippi, to prepare the mix design and develop and coordinate a quality-control plan as required by the specifications. The mix design approved for the project was a five-bag, 4,000-psi mix containing an air-entraining agent and a water reducer. The quality-control plan submitted merely incorporated the minimum requirements set forth in the contract.

Diverting Traffic

The contractor requested and was granted permission to divert all traffic to the southbound lanes before performing any work on the northbound lanes. The traffic-control plan described earlier called for all patching and placement of the bond breaker/leveling course to be done under traffic. This was to have been done by performing the patching and asphalt placement one lane at a time while accommodating traffic in the other lane. The contractor reasoned that the time period during which all traffic would be forced to use the southbound lanes could be shortened by doing all the preliminary work on the northbound lanes after traffic had been diverted. Having the entire roadway open for construction activities would vastly improve efficiency and thus save time, and several operations could be carried out concurrently. Another advantage of this method was that it would eliminate the possibility of accidents between highway travelers and construction equipment.

Work began in mid-November 1980 on construction of the crossovers at each end of the project. Northbound traffic was diverted to the southbound lanes on December 12. Figures 7-9 show crossover barriers, traffic-control devices, impact attenuators, and lighting. Figure 10 shows a typical view of the southbound lanes, including the double yellow center stripe and snap-back delineators. Portable electric message-board signs were placed at four locations along the southbound lanes to remind motorists of the two-way traffic.

Repair of Distressed Areas

As soon as traffic was diverted from the northbound lanes, the work crew began removing concrete in the transition areas and in the distressed areas. Concrete saws were used in both areas. Numerous cuts were made in the transition areas at 10- to 20-ft intervals to facilitate removal of the concrete and steel. The depth of the saw cuts varied but always extended below the level of the steel reinforcement. Jackhammers and hydraulic rams were then used to break the concrete into pieces small enough to be picked up by a front-end loader. In the 62 distressed areas, saw cuts were made on each side of the distress into sound concrete. A hydraulic ram was then used to break the concrete and asphalt in the distressed area into small pieces for removal. The soil-cement base was generally in good condition and did not require removal. In areas where the soil-cement was unsuitable, it was removed and replaced with hot-plant-mix bituminous pavement.

Figure 7. Crossover barrier and attenuator.

Figure 8. Crossover directional arrow.

Figure 9. Crossover marking and lighting.

Figure 10. Typical section for two-way traffic on southbound lanes.

After the distressed areas were cleaned and a tack coat was applied to the edges of the existing concrete pavement, hot-plant-mix was placed in three lifts and compacted to complete the patch.

End Anchorage and Transition

Crews began work to provide end anchorage for the overlay as well as a smooth transition from the overlay to the adjoining pavement on each end of the project. The existing concrete pavement was removed over a distance of 375 ft at each end of the project. The existing lug anchors were left in place and prepared for use in anchoring the overlay. Approximately 75 ft of pavement removal exposed the four lug anchors, and the remaining 300 ft allowed for the grade transition. Hot-plant-mix bituminous pavement was placed in the 300-ft transition area.

Placing Bituminous Bond Breaker/Leveling Course

A combination bond breaker and leveling course with a 1.5-in. nominal thickness was placed across the entire width of the traffic lanes and shoulders for a width of 38 ft. The bituminous mixture conformed to MSHD specifications for hot-plant-mix surface course. The bond breaker/leveling course was placed in two lifts and conformed to the same cross-slope requirements as the CRC overlay. The thickness of the bond breaker/leveling course was variable to allow for the correction of minor grade problems.

Overlay Construction

The overlay construction began on January 5, 1981, at the northern end of the project. It should be remembered that the first 375 ft of pavement was of variable thickness and that transverse reinforcement, consisting of full-length No. 4 bars spaced 36 in. on center, was used. Thereafter, the longitudinal reinforcement was placed through tubes on the concrete spreader and the full-length transverse reinforcement was replaced with 30-in. tie bars.

The contractor used a modified concrete spreader (see Figure 11) to spread concrete about 8 in. thick across the two 12-ft-wide traffic lanes and to place both longitudinal reinforcement and the centerline tie bars. The longitudinal bars were fed through tubes and positioned at the proper height. Plans called for 2.5 in. of concrete cover above the reinforcement. To reduce the possibility of the deformed bars becoming stuck in the feeder tubes, a mechanical system was installed on the spreader to rotate the tubes slowly back and forth. The centerline tie bars were inserted by a reel-like device, and the shoulder tie bars were placed by hand. Steering of the spreader was controlled by an electronic sensor traveling on a string line. Concrete was delivered to the spreader in side dump trucks.

The slip-form paver used was modified to pave the entire 38-ft-wide roadway in one pass (see Figure 12). Twin augers on the front of the paver spread the extra-thick concrete placed by the spreader an additional 4 ft on one side and 10 ft on the other side to form the plain concrete shoulders. To increase production, concrete was also dumped directly in front of the paver on the 12-ft outside lane and on the 10-ft shoulder (see Figure 13). The paver inserted plastic crack initiators longitudinally into the 6-in.-thick slab along the centerline and along the joints between the traffic lanes and the shoulders. An oscillating belt and a burlap drag were used immediately behind the paver. Horizontal alignment of the paver was controlled by a string line. Vertical alignment was controlled by a rolling string line in tangent sections and by separate stationary string lines on each side of the roadway in the superelevated sections and in transitions.

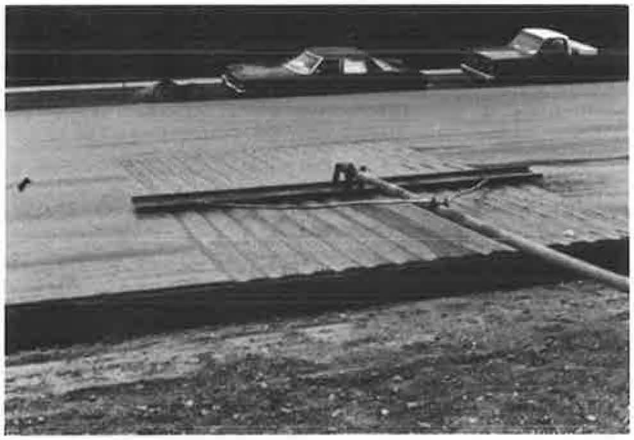
Concrete finishers standing on work bridges towed by the paver continued the finishing operation. A tube finisher followed the work bridges. Rumble strips were then formed in the concrete shoulders by hand with a special float (see Figure 14). The

Figure 11. Rear view of concrete spreader.

Figure 12. Front view of concrete paver.

Figure 13. Placing additional concrete on outside lane and shoulder.

Figure 14. Forming rumble strips in shoulder.



pavement was then given a transverse tined finish and sprayed with white pigmented curing compound at a rate of 1 gal/90 ft^2. If freezing temperatures were anticipated during the night, the pavement would be covered with plastic sheeting.

Transverse contraction joints were sawed into the plain concrete shoulders when the concrete was 1 day old. These joints were later cleaned with compressed air and sealed with grade AC-13 asphalt cement.

Construction joints were made at the end of each day's work. A piece of No. 5 reinforcing bar 5 ft in length was tied to each longitudinal bar so that half the length was in the current day's slab and the rest was in the next workday's slab to increase the bonding area across the cold joint. This is a standard practice for all CRCP construction by the MSHD.

Concrete for the project was produced in the contractor's double-drum plant located on property adjacent to the highway right-of-way and about 0.5 mile north of the beginning of the project. The maximum haul was about 6 miles. Batch size was 5.5 yd^3. The concrete was delivered to the site in side dump, standard dump, and agitator trucks.

In view of the fact that it was the middle of winter, the work progressed satisfactorily. The 6.7-mile project was paved in 14 workdays over a 3-week time span, and concrete placement was completed on January 24. The average maximum and minimum ambient temperatures for the month of January were 55.4°F and 28.5°F, respectively. The overall average ambient temperature was 42.0°F, which is 6.2°F below normal. One workday was lost due to low temperatures. The total precipitation (all rainfall) for January was 1.61 in., which is 3.23 in. below normal. One workday was also lost due to rainfall. Average daily production was 10,670 yd^2 or 2,527 linear ft of pavement and shoulders.

In conjunction with the CRC overlay, the exit and entrance ramps for the two interchanges located within the project limits were improved. Alignment and length changes were made to reflect current design standards. Grade changes were also made to match the new overlay grade. The 10-ft-wide outside concrete shoulder was deleted in the ramp areas, and asphalt pavement was used for the entire ramp.

After ramp improvements, striping, and signing had been completed, the northbound lanes were opened on February 11.

Shortly after the roadway was opened to traffic, excessive spalling of the concrete was noticed adjacent to the construction joint approximately 2.5 miles north of the beginning of the project. The concrete had deteriorated to an average depth of 1.5 in. Spalling was confined to only one side of the construction joint and was attributed to a bad batch of concrete used at the start of that day's construction. Only the traffic lanes were affected. Saw cuts were made on both sides of the spalled area across the traffic lanes. The 1.5-in.-deep cuts were approximately 2 ft apart, and one cut was directly above the construction joint. A jackhammer was used to remove the concrete between the saw cuts to an average depth of 1.5 in. After cleaning, the entire area was coated with epoxy cement and a rapid-setting concrete was used to restore the section. This repair was done one lane at a time while traffic was diverted to the other lane. Curing time for the patch was 2 hr.

POSTCONSTRUCTION EVALUATION

Performance Testing

Pavement performance testing was conducted on the completed overlay by using the MSHD skid trailer, the Mays Ride Meter, and the Dynaflect. The skid test resulted in a skid number of 51. The PSI rating of the pavement, based entirely on Mays Ride Meter measurements, was 3.1. Dynaflect deflections showed an average maximum deflection of 0.279 mil and a standard deviation of 0.078 mil.

Crack Pattern

The crack pattern of the CRC overlay was slow in developing. It is thought that the cool temperatures may be responsible for this. Also noteworthy is the fact that even on sunny days most of the pavement was shaded during most of the daylight hours. Both transverse cracks and longitudinal cracks at the centerline and at the edge between pavement and shoulder were not generally noticed until about 3 weeks after construction. It should be pointed out that the tight cracks were especially hard to see amid the transverse tining and the curing compound. Two 1,000-ft sections were mapped before placement of the overlay, and the corresponding sections of the overlay are to be mapped periodically. One section is near the south end of the project, and the other is approximately at midpoint of the project.

The first transverse cracks that were noticed extended across the pavement between the sawed contraction joints in the shoulders for a crack spacing of approximately 20 ft. The next transverse cracks generally appeared approximately midway between the contraction joints in the shoulders. In most cases, although these cracks remained tight, they extended through the plain concrete shoulders.

One of the mapped sections has two areas where five or six transverse cracks are closely spaced over a 10-ft section. It is interesting to note that these two areas are located above bituminous patches in the underlying CRCP. The method for constructing these bituminous patches was described earlier. In light of this finding, the entire project was examined. Eight additional areas of closely spaced cracks were found, six of which are directly above bituminous patches in the underlying CRCP. At the time this paper was prepared, all the cracks in these areas were tight and showed no spalling or other distress.

Longitudinal cracking has been found in three locations on the project. These longitudinal cracks are situated at about midpoint of each traffic lane and vary from about 50 to about 200 ft in length. All are tight at this time.

Specifications

No glaring deficiencies were noticed in the content of the new specifications dealing with rigid pavement. In our opinion, the specifications were not enforced as rigidly as they should have been. Quality assurance reporting, and possibly testing, were lax. Hereafter, an effort should be made to compel strict compliance with the requirements of the specifications.

Several factors probably affected the failure of the MSHD to follow the letter of the specifications:

1. Because few concrete highways are currently being built in Mississippi, MSHD personnel do not have extensive experience with this type of construction.
2. Because of the traffic-control situation, this project was built on a fast-track schedule, which made it difficult to keep up with all of the construction activities.
3. Because this was the first time these specifications had been used, various people involved were unfamiliar with their responsibilities.

One major factor of the construction directly attributable to these new specifications was the concrete mix design. The contractor submitted a mix design with a cement factor of 1.25, the minimum allowed. This mix design met MSHD criteria and was approved. Under the previous specifications, the MSHD would have used the aggregate and cement sources selected by the contractor in designing the mix and would probably have designed a mix with a cement factor of 1.45. This is one reason for the end-result type of specification. It allows the MSHD to set minimum criteria and frees contractors to meet the requirements in what they perceive to be the most economical way. Acceptance criteria spell out the penalties for noncompliance.

In this case, based on the asumption that a cement factor of 1.45 would have been used under the previous specifications, the contractor saved almost 1 bag of cement/yd^3 of concrete but was penalized for insufficient core strength on four lots, all located in the traffic lanes. Three lots were penalized 20 percent each, and one lot was penalized 5 percent for low compressive strengths on roadway cores. One of the lots that was penalized 20 percent for low compressive strength was also penalized 20 percent for deficient thickness. Because the contract price on which a penalty is calculated is also reduced by other penalties, this resulted in a penalty of 36 percent. A lot consisted of 1,500 linear ft of the two traffic lanes, or 3,000 linear ft of each shoulder. The shoulder pavement met all acceptance criteria.

An unusual aspect of the specifications used for this project was the method of determining contract time and contract award, which was discussed earlier. These provisions proved useful in limiting closure of the northbound lanes to the shortest time possible. In view of the rapid completion of this project, the use of such provisions should be strongly considered in future projects where lane closures are necessary.

CONCLUSIONS AND RECOMMENDATIONS

Based on observations made during and after construction, the following conclusions and recommendations are made:

1. The statistically oriented quality assurance specifications are sound, and strong consideration should be given to their implementation as the standard specifications for rigid pavement construction; however, measures need to be taken to ensure that the provisions of the specifications are rigidly followed.
2. Where traffic-control procedures permit, a less expensive leveling course/bond breaker should be used. Where a leveling course is required, sand asphalt is suitable if high skid resistance is not required. Where a leveling course is not necessary, some type of fabric or liquid membrane should be investigated for possible use as a bond breaker.
3. The use of specifications, such as those described in this paper that relate to contract time determination and contract award, should be considered when it is in the public interest to reduce construction time to the minimum.

The project will continue to be monitored, and the adequacy of the design and construction procedures will be assessed at the end of the study period.

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Construction of Thin Bonded Concrete Overlay

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In 1981 a bonded concrete overlay 3 in. thick was placed on I-81 north of Syracuse under a contract with the New York State Department of Transportation. I-81 is a six-lane divided Interstate highway that has an annual average daily traffic of 23,000 with 8 percent trucks. The existing concrete pavement was overlaid for a length of 3 miles in all northbound and southbound lanes. Two lanes in each direction were closed to traffic while the overlay was placed. Traffic was maintained on the third lane and a thickened asphalt-concrete shoulder. The concrete overlay was placed to remedy widespread longitudinal and transverse joint deterioration caused by porous coarse aggregate in the existing concrete pavement. The freezing and thawing of water in the coarse aggregate caused a surface spalling problem and layered cracking beneath the surface similar to D-cracking. Deteriorated pavement at the joints was removed to a 3-in. depth by using a milling machine. A nominal 3-in.-thick concrete overlay was bonded to the existing pavement with a cement-sand grout after scarification, sandblasting, and cleaning. The resulting 6-in.-thick lift of concrete at the deteriorated joints was designed to bridge the deterioration and provide a long-lasting overlay. Pavement blowups were occurring on the 23-yr-old existing pavement, which dictated the installation of pressure relief joints before overlaying. The surface preparation and cement-sand grout have resulted in an adequate bond. The thicker concrete overlay is bridging the deterioration. Shrinkage cracking, which developed during paving in hot weather, and reflection cracks over existing pavement cracks have not resulted in performance problems to date. Dust control during surface preparation needs improvement. Pavement friction generally is adequate but needs further study. Overall rideability is excellent.

The original I-81 concrete pavement was constructed by the New York State Department of Transportation (NYSDOT) in 1957 and was part of a contract for 12 miles from North Syracuse to Oneida Lake. A 40-ft-wide grass median separated the three northbound lanes from the three southbound lanes. The concrete pavement was placed 9 in. thick. Steel mesh reinforcement was placed between middepth and 3 in. below the surface of the concrete slab. Lanes were 12 ft wide. Transverse contraction joints were sawcut at 43-ft intervals. Steel dowels 1.25 in. in diameter by 18 in. long at 1-ft intervals provide load transfer at the transverse joints. The driving lane and middle lane were integrally paved 24 ft wide. The longitudinal contraction joint was sawcut, and a 5/8-in.-diameter deformed steel bar was used at 40-in. intervals as a longitudinal joint tie. The median lane was paved 12 ft wide. The longitudinal joint between the median lane and the previously placed lanes consisted of a formed keyway and a two-piece threaded longitudinal joint tie at 40-in. intervals. A bituminous sealer was used for both the longitudinal and transverse joints.

The concrete used in the original construction had a cement factor of 6.2 bags/yd^3 and a water/cement ratio of 0.42 and was air entrained. A range of 3 to 6 percent total air content was specified. The coarse aggregate was a crushed stone graded from 3/8 to 2.5 in. The coarse aggregate was an argillaceous dolomite from a nearby stone quarry.

An attempt was made in 1972 to solve the spalling deterioration, which was occurring at the transverse joints. The entire transverse joint was removed after approximately 2 ft had been sawcut on each side of the joint. A new dowel load-transfer device was installed. Steel dowels were grouted into holes drilled into the existing slabs, new concrete was placed, and the joint was sawcut and sealed. This joint replacement was performed up to the funding limits for the contract. Project engineers found that many additional joints needed treatment at that time.

EXISTING PAVEMENT CONDITION

A pavement evaluation that included pavement coring was performed in 1980. Observation of pavement condition and inspection of full-depth cores confirmed that the coarse aggregate was the cause of the deterioration of this concrete pavement. The deterioration consisted of popouts of the coarse aggregate near the surface and extensive spalling at the longitudinal and transverse joints. The joint spalling was more severe at the pavement edges and at the intersection of the longitudinal and transverse contraction joints. A large number of asphalt patches had to be used to repair the spalls.

The argillaceous dolomite used was highly absorptive. Water that had penetrated from the pavement surface or from the joint faces was trapped in this absorptive stone. During the winter, this water froze and expanded and developed enough internal pressure to cause the surface popouts and the surface spalling.

Cores taken in deteriorated areas at the joints showed horizontal layered cracking for the full 9-in. depth of the core. Only broken pieces were retrieved from cores taken directly over the transverse joints; these cores indicated more deterioration than cores taken farther away from the joint. Cores taken in the center of the slab were intact, consisted of sound concrete, and only showed deterioration at the surface where a surface aggregate popout coincided with the core. The surface where exposed to water and where water could be held and trapped, such as the pavement edges and joint faces, were the places that showed the most deterioration. The spalling was continuous; it started on the outer surfaces and progressed inward as new faces and the porous coarse aggregate were exposed to water and freezing. During construction when pavement edges were visible and pavement sections were removed, the classic hourglass shape of D-cracking deterioration at the pavement joint was observed.

Deterioration was also occurring where the transverse joints had been replaced. After 8 yr, this joint-replacement treatment was found to have separated at the interface between new and existing concrete. Water was entering the pavement through this separation, and spalling deterioration of the original concrete, which contained the porous coarse aggregate, was occurring. This trial construction technique showed that extensive repairs were necessary and a different repair method was needed.

Blowups were another type of deterioration present in the existing pavement. Fines and incompressibles had infiltrated the transverse joints with time. Winter sanding for traction and lack of a durable and maintainable joint sealer had caused this problem. The pavement blowups occur when temperatures are warm; the concrete expands and the filled joints cannot accommodate the expansion. The blowups that had occurred were patched with asphalt concrete after removal of the buckled and shattered concrete.

In some instances the blowup occurred across all three lanes of pavement. Transverse joints closest to the blowup opened wider as a result of the pressure relief afforded by the blowup. At other times, blowups occurred in one or two of the three lanes. When these partial-pavement-width blowups occurred, displacements also occurred in several adjacent transverse joints. The pavement slabs in the affected lanes moved in toward the blowup. The blowup-free lane or lanes remained stationary. Transverse-joint misalignment of as much as 9 in. was noted.

Inspection of cores and observations of the pavement showed a difference in performance in the longitudinal joints. The joint with a formed keyway and two-piece tie was weaker than that with the deformed-bar tie used in integral-lane paving. Corrosion of the two-piece tie weakened its threaded connection. When the blowup occurred, the ties pulled apart and broke or bent. When this occurred, the longitudinal joint was no longer tied together and could separate because of infiltration of fines. Separation had not occurred as yet on this project; some edge support was still being provided by the formed keyway.

A third type of pavement distress was slab cracking. In general, these cracks were located over culverts crossing underneath the pavement. Such cracks were caused by differential movements in freezing temperatures due to the difference between temperature in the soil near the culvert and that further away from the culvert. Usually only a single crack was present, which followed the direction of the culvert underneath. In general, the pavement at the crack was being held together by the steel-mesh reinforcing.

The 3-mile section chosen for rehabilitation has one overhead structure at the southern end of the project near the I-481 interchange. There are two mainline structures, northbound and southbound, over NY-31 2 miles from the project beginning. The mainline structures were also rehabilitated under this contract with a new high-density-concrete bonded wearing course.

DESIGN

Based on favorable experience of other states with a bonded concrete overlay, success in New York State with bonded concrete pavement inlays and bonded high-density-concrete bridge deck overlays, and the need for a longer-lasting solution than an asphalt overlay, a concrete overlay was chosen. Asphalt concrete was assumed to have a 7-yr service life before it became necessary to use another overlay, whereas a concrete overlay was assumed to have a 15-yr life. Comparisons showed that the concrete overlay and the two asphalt overlays were nearly equal in cost. The design chosen was a 3-in.-thick bonded concrete overlay.

To address the deterioration at the longitudinal and transverse joints, the specifications called for milling to a depth of 3 in. to increase the concrete thickness to 6 in. at these distressed locations. The milling depth was determined in advance and planned to stay above the existing steel-dowel load-transfer devices. In this way, the dowels would remain in place, provide load transfer, and not have to be replaced at high cost. Because no faulting had occurred at the transverse joints, it was assumed that enough dowel embedment in solid concrete and dowel strength existed to provide load transfer. Figure 1 shows a cross section of the thickened section.

To relieve the pressures built up in the pavement, which caused the blowups, and to prevent future blowups, asphalt-concrete pressure-relief joints at full pavement depth and full width (three lanes) were specified. The pressure-relief joints were located at blowup locations and at the ends of the northbound and southbound mainline structures over NY-31.

Cracking in the existing pavement was handled by specifying wire mesh in areas of wide cracking where the existing mesh no longer functioned in holding the pavement together. The wire mesh was placed to span the cracks and at the interface between new and old concrete. Cracks were expected to reflect through the new overlay, and the new wire-mesh reinforcement was depended on to hold the cracked pavement tightly together. Faulting had not occurred at these cracks, which indicated that load transfer from aggregate interlock still existed. The alternative to correcting the cracking, which would have been complete slab replacement and installation of control joints at the location of the cracks, was deemed too expensive for the benefit.

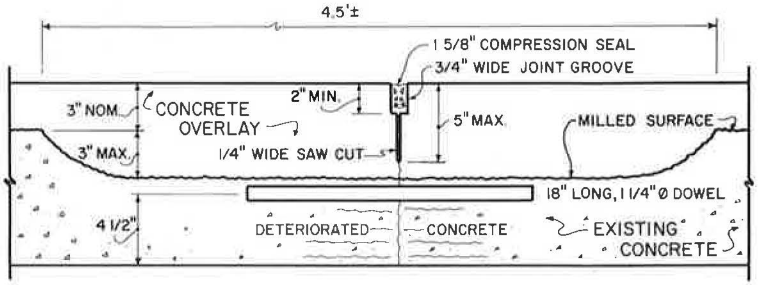
In order to maintain traffic during construction, the following scheme was used. The right-hand, 10-ft-wide shoulder was thickened with a wedge of asphalt concrete. The median and middle lanes were then closed to traffic, and two lanes of traffic were maintained on 10 ft of the driving lane and the 10-ft right-hand shoulder. Traffic was allowed on the new concrete overlay after placement, curing, and joint sealing in the median and middle lanes. The 4-ft-wide median shoulder was brought to the grade of the new concrete overlay by using asphalt concrete. The remaining driving lane was closed to traffic and the remaining lane of concrete overlay constructed. Two lanes of traffic were maintained on a 22-ft-wide portion of the new concrete overlay. The right-hand shoulder was then brought to the new overlay grade by using asphalt concrete.

To prepare the existing concrete surface for overlaying, the following operations were specified. The entire surface was to be scarified by using a milling machine to a depth of 1/4 to 1/2 in. After concrete millings had been swept or vacuumed from the pavement, this same surface was to be thoroughly sandblasted to remove loose chips and contaminants that would interfere with bond.

Milling was specified to be done to a 3-in. depth at transverse and longitudinal joints. The amount of surface distress present was used to estimate the quantity of deep milling.

A cement-sand grout was specified as a bonding medium. The grout specification called for mixing to a thick slurry in a mortar mixer by using 1 part of portland cement to 1 part of concrete sand by

Figure 1. Transverse contraction-joint cross section.



volume. Water was added to achieve the desired fluidity.

The concrete used for the overlay was a modified NYSDOT class D mix. This mix had a cement factor of 725 lb/yd³, a water/cement ratio of 0.44, and a specified air content of 5.5 to 9.5 percent. The coarse aggregate used in the mix had a maximum size of 1 in.; most stone had a nominal size of 3/8 in. The mix was modified by specifying a water-reducing retarder in all concrete. The water reducer was used for workability at lower slumps and to take advantage of higher strengths due to any water reduction. This class D concrete mix was used in the past as a concrete pavement inlay and bridge deck overlay in thicknesses of 2 to 3 in.

Sealing of the new transverse and longitudinal joints, which were reestablished by transverse saw-cutting over the existing joints, was accomplished by specifying a preformed neoprene compression sealer. Dimensions of the transverse joint sealer are given in Figure 1.

A finished grade 2.75 in. above the original theoretical grade was specified. To achieve the new theoretical grade, the contractor was given the choice of deeper scarification or thicker overlay placement. The clearance of the one overhead structure was adequate and was not affected by the new overlay thickness.

Plans and special specifications were prepared and were advertised for bids. The low bidder of the eight contractors submitting bids was determined on September 4, 1980. The contract was officially awarded on October 14, 1980. Full-scale construction did not begin until the following spring.

CONSTRUCTION

Construction of the concrete overlay began April 6, 1981. Paving and associated work was completed October 31, 1981. Signs, arrow boards, and plastic post delineators were used to route traffic onto the median and middle lanes in both the northbound and southbound directions, limiting traffic to two lanes in each direction. When traffic rerouting was complete, a wedge of asphalt concrete was placed over the existing 10-ft-wide right-hand shoulder. This wedge flattened the cross slope of the shoulder and thickened the existing asphalt concrete. When this asphalt paving was complete, traffic was routed onto two 10-ft lanes made up of the right-hand shoulder and a 10-ft-wide portion of the right-hand driving lane. This left the median and middle lanes free of traffic so that construction could begin.

The following construction sequence was used. Scarification of the entire concrete surface to a depth of 1/4 to 1/2 in. was accomplished by using a large milling machine with a mandrel 78 in. wide. To meet the proposed grade and to remove irregularities, some sections of pavement that were high were milled to approximately an inch in depth. Most millings were picked up by the self-loader during milling and were trucked to a disposal area off the highway. Some of the millings were sold as is by the contractor for use in fills and driveways. A rotary broom mounted on a small tractor swept and windrowed the remaining millings for removal. This operation generated dust, because the sweepings were dry and not picked up and removed. Street sweepers were used to reduce the dust. Although the dust was reduced, the water used created a slurry with the fine millings, which later dried. Additional sandblasting was required to remove the dried slurry.

While the transverse-joint reservoirs were still visible, their locations were marked by using steel pins offset in the shoulders.

Deep milling was performed on transverse joints that emitted a hollow sound when sounded with a chain drag or a hammer. Hollow regions were delineated with paint. Except for a few joints, all emitted a hollow sound. Deep milling generally was ±2 ft on each side of the transverse joint, full lane width, and 3 in. deep. This deep-removal operation did not break up the layered concrete in the lower portion of the slab. The deep milling did expose the upper portion of the steel dowels as the concrete was milled. The exposed dowels were not dislodged and incurred only minor gouging due to the milling operation. No corrective action was necessary.

Figure 2. Pavement sandblasting machine in operation.

Deep milling was also done on the longitudinal joints. A smaller milling machine that had a mandrel 24 in. wide was centered over and milled the longitudinal joint 3 in. deep. Ten percent of the longitudinal joint was in good condition and did not require the deep milling. High pavement mesh that was encountered during milling did not hamper production. The steel wires were broken, pulled out, or flattened in place. Loose wire or exposed ends were later cut off by hand. A vacuum truck and rotary broom were used to remove millings not picked up by a self-loader on the milling machine. The vacuum truck had a specially made high-capacity vacuum capable of picking up fines and stone in an 8-ft-wide pickup. A water spray system for dust control worked well, but continuous adjustments of the water spray and care not to exceed holding capacity were found to be necessary.

The holding capacity of these vacuum trucks had an effect on the dust emissions. Dust emissions increased when the holding tank was more than half full. The single truck on this project was able to maintain production and stay within the limit for capacity.

The milled surface was then sandblasted by using a unique sandblasting machine. This machine was specially made for this type of operation and is the only one in the United States (Figure 2). The sandblaster was mounted on a trailer and consisted of a large air compressor, a vacuum unit, the sand pressure chamber, and four sandblast nozzles mounted in the rear of the trailer. The nozzles were constructed to sweep in arcs while pointing down at the pavement. Skirts, enclosures, and vacuum hoses were used to contain and pick up the sandblasting sand, concrete chips, and dust. In spite of the enclosures and vacuum system, dust did escape to affect nearby homes and adjacent traffic. Continuous efforts resulted in improvements to the operation, but additional improvements are still needed. Sandblasting is necessary for bond and to remove partially loosened concrete chips, contaminants, and the dried slurry that forms when the millings are wetted and then dried.

Milling, sandblasting, and vacuum removal were operations handled by subcontractors. The concrete and asphalt paving and related work were performed by the general contractor.

Pressure-relief joints were installed after milling operations. They consisted of sawcutting out a 7- to 17-ft length of pavement to remove, reshape, and compact the subgrade damaged during removal and then filling and compacting dense-graded asphalt concrete in this opening to the top of the existing pavement. There was a need for pressure relief during sawcutting because the pressure in the pavement caused the circular saw to bind as its sawcut kerf closed behind it. The blade had to be freed by using a jackhammer so that operations could continue. After the blade was freed, a wheel cutter was used for the initial cuts. A 4-in.-wide cut was made across all the lanes to relieve the pressures. Traffic was halted temporarily while cutting took place. Two cuts were made by using the wheel cutter. Sawcuts were then used to provide a neat edge for the pressure-relief joint. When the concrete overlay was placed later, work was not stopped to install these pressure-relief joints; paving was done directly over the asphalt concrete. A strip of the concrete overlay was shoveled out while still plastic to break the continuity of the overlay. The edges were sawcut after the concrete had set, the excess overlay concrete was removed, and the space was filled and compacted with a dense asphalt concrete to the surface of the overlay.

After sandblasting and vacuum removal of any residue, the concrete paving operation began. A portland-cement grout was spread by hand and broomed into the prepared surface. Grout was placed only 10 to 20 ft ahead of the paver to prevent drying of the grout before paving. The grout consisted of equal parts by volume of portland cement and concrete sand. It was mixed in a mortar mixer with enough water to give a flowable, slurrylike consistency. The mortar mixer and cement, sand, and water containers were mounted on a flatbed truck that moved along with the paving operation. Hand placement of the grout was rapid enough to stay ahead of the paver. Estimates by the contractor showed that this operation could handle paving 24 ft wide.

The overlay was slipformed a lane at a time beginning with the median lane. A standard slipform paver with a leveling auger, internal vibrators, and an extrusion screed was used. For the initial placement, grade and alignment for paving were provided by a previously set, taut stringline. In subsequent lane placement, grade was taken from the previously placed lane. Deep-milled areas were paved simultaneously with the overlay. Adjacent traffic was narrowed to one lane during paving because the extra lane was needed to give the paver room to work. It was also used by the transit mix trucks to reach and supply the paver with their chutes. This lane was available only during off-peak traffic hours (Figure 3).

The class D concrete was supplied from an existing concrete batch plant, which was located 17 miles from the project. Depending on traffic, the driving time was about 1/2 hr. Some delays were experienced when the concrete trucks were caught in the traffic delays caused by the project.

After the ingredients had been batched into the trucks, 90 percent of the mix water was added and the concrete was mixed. The concrete was agitated while in transit. When the trucks arrived at the project, water was added and the concrete mixed to achieve the desired 2.5-in. slump. The average slump produced was 2.25 in. The concrete had an average entrained-air content of 7 percent. A set-retarding, water-reducing admixture was used in all

Figure 3. Middle-lane grouting and paving adjacent to previously overlaid median lane.

the overlay concrete. Compressive strengths of 3,900 psi at 4 days were achieved with this mixture; 28-day compressive strengths were 5,500 psi. Compressive-strength test results were used as a basis to open portions of the concrete overlay to traffic after 4 days. This was necessary at the NY-31 interchange ramps due to the maintenance of the traffic scheme.

Immediately after paving, the plastic concrete was scored 1/8 in. wide and 1/2 in. deep with a straightedge and an edging tool over the transverse joints. The previously marked joint locations were used. These location pins were necessary because the narrow contraction crack was not visible in the irregular, scarified, milled surface. The scoring was an extra precaution taken by the contractor to ensure correct placement of the contraction crack if sawing delays occurred. This 1/2-in. scoring may not be sufficient to initiate a contraction crack. Sawcutting was not delayed; therefore this shallow scoring was not tested.

Transverse texturing was done with a motor-driven tine rake mounted on the same machine as the white-pigmented curing-compound sprayer. A tine texture of grooves 3/16 in. wide and about 1/16 to 1/8 in. deep at 3/4-in. spacing resulted from this operation. Tining was stopped 2 to 3 in. from the edge that would form a side of the longitudinal joint. This was done to prevent rounding that edge. When adjacent lanes were paved, the concrete was butted against the previously placed lane. No keyway or joint ties were used.

Immediately after texturing, the white-pigmented curing compound was applied at a rate of 75 ft^2/gal. This was twice the normal rate.

Transverse-joint sawing was done as soon as the concrete had hardened sufficiently to allow sawing equipment on it, and sawing could be done without concrete raveling. In most cases this sawing was done within 5 to 6 hr. An initial transverse sawcut was made 5 in. deep at the thicker areas of the joints. This depth of cut was made to ensure formation of a joint at the existing contraction crack. The sealer reservoir was formed by sawcutting after the adjacent lane had been placed.

Preformed neoprene transverse joint sealers were installed after the sawcut joint reservoir faces had been sandblasted clean and a lubricant adhesive applied. A hand-pushed machine was used for compressing and pushing the seal in the slot. The seal was installed after two lanes had been completed.

The remaining 12-ft end of the 36-ft-long seal was coiled and tied. The sealer was stored in the

Table 1. Friction numbers.

Parameter	Northbound Lane			Southbound Lane		
	Driving	Middle	Median	Driving	Middle	Median
No. of tests	15	15	15	14	14	14
Average	31	29	43	29	24	37
Standard deviation	3	3	4	3	2	4
Range	25-37	24-35	32-49	25-33	21-27	30-42

Note: Friction numbers in the table are dynamic coefficient of friction multiplied by 100. The dynamic coefficient of friction was determined by using the locked-wheel test trailer with full-scale ribbed tire according to ASTM E274. Testing was performed at 55 mph on July 12, 1982.

space between the construction work and the traffic. When construction of the third lane had been completed, this remaining end of the seal was installed. Both ends of the seal were wrapped around the edges of the overlay.

In those places where blowups had occurred and had caused transverse-joint misalignment between lanes, a hot poured liquid polyvinyl chloride sealer was used rather than the neoprene, which would have had to be cut at the misalignment intersection. Experience showed that a cut end of neoprene sealer will allow water and fines to infiltrate. The fines will build up and force the sealer from its reservoir where it will be exposed to traffic and be destroyed or torn out. With the loss of the sealer, the entire joint becomes accessible to water, salts, fines, and incompressibles. The liquid sealer was also used to seal the transverse joint constructed over the separated edges of the transverse-joint replacement done under the previous contract.

The longitudinal joints were sealed with a preformed neoprene seal placed in a sawcut reservoir in the same manner as the transverse seals.

The shoulders on either side of the pavement were paved with asphalt concrete to match the new grade of the concrete overlay. The shoulder paving was done while traffic was diverted to complete the construction.

POSTCONSTRUCTION FINDINGS

To determine whether the minimum thickness had been achieved and also to test the bond of the overlay, 4-in.-diameter cores were taken. The cores showed that the desired thickness had been achieved. To determine bond strength, the cores were subjected to a modified shear test. The core was supported on steel rollers 1.5 in. apart. A load was applied vertically through another steel roller centered on the bond interface. It was assumed that flexural effects were minimized. The shear force divided over the specimen's cross-sectional area was reported as its shear strength. Testing on 96 cores showed an average shear strength of 147 psi, a standard deviation of 36 psi, and a range of 45 to 250 psi. The fracture resulting from testing always occurred in the original concrete. The presence of the coarse aggregate affected the shear strength. Coarse, laminated, porous stone in the plane of the fracture resulted in lower strengths. The mortar grout and surface preparation resulted in an adequate bond to the existing concrete.

Additional testing was performed in October 1982 with a shear collar device similar to that used by the state of Iowa. Testing of ten 4-in.-diameter cores, which were approximately 1 yr old, showed an average strength of 575 psi, a standard deviation of 126 psi, and a range of 363 to 808 psi.

The difference in values is due to the differences in test methods. Nevertheless, both methods and visual observations show an adequate bond.

Narrow transverse cracking was noted during construction of the overlay. The cracks are ±0.008 in. wide and extend for the full depth and lane width of the overlay placement. Crack spacing as close as 1.5 ft was noted. In general, most slabs had four to six cracks in the overlay. Cores showed that the cracks formed near the surface while the concrete was still plastic and then progressed and fractured full depth after the concrete had hardened. Observations after construction was completed showed that the cracking predominated in the lanes placed during the hot summer months, whereas the lanes placed in the cooler fall temperatures were nearly crack free. The observations show that high temperatures, normal drying shrinkage, and slab curling due to day and night temperature differentials combined to cause the transverse cracking. Because the cracking is so narrow, no repairs are practical. Pavement in another state had shown similar cracking and had not experienced performance problems. It was therefore decided to monitor the pavement and determine the effect of the cracks on performance.

A detailed preconstruction survey was carried out for future reference on test sections set aside to represent the project. A California-style profilograph was used to measure roughness in the overlaid driving lanes of the two 2,000-ft-long test sections. The results show a profile index (PI) of 14.4 in. in the northbound driving lane and 14.8 in. in the southbound driving lane. A maximum PI of 12 is specified for new concrete pavement construction. These test results are adequate for this type of construction. Paving grade taken from the previously placed lane and the paving delays, which result in starting and stopping the paver, caused the roughness.

The rideability of the completed overlay was measured by using New York's test vehicle. The present rideability index was determined for the length of all three lanes in each direction. The test results show excellent weighted average values for each lane.

Friction testing with New York's locked-wheel pavement friction test trailer was performed at the posted speed of 55 mph. The testing was performed at 0.2-mile intervals throughout the 3-mile length of the project in each of the lanes. Testing took place after 8.5 months of traffic, including one winter.

As can be seen from Table 1, friction numbers lower than expected values were found in the southbound middle lane. The average friction number of 24 was below the expected minimum value of 31 at this test speed. A marginal friction number (29) was also found in the southbound driving lane and the northbound middle lane.

Both median lanes, which had less traffic, show higher friction numbers than the other heavily used lanes. It appears that the initial rough mortar ridges formed when tine texturing was done had worn away and caused this differential between lanes. Observations of the pavement on November 3, 1982, showed a loss of microtexture in the lane between the tine grooves. The median lanes had the coarsest microtexture, whereas the driving and middle lanes were noticeably smoother. Friction testing will be performed in the future to determine changes with time under traffic and to determine whether any corrective action may be necessary.

CONCLUSIONS

A bonded concrete overlay can be constructed while traffic is maintained on adjacent lanes and shoulders of this six-lane highway.

An adequate bond was achieved by using surface preparation consisting of scarification, sandblasting and cleaning, and a portland-cement and sand bonding grout.

As shown during construction, pressure relief in the existing pavement is necessary before the overlay is constructed.

To reduce delays in transporting concrete due to traffic and haul times and for faster response to problems, an on-site concrete plant is necessary.

Control of dust generated by the surface-preparation stage, although improved during the project, needs additional work on equipment development. Further work is also needed on equipment limitations and tolerable dust limits so that practical specifications for dust control may be developed.

Acceptable results on roughness, as measured by a profilograph, have been obtained.

Overall rideability of the overlay, as shown by New York State's present rideability index, was excellent.

Friction testing with New York's locked-wheel pavement friction-testing trailer generally showed acceptable friction numbers. Nevertheless, additional future testing is necessary to determine changes with time and any necessary action on some low test values found.

After exposure to one winter, including 8.5 months of traffic, the expected reflection cracks and the narrow transverse cracking, which occurred during hot weather, had not created any problems.

To determine project performance for cost-effectiveness, the overlay should be monitored annually for at least 5 yr.

ACKNOWLEDGMENT

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Portland-Cement Concrete Inlay Work in Iowa

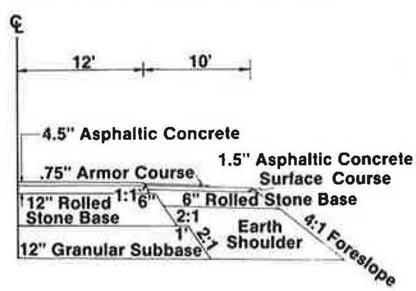
GEORGE CALVERT

High maintenance costs and continuing inconvenience to the traveling public have forced Iowa to take drastic measures in resolving a long-standing problem on Interstate 80 in western Iowa. It was found necessary to remove a section of asphalt and replace it full width with a portland-cement concrete section 10 in. deep. The removal and replacement operation led to the conclusion that in the future major problems could be corrected by replacement of the 12-ft travel lane only. Construction of the 12-ft travel lane proved to be cost-effective and no major problems were encountered. Through traffic was maintained in the normal passing lane and the contractor was limited to use of the 10-ft outside shoulder only. Included are details of the reasoning leading to the decision to reconstruct the travel lane only. Minor problems associated with smoothness of ride were corrected by use of a heavier finishing machine.

During the height of Interstate construction in Iowa in the late 1950s, a section of Interstate approximately 13.7 miles long was constructed through Adair, Dallas, and Madison Counties by using existing design practices for the construction of asphalt pavements. This section of roadway was constructed by using 12 in. of crushed-stone granular subbase overlaid with 12 in. of rolled-stone base followed by 3 in. of asphalt binder material and 1.5 in. of asphalt surface course. This construction was completed in 1959 and opened to traffic in 1960 (Figure 1).

This section of I-80 is now handling approximately 15,000 vehicles/day; trucks make up 35 percent of this traffic. Many of them are cross-country, heavy semitrailer loads. Because of the type and volume of traffic as well as because those grades were laid to a lower standard than that used now and the water table is higher than desirable, it was recognized that this was an inadequate design.

Figure 1. Interstate construction, 1958.

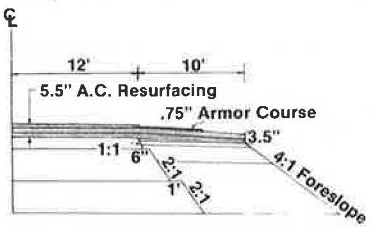


FIRST OVERLAY

The enormous buildup in cross-country truck traffic and the high water table caused premature damage to this thin section. A major overlay with asphalt materials was scheduled in 1964. With the encouragement of both FHWA and the asphalt industry, this section of roadway was overlaid with 5.5 in. of asphalt material, which included a 2-in. leveling course and a 2-in. layer of binder course. Both courses had a maximum particle size of 1.5 in., which caused some problems. This was overlaid with 1.5 in. of 3/4-in. surface course (Figure 2).

The problem encountered with the 1.5-in. leveling and binder course was based on the use of a limestone aggregate. Because this aggregate had relatively high absorption, it was difficult to completely dry the particles that were between 0.5 and 1.5 in. The particles continued to give off mois-

Figure 2. Overlay of Interstate pavement, 1964.



ture even after they had been taken from the dryer and had been coated with asphalt. The result was bad stripping of the coarse aggregate particles even at the time of laying. This did not cause concern, even though there was an enormous drop in temperature between the mixing operation and the actual laying operation, because the moisture was driven off and condensed; it actually streamed out of the trucks. The reason for the lack of concern was that while the material went through the spreading operation, the paving hoppers, and the auger systems, it appeared that the coarse aggregate particles were being recoated. They lost their brown color and reverted to a blacker shade as the asphalt was redistributed on them. This, however, laid the groundwork for future problems.

When this 4-in. layer of 1.5-in. maximum-size particle mix was consolidated and the surface course was put on top, it was discovered within a matter of months that there were further problems with stripping of the coarse aggregate particles. This was caused by the heavy-truck traffic and the high moisture table, which fed large quantities of water into this layer. The hydraulic pressures exerted on these particles by repeated truck loadings and the presence of excessive amounts of water caused stripping. The surface shoved badly in those areas where stripping occurred and where the binder-level courses lost their strength.

SECOND OVERLAY

These premature failures the second time caused a desperate attempt in 1970 to remedy what appeared to be a bad situation. Still more asphalt was placed on top of this section--an additional 2 in. of binder course and a 1-in. surface course (Figure 3). By this time the mainline roadway was approaching a thickness of about 13 in. of composite asphalt, a reasonable design section for the traffic volume. The problem was that each of the layers had broken up before being overlaid, so the composite thickness was not nearly so impressive as it should have been because each succeeding layer had failed at an early date.

Again this 3-in. asphalt overlay lasted a few short years and the roadway started to show failures due to the underlying 4 in. of 1.5-in. mix. There was insufficient asphalt overlay over this material to support the heavy truck loading, and continual stripping of this aggregate in the saturated condition caused failures to work through the surface.

The traveled lane showed the worst distress and the passing lane showed little distress. The shoulders in this section, which were now a total of 10 in. thick, showed no distress at all. These layers had not failed, so there was 10 in. of good asphalt on the shoulders throughout the entire project. There was almost continual failure for the full 13.7-mile length of this project in the truck lane. There were only isolated spots of failure in the passing lane, where the moisture table was

Figure 3. Additional overlay of Interstate pavement, 1970.

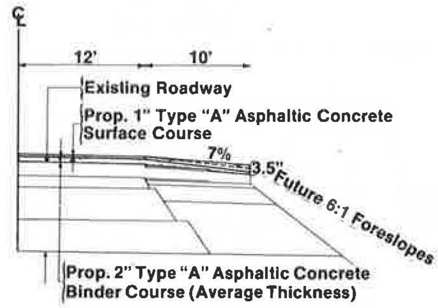


Figure 4. Trenching 42 in. deep for placement of longitudinal drain.

higher, and at one location on the inside of a superelevated curve.

By the late 1970s it was obvious that the previous approach to a permanent solution to this problem was in error. Additional overlays of asphalt were forcing the elevation of structures to maintain clearance, and any solution to the problem was temporary in nature. The engineering staff of the Iowa Department of Transportation did much brainstorming to effect a permanent repair on this section of roadway.

INSTALLATION OF LONGITUDINAL DRAINS

There was some disagreement on a permanent low-maintenance repair, but the staff was unanimous that the water problem must be solved first. A part of any repair instituted would involve installation of longitudinal drains in porous backfill, which would drain the bottom of the lower lift of crushed stone. The plan called for cutting a trench from the surface to the bottom of the crushed stone approximately 42 in. deep and 10 in. wide as the first stage of construction on this project. This trench was to be located under the outside edge of the slab for the full length of the project (Figure 4). The material removed with the trenching operation was to be placed over the outside 10-ft shoulder edge to form a rock fillet in this area. The longitudinal drain was then to be placed 3 in. above the bottom of the trench and the porous backfill was placed over it to near the lower surface of the roadway. This proved to be a satisfactory method of lowering the water table and keeping it to a much more acceptable level under the roadway surface (Figure 5).

CONSTRUCTION OF INLAY

After much discussion, it was agreed to mill out the

Figure 5. Installation of longitudinal drains, 1979.

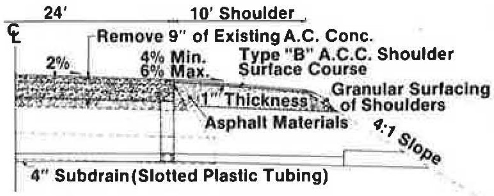
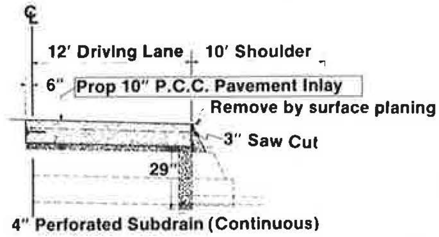


Figure 6. Milling asphaltic concrete 10 in. deep and 12.5 ft wide.

Figure 7. Construction of inlay, 1981.



top 9 in. of the asphalt now in place down to the bottom of the questionable 1.5-in. maximum-size particle mix that had caused problems because of stripping and shoving. This made a trench 24 ft wide and 9 in. deep for the 6-mile project from the Redfield interchange west to the Stuart interchange. Nonreinforced concrete paving was then placed in this trench area to a depth of 10 in. Load-transfer devices were placed in joints at 15-ft intervals. The asphalt-concrete material removed was stockpiled for recycling on other projects in the area.

During the milling and removal of the 9 in. of asphalt, it became apparent that the passing lane was not nearly so deteriorated as the traveled lane. The contractor was allowed to use the 10-in. asphalt shoulder for construction traffic, and this worked well. There were no failures or signs of distress on the shoulder during the hauling of the wet batches out to the paving job. The shoulder material had been built up over the years as the mainline was overlaid. Even though the shoulders had showed no distress, it was necessary to keep them up to grade to have a uniform cross section. This was a stable, strong shoulder that was not to be wasted on this or future projects. It was planned, therefore, to pave the mainline 1 in. higher than the shoulders and, after the paving had been completed, to bring the shoulders up to match the edge of the portland-cement concrete inlay with asphalt material. Consequently, the inside 6-ft and outside 10-ft shoulders were overlaid with 1 in. of asphalt.

The 10-in. concrete inlay in the center 24 ft had as a base 4.5 in. of asphalt, which was laid during the initial construction in 1958, underlaid by 2 ft of crushed stone. It was believed at that time, which has since been confirmed, that this section would provide sufficient strength so that little maintenance would be required in the future. To date there has been no distress or problems with this mainline paving or the shoulders on this project. The shoulders on this section of roadway now consist of 6 in. of rolled-stone base overlaid by 11 in. of various courses of asphalt, all of which are performing well.

The design and construction of this section of roadway involved a learning process from which there has been considerable benefit. This was a small segment of the total problem in this area. It was discovered during the construction of this inlay project that the passing lane was in relatively good condition; there were failures only in isolated areas. Successive overlays of asphalt had left a good section of asphalt and crushed stone in this area. The shoulders were in excellent condition and were to be salvaged if at all possible. The engineering staff was still faced with the problem of what to do with the remainder of this roadway.

In discussion sessions, it was decided to take the bold step of replacing only that section of roadway that was badly damaged. This decision was made because of a lack of funds to replace the full roadway width and the firm belief that by replacing only the traveled lane, the immediate problem could be solved. The other half of the roadway could be replaced at a later time with little additional cost.

The project began at the western termini of the 24-ft inlay at the Stuart interchange and extended west of the I-25 interchange, a distance of slightly more than 8 miles. In this instance, it was decided to inlay 10 in. of concrete 12.5 ft wide. This extended the new concrete inlay over the centerline joint 6 in. and, it was hoped, would mitigate the problem of maintaining the centerline joint. This time 10 in. was milled and removed in order to match the centerline joint and the shoulder profile (Figure 6). It was planned to true up the longitudinal profile of the 10-ft shoulder edge and the centerline before the 12.5-ft trench 10 in. deep was milled and removed. This proved to be a good operation also and made it much easier for the padline of the paver to run on a good profile surface. A longitudinal drain was again placed under the outside shoulder of the slab to drain water from the base and subbase sections. The end product again provided 24 in. of crushed stone topped by 3.5 in. of old asphalt and 10 in. of portland-cement concrete. This total thickness is more than adequate to provide a roadway section that will require minimum maintenance in the future (Figure 7).

To provide for possible future replacement of the inside lane, a keyed tied joint was placed the full length of the project near the centerline of the new pavement. If the passing lane needs to be replaced in the future, it can be milled out and replaced and the tie bar can be straightened out on the keyed tied joint assembly to tie in the two lanes of roadway (Figure 8).

This turned out to be a structurally adequate design. The contractor had problems with equipment early in this project, which caused some rough pavement that had to be corrected. If future designs of this type are used, the industry has the necessary expertise to produce a good inlay.

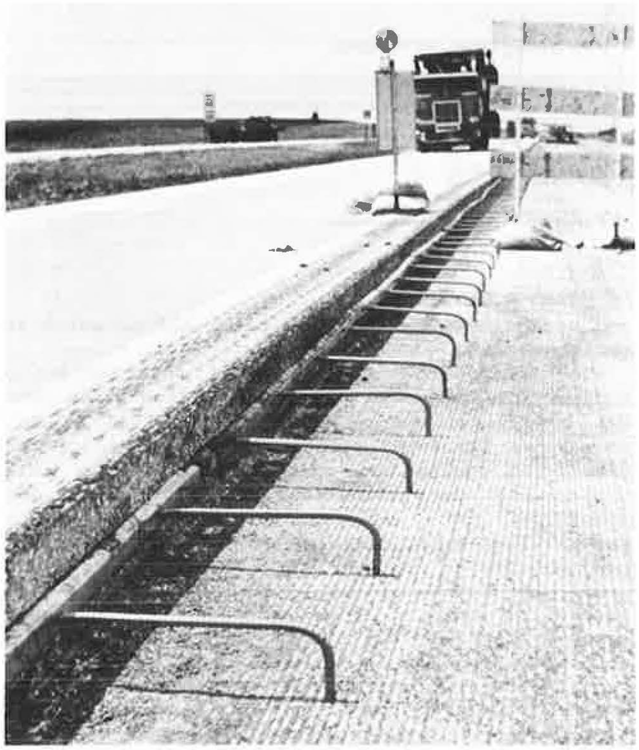


Figure 8. Tie bars in key and dowel assembly at centerline.


Figure 9. Finished portland-cement concrete inlay.

This project was constructed under traffic. The contractor was required to restrict operations to the trenched area plus 2 ft of adjacent roadway and to keep construction traffic on the 10-ft shoulder (Figure 6). This worked well and brought about a real cost savings. Where traffic volumes are as low as they are on this section--approximately 12,000 vehicles/day with 27 percent trucks--this procedure is adequate.

CONCLUSIONS AND RECOMMENDATIONS

It is believed that there is considerable merit in repairing only that part of the roadway that is extensively damaged. In this instance, and on the many hundreds of miles of Interstate showing distress in the traveled lane only, there is real justification in repairing the distressed section only. Replacing a 12-ft lane as opposed to replacing 40 ft of roadway is obviously a cost savings. Iowa strongly recommends that other states consider reconstruction of the traveled lane only. Potential cost savings nationwide are enormous.

Traffic control is costly when a full 24-ft roadway is being replaced. The ease of doing this under traffic with minimum disruption to the traveling public has been demonstrated on this project; the technique will be used on other projects with the cooperation of FHWA. The replacement of only the 12-ft traveled lane on other sections of Interstate is being considered in Iowa. Such sections may be replaced with materials other than portland-cement concrete through the recycling process (Figure 9).

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Comparative Analysis of Dowel Placement in Portland-Cement Concrete Pavements

JAMES L. BURATI, JR., MICHAEL G. BEESON, AND HOKE S. HILL, JR.

No statistically valid proof has been found to indicate that dowel-bar alignment is better or worse by either the basket-assembly or the implanted-dowel-placement method. Also, no significant difference in the amount of joint-related distress in the basket-assembly projects compared with the implanted-dowel projects has been identified. There is no evidence that the joint-related distress on the projects studied can be directly linked with the type of dowel misalignment studied, because alignment error does not appear to be the sole determining factor of the distress. To provide the distress data for the study, visual surveys were conducted at selected concrete pavement locations constructed in the early 1960s in Alabama. Then an electronic metal detector was used to gather dowel alignment data from the same locations. Statistical tests and comparison procedures were employed to analyze the alignment data, and attempts were made to relate the alignment results to the distress data. Conclusions were drawn indicating that neither placement method appears to be superior, but statistical proof could not be established. Many uncontrollable variables, such as work crew difference, equipment difference, and inspection, may be as important to dowel alignment as the examined variables, e.g., contractor, form type, and dowel-placement method. Alignment data regarding dowel depth were unobtainable due to the electronic detection method of data collection employed and a prohibition against coring. Futher research should address this limitation.

Past practice of dowel installation in jointed plain portland-cement concrete (PCC) pavement has been to place the dowels either by a basket-assembly method or by mechanical implantation. Until recently most federal and state specifications have allowed the individual contractor performing the work the choice of dowel-placement method. Recent strict enforcement of tolerances for dowel alignment has virtually eliminated mechanical implanting. Some states, such as Georgia and Florida, have forbidden implanting and require basket assemblies. Dowels have been successfully implanted since the 1950s, but little is known of the actual bar position achieved by either the basket method or the implanting system.

Several recent studies by the Transportation Research Board (TRB) and others (1) have indicated that current dowel tolerance specifications are not realistic and that broader specifications can be tolerated. Due to the increased cost of approximately $30,000/Interstate mile for basket-assembly installation compared with that of the implanting method, it is important to investigate dowel alignment in existing pavements and the relationship of dowel location to joint distress. Little evidence linking distress to alignment error currently exists. These factors led to the initiation of this current study of dowel installation. After careful consideration, Alabama was deemed by TRB's Committee on Rigid Pavement Construction the most promising location for such a study because of the existence there of 67 miles of 20-yr-old PCC pavement. In this paper a description of the research effort and relevant findings are presented.

RESEARCH OBJECTIVES

The objectives of the study were

1. To determine from field investigations of existing pavements the alignment that had been achieved in the placement of dowel bars,
2. To determine the pavement distress that had developed at the joints, and
3. To determine whether a relationship exists between dowel-bar alignment and joint-related pavement distress.

In addition to the preceding objectives, other information was desired from the study. Also of interest was whether there is a significant difference in dowel alignment by using the implanting method compared with the basket procedure. Ultimately, an overall comparison of the basket-assembly method with the implanting method with respect to both distress and alignment was desired. Additional useful knowledge included the distress types present, their rates of occurrence, the severity, and the cause.

RESEARCH PROCEDURE

The overall research procedure consisted of four interrelated and progressive steps. The first step consisted of a thorough literature search; it was followed by the collection of initial background information on the pavement in question. The third step was a visual survey of distress; the final step consisted of actual field sampling of dowel alignment. This process was viewed as orderly and consistent with established procedures, because additional facts were gathered during each step.

Literature Review

A literature review was conducted to obtain pertinent background material related to pavement distress and dowel-bar alignment. The literature deemed beneficial was divided into the categories of distress, dowel alignment, and pavement surveys.

Past research has revealed several defects caused by load transfer, dowel alignment, and joint-forming problems. The most prominent defects caused are faulting, spalling, transverse cracking, restraint cracking, and raveling.

Joint-forming methods often lead to raveling and spalling when improperly performed (2). The three methods commonly employed to form joints are sawing, hand forming, and using inserts. Metal inserts that remain in place may fatigue or corrode (2), which can lead to spalling. Inserts may also tip from the vertical or be left above or below the riding surface. Defects in joints formed with inserts usually result from improper positioning of the insert (3).

Misaligned dowels can cause transverse cracking at midslab as well as spalling at the pavement surface above the dowels (2). Corrosion of dowels is a source of frozen joints; it can also cause transverse cracking (4). Faulting is related mainly to load-transfer problems. There are three major types of failure related to dowel misalignment (5, pp. 27-37): (a) transverse cracking at midslab, (b) local spalling at the joint, and (c) flexural cracks between the joint and midslab. Misaligned dowels can lock the joint (6). One research study (2) concluded that a 1-in. vertical misalignment can cause serious spalling. It was also determined that a 0.25-in. vertical misalignment or 0.75-in. horizontal misalignment was tolerable for a 0.50-in. joint opening. This is the basis for the dowel misalignment tolerance of 0.25 in./18-in. length now employed (2). Another study (5, pp.27-37), conducted in Alabama under laboratory conditions, concluded that alignment errors in the vertical plane were more critical than errors of equal magnitude in the horizontal plane. This same research also concluded that serious spalling failures could result from vertical alignment errors of 1 in. and horizontal errors of 3 in.

The final concern in the literature search was pavement surveys. The major objective of pavement surveys is to legitimately rate performance without biased opinions. Carey and Irick (7) state that rating consistency is important and that a replication of results is desired. The concrete-pavement-condition rating system developed by Majidzadeh and Ilves (8) consists of two steps: rating the pavement with a riding comfort index at highway speed and close inspection from the shoulder at predetermined random locations. This method does not require the closure of traffic lanes. The pavement-rating system used in the current study is a modification of the system of Majidzadeh and Ilves.

Collection of Background Information

The first step in organizing the field survey was a collection of all available pertinent information on the pavement in question.

Contacts with the Alabama Highway Department and the dowel supplier for the paving projects in question provided initial facts. After the old specifications and plans had been reviewed, the many variables of each project were studied, and the actual screening process to determine representative projects was performed. The screening process consisted of visually inspecting each project, recording outstanding characteristics, and analyzing the results. After close examination of the visual data, projects were labeled as either potentially beneficial or not applicable. This initial stage was followed by a more exact visual survey of selected projects.

Visual Surveys

The actual field visual surveys consisted of a preliminary visual survey, a detailed visual survey, and a visual survey of the joints actually tested for alignment. Each type was intended to garner progressively more definite results. This process was used to first generalize then to specify details.

The preliminary visual survey consisted of completing a rating form on the project in question. This form was used to record the distress type, rate of occurrence, severity, location of distress, general condition of the pavement, and any relevant comments. The survey was carried out in a car traveling 55 mph. The projects were divided into 0.5-mile segments. After each segment, the rater would stop and complete the section of the form coinciding with the appropriate location. The entire project was covered. The distress was estimated for the right and left traffic lanes for both directions.

The detailed visual survey was a much closer view of the projects that had been visually inspected by the preliminary method. It also encompassed both directional lanes and the right and left traffic lanes. The projects were divided into 0.5-mile segments. Each segment was further separated into 132 sections, consisting of the 20-ft slabs between the joints. Ten consecutive joints from the 132 were surveyed. The precise location of the detailed section was selected by a random process. This procedure stratified the projects, ensuring that at least one 200-ft section out of each 0.5-mile segment would receive close scrutiny.

Forms were prepared to record the actual project location, the distress type, the general condition, any unusual distress, and pertinent comments. Within each distress type, the severity, actual joint or slab location, quantity, and type of repair were recorded. The distress was estimated by actually walking the 200-ft section and by analyzing it from the shoulder. Each inspection location was randomly predetermined and marked at the shoulder before the actual inspection.

The sampled-joint visual survey consisted of inspecting the joints that were tested for dowel alignment. This survey did not cover all of the projects surveyed by the detailed and preliminary visual surveys. Also, only one directional lane was examined but both right and left traffic lanes were examined. The joints were inspected from the shoulder and the estimated distress was recorded on prepared forms. The forms were devised to account for the project surveyed, location, joint number, distress type, severity, general condition, and relevant comments.

Dowel Alignment Testing

The overall procedure for actual alignment testing consisted of randomly selecting the joints and dowels and then measuring the alignment with an electronic metal detector. According to Gary Fowler of the Georgia Department of Transportation, similar procedures have been used in studies performed in Georgia with the same detector model.

Several methods were considered for the alignment testing. After discussion with Georgia Department of Transportation officials, it was concluded that the metal detector was the most suitable method without actually coring the pavement. Coring was specifically ruled out by the Alabama Highway Department. The detector was considered accurate for horizontal measurements, but vertical (depth) measurements could not be determined.

The field measurement process consisted of marking the ends of the dowel bar on the pavement and then measuring the distance from the right-hand pavement edge to the end marks. This established the lateral positioning of the bar across the joint. Next, a measurement from the joint to the front dowel mark was made to determine the longitudinal position of the bar with respect to the joint.

The field sampling performed in Alabama was executed on the most representative projects from the visual surveys. One directional lane was tested; only the right traffic lane was used. Each project was divided into segments consisting of 20 joints. One randomly selected joint of the 20 was sampled. This was equivalent to a 5 percent sample stratified in 20 joint segments. The joints were marked at the right shoulder before the actual sampling. Because only the right lane was tested, there were 12 possible dowel bars to examine. The first and last bars, i.e., the bar closest to the pavement edge and the bar closest to the left lane, were omitted. This was done for several reasons. It reduced electronic interference, was safer, and simplified numerical calculations. Of the 10 available dowels, only 4 were inspected. These were selected by a random process. For each joint sampled, the joint location, grade, bar measurements, and relevant comments were recorded.

ANALYSIS OF DATA

Distress-related data were gathered from the three visual surveys, whereas dowel-alignment data were collected by using the electronic metal detector. Each data type was analyzed differently.

The preliminary visual survey, the detailed visual survey, and the sampled-joint visual survey were all analyzed similarly. The raw data were separated into categories by project, type of dowel installation, contractor, location, and grade. The distress types were categorized as spalling, faulting, transverse cracking, restraint cracking, and other distress. The results were summarized into tables, and brief descriptions were made.

The data collected from actual field testing were analyzed in more detail than the distress data because quantitative measurements rather than subjective ratings were obtained. Before a statistical analysis technique could be devised, the raw numerical measurements were converted into the desired variable characteristics.

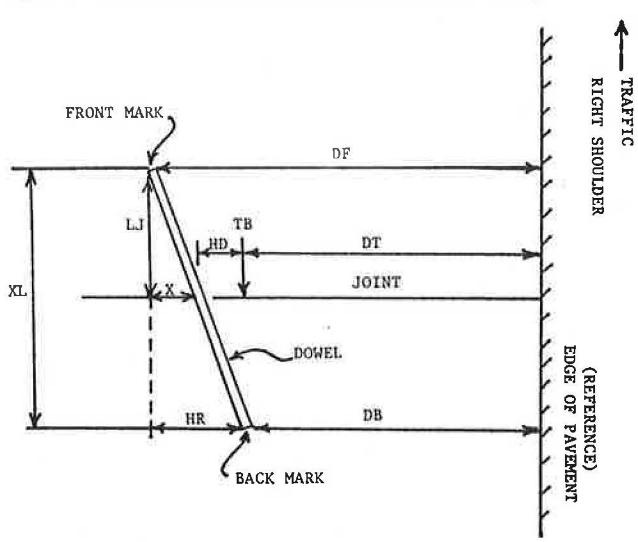
Calculation of Variable Characteristics

As previously noted, three measurements--distance from the pavement edge to the front mark of the dowel (DF), distance from the pavement edge to the back mark of the dowel (DB), and distance from the joint to the front mark of the dowel (LJ)--were made for each dowel. These measurements were used to calculate the three desired variables: horizontal rotation (HR), horizontal displacement (HD), and longitudinal displacement (LD). Each measurement was to the nearest 0.125 in. In Figure 1 the following theoretical, measured, and calculated variables are given:

1. TB = theoretical horizontal position of the dowel at the joint,
2. DT = distance from the edge of the pavement to TB,
3. HR = DF - DB,
4. $XL = (16^2 - HR^2)^{1/2}$,
5. X = HR(LJ)/XL,
6. HD = (DF - X) - DT, and
7. LD = LJ - 8.

HR, HD, and LD could result in either positive or negative values; absolute horizontal rotation (ABSHR), absolute horizontal displacement (ABSHD), and absolute longitudinal displacement (ABSLD) were defined as the absolute values for these variables, respectively. The horizontal rotation is a measure of the skewness of the bar in the horizontal plane. The longitudinal displacement measures the distance the bar is off center longitudinally. The horizontal displacement is a measure of the distance the dowel is left or right of the theoretical horizontal position at the joint. After the six computations had been performed, the analyses followed.

Figure 1. Measured and calculated variables for each dowel tested.



Methods of Analysis

Two approaches, labeled preliminary and final, were considered in the analysis of the alignment data. The preliminary method of analysis treated each dowel from a project as an independent observation and ignored probable within-joint as well as possible within-position dependencies. In the final method of analysis, the dowels within a joint were treated as subsamples and averaged to provide a single observation per joint.

The most important comparisons performed by means of the preliminary method of analysis were project, dowel installation type, form type, and contractor comparisons. The comparisons were made by a t-test procedure. The average values of the six characteristics of one category were compared with those of another category. Three additional factors were also analyzed by employing the preliminary analysis concept: the effect of dowel position on alignment, the effect of grade on alignment, and the interrelationship of dowels within a joint. These conditions were examined to illustrate possible additional causes of dowel alignment errors.

The final method of analysis was similar to the preliminary method except that it regarded the dowels within joints as more closely related to each other than to the dowels contained in other joints. The characteristics of the four dowels within each joint were averaged, and then the joints were averaged. Thus, each joint rather than each dowel was considered as a single observation. This reduced the sample size to one-fourth the size of the preliminary method. The same comparison procedure that was performed in the preliminary method of analysis was also employed in this final method.

RESULTS OF ANALYSIS

The projects considered were labeled with letters of the alphabet; the letters ranged from A to S. Of the 29 projects, 10 were labeled with both letters and numbers, as shown in Table 1. For project G-2 the G means that this project was constructed by the same contractor who built project G and during the same time span as project G; 2 means that this project was located adjacent to project G and differed only by federal contract number. The actual contractor identities were changed to fictitious names. The various geographic locations of the projects within the state were designated 1-4. In Table 1 the types of variables that affect both dowel alignment and joint-related distress are defined and the values of these variables are given for each project. After the initial background material had been prepared, the visual surveys were conducted.

Visual Survey Results

The results of all three visual surveys for the three major distress types (faulting, spalling, and transverse cracking) are summarized in Table 2.

The results of the preliminary visual survey were used as a barometer for the general condition of each project. Each was classified as being in poor, fair, or good general condition. The distress in all three visual surveys was subjectively rated as being very minor, minor, moderate, or severe. Also, the significant types of distress were recorded for each project.

Table 1. Variables affecting dowel alignment and joint-related distress.

Project	Contractor[a,b]	Type of Dowel Installation[a,b]	Type of Form[a,b]	Year Constructed[b]	Type of Base[b]	Joint-Forming Method[b]	Location[b]	1980 ADT[b]	Thickness[b] (in.)
A	Barnes	Implant	Rigid	1962	CTB	Unitube	1	7,620	9
B	Barnes	Basket	Rigid	1967, 1968	CTB	Saw	1	8,500	9
C	Nelson	Implant	Slip	1968, 1969	CTB	Unitube	1	9,130	9
D	Jones	Implant	Slip	1967	CTB	Unitube	1	9,130	9
D-2	Jones	Implant	Slip	1967, 1968	CTB	Unitube	1	9,130	9
E	Barnes	Basket	Rigid	1959, 1960	CTB	Saw	2	14,500	10
E-2	Barnes	Basket	Rigid	1959	LMST	Saw	2	14,500	10
F	Finley	Implant	Rigid	1960	CTB	Saw	2	14,500	10
G	Barnes	Implant	Rigid	1965	LMST	Saw	2	12,460	10
G-2	Barnes	Implant	Rigid	1965	LMST	Saw	2	12,460	10
G-3	Barnes	Implant	Rigid	1966	LMST	Saw	2	12,460	10
H	Barnes	Basket	Rigid	1960	LMST	Saw	2	13,150	10
I	Collins	Basket	Rigid	1958, 1959	LMST	Saw	2	16,000	10
I-2	Collins	Basket	Rigid	1958, 1959	LMST	Saw	2	16,000	10
J	Barnes	Implant	Rigid	1967	LMST	Saw	3	7,300	10
K	Barnes	Implant	Rigid	1961	LMST	Saw	3	7,300	10
L	Finley	Implant	Rigid	1964, 1965	LMST	Saw	3	6,000	10
L-2	Finley	Implant	Rigid	1964, 1965	LMST	Saw	3	6,000	10
L-3	Finley	Implant	Rigid	1964	LMST	Saw	3	6,000	10
M	Barnes	Implant	Rigid	1964	LMST	Saw	3	6,000	10
N	Rogers	Implant	Rigid	1965	LMST	Saw	3	7,000	10
N-2	Rogers	Implant	Rigid	1965	LMST	Saw	3	7,000	10
O	Finley	Implant	Rigid	1960, 1961	LMST	Saw	3	5,700	10
P	Smith	Implant	Rigid	1961	LMST	Saw	3	5,500	10
P-2	Smith	Implant	Rigid	1961	LMST	Saw	3	5,500	10
Q	Smith	Implant	Rigid	1966	LMST	N/A	4	11,000	10
Q-2	Smith	Implant	Rigid	1968	LMST	N/A	4	11,000	10
R	Barnes	Implant	Rigid	1963	LMST	N/A	4	12,000	10
S	Finley	Implant	Rigid	1961	LMST	N/A	4	13,500	10

Note: ADT, average daily traffic; CTB, cement-treated base course; LMST, limestone; rigid, rigid-formed; slip, slipformed; N/A = not available.

[a]Variable affecting alignment.

[b]Variable affecting distress.

Table 2. Distress results from all visual surveys.

Project	Survey Type	Avg Faulting[a] (in.) Right Lane	Avg Faulting[a] (in.) Left Lane	Spalling[b] (%)	Transverse Cracking[b] (%)	Project	Survey Type	Avg Faulting[a] (in.) Right Lane	Avg Faulting[a] (in.) Left Lane	Spalling[b] (%)	Transverse Cracking[b] (%)
A	Preliminary	0.15	0.08	3.3	6.6	G-2	Preliminary	–[c]	0.06	0.8	8.5
	Detailed	NP	NP	NP	NP		Detailed	–[c]	0.06	0.0	3.3
	Sampled	0.13	0.07	19.5	13.8		Sampled	–[c]	NP	NP	NP
B	Preliminary	0.19	0.06	0.0	0.5	G-3	Preliminary	0.15	0.06	0.0	0.0
	Detailed	0.16	0.08	1.2	4.6		Detailed	0.13	0.06	0.0	0.0
	Sampled	0.15	0.07	0.0	2.3		Sampled	NP	NP	NP	NP
C	Preliminary	0.03	0.08	2.4	0.0	H	Preliminary	0.15	0.06	1.4	0.8
	Detailed	0.14	0.07	4.8	0.0		Detailed	0.13	0.03	7.5	0.0
	Sampled	0.07	0.02	6.0	1.0		Sampled	NP	NP	NP	NP
D	Preliminary	0.14	0.06	8.1	0.4	J	Preliminary	0.03	0.00	0.2	0.0
	Detailed	0.14	0.06	8.4	1.1		Detailed	NP	NP	NP	NP
	Sampled	0.12	0.05	7.5	7.5		Sampled	NP	NP	NP	NP
E	Preliminary	0.06	0.00	0.8	1.0	K	Preliminary	0.10	0.03	0.0	0.2
	Detailed	0.03	0.00	0.0	2.5		Detailed	NP	NP	NP	NP
	Sampled	0.07	0.00	1.3	5.2		Sampled	NP	NP	NP	NP
E-2	Preliminary	0.05	0.03	1.8	1.4	A-G[d]	Preliminary	0.11	0.05	3.0	5.3
	Detailed	0.05	0.01	0.8	4.4		Detailed	0.10	0.05	2.7	5.0
	Sampled	NP	NP	NP	NP		Sampled	0.11	0.05	5.1	9.8
F	Preliminary	0.06	0.00	2.6	1.8	A-G	Average (all types)	0.11	0.05	3.6	6.7
	Detailed	0.03	0.00	0.8	0.0						
	Sampled	0.07	0.00	2.6	0.0						
G	Preliminary	–[c]	0.06	3.6	26.5						
	Detailed	–[c]	0.06	1.0	22.0						
	Sampled	–[c]	0.06	0.0	44.0						

Note: NP = survey type not performed.

[a]Faulting estimated to nearest 0.062 in.

[b]Actual number of joints distressed per 100 joints (distressed and nondistressed) in each project.

[c]Ground.

[d]Projects tested for dowel alignment.

Table 3. Average alignment values for all projects sampled.

Project	Placement Method	ABSHR (in.)	HR (in.)	ABSLD (in.)	LD (in.)	ABSHD (in.)	HD (in.)
		Alignment Value (in.)					
A	Implanted	0.431	-0.180	1.689	-1.391	0.375	0.109
B	Basket	0.391	0.100	1.493	0.195	1.432	0.371
C	Implanted	0.389	0.140	0.612	-0.356	0.540	-0.473
D	Implanted	0.402	-0.213	0.523	0.311	1.162	-0.543
E	Basket	0.373	0.043	1.178	0.413	0.880	-0.720
F	Implanted	0.360	-0.106	1.401	-1.158	0.580	-0.095
G	Implanted	0.344	0.170	2.862	2.555	1.177	-0.851
All							
Avg	Basket	0.388	0.079	1.371	0.275	1.226	-0.028
Avg	Implanted	0.386	-0.021	1.084	-0.330	0.696	-0.355
Avg	Both	0.387	0.019	1.200	-0.086	0.910	-0.224
Minimum	Both	0.344	-0.213	0.523	-1.391	0.375	-0.851
Maximum	Both	0.431	0.170	2.862	2.555	1.432	0.371
Range	Both	0.087	0.383	2.339	3.946	1.057	1.222

Note: ABSHR, absolute horizontal rotation; HR, horizontal rotation; ABSLD, absolute longitudinal displacement; LD, longitudinal displacement; ABSHD, absolute horizontal displacement; and HD, horizontal displacement.

Projects J and K, both implanted projects, were clearly superior to the remaining projects, but projects A, G, and G-2, also implanted projects, were the worst. The reason for this was identified during the alignment sampling. It was discovered that these projects possessed numerous joints in which the dowels had been omitted altogether. The remaining projects were generally rated as being in good overall condition.

The detailed visual survey produced the most extensive results of the three types of visual surveys. Only projects B, C, D, E, E-2, and H were included. These projects were considered to be the most representative projects that could be tested during the allotted sampling time span. On projects B, C, and D, all at location 1, faulting was measured. This was not possible at the other sites because of high traffic volumes. On these projects, faulting was estimated.

The sampled-joint visual survey was performed on the seven projects tested for dowel alignment. Projects C and D revealed more spalling than other projects. The faulting severity of location 1 compared with that of location 2 was also similar to the detailed survey results.

Of the 469 joints surveyed, only 7.9 percent were found to be distressed. Furthermore, 2.9 percent of the joints were classified as very minor in distress, 2.5 percent were listed in the minor category, 1.9 percent of the joints were moderately distressed, and 0.6 percent of the joints were categorized as severely distressed.

Dowel-Alignment Results

A total of seven projects was sampled for dowel alignment: projects A, B, C, D, E, F, and G. Projects A and G, those lacking dowels, were tested less extensively than were the remaining five projects. A total of 511 joints and 2,035 dowels was included in the analysis. Six characteristics were analyzed: ABSHR, HR, ABSLD, LD, ABSHD, and HD.

The results of each project along with the overall results are given in Table 3. In addition, the average results of the basket and implanted projects are shown. The ABSHR values were virtually the same for both the implanted and the basket projects, but the ABSLD and ABSHD characteristics were noticeably better in the implanted projects.

Statistical Comparison Procedure of Dowel-Alignment Results

In an effort to evaluate the data statistically, t-tests, analysis of variance (ANOVA), and pairwise comparisons of means (averages) were performed. To compare the means between two variables, t-tests were used; the ANOVA was used when more than two variables were compared, and pairwise comparisons of means were used to determine category differences when ANOVA indicated significant differences. The variables compared were project, contractor, type of dowel installation, type of form, grade, location, and joint-forming method. The level of significance employed was 0.05 (5 percent).

Table 4. Summary of alignment variables for all projects sampled.

Project	Location	Placement Method	Type of Form	Joint-Forming Method	Base	Thickness (in.)
A	1	Implanted	Rigid	Unitube	CTB	9
B	1	Basket	Rigid	Saw	CTB	9
C	1	Implanted	Slip	Unitube	CTB	9
D	1	Implanted	Slip	Unitube	CTB	9
E	2	Basket	Rigid	Saw	CTB	10
F	2	Implanted	Rigid	Saw	CTB	10
G	2	Implanted	Rigid	Saw	CTB	10

Note: CTB, cement-treated base course.

Before the statistical comparison procedure was executed, Table 4 was devised to summarize the similar and dissimilar variables of each project. These variables and the contractor variable were used to establish groupings, and the statistical tests were employed to analyze the groupings.

The actual comparison procedure was an orderly process; each level, or step, was dependent on the preceding step or steps. Each step produced one of two results: (a) a statistically significant difference between the compared variables, which indicated that the process should not progress further, or (b) no statistically significant difference between the compared variables, which indicated that the comparison analysis should proceed. Table 5 should help clarify the process. The desired overall objective was a statistically valid comparison between the basket and implanted projects.

Results of Preliminary Analysis Techniques

The comparison process described in Table 5 was performed by using the preliminary analysis technique. A summary of the results is presented in Table 6. For the variables compared in the respective steps to be classified as similar for grouping, it was required that there be no difference between them with regard to any of the three characteristics (ABSHR, ABSLD, and ABSHD). If any one characteristic produced a difference, this would indicate that the compared variables were not similar for grouping, and the comparison procedure could be stopped at that step. Once the three base steps (steps 1, 2, and 4, as shown in Table 6) had produced a difference between the compared variables, the remaining steps were automatically rendered invalid. Thus, a statistically legitimate comparison between basket projects and implanted projects never materialized when the preliminary analysis concept was used. Other conditions, however, were examined by this technique.

Table 5. Comparison procedure employed by preliminary and final analyses.

Step	Projects Compared	Variable Compared	Similar Variables
1	B versus E	Project	Basket installation, rigid-formed, contractor
2	A versus G	Project	Implanted installation, rigid-formed, contractor
3[a]	B, E versus A, G	Implanted versus basket	Contractor, rigid-formed
4	C versus D	Contractor	Implanted installation, slipformed
5[b]	A, G versus F	Contractor	Implanted installation, rigid-formed
6[c]	C, D versus A, G, F	Rigid-formed versus slipformed	Implanted installation
7[d]	C, D, A, G, F versus B, E	Implanted versus basket	None

[a]Based on steps 1 and 2.
[b]Based on step 2.
[c]Based on steps 2, 4, and 5.
[d]Based on steps 1, 2, 4, 5, and 6.

Table 6. Results of preliminary analysis technique.

Step	Projects Compared	Variable	Project Results (in.) by Characteristic Compared		
			ABSHR	ABSLD	ABSHD
1	B versus E	B	0.391	1.493	1.432
		E	0.382	1.160	0.869
		Difference[a]	No	Yes	Yes
		Conclusion[b]	Dissimilar	Dissimilar	Dissimilar
2	A versus G	G	0.344	2.862	1.177
		A	0.427	1.798	0.387
		Difference	No	Yes	Yes
		Conclusion	Dissimilar	Dissimilar	Dissimilar
3	B, E versus A, G	B, E	0.388	1.371	1.226
		A, G	0.395	2.203	0.688
		Difference	No	Yes	Yes
		Conclusion	Dissimilar	Dissimilar	Dissimilar
4	C versus D	C	0.389	0.612	0.540
		D	0.402	0.523	1.162
		Difference	No	Yes	Yes
		Conclusion	Dissimilar	Dissimilar	Dissimilar
5	F versus A, G	A, G	0.395	2.203	0.688
		F	0.360	1.401	0.580
		Difference	No	Yes	Yes
		Conclusion	Dissimilar	Dissimilar	Dissimilar
6	C, D versus A, G, F	C, D	0.393	0.582	0.747
		A, G, F	0.376	1.758	0.628
		Difference	No	Yes	Yes
		Conclusion	Dissimilar	Dissimilar	Dissimilar
7	C, D, A, G, F versus B, E	C, D, A, G, F	0.386	1.084	0.696
		B, E	0.388	1.371	1.225
		Difference	No	Yes	Yes
		Conclusion	Dissimilar	Dissimilar	Dissimilar

Note: ABSHR, absolute horizontal rotation; ABSLD, absolute longitudinal displacement; ABSHD, absolute horizontal displacement.

[a]Determination of whether there was or was not a statistically significant (at 0.05 level of significance) difference.
[b]"Dissimilar" means that compared projects should not be grouped.

The effect of dowel position on alignment was not a major objective of the study, but an analysis of this effect did produce interesting results. It was desired to determine whether the actual position of the individual dowel influenced alignment. The primary interest concentrated on the slipformed implanted projects. It was hypothesized that these projects would be more readily influenced by dowel position than the other types of projects. The reasoning was that the dowels closest to the pavement edge (or farthest away from the centerline) would tend to become more easily misaligned than the dowels near the centerline. With no rigid forms anchoring the edge of the pavement and no basket connections between the dowels, it was believed that the edge dowels might move or slip. Nevertheless, all projects were analyzed, and groupings were classified into three categories: rigid-formed basket projects, rigid-formed implanted projects, and slipformed implanted projects. Table 7 provides a summary of the alignment results for the individual dowel positions. The results indicated no substantial evidence that individual dowel position had an effect on the alignment achieved.

Results of Final Analysis Technique

The step-by-step comparison procedure employed for the preliminary method was also used for the final analysis technique. The identical steps, comparisons, and statistical tests were performed. Although there were minor changes in results between the final and preliminary methods with respect to the individual characteristics, the overall conclusions for each of the comparisons, and ultimately

Table 7. Results of alignment for individual dowel positions.

Dowel Position	ABSHR (in.)	ABSLD (in.)	ABSHD (in.)
Rigid-Formed Basket Projects			
2	0.402	1.174	0.964
3	0.385	1.344	0.993
4	0.358	1.161	0.812
5	0.422	1.189	0.967
6	0.405	1.181	0.896
7	0.368	1.233	0.889
8	0.429	1.160	0.899
9	0.317	1.178	0.917
10	0.369	1.093	0.917
11	0.412	1.295	0.830
Rigid-Formed Implanted Projects			
2	0.400	1.680	0.825
3	0.277	1.910	0.744
4	0.341	1.595	0.700
5	0.373	2.061	0.622
6	0.373	2.028	0.522
7	0.357	1.841	0.548
8	0.554	1.679	0.625
9	0.314	1.683	0.696
10	0.344	1.450	0.562
11	0.424	1.569	0.446
Slipformed Implanted Projects			
2	0.502	0.748	0.630
3	0.438	0.636	0.792
4	0.380	0.647	0.604
5	0.442	0.532	0.798
6	0.409	0.539	0.874
7	0.365	0.577	0.647
8	0.399	0.589	0.804
9	0.262	0.564	0.955
10	0.350	0.507	0.671
11	0.395	0.484	0.654

Note: ABSHR, absolute horizontal rotation; ABSLD, absolute longitudinal displacement; ABSHD, absolute horizontal displacement.

Table 8. Results of final analysis technique.

Step	Projects Compared	Variable	Project Results (in.) by Characteristic Compared		
			ABSHR	ABSLD	ABSHD
1	B versus E	B	0.391	1.493	1.432
		E	0.373	1.178	0.880
		Difference[a]	No	No	Yes
		Conclusion[b]	Dissimilar	Dissimilar	Dissimilar
2	A versus G	A	0.432	1.689	0.375
		G	0.344	2.862	1.177
		Difference	No	Yes	Yes
		Conclusion	Dissimilar	Dissimilar	Dissimilar
3	B, E versus A, G	A, G	0.395	2.203	0.688
		B, E	0.388	1.371	1.225
		Difference	No	Yes	Yes
		Conclusion	Dissimilar	Dissimilar	Dissimilar
4	C versus D	C	0.389	0.612	0.540
		D	0.402	0.523	1.162
		Difference	No	No	Yes
		Conclusion	Dissimilar	Dissimilar	Dissimilar
5	F versus A, G	A, G	0.395	2.203	0.688
		F	0.360	1.101	0.580
		Difference	No	Yes	No
		Conclusion	Dissimilar	Dissimilar	Dissimilar
6	C, D versus A, G, F	C, D	0.393	0.582	0.747
		A, G, F	0.376	1.758	0.628
		Difference	No	Yes	No
		Conclusion	Dissimilar	Dissimilar	Dissimilar
7	C, D, A, G, F versus B, E	B, E	0.388	1.371	1.225
		C, D, A, G, F	0.386	1.084	0.696
		Difference	No	Yes	Yes
		Conclusion	Dissimilar	Dissimilar	Dissimilar

Note: ABSHR, absolute horizontal rotation; ABSLD, absolute longitudinal displacement; ABSHD, absolute horizontal displacement.

[a] Determination of whether there was or was not a statistically significant (at 0.05 level of significance) difference.

[b] "Dissimilar" means that compared projects should not be grouped.

the entire study, were not affected. The results of the final method are summarized in Table 8.

The final step, the overall comparison between the basket projects and the implanted projects, showed that the implanted dowels were better aligned than the basket-assembly dowels with respect to ABSLD and ABSHD. In ABSHR, the implanting system and the basket method gave virtually identical results. Thus, the overall result was that the implanted projects were superior to the basket projects. Because of contractor inconsistencies and contractor differences, however, this conclusion must be taken with some degree of skepticism. The most interesting trait emerging from the results involved ABSHR. Neither form type, contractor, project, nor type of dowel installation generated a significant effect on ABSHR.

Correlation of Alignment to Distress

After the distress and alignment results had been reviewed, an attempt was made to relate the types of distress (spalling, faulting, and transverse cracking) to the alignment characteristics (ABSHR, ABSLD, and ABSHD). Only projects B, C, D, E, and F were included. Projects A and G were omitted due to the missing dowels. Table 9 presents a comparison of the distress results and the alignment results, and Table 10 gives the ranking of projects B-F according to results for distress and alignment.

The ABSHR results produced an extremely small range, only 0.087 in. Thus, it was concluded that no valid correlation could be demonstrated with ABSHR. Nevertheless, an attempt was made to relate ABSLD and ABSHD with the different types of distress. It was theorized that the higher (worse) the alignment value, the more frequent or severe should be the distress, but no consistent trend was discovered. In summary, no characteristic was directly linked with a specific type of distress.

Another subjective trial was attempted in an effort to relate alignment with distress. All distressed joints from the sampled-joint visual survey were compared with the nondistressed joints. As is clearly shown in Table 11, the alignment results of the distressed joints were not any worse or any better than those of the nondistressed joints. With this under consideration, it was subjectively determined that the alignment results could not be directly linked with the distress.

Table 9. Comparison of alignment and distress results.

Project	Distress Results: Faulting[a] (in.)	Spalling[b] (%)	Transverse Cracking[b] (%)	Alignment Results (in.): ABSHR	ABSLD	ABSHD
A	0.14	9.2	8.2	0.431	1.689	0.375
B	0.17	0.4	1.9	0.391	1.493	1.432
C	0.11	4.2	0.3	0.389	0.612	0.540
D	0.13	7.0	3.0	0.402	0.523	1.162
E	0.06	1.2	3.6	0.373	1.178	0.880
F	0.05	0.3	0.5	0.360	1.401	0.580
G	–[c]	1.3	28.5	0.344	2.862	1.177

Note: ABSHR, absolute horizontal rotation; ABSLD, absolute longitudinal displacement; ABSHD, absolute horizontal displacement.

[a] Faulting estimated to the nearest 0.062 in.

[b] Actual number of distressed joints per 100 joints (distressed and nondistressed) in each project.

[c] Ground.

Table 10. Ranking of projects B-F by distress and alignment results.

Rank	Project by Distress Type: Faulting	Spalling	Transverse Cracking	Project by Alignment Characteristic: ABSHR	ABSLD	ABSHD
1	F	F	C	F	D	C
2	E	B	F	E	C	F
3	C	E	B	C	E	E
4	D	C	D	B	F	D
5	B	D	E	D	B	B

Table 11. Alignment results from distressed and nondistressed joints.

Project	Joint Condition	No. of Joints	Alignment Results[a] (in.): ABSHR	ABSLD	ABSHD
B	Distressed	3	0.333	0.688	0.973
	Nondistressed	127	0.392	1.512	1.443
	Both	130	0.391	1.493	1.432
C	Distressed	7	0.322	0.594	0.830
	Nondistressed	109	0.393	0.613	0.521
	Both	116	0.389	0.612	0.540
D	Distressed	10	0.381	0.400	1.281
	Nondistressed	48	0.406	0.549	1.137
	Both	58	0.402	0.523	1.162
E	Distressed	4	0.352	0.633	0.878
	Nondistressed	72	0.384	1.189	0.870
	Both	76	0.382	1.160	0.870
F	Distressed	10	0.400	1.434	0.655
	Nondistressed	62	0.354	1.396	0.568
	Both	72	0.360	1.401	0.580
B, C, D, E, F	Distressed	34	0.367	0.797	0.929
	Nondistressed	418	0.387	1.094	0.939
	Both	452	0.385	1.072	0.938

Note: ABSHR, absolute horizontal rotation; ABSLD, absolute longitudinal displacement; ABSHD, absolute horizontal displacement.

[a] Average values.

CONCLUSIONS

After the data that were obtained from the three visual surveys and the dowel alignment measurements had been analyzed, the following results and conclusions were reached:

1. There was no significant difference between basket-assembly and implanted projects with respect to joint-related distress. Neither method was clearly superior to the other.

2. Although faulting was not severe, the pavement 10 in. thick exhibited less faulting than did the 9-in. pavement.

3. Spalling was only noticeable on the metal-insert projects. The metal-insert joints appeared to be the cause of the spalling, because there was much evidence of corroded insert segments. Raveling was also attributed to the metal inserts.

4. The results of the visual surveys revealed minor distress in all projects except for two special cases, projects A and G. Transverse cracking was not a problem on any project except those two. Both projects A and G were severely distressed. The reason for this distress was probably the high percentage of joints that contained no dowels. The omission of these dowels is the likely cause of the widespread transverse cracking and severe faulting encountered on these projects.

5. The results of the analyses on the alignment data indicated that no valid statistical conclusion could be reached for the comparison between the implanting and basket-assembly dowel-placement methods. The overall results, which must be viewed with some skepticism, showed that the dowels in the implanted projects, on the average, were better aligned than were the dowels in the basket projects. It could not be shown statistically, however, that this difference could be attributed strictly to the dowel-placement method (implanted versus basket) and that other factors, such as contractor difference and contractor inconsistency, did not also contribute to the difference.

6. The alignment error of the dowel in the horizontal plane, or skewness, averaged approximately 0.375 in. for both basket and implanted projects.

7. With respect to the longitudinal position of the dowel at the joint, the implanted projects produced an average error of approximately 1 in., or about 0.375 in. better than the results of the basket projects.

8. Regarding the horizontal position of the dowel at the joint, the error for the implanted projects, averaging approximately 0.75 in., was 0.50 in. better than the average error found on the basket projects.

9. The position of the dowel, whether it was the second, third, fourth, and so on, from the edge of the pavement, did not have an effect on the alignment results for the dowels.

10. There was also no effect from pavement grade--uphill, level, or downhill--on the dowel alignment or joint-distress results.

11. No correlation could be found between the alignment characteristics considered--absolute horizontal rotation (ABSHR), absolute longitudinal displacement (ABSLD), and absolute horizontal displacement (ABSHD)--and the various distress types (faulting, spalling, and transverse cracking).

12. There was no evidence to identify dowel-bar alignment errors as the cause of pavement distress. Of the distressed joints that were actually tested for dowel alignment, the alignment was neither better nor worse than was the alignment of the dowels in the joints that exhibited no distress. There was therefore no evidence to conclude that the distress was directly linked with the type of dowel misalignment that was measured in this study. It should be noted, however, that the literature search indicated that vertical dowel misalignment may be more significant than horizontal alignment errors in leading to joint-related distress. Because the limitation imposed on coring precluded the opportunity of measuring vertical alignment, there is no way of knowing from this study whether or not vertical dowel misalignment was the cause of the joint distress that was observed.

13. There is no statistical proof that either basket projects or implanted projects are superior. There are indications that dowel alignment and joint-related distress may be more influenced by factors other than dowel-placement method. Contractor difference and inconsistency by the same contractor appear to be major factors in dowel alignment. There was a wide variation in dowel alignment between projects, even when the identical type of dowel installation (basket versus implanted), form type (rigid form versus slipform), and contractor were present. Inspection, supervision, work-crew difference, and equipment difference may be important factors in determining dowel alignment. Also, the attitude of the contractor toward quality construction and the need to finish a project quickly may also influence alignment. The type of dowel installation does not appear to be the sole determining factor in dowel alignment.

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Effect of Hot Climate on Shear Strength of Concrete

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Many construction projects are being carried out in countries known to have a hot climate during the major part of the year. High-temperature conditions create problems in preparation, placement, and curing of concrete and adversely affect the properties of concrete. Results are presented of tests on reinforced-concrete beams of different sizes prepared and cured at various temperatures; the tests were performed under both natural atmospheric conditions and controlled laboratory conditions. Tests have shown that even if the concrete mix is so designed to give the required compressive strength of concrete in high-temperature conditions, the shear strength of the concrete is still reduced by 7 to 20 percent in the temperature range of 90 to 113°F.

It is generally known that special problems are created when concreting is done in hot climatic conditions and that the quality of the concrete is adversely affected by high temperature during mixing, placing, and curing. Rapid evaporation of water at the time the hot ingredients are mixed occurs at high temperatures; this results in lower slump, which is generally restored by adding more water. This increased demand for water is considered to be largely responsible for the reduced strength of the concrete (1,2).

Furthermore, under high-temperature conditions cement sets faster, and it becomes difficult to compact and finish the concrete (3). The American Concrete Institute (ACI) Manual of Concrete Practice (4) gives the harmful effects of concreting under high-temperature conditions, and the necessary precautions to be taken in advance to minimize these effects are also given in various books and publications (1-5). In the construction specifications for regions with hot climates, it is required that the concrete temperature not exceed 90°F; hence either ice is added to reduce the temperature of the concrete or concreting is done in the evening when the atmospheric temperature is low enough so that the concrete can be prepared and placed at a temperature not exceeding 90°F. Sometimes it is impossible to avoid concreting under high-temperature conditions; precautions are then required, which not only are difficult to follow but add to the cost of the concrete. Even if the temperature of the concrete is lowered to 90°F or less and the workability is restored by adding the proper amount of extra water, the curing problem remains, because the process is to be carried out in hot weather for at least 7 days, preferably longer.

The compressive strength of concrete is the most important property for designing concrete structures. The shear, tensile, and bond strengths of concrete are expressed in terms of its compressive strength (6), which in turn is greatly influenced by the effective water/cement ratio. Although the extra quantity of water required at high temperatures can be estimated by using the information given in the ACI Manual of Concrete Practice (4), it is difficult to do so accurately under changing atmospheric conditions; the result is that too much extra water is added, which yields a higher effective water/cement ratio and lower compressive strength.

Moreover, it must be emphasized that natural atmospheric conditions in a hot climate are different from controlled laboratory conditions, mainly because the atmospheric temperature does not remain constant throughout the day. The high-temperature conditions prevail for only a few hours during the middle of the day, whereas the temperature at night may even be lower than 86°F. Therefore, the test specimens must be cured at varying daily temperatures. This is perhaps one of the main reasons for the conflicting results obtained about the effect of high temperature on the compressive strength of concrete (3,5,7-11).

Tests conducted at the University of Petroleum and Minerals, Dhahran, Saudi Arabia (12), have shown that if just sufficient extra water is added to compensate for the loss caused by evaporation in high-temperature conditions, which keeps the effective water/cement ratio unchanged, and if curing of the concrete is done properly at varying daily temperatures, the compressive strength of the concrete is unaffected even at concrete temperatures as high as 113°F. On the other hand, even if the concrete temperature is lowered by taking necessary precautions, improper curing in hot climatic conditions results in lower concrete compressive strength.

Therefore, if a concrete mix could be so designed to give the specified compressive strength when the mix is prepared at the prevailing high temperature and in the curing conditions of the natural atmosphere, the other properties of concrete that are known to depend on the compressive strength, such as bond strength (development length), shear strength, modulus of rupture, and tensile (split-cylinder) strength, should remain unaffected. Tests on reinforced-concrete beams with varying lengths of embedment (13) have shown that when beams are prepared and cured in hot weather, a reduction in the bond between steel and concrete results. This reduced bond causes a reduction in the moment capacity of these beams compared with that of beams prepared and cured under normal laboratory conditions even if the moment capacity is computed by using the actual compressive strength of the concrete prepared under both conditions, which eliminates the effect of hot weather. Consequently, the following test program was carried out to determine the effect of hot weather on the shear strength of concrete.

TEST PROGRAM

Two series of specimens were prepared at different temperatures and different mix proportions as described in the following.

Series I

Beams 5x5x25 cm reinforced with one 10-mm-diameter deformed bar in the tensile zone were prepared in the laboratory and tested over a simple span of 20 cm. The concrete temperatures at preparation and placement ranged from the normal laboratory temperature of 75° to 113°F. Curing of specimens prepared at higher temperatures was done in an oven with a 24-hr cycle of varying temperature. The specimens prepared at normal laboratory temperature were cured at the same constant temperature. For each temperature at which the mix was prepared, three beams were formed.

Three 2-in. cubes were also prepared from the same mix and cured in the same way as the beams to determine the compressive strength of the concrete and hence compute the nominal shear strength of the concrete according to the ACI code. In the concrete mixes for high-temperature specimens, an extra quantity of water was added to compensate for evaporation and hence get approximately the same kind of workable mix.

Series II

Beams 4x4.5x30 in. and 6x6x30 in. reinforced with two No. 4 deformed bars in the tensile zone were prepared and cured in the natural atmosphere in hot weather as well as at normal laboratory temperature and tested over a simple span of 24 in. The same mix proportions were used for all the specimens of this series, but an extra quantity of water was added to the mixes at high temperatures to compensate for the loss of water due to evaporation. Cylinders 3x6 in. were also prepared from the same mix and compacted and cured in the same way as the beams to determine the compressive strength of the concrete.

The specimens are described in Table 1, and the properties of the materials are given in Table 2.

Table 1. Description of specimens.

Specimen No.	Size (in.)	Effective Depth (in.)	Span (cm)	Concrete Temperature at Preparation (°F)	Type of Curing
Series I					
1	2x2x10	1.303	9	75	Inside laboratory
2	2x2x10	1.303	9	99	Inside oven
3	2x2x10	1.303	9	102	
4	2x2x10	1.303	9	110	
5	2x2x10	1.303	9	113	
6	2x2x10	1.303	9	75	
Series II					
1	4x4.5x30	3.5	24	75	Inside laboratory
2	6x6x30	5.0	24	91	Outside in natural atmosphere
3	4x4.5x30	3.5	24	100	
4	4x4.5x30	3.5	24	102	
5	4x4.5x30	3.5	24	111	

Note: $t^\circ F = (t^\circ C \div 0.55) + 32$; 1 in. = 2.5 cm.

[a]Size = bxhxL,
where
b = width of section,
h = overall depth of section,
d = effective depth of section, and
L = length of beam.

Table 2. Material properties.

Property	Series I	Series II
Cement	Type I	Type I
Water/cement ratio	0.55	0.54 to 0.59
Aggregate/cement ratio	4.0	3.0
Coarse aggregate		
Type	Limestone from Dhahran area	Limestone from Dhahran area
Percent passing		
3/8-in. to No. 4 sieve	70	40
3/4- to 3/8-in. sieve	-	50
No. 4 to No. 8 sieve	30	10
Fine aggregate		
Type	Dune sand near Halfmoon Beach	Dune sand near Halfmoon Beach
Fineness modulus	1.60	1.60
Ratio fine aggregate to coarse aggregate	2/3	2/3
Reinforcement		
No. of bars	One	Two
Diameter	0.39 in.	1/2 in.
Yield stress	55,300 psi	50,500 psi
Clear cover to bars	1/2 in.	3/4 in.
Relative humidity (%)	-	30-50

Note: 1 in. = 2.5 cm; 1 psi = 145 MPa.

Preparation of Specimens

Series I

The ingredients for all the specimens of this series were first mixed dry in the required proportions with a hand trowel. For the high-temperature specimens, the dry, mixed ingredients were put in an oven and heated to a temperature such that for different specimens different concrete temperatures were obtained after the specimens were mixed with water. The required amount of water was added to the dry mix and the ingredients were mixed thoroughly with a hand trowel for about 3 min until a uniform mix was obtained. The concrete temperature was noted after the ingredients were mixed. The beam and cube molds were then filled in three layers. Each layer was compacted by placing the molds on a small vibrating table. After the specimens were finished, the concrete temperature was again recorded. The average of the two temperatures was taken as the concrete temperature during the preparation of the specimens. The normal laboratory specimens were also prepared in the same way with all the ingredients at the normal laboratory temperature of 75°F. After being finished, the specimens were covered with thin polyethylene sheets; the normal-temperature specimens were left in the molds in the laboratory and the high-temperature specimens, in the oven; the oven temperature had been adjusted to the range 113° to 122°F. The specimens were removed from the molds after 18 to 24 hr.

Series II

The ingredients for the high-temperature specimens of series II were kept outside in the sun for about 3 hr to heat them before mixing. The materials for all the specimens were mixed for about 3 min in a portable mixer outside in the natural atmosphere in the early afternoon between 1:00 and 2:00 p.m. Concrete was placed in the molds in three layers and compacted by vibration on a vibrating table. The temperature of the concrete was recorded in the middle of the concreting operation. The normal-laboratory-temperature specimens were prepared in the same way inside the laboratory at about 75°F. After being finished, the specimens were covered with thin polyethylene sheets and left in the molds at the casting place until removed from the molds on the next day. The relative humidity ranged between 30 and 50 percent during the period of the test program.

Curing of Specimens

All the specimens of series I were moist cured up to the age of 14 days by being kept in polyethylene bags containing water. For the remaining 14 days, the specimens were kept in air. The normal-laboratory-temperature specimens were cured in the laboratory at a constant temperature of about 75°F, whereas the high-temperature specimens were cured in an oven at daily varying temperatures as given in the next section.

All the specimens of series II were moist cured up to the age of 7 days and in air for 21 days in the same way as those of series I except that the high-temperature specimens were cured by being kept in the natural atmosphere instead of the oven.

Oven Temperatures

The high-temperature specimens of series I were cured in an oven at varying temperatures to simulate the temperature conditions in the natural atmosphere. The temperature of the oven was changed four times in a cycle of 24 hr, i.e., 8:00 to 11:00 a.m., 95° to 104°F; 11:00 a.m. to 5:00 p.m., 113° to 122°F; 5:00 to 8:00 p.m., 95° to 104°F; and 8:00 p.m. to 8:00 a.m. (the next day), laboratory temperature of 75°F.

Testing of Specimens

All the beams were tested at 28 days under a concentrated load at midspan. The typical diagonal tension failure occurred in the beams. The beams were tested in triplicate for each of the specimens of series I and in duplicate for series II. The compressive strength of the concrete was determined by testing three 2-in. cubes for series I, the corresponding cylinder strength being taken as 80 percent of the cube strength, and three 3x6-in. cylinders for series II.

TEST RESULTS

For all the specimens, the shear force at failure (V_u) was determined from the average of the load at failure (P_u) for each set of beams. The shear strength of the specimens (V_c) was calculated from the average compressive strength (f_c') obtained for each set of three cubes or cylinders by using Equation (11-3) from the ACI code (6):

$$V_c = 2(f_c')^{1/2}bd \quad (1)$$

where b is the width of the beam and d is the effective depth of the beam.

The ratio V_u/V_c was calculated for each specimen and compared with that of the corresponding normal-temperature specimen of each series to determine the effect of high temperature on shear strength of concrete. Table 3 gives a summary of the test results and comparison of shear strengths for all the specimens.

DISCUSSION OF TEST RESULTS

Because the compressive strength of concrete varied from specimen to specimen--the range was 3,000 to 5,000 psi--it was not possible to directly compare the experimentally determined shear strengths (V_u) of specimens at different temperatures. Hence to eliminate the effect of compressive strength, the ratios of V_u to V_c (the shear strength estimated as recommended by the ACI code) have been compared at different temperatures as shown in Figures 1 and 2 for series I and II, respectively. This has another advantage in that the effect of temperature on compressive strength is also eliminated because the actual compressive strength of concrete at different temperatures has been used in calculating V_c.

Figures 1 and 2 show that for both series of specimens, prepared and cured under the controlled laboratory conditions as well as in natural atmospheric conditions, the ratio V_u/V_c decreases with an increase in temperature. In order to compare the results of series I with those of series II and also to determine the percentage of reduction in shear strength at different temperatures, the ratios of V_u/V_c for each specimen to that of the corresponding specimen at normal laboratory temperature of 75°F

Table 3. Test data.

Specimen	Temperature of Concrete (°F)	Avg Compressive Strength of Concrete[a] (psi)	Avg Load at Failure[b] (lb)	Shear Force at Failure (lb)	Shear Strength (lb)	V_u/V_c	$[(V_u/V_c)$ for Specimen at T_c °F] ÷ $[(V_u/V_c)$ for Specimen at 75°F]	Reduction in Shear Strength (%)
Series I								
1	75	3,950	2,183	1,092	328	3.33	1.0	0
2	99	3,837	1,992	996	323	3.08	0.925	7.5
3	102	4,458	2,000	1,000	348	2.87	0.863	13.7
4	110	4,205	2,000	1,000	338	2.96	0.888	11.2
5	113	3,847	1,633	817	323	2.63	0.790	21.0
6	75	5,407	2,372	1,186	383	3.10	0.930	7.0
Series II								
1	75	5,411	8,770	4,385	2,060	2.13	1.0	0
2	91	5,022	16,580	8,290	4,252	1.95	0.915	8.5
3	100	5,000	7,550	3,775	1,980	1.91	0.897	10.3
4	102	4,607	6,970	3,485	1,900	1.83	0.859	14.1
5	111	2,928	5,763	2,881	1,618	1.78	0.836	16.4

Notes: Temperature of concrete, T_c; average compressive strength of concrete, f_c'; average load at failure, P_u; shear force at failure, $V_u = P_u/2$; shear strength, $V_c = 2(f_c')^{1/2}bd$.
$t°F = (t°C \div 0.55) + 32$.
1 psi = 145 MPa. 1 lb = 0.45 kg.

[a] Average of three specimens for both series.
[b] Average of three specimens for series I and two specimens for series II.

Figure 1. Comparison of shear strength of concrete at different temperatures: series I.

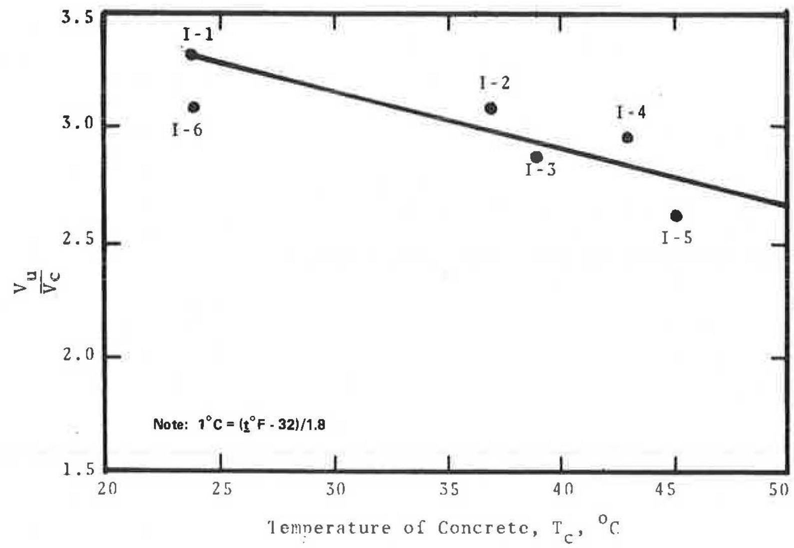


Figure 2. Comparison of shear strength of concrete at different temperatures: series II.

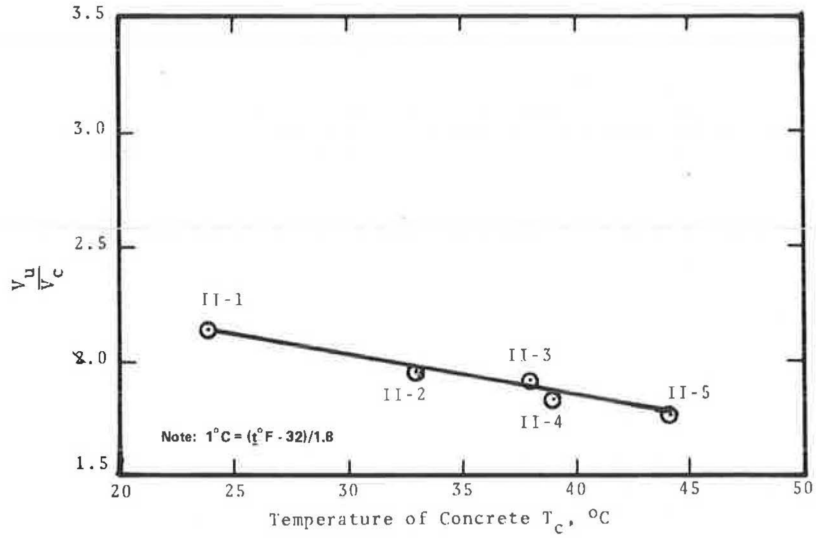
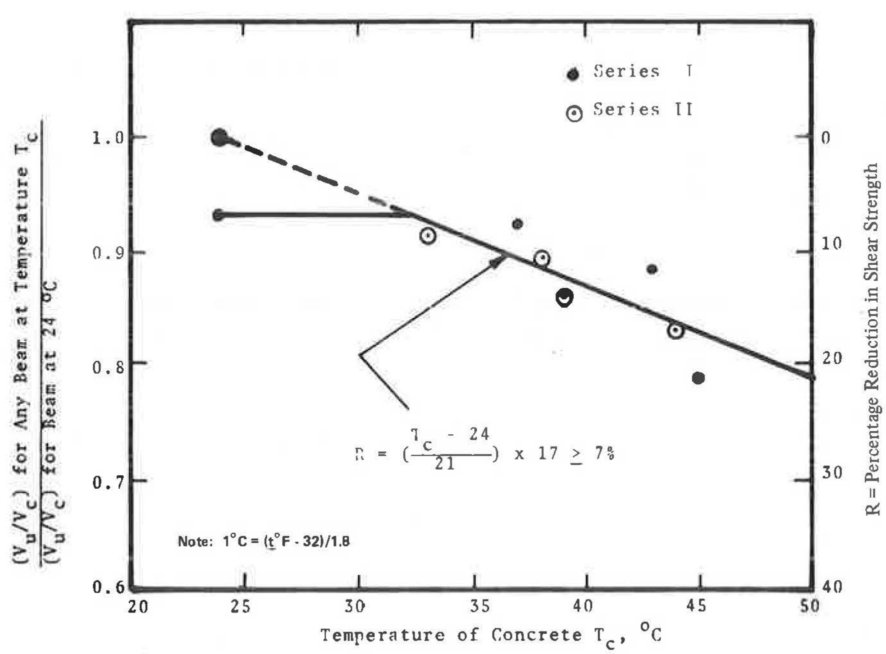


Figure 3. Reduction in shear strength of concrete at different temperatures.



have been calculated as given in Table 3 and are shown in Figure 3, which also shows the percentage of reduction in shear strength at different temperatures. It can be determined from Figure 3 that the percentage of reduction in shear strength at any temperature T_c°C of concrete is given as follows:

$$R = [(T_c - 24)/21] \times 17 \quad (2)$$

It is pertinent to emphasize here that because the actual f_c' of concrete at different temperatures was used in analyzing the test results, the previously mentioned reduction in shear strength will be obtained even if necessary precautions are taken or the concrete mix is so designed to give the required compressive strength in a hot climate.

The test results for specimen I-6, which was prepared at the normal laboratory temperature of 75°F but cured in the oven at high temperature simulating hot climatic conditions, show that the shear strength is reduced by 7 percent. This corresponds to the reduction obtained at 90°F by using Equation 2. Thus, even if all the necessary precautions are taken to reduce the temperature of the concrete to 90°F as laid down in the construction specifications, the shear strength of the concrete will still be reduced by about 7 percent, mainly due to the curing effect in hot climatic conditions. Thus, Equation 2 can be restated as follows:

$$\text{Percentage reduction in shear strength of concrete at any temperature } T_c{}^\circ\text{C} = [(T_c - 24)/21] \times 17 \geq 7 \text{ percent} \quad (3)$$

Hence, the shear strength (V_{ct}) of concrete at any temperature T_c can be expressed by any of the following equations:

$$V_{ct}/V_c = 1.0 - [(T_c - 24)/21] \times 0.17 \leq 0.93 \quad (4)$$

$$V_{ct}/V_c = 1.194 - 0.0081T_c \leq 0.93 \quad (5)$$

$$V_{ct} = V_c(1.194 - 0.0081T_c) \leq 0.93V_c \quad (6)$$

where V_c is the shear strength of concrete obtained from Equation (11-3) of the ACI code.

CONCLUSIONS AND RECOMMENDATIONS

The shear strength of concrete prepared and cured under hot climatic conditions is reduced with an increase in the temperature of the concrete. This is true even if all the necessary measures are taken to obtain the required compressive strength of concrete under hot climatic conditions.

The shear strength of concrete (V_{ct}) at any temperature of concrete (T_c) between 75° and 113°F can be obtained from Equation 6.

It is recommended that Section 11.3 of the 1977 ACI code be suitably amended to incorporate Equation 6 to modify Equation (11-3) of the code for shear strength of concrete in hot climatic conditions.

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Discussion

Waheed Uddin*

The authors should be commended for initiating laboratory research in Saudi Arabia on the effects of local environment on the quality and placement of portland-cement concrete. I have worked for several years as a materials engineer on civil aviation projects and in pavement research in Saudi Arabia and therefore am interested in this research. There are several points that need further clarification from the authors.

They mention the practice of adding water to improve the workability of concrete in hot weather conditions. I recall that while I was supervising the construction of a portland-cement concrete apron at Dhahran International Airport in 1978, addition of water in the transit mixer was strictly prohibited once it had left the batch plant. The workability of the low-slump concrete on such jobs was attained within desirable limits by designing a concrete mix with suitable admixtures, e.g., plasticizers. It is common practice, followed in all parts of Saudi Arabia, to use plasticizer in the concrete mix designed for any major construction where the quality is controlled through a materials testing laboratory.

The authors fail to describe the concrete mixes that were used in the preparation of laboratory specimens. It is well known that coarse aggregate is the most important component of the concrete mix that is affected by temperature variation. Was the same type of aggregate used in all specimens?

I have had wide experience in the availability of aggregate types in the quarries around Dhahran, which are evidently the source of the aggregates used in this study. Basically limestone, dolomite, or dolomitic limestone are the rock types crushed in the quarries of the Dhahran region. The limestone can be very soft and highly water absorptive. The dolomitic limestone is scarce, relatively hard, less water absorptive, and if used in concrete yields higher compressive and flexural strength. It has been reported that the percentage loss from the sodium sulfate soundness test (AASHTO T-104) for soft limestone is as high as 27 percent, and for dolomitic limestone it can be as low as zero (14). It is suggested that the authors include a data summary on the physical and engineering properties of the aggregate types used in their study.

The authors recommend amending the ACI code of 1977 [Equation (11-3)] based on the equation developed in their paper. It is pointed out that their relationship may not be unique. Apparently they did not consider three important variables in their testing program, namely, aggregate type, mix type, and presence of plasticizers.

The inferences made by the authors are obviously limited to the aggregate samples and testing conditions used in their study. Additional work is desirable before any amendment to the existing ACI code is considered.

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Authors' Closure

We appreciate the valuable comments by the discussant. No doubt suitable admixtures are used to improve the workability of concrete in hot weather. With regard to the practice of adding extra water to a concrete mix to compensate for evaporation of water from the mix, which was mentioned in our paper, the discussant may refer to papers by Scanlon (1) and Newman (2). Furthermore, if no extra water is added and if workability is not a problem, the net effective water/cement ratio is reduced due to evaporation of water. The results of the tests conducted at the University of Petroleum and Minerals (12) have shown that in this case the compressive strength is increased. In any case the results of the shear tests have been so analyzed and presented in our paper that the effect of hot weather on compressive strength is eliminated.

The necessary information on concrete mixes has been given in Table 2 of our paper, which gives the type and size of the coarse aggregate used in the mixes. The coarse aggregate used is limestone available in the quarries around Dhahran, which is used extensively in construction work. Its mineralogical composition is calcite, 73 percent; quartz, 10 percent; and dolomite, 13 percent.

Different mix proportions were used in the two series of tests. The water/cement ratio and the conditions for preparation and curing were varied to obtain compressive strengths from 3,000 to more than 5,000 psi. Equation 6 is developed on the basis of the test results obtained. Although not unique, it is believed to give a reasonable relationship considering the highly unpredictable conditions in hot weather. Any additional work will probably give similar results.

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Effect of Hot Climate on Slump Loss and Setting Times for Superplasticized Concretes

V. RAMAKRISHNAN AND V. PERUMALSWAMY

The results of a laboratory investigation of slump loss, setting times, and workability at low and high concrete temperatures for superplasticized concretes are presented. There was a substantial reduction in the initial and final setting times when there was a 16°F increase in the initial concrete temperature. Results of the slump-loss study indicated that there was adequate working time (2 to 3.5 hr) when a superplasticizer was used for concretes made at a temperature of 70°F (21.2°C). For the same concretes mixed at a temperature of 86°F (30°C), however, there was a considerable reduction in slump and available working time.

Concrete having desirable properties in the hardened state is normally made with a low water/cement ratio and with the least possible amount of cement paste in the mix. Such a concrete usually has a low slump and requires intensive and careful compaction. In order to produce concrete of the same quality with less vibration, very effective plasticizers, known as superplasticizers, have been developed for making flowing and self-compacting concretes. Superplasticizers are added to concrete to cause a vast increase in its workability or allow a large reduction in mixing water and thus produce high-strength concrete. Such a change in concrete properties would result in reduced placement costs or reduction in the cement requirement. A well-designed mix with a superplasticizer will have good flowability and sufficient cohesiveness and would not cause bleeding or segregation or strength reduction either during or after placement of the concrete.

The introduction of superplasticizers has opened up new possibilities for the use of concrete in construction, particularly for bridge-deck repair and resurfacing, pavement rehabilitation, and construction of other highway facilities.

Slump loss is an inherent property of concrete even with the addition of superplasticizers. The high slumps of superplasticized concretes are not sustained over long periods (1-3; 4, pp. 389-402; 5, pp. 137-157; communication from D.A. Whiting, Portland Cement Association), especially at higher temperatures. Hence a delay in the discharge of concrete from truck mixers could cause stiffening to the point of unworkability and loss in air content, which would affect the desired air-void system.

OBJECTIVES AND RESEARCH PROGRAM

The objectives of this research program were to study

1. The initial and final setting times at two different concrete temperatures approximately equal to the spring and summer concreting conditions in Rapid City, South Dakota, and
2. Slump and air-content losses for superplasticized concrete with time at two different concrete temperatures.

Two concretes, one with a high workability and medium cement content [6.5 sacks/yd^3 (363 kg/m^3)] and another with medium workability and high cement content [8.5 sacks/yd^3 (474 kg/m^3)] were studied. The former (mix 13) will be suitable for general structural work and for construction of highway pavements and airport runways, and the latter (mix 33) will be suitable for bridge-deck overlays and for construction where high strength and highly impermeable concretes are needed. Identical mixes were made in the spring (March) and in the summer (July).

The research program consisted of two parts. In the first, the initial and final setting times for the selected concretes were determined at different temperatures. In the second part, a study of slump-loss characteristics of superplasticized concrete was carried out.

MATERIALS AND MIXES

Cement

Type 1 portland cement satisfying ASTM C 150 was used.

Aggregates

The fine aggregate used was natural sand. A sample sieve analysis of the fine aggregate is shown in Table 1. It had a water absorption coefficient of 1.6 and a saturated surface-dry specific gravity of 2.62. The coarse aggregate used was crushed limestone. A sample sieve analysis of the coarse aggregate is also given in Table 1. It had a water absorption coefficient of 0.45 and a saturated surface-dry specific gravity of 2.69.

Water

The water used was from the municipal water supply.

Air-Entraining Agent

The air-entraining agent (AEA) used was neutralized vinsol resin (Protex).

Table 1. Sieve analysis of aggregates.

Sieve Size		
Passing Through	Retained on	Percent by Weight
Fine Aggregate		
1/4 in.	No. 4	0
No. 4	No. 8	14.30
No. 8	No. 16	24.94
No. 16	No. 30	28.87
No. 30	No. 50	17.88
No. 50	No. 100	10.55
No. 100	No. 200	2.80
No. 200	Pan	0.66
Coarse Aggregate		
1 1/2 in.	1 in.	1.0
1 in.	3/4 in.	28.8
3/4 in.	1/2 in.	50.8
1/2 in.	3/8 in.	15.3
3/8 in.	1/4 in.	3.0
1/4 in.	No. 4	0.5
No. 4	No. 8	0.3
No. 8	Pan	0.3

Note: 1 in. = 25 mm.

Table 2. Properties of superplasticizer.

Property	Lot No. 1	2	3	4	5
pH	9.6	9.7	9.7	9.7	9.7
Specific gravity	1.23	1.25	1.23	1.24	1.22
Percent residue by oven drying	44.50	42.85	42.41	42.48	42.49

Table 3. Mix proportions for selected mixes.

Item	Mix 13	Mix 33
Water/cement ratio by weight	0.38	0.28
Aggregate content (%)		
Coarse	54	51
Fine	46	49
Cement content (sacks/yd^3)	6.5	8.5
Superplasticizer	Mighty RDI	Mighty RDI
Dosage (% by weight of cement)	1.0	1.2
Air-entraining agent	NVR (Protex)	NVR (Protex)
Dosage (% by weight of cement)	0.08	0.18

Notes: NVR, neutralized vinsol resin.
1 sack/yd^3 = 55.77 kg/m^3.

Superplasticizer

The superplasticizer used for this study was Mighty RDI. It is a salt of a naphthalene sulfonate formaldehyde condensate and a standard retarding agent. It is a dark-brown liquid with a viscosity of 35 in./sec (90 cm/sec) at 150°F (68°C), pH value 10 ± 1.0 (5 percent aqueous solution), and a specific gravity of 1.2 ± 0.1. Five lots of superplasticizers received on different occasions were used in the entire investigation. They were tested and found to have the same chemical composition and physical properties within the allowable variations according to ASTM requirements (Table 2). All the lots showed the same characteristic absorption peak at the same wavelength in the infrared analysis.

Mixes

The mix proportions used for the selected concretes are given in Table 3. All the mixes were blended in a drum that had 6 ft^3 (0.17 m^3) of mixer capacity. The mixing sequence was as follows:

1. Coarse aggregates,
2. Fine aggregates,
3. Two-thirds of water and mix for 1 min to allow for absorption of water,
4. Cement,
5. Remaining water with air-entraining admixture,
6. Superplasticizer,
7. Mix for 3 min,
8. Rest for 3 min, and
9. Additional mixing for 2 min.

A total of 12 mixes was made by using the mix proportions designated 13 and 33. Six mixes were made to study the setting times of concrete. Original mixes prepared in the spring were designated FSET and the replicate mixes were designated FRSET. Mixes made in the summer were designated HTSET.

Six mixes were made for the slump-loss study. Original mixes made in the spring were designated SL and replicate mixes were designated SLR. Mixes made in the summer were designated HTSL.

TESTS AND SPECIMENS

The properties of fresh concrete--temperature, slump (ASTM C 143-78), air content (ASTM C 231-81), vebe time (ACI Standard 211-65), flow-table spread, and unit weight (ASTM C 138-81)--were determined immediately after mixing. To determine the rate of slump and air-content loss, slump and air content were determined and recorded at various time intervals. These tests were conducted until a zero slump was reached.

The setting time of the concrete was determined according to ASTM C 403-77.

The test specimens, 4 x 8-in. (101.6 x 203-mm) cylinders, were cast in steel molds immediately after mixing and at other time intervals when air content was determined. The cylinder-casting sequence and the time of casting of these cylinders are given in tables in the next section. Cylinders were unmolded after 24 hr and cured in lime-saturated water according to ASTM C 192-76.

After the dry unit weight had been determined, the cylinders were tested for compressive strength at the age of 3, 7, and 28 days.

ANALYSIS AND DISCUSSION OF RESULTS

Setting Times

First, during the month of March [concrete temperature 69°F (20.6°C)] the concrete setting times (ASTM C 403) were studied for the two selected mixes FSET13 and FSET33, which had water/cement ratios of 0.38 and 0.28, respectively. The setting-time studies were repeated for replicate mixes FRSET13 and FRSET33. Then during the month of July [concrete temperature 84°F (28.9°C)] the setting-time study was repeated for the same two mixes (HTSET13 and HTSET33).

The wet concrete mix was passed through sieve No. 4, which has 0.19-in. (4.75-mm) openings, to remove the coarse aggregates and the resulting mortar was used for the setting-time study. The mix proportions used are given in Table 3. The details of elapsed time, concrete temperature, air temperature, relative humidity, and penetration resistance are given for original and replicate mixes in Tables 4 and 5 and for the summer mixes in Table 6. The plastic properties of all the mixes are given in Table 7. The setting-time curves for mixes 13 and 33 are shown in Figures 1 and 2, respectively. The 3-, 7-, and 28-day compressive strengths for these concretes are plotted in Figure 3.

The slump and air content for the original mixes, FSET13 and FSET33, were 7 in. (178 mm) and 9.2 percent and 3 in. (76 mm) and 8.6 percent, respectively. The slump and air content for the replicate mixes FRSET13 and FRSET33 were 6 in. (152 mm) and 8.6 percent and 4 in. (102 mm) and 8.6 percent, respectively. The differences in initial and final setting times for the mixes FSET13 and FRSET13 were 48 and 25 min, respectively. The difference in initial and final setting times for the mixes FSET33 and FRSET33 were 55 and 34 min, respectively. The averages of the original and replicate mix results were taken as final values.

The slump and air content for the summer mixes HTSET13 and HTSET33 were 2.5 in. (64 mm) and 3.2 percent and 0 in. and 4 percent, respectively. The concrete temperature for these mixes was relatively high [84°F (28.9°C)] compared with the original and replicate mixes [70°F (21°C)]. The higher concrete temperature caused a considerable reduction in the slump, the air content, and the setting times.

Table 4. Setting time of concrete for original mixes.

Mix FSET13					Mix FSET33				
Elapsed Time (hr:min)	Air Temperature (°F)	RH (%)	Concrete Temperature (°F)	Penetration Resistance (psi)	Elapsed Time (hr:min)	Air Temperature (°F)	RH (%)	Concrete Temperature (°F)	Penetration Resistance (psi)
0:00	69	49	69	0	0:00	70	51	70	0
3:10	69	51	63	0	3:30	67	52	63	11
5:00	66	56	63	0	4:00	66	56	63	16
5:50	64	69	63	26	5:10	64	68	63	24
6:10	64	68	63	33	5:36	65	61	63	30
6:30	65	61	63	60	5:56	66	60	63	52
6:50	66	60	63	90	6:29	64	59	63	122
7:20	64	59	63	150	7:10	64	59	64	320
7:40	65	59	64	240	7:26	64	59	64	400
8:03	64	59	64	500	7:41	64	59	64	520
8:40	64	59	64	880	8:16	66	58	64	910
9:10	66	58	64	1,240	8:46	64	58	64	1,480
9:40	64	58	64	1,960	9:06	64	58	64	2,200
10:00	64	58	64	3,360	9:20	62	58	64	3,520
10:10	62	58	64	4,120	9:40	63	58	64	4,160

Notes: RH, relative humidity.
1 psi = 145 MPa.
1°F = (1°C ÷ 0.55) + 32.

Table 5. Setting time of concrete for replicate mixes.

Mix FRSET13					Mix FRSET33				
Elapsed Time (hr:min)	Air Temperature (°F)	RH (%)	Concrete Temperature (°F)	Penetration Resistance (psi)	Elapsed Time (hr:min)	Air Temperature (°F)	RH (%)	Concrete Temperature (°F)	Penetration Resistance (psi)
5:22	70	51	64.0	0	5:17	69	53	64	0
6:19	67	54	64.0	0	6:02	70	51	65	22
6:45	68	54	64.5	22	6:27	68	53	65	43
7:16	69	53	65.0	52	6:57	67	54	65	70
7:42	69	53	65.0	90	7:27	68	54	65	148
8:07	68	53	65.0	200	7:57	69	53	65	244
8:47	67	53	65.0	360	8:22	69	53	66	400
9:07	67	53	65.0	570	8:37	68	53	66	540
9:27	66	53	65.0	800	9:30	67	53	66	1,220
9:47	66	53	65.0	1,320	9:49	67	53	66	2,120
10:28	66	53	65.0	3,200	10:10	66	53	66	3,200
10:34	66	53	65.0	4,120	10:20	66	53	66	4,000

Notes: RH, relative humidity.
1 psi = 145 MPa.
1°F = (1°C ÷ 0.55) + 32.

Table 6. Setting time of concrete for summer mixes.

Mix HTSET13					Mix HTSET33				
Elapsed Time (hr:min)	Air Temperature (°F)	RH (%)	Concrete Temperature (°F)	Penetration Resistance (psi)	Elapsed Time (hr:min)	Air Temperature (°F)	RH (%)	Concrete Temperature (°F)	Penetration Resistance (psi)
2:00	83	40	81	40	1:05	83	40	80	0
2:25	85	40	80	50	1:25	85	40	80	30
2:45	86	38	81	81	1:45	86	38	81	60
3:15	85	37	82	103	2:00	86	38	81	100
3:45	87	36	83	178	2:15	86	37	82	184
4:15	88	34	84	610	2:45	87	36	83	340
4:30	89	33	86	790	3:15	88	34	84	740
4:50	90	33	87	1,960	3:30	89	33	84	1,110
5:05	90	33	87	3,600	3:45	90	33	85	1,420
5:15	90	33	87	4,440	4:00	90	33	85	2,560
					4:15	90	33	85	3,760
					4:25	90	33	85	4,320

Notes: RH, relative humidity.
1 psi = 145 MPa.
1°F = (1°C ÷ 0.55) + 32.

Table 7. Plastic properties of mixes used in setting-time study.

Mix No.	Superplasticizer Dosage (% by weight of cement)	AEA (% by weight of cement)	Water/Cement Ratio by Weight	After Mixing: Temperature (°F)	Slump (in.)	Unit Weight (lb/ft^3)	Air Content (%)	Vebe Time (sec)	Flow-Table Spread (in.)
FSET13	1.0	0.08	0.38	69	7.0	138.2	9.2	0	16.54
FSET33	1.2	0.18	0.28	70	3.0	140.8	8.6	2.0	11.81
FRSET13	1.0	0.08	0.38	71	6.0	140.3	8.6	0.5	14.57
FRSET33	1.2	0.18	0.28	72	4.0	141.2	8.6	2.0	14.17
HTSET13	1.0	0.08	0.38	83	2.5	150.15	3.2	3.8	11.02
HTSET33	1.2	0.18	0.28	84	0.0	150.18	4.0	7.6	7.87[a]

Notes: FR = replicate mixes.
1 in. = 25.4 mm.
$1^\circ F = (1^\circ C \div 0.55) + 32$.
$1\ lb/ft^3 = 16.03\ kg/m^3$.

[a]No flow of concrete; concrete crumbles.

Figure 1. Setting time of concrete for mix 13.

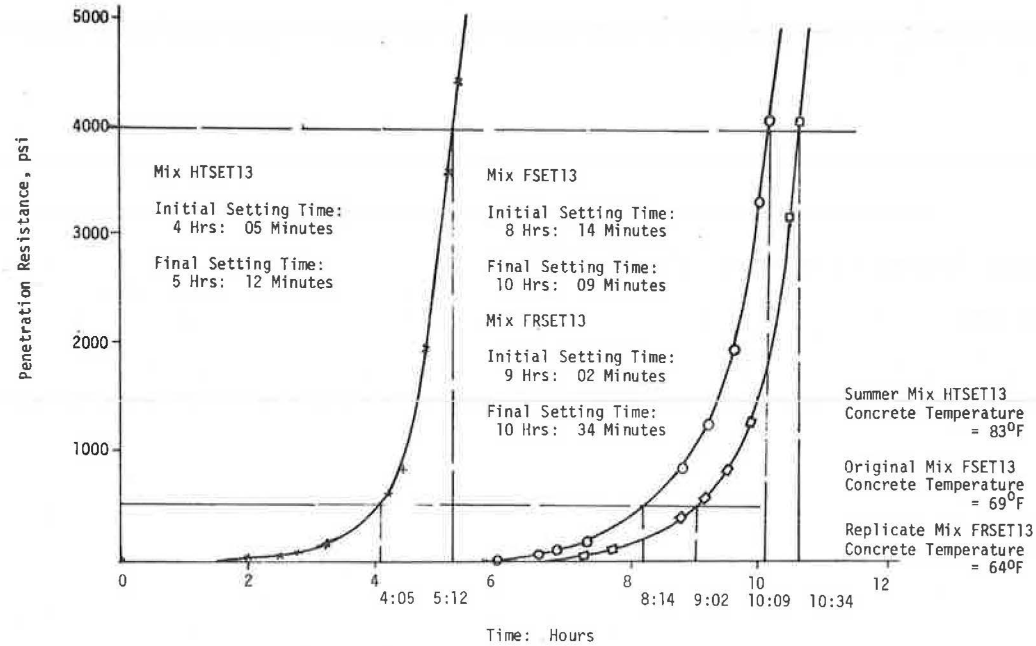


Figure 2. Setting time of concrete for mix 33.

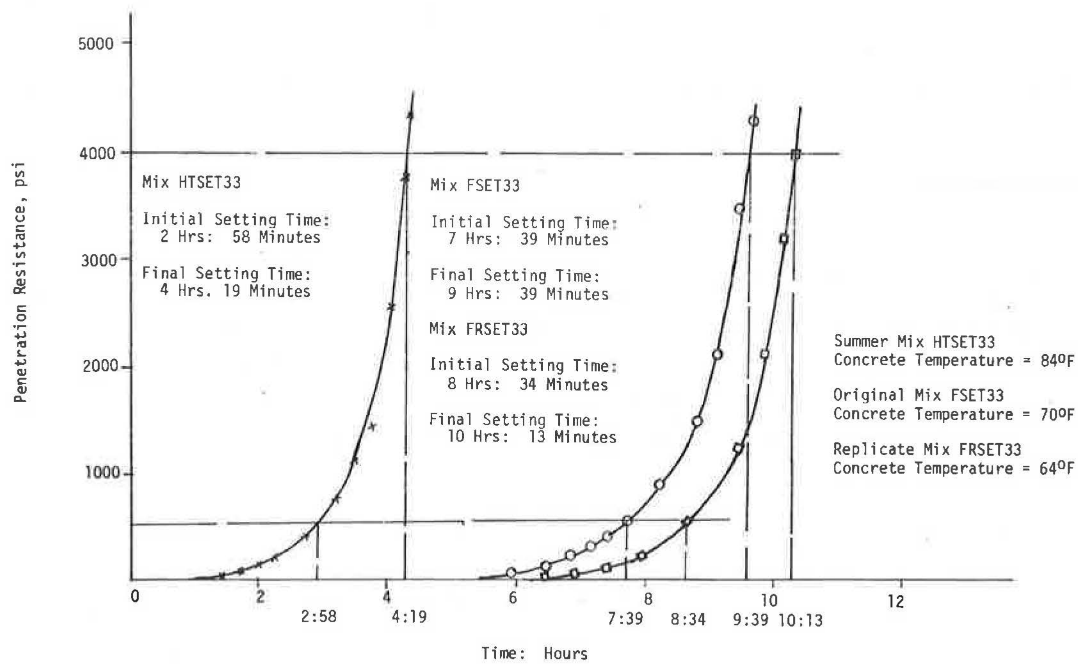


Figure 3. Compressive strength.

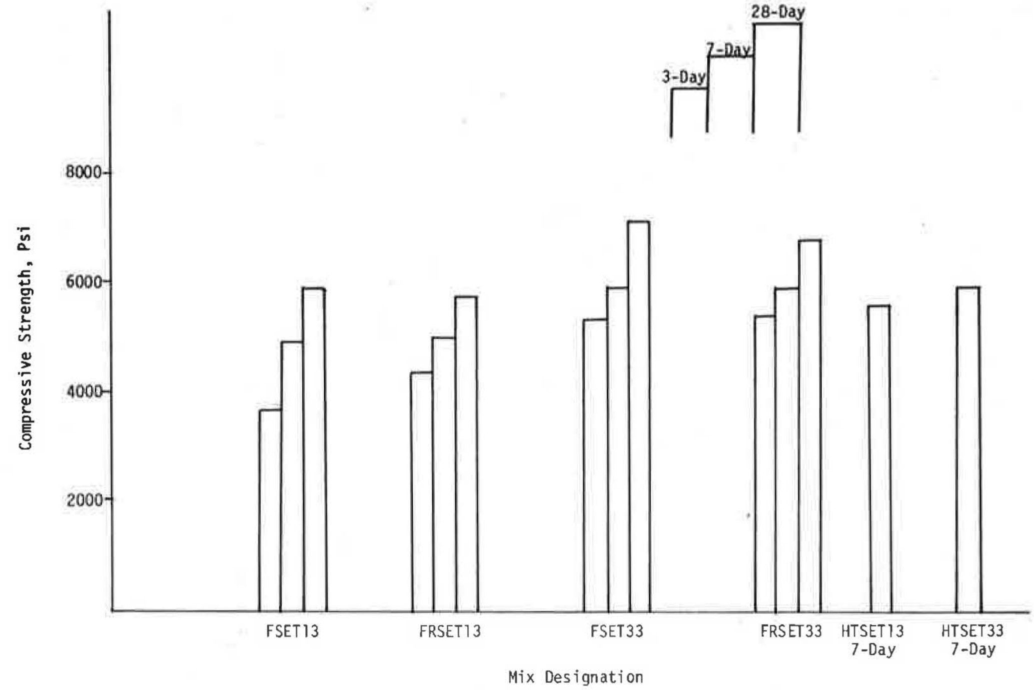


Table 8. Properties of fresh concrete mixes used in slump-loss study.

					After Initial Mixing							
					Temperature (°F)							
Mix No.	Superplasticizer	Dosage of Superplasticizer (% by weight of cement)	AEA Dosage (% by weight of cement)	Water/Cement Ratio	RT	CT	Slump (in.)	Unit Weight (lb/ft^3)	Air Content (%)	Vebe Time (sec)	Flow-Table Spread (in.)
SL33	Mighty RDI	1.2	0.18	0.28	65	72	5.125	148.41	6.4	2.0	12.96
SL13	Mighty RDI	1.0	0.08	0.38	65	70	9.000	133.80	14.4	0	20.82
SLR33	Mighty RDI	1.2	0.18	0.28	78	78	4.000	148.24	5.8	3.0	12.57
SLR13	Mighty RDI	1.0	0.08	0.38	67	70	9.250	136.71	10.4	0.0	16.89
HTSL13	Mighty RDI	1.0	0.08	0.38	84	84	6.750	144.15	8.0	0.5	14.14
HTSL33	Mighty RDI	1.2	0.18	0.28	91	88	0.000	150.84	3.8	7.2	7.86

Notes: RT, room temperature; CT, concrete temperature; AEA, air-entraining agent, vinsol resin (Protex).
1 in. = 25.4 mm.
°F = (°C ÷ 0.55) + 32.
1 lb/ft^3 = 16.03 kg/m^3.

Figure 4. Slump versus time for mix 13.

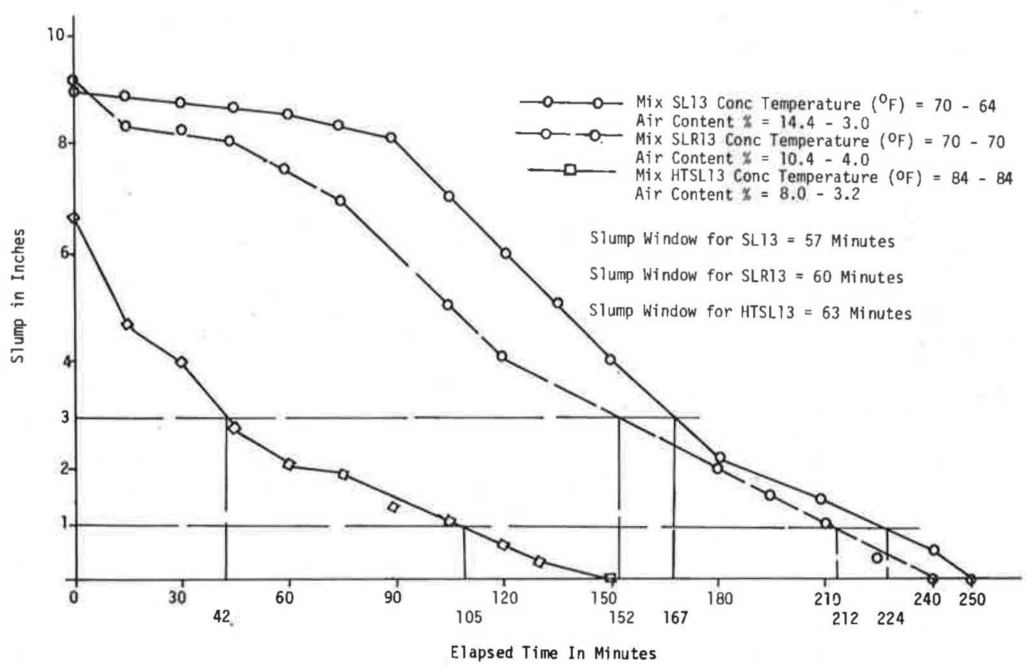


Figure 5. Slump versus time for mix 33.

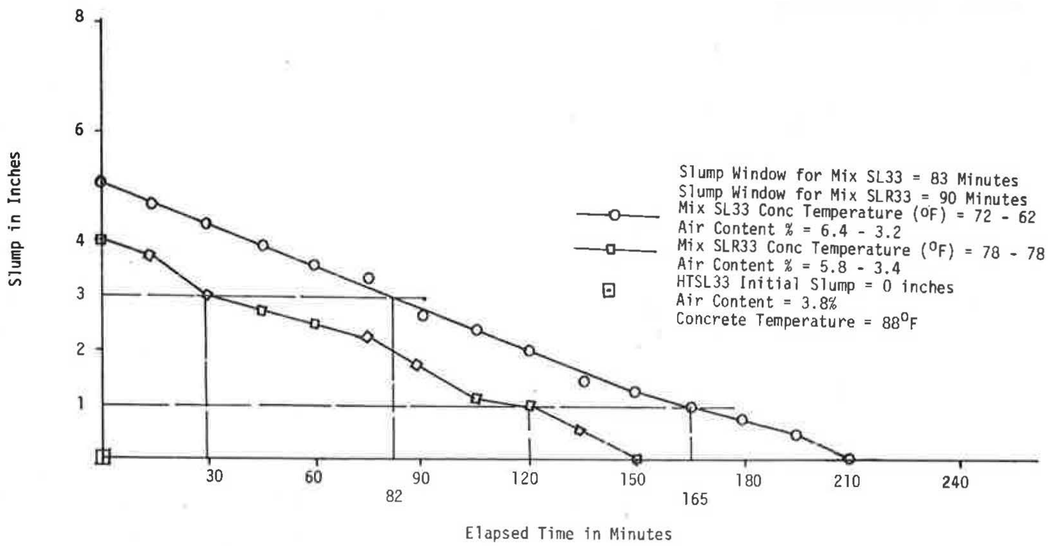
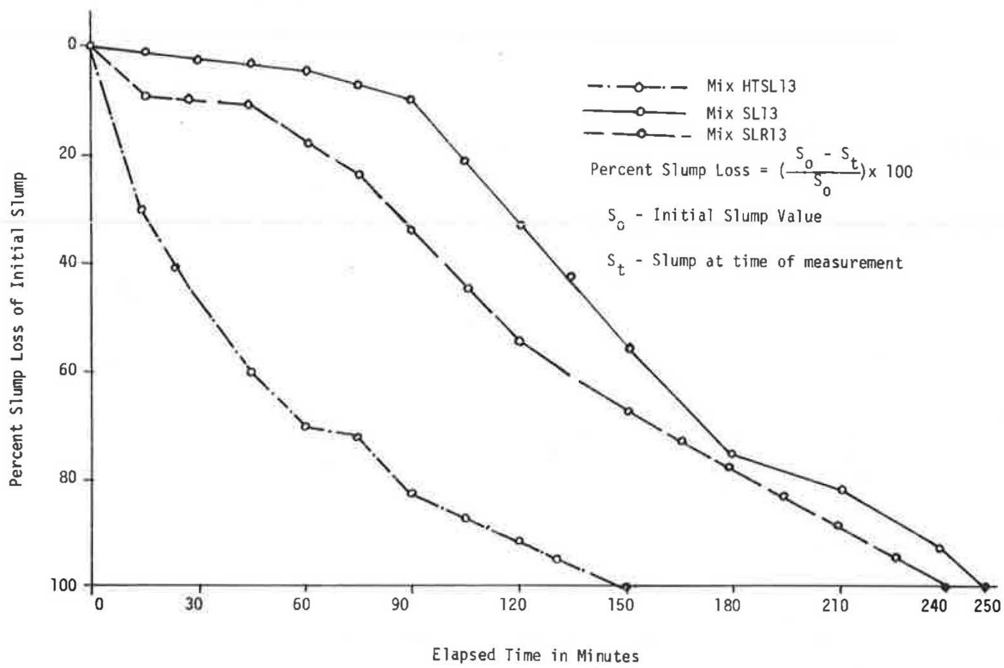


Figure 6. Percent slump loss versus time for mix 13.



Slump-Loss Study

During the spring (March) four mixes--SL13, SL33, and their replicates, SLR13 and SLR33--and during the summer two mixes--HTSL13 and HTSL33--were made for the slump-loss study. The mix proportions used are given in Table 8. Immediately after mixing, the concrete was transferred to a wheelbarrow and tests for slump, air content, flow-table spread, unit weight, and vebe time were made. The slump was measured at 15-min intervals. Before each slump measurement, the concrete was hand mixed. The concrete was kept covered in the wheelbarrow until the slump-loss study was over. The curves for slump loss with time are shown in Figures 4 and 5 for mixes 13 and 33, respectively. The initial slumps for mixes 13 and 33 were approximately 9 in. (228.6 mm) and 5 in. (127.0 mm), respectively. The initial slumps for mixes HTSL13 and HTSL33, which were made at a relatively higher temperature and lower humidity, were 6.75 in. (171.5 mm) and 0, respectively.

The initial slump for mix SLR13 was 9.25 in. (235 mm), corresponding to a concrete temperature of 70°F and relative humidity of 52 percent. The initial slump for mix SL13 was 9 in. (228.6 mm), corresponding to a concrete temperature of 70°F and relative humidity of 52 percent. The initial slump for mix HTSL13 was 6.75 in. (171.5 mm), corresponding to a concrete temperature of 84°F and relative humidity of 40 percent.

The effect of concrete temperature on initial slump loss was clearly seen in the case of mix 33. The initial slumps for mixes SL33 and SLR33 were 5.125 in. (130 mm) and 4 in. (102 mm), corresponding to concrete temperatures of 72°F (22.2°C) and 78°F (25.6°C) and relative humidities of 51 and 32 percent. The initial slump of mix HTSL33 was 0, corresponding to a concrete temperature of 88°F (31°C) and relative humidity of 33 percent. This clearly shows the effect of the temperature and humidity on the initial slump of concrete.

The mixes that were made at lower temperatures

Table 9. Slump, temperature of concrete, air content, and unit weight at various time intervals for mix SL13.

Cylinder-Casting Sequence	Time After Mixing (min)	Concrete Tempera-ture (°F)	Slump (in.)	Slump Loss (%)	Air Content (%)
1	0	70	9.000	0	14.4
	15	70	8.875	1.4	–
	30	67	8.750	2.8	9.8
	45	66	8.750	2.8	–
	60	66	8.625	4.2	–
	75	66	8.375	7.5	–
	90	66	8.125	9.2	7.2
	105	66	7.125	20.8	–
2	120	65	6.000	33.3	6.0
	135	65	5.125	43.1	–
	150	65	4.000	55.6	–
	165	65	2.375	73.6	–
3	180	65	2.250	75.0	4.4
	210	64	1.625	81.9	–
	240	64	0.500	94.4	–
4	250	64	0.000	100.0	3.0

Notes: Air temperature, 65°F; RH, 52 percent; and unit weight, 133.8 lb/ft³.
1 in. = 25.4 mm. 1°F = (1°C ÷ 0.55) + 32. 1 lb/ft³ = 16.03 kg/m³.

had higher initial air contents, whereas the mixes that were made at high temperatures had low initial air contents. The loss in initial air content due to higher temperatures was also a contributing factor to the reduced initial slumps at higher temperatures.

The loss in slump was not rapid at low temperatures for the mixes evaluated, as shown in Figures 4 and 5. The percent slump loss is shown in Figures 6 and 7. For mixes SL13 and SLR13 the time taken for 100 percent slump loss was about 4 hr, whereas for mixes SL33 and SLR33 the time taken for 100 percent slump loss was about 3 hr. For mix HTSL13 the time taken for 100 percent slump loss was about 2.5 hr.

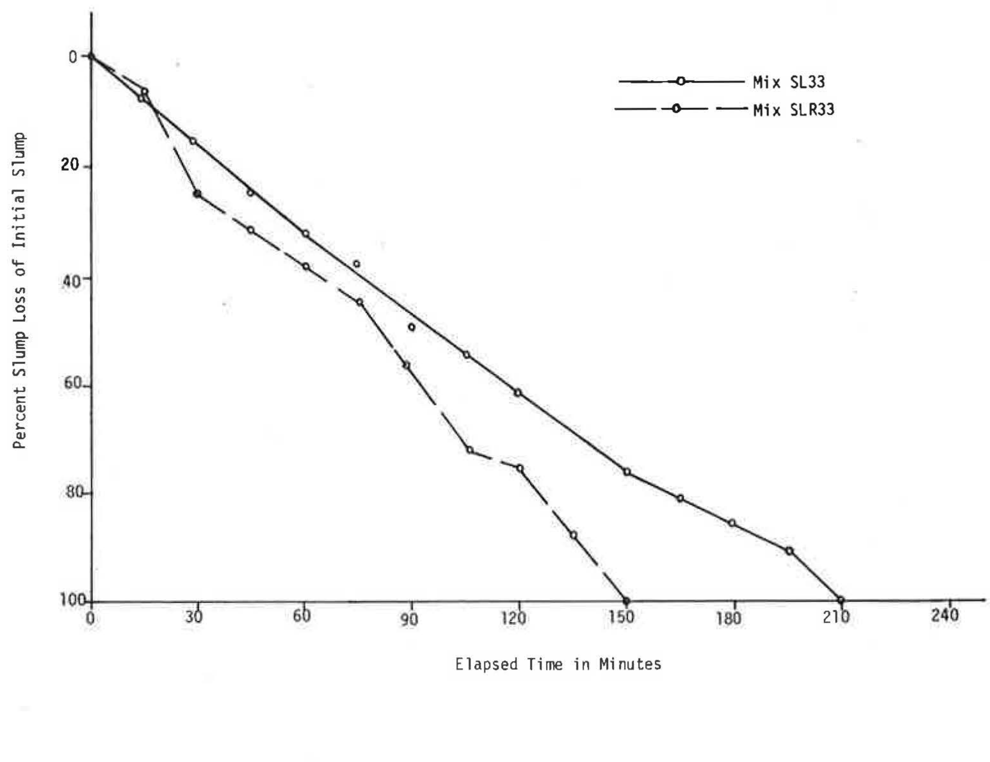
Room temperature and humidity affect the rate of loss of slump. For mixes SL13 and SLR13 the initial slump was the same, but the rate of loss of slump for SLR13 was higher compared with that for SL13 (Figure 6). The room temperature and concrete temperature for SLR13 were higher than those for SL13 (Tables 9 and 10). The same trend was observed for other mixes also (Tables 11-13). Similar findings of higher rate of slump loss at higher temperatures were reported by Mailvahanam (4).

Table 10. Slump, temperature of concrete, air content, and unit weight at various time intervals for mix SLR13.

Cylinder-Casting Sequence	Air Content (%)	Unit Weight (lb/ft³)	Time After Mixing (min)	Air Tempera-ture (°F)	Concrete Tempera-ture (°F)	Relative Humidity (%)	Slump (in.)	Slump Loss (%)
1	10.4	136.71	0	67	70	52	9.250	0.0
			15	68	71	52	8.375	9.5
			30	69	71	51	8.375	9.5
			45	69	71	50	8.125	12.2
2	7.0	145.43	60	70	70	49	7.500	18.9
			75	70	70	48	7.000	24.3
			90	70	69	44	5.375	41.9
			105	71	70	41	5.000	45.9
3	5.2	148.27	120	73	70	39	4.125	55.4
			135	73	70	39	3.750	59.5
			150	73	70	40	3.000	67.6
			165	73	70	40	2.500	73.0
4	4.0	151.19	180	73	70	40	2.000	78.4
			195	73	70	40	1.500	83.8
			210	73	70	40	1.000	89.2
			225	73	70	40	0.375	95.9
5		151.50	240	73	70	40	0.000	100.0

Notes: 1 in. = 25.4 mm. 1°F = (1°C ÷ 0.55) + 32. 1 lb/ft³ = 16.03 kg/m³.

Figure 7. Percent slump loss versus time for mix 33.



Although useful trends can be deduced from such visual examination of the slump-loss curves, more quantitative parameters are needed. Two such parameters that have been found useful are the slump window and the total working time (5). The slump window is defined as the time taken for the slump to decay from 3 in. (76.2 mm) to 1 in. (25.4 mm) and would be useful to those interested in slipform operations, where high slumps could not be tolerated.

Table 11. Slump, temperature of concrete, air content, and unit weight at various time intervals for mix SL33.

Cylinder-Casting Sequence	Time After Mixing (min)	Concrete Temperature (°F)	Slump (in.)	Slump Loss (%)	Air Content (%)
1	0	72.0	5.125	0	6.4
	15	71.5	4.750	7.3	5.8
2	30	71.0	4.375	14.6	5.3
	45	70.5	3.875	24.4	4.6
3	60	68.0	3.500	31.7	4.4
	75	67.0	3.250	36.6	4.2
	90	67.0	2.625	48.8	–
	105	67.0	2.375	53.7	–
	120	65.0	2.000	61.0	–
	135	64.0	1.375	73.2	–
	150	64.0	1.250	75.6	–
	165	64.0	1.000	80.5	–
	180	63.0	0.750	85.4	–
	195	62.0	0.500	90.2	–
4	210	62.0	0.000	100.0	3.2

Notes: Air temperature, 65°F; RH, 51 percent; and unit weight, 148.41 lb/ft³.
1 in. = 25.4 mm.
1°F = (1°C ÷ 0.55) + 32.
1 lb/ft³ = 16.03 kg/m³.

The total working time is defined as the time needed for the slump to go from the initial value to 1 in. These two parameters are plotted in Figures 4 and 5, respectively, for mixes 13 and 33.

The total working time for mix 13 was 218 min (average of mixes SL13 and SLR13) and for mix 33, 143 min (average of mixes SL33 and SLR33). The total working time for mix HTSL13 was 105 min. These high total working times are possible because of the use of superplasticizer with a retarder.

Figures 4 and 5 show that for higher initial slump, the rate of slump loss is higher. Mix 13 took 240 min to go to zero slump from a high initial slump of 9 in. (228.6 mm), whereas mix 33 took approximately 180 min to go to zero slump from an initial high slump of 5 in. (127.0 mm). Similar findings were reported by Whiting (5) and Ramakrishnan, Coyle, and Pande (1).

The curves for loss of air content with time are shown in Figure 8. The higher the initial air content is, the higher is the rate of air-content loss. Mixes SL13 and SLR13 had high initial air contents of more than 10 percent when tested immediately after mixing and had about 3 percent air content when tested at zero slump. Mixes SL33 and SLR33 had initial air contents of about 6 percent and final air contents of about 3 percent at zero slump.

Properties of Hardened Concrete

The 7-day compressive strengths of cylinders cast at different time intervals after mixing are plotted in Figure 9. The compressive strength and dry unit weights were higher for cylinders that were cast long after mixing when compared with the compressive

Table 12. Slump, temperature of concrete, air content, and unit weight at various time intervals for mix SLR33.

Cylinder-Casting Sequence	Air Content (%)	Unit Weight (lb/ft³)	Time After Mixing (min)	Air Temperature (°F)	Concrete Temperature (°F)	Relative Humidity (%)	Slump (%)	Slump Loss (%)
1	5.8	148.24	0	78	78	32	4.000	0
			15	80	76	29	3.750	6.3
2	5.2		30	80	76	28	3.000	25.0
			45	80	78	29	2.750	31.3
	4.2	149.65	60	80	77	31	2.500	37.5
			75	80	77	32	2.250	43.8
			90	80	77	32	1.750	56.3
			105	80	78	29	1.125	71.9
3	4.0	151.50	120	80	78	28	1.000	75.0
			135	81	78	27	0.500	87.5
4	3.4	151.81	150	82	78	26	0.000	100.0

Notes: 1 in. = 25.4 mm.
1°F = (1°C ÷ 0.55) + 32.
1 lb/ft³ = 16.03 kg/m³.

Table 13. Slump, temperature of concrete, air content, and unit weight at various time intervals for mix HTSL13.

Cylinder-Casting Sequence	Air Content (%)	Unit Weight (lb/ft³)	Time After Mixing (min)	Air Temperature (°F)	Concrete Temperature (°F)	RH (%)	Slump (in.)	Slump Loss (%)
1	8.0	144.15	0	84	84	40	6.750	0.0
			15	86	85	37	4.750	29.6
2	5.0	146.50	30	86	85	37	4.000	40.7
			45	86	85	37	2.750	59.3
			60	87	84	36	2.063	69.4
			75	87	84	35	1.075	72.2
3	4.2	150.49	90	88	84	35	1.125	83.3
			105	88	84	34	0.875	87.0
			120	89	83	33	0.563	91.7
			130	90	84	33	0.313	95.4
4	3.2	151.75	150	90	84	33	0.000	100.0

Notes: RH, relative humidity.
1°F = (1°C ÷ 0.55) + 32.
1 in. = 25.4 mm.

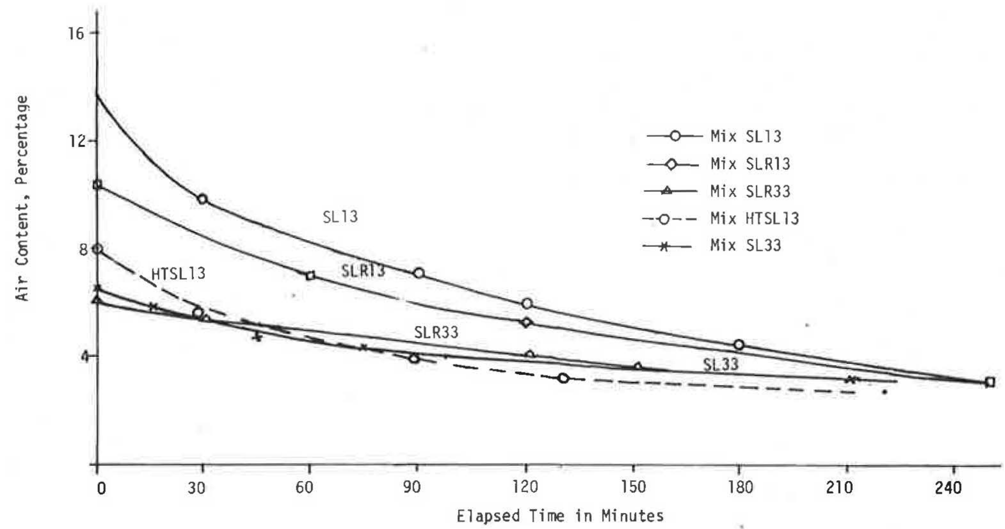


Figure 8. Air content versus time.

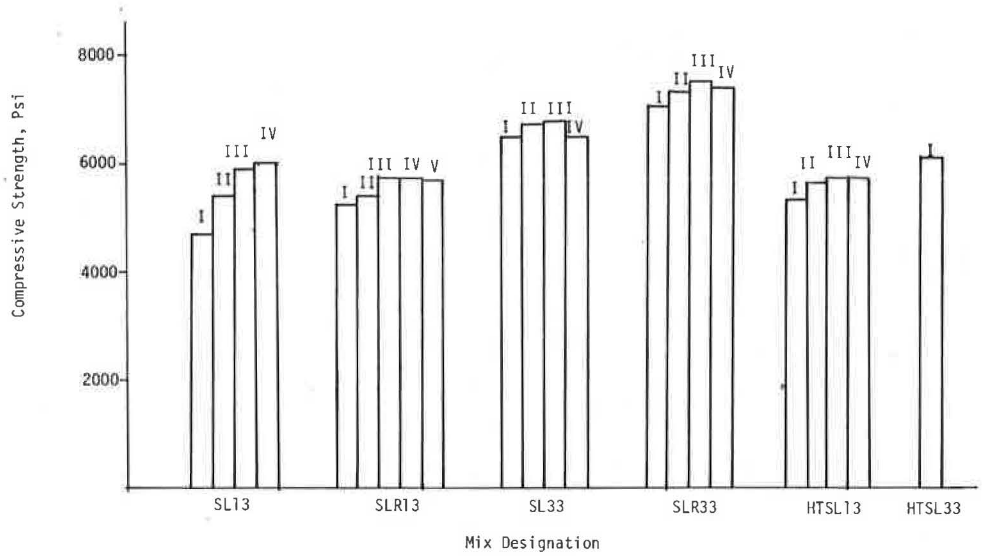


Figure 9. Seven-day compressive strength.

strengths of cylinders that were cast immediately after mixing. The compressive strengths and unit weights increased as the time after initial mixing increased. This trend was observed for all the mixes. The gain in compressive strength and unit weight can be attributed to the loss in air content with time. In contrast to this, the cylinders that were cast when the slump was zero show a drop in compressive strength compared with the compressive strength of previously made cylinders. This can be attributed to the poor workability of concrete at zero slump, because all the cylinders were hand compacted.

The increase in compressive strength from initial casting to final casting is about 15 percent for mix 13 and 5 percent for mix 33.

CONCLUSIONS

Based on the analysis of experimental results, the following conclusions are drawn:

1. Due to a 16°F increase in the initial concrete temperature and 10 percent reduction in the relative humidity, the initial setting time was reduced by 53 and 63 percent, respectively, for medium-cement-content and high-cement-content concretes; the corresponding reductions in the final setting times were 50 and 57 percent.

2. The rate of slump loss is low at low temperatures and the workability is maintained for several hours after mixing for the concrete mixes that were made at relatively low temperatures. The mixes with a water/cement ratio of 0.38 and 1.0 percent superplasticizer dosage maintained their workable slumps for 3 hr at low temperatures, whereas the mixes with a ratio of 0.28 and 1.2 superplasticizer dosage maintained their workable slumps for 2 hr.

3. The slump loss is proportional to the initial slump level for all the mixes; the higher the initial slump is, the higher the slump loss. The total time span to which concrete could be kept workable, however, is more for a concrete with a higher initial slump.

4. The air-content loss is proportional to the initial level of air content for all the mixes; the higher the initial air content is, the higher is the air-content loss.

5. Room temperature and humidity have a great influence on the level of initial slump as well as on the rate of loss of slump. The higher the room temperature is and the lower the relative humidity

is, the lower is the initial slump and the higher is the rate of loss of slump.

6. The compressive strength and the dry unit weight are higher for cylinders that were cast long after the mixing compared with the cylinders that were cast immediately after mixing. The gain in compressive strengths and dry unit weights can be attributed to the loss in air content with time.

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Effects of High Temperatures on the Properties of Fresh Concrete

M. SAMARAI, S. POPOVICS, AND V.M. MALHOTRA

The effects of high temperature on the properties of fresh concrete and the mechanism of the setting of cement paste and concrete are described. The undesirable effects of high temperature on fresh concrete mentioned include increased water demand, increased rate of slump loss, increased rate of setting, and increased tendency for plastic shrinkage cracking. This is followed by a discussion of the methods to minimize the above effects; the roles of water-reducing and set-retarding admixtures, superplasticizers, and retempering of concrete are described.

The effects of high temperatures (that is, a hot climate) on the properties of fresh concrete are usually undesirable from the standpoint of construction. Frequently occurring phenomena such as accelerated slump loss or increased possibility of excessive moisture loss and the resulting shrinkage cause extra problems for the construction engineer. Such problems can be eliminated only by careful and sometimes expensive preventive measures.

The current status of the knowledge is still incomplete concerning the effects of high temperatures on the behavior of fresh cement paste or concrete. Some aspects have been published, such as those related to setting or effects of admixtures (1,2). Nevertheless, the seriousness of the problems of construction in tropical climates makes it worthwhile to provide additional details in a systematic fashion. In addition, the emphasis in this paper is on new aspects of the temperature effects.

EFFECTS OF TEMPERATURE ON HYDRATION OF CEMENT

Definitions

When portland cement is mixed with a limited amount of water, the cement particles become dispersed in the water. The result is cement paste, which is a material of considerable plasticity.

The setting and hardening processes are the results of a series of simultaneous and consecutive reactions between water and the constituents of portland cement. These reactions are described as the hydration of portland cement. The hydration of cement compounds is exothermic. The heat developed is called the heat of hydration.

Reactions in Early Hydration and Setting

The measurement of heat evolution is particularly suitable for the investigation of the early stages of hydration (3). During a short period beginning when portland cement and water are first brought into contact at room temperature and during the time of mixing, relatively rapid chemical reactions occur, primarily between the water and the tricalcium aluminate (C_3A) of the cement.

When portland cement is insufficiently retarded, the time of initial setting is considerably less than 1 hr at normal temperature.

Another factor that can cause rapid setting is elevated temperature. The higher the curing temperature is, the faster are the reactions between cement and water, and consequently the shorter becomes the setting time. The effects of curing temperature on the intensity (rate) of hydration can be seen in Figures 1 through 4 (4;5, pp. 1-32; 6, pp. 259-273). It can also be seen from Figure 1 that a change appears in the hydration process at a temper-

Figure 1. Hydration of tricalcium silicate: effect of temperature.

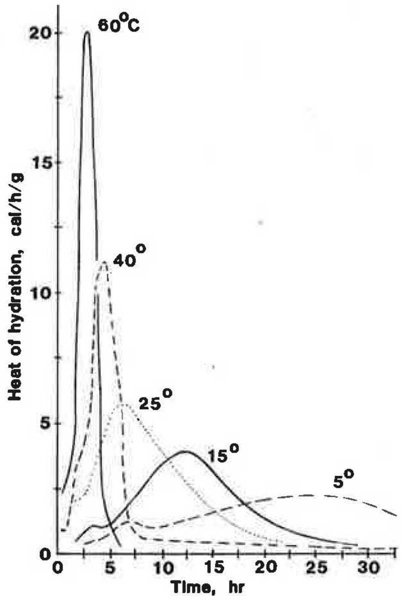


Figure 2. Initial rate of cement hydration as measured by the rate of heat development as a function of curing temperature.

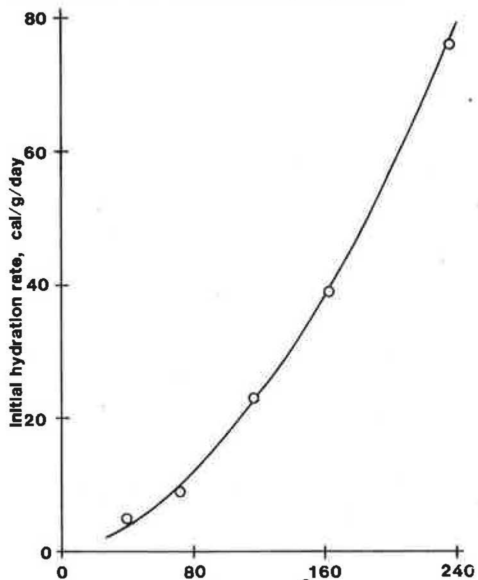


Figure 3. Rate of heat of hydration of three portland cements of different C_3A contents at 104°F (40°C): 1, reactions of silicate phase; 2, reactions of aluminate phase (A, 12.5 percent C_3A; R, 7.6 percent C_3A; E, 3.7 percent C_3A).

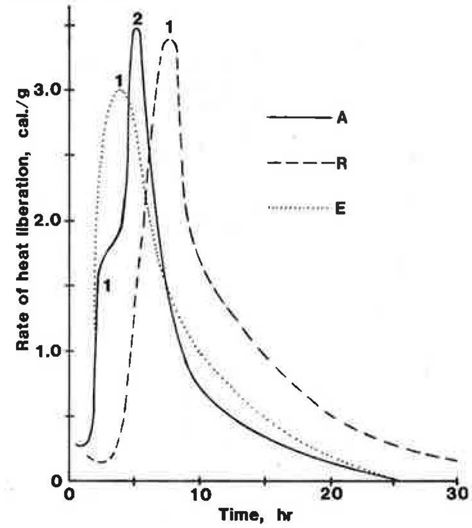
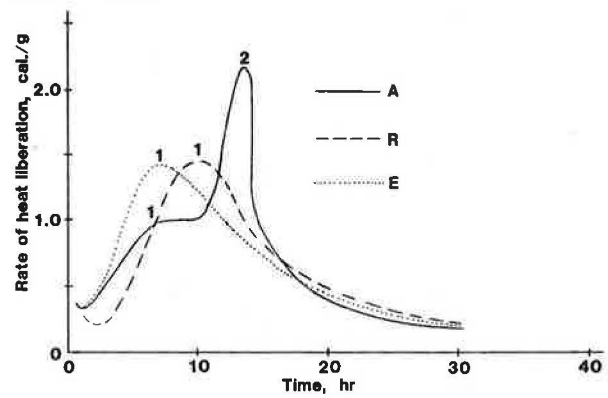


Figure 4. Rate of heat of hydration at 77°F (25°C) of three portland cements shown in Figure 3.



ature of about 95°F (35°C), which is manifested in a decrease in the apparent activation energy from 8.45 to 5.70 kcal/mole (4).

Methods for counterbalancing the acceleration of setting are discussed in the following sections.

SETTING OF CEMENT PASTE AND CONCRETE

Definitions

When cement is mixed with approximately 20 to 35 percent of water, the result is a paste. This mixture displays considerable plasticity that is maintained for a period of time called the dormant period. After a while, however, the paste starts to stiffen, less and less plasticity can be observed, and finally all the plasticity is gone and the paste becomes brittle, although still without any great strength. This stiffening process is called setting and is the result of a series of reactions between the cement and the water, which was discussed earlier. One should recognize that the stiffening is not a drying process; it takes place even if the fresh cement paste is kept under water. The gain of strength--that is, the hardening process--takes place subsequent to the setting.

It is customary to refer to initial setting, which is basically the beginning of the stiffening, and final setting, which is marked by the disappearance of plasticity. The setting process should not start too early because the freshly mixed concrete should remain in a plastic condition for a sufficient period to permit satisfactory compaction and finishing after transportation and placement. On the other hand, too long a setting process is also undesirable because this would cause a useless delay in the strength development after the finishing. Of the two concepts the initial setting has far greater significance.

There is no strict dividing line between setting and hardening. Any distinction is more or less arbitrary and poorly defined. So are the terms initial setting and final setting. Nevertheless, for practical purposes, it is convenient to have test methods for the approximate determination of the time when the stiffening starts and when the plasticity is gone.

The time of set of a paste or concrete is usually determined by measuring repeatedly the changes in its resistance to penetration by specified small rods or needles, although slump, heat development, pulse velocity, shearing test, and electrical resistivity measurements are also applicable for this purpose (1,2). There are two standard penetration methods for portland-cement pastes in the United States. One applies the Vicat apparatus (ASTM C 191-74), the other the Gillmore needles (ASTM C 266-74). The Vicat apparatus is primarily for laboratory use, whereas the Gillmore needles are for field tests. For blended portland cement only the Vicat method is specified.

ASTM C 403-70 provides a procedure for determining the time of setting of concrete with slump greater than zero by measuring the penetration resistance of the mortar that is sieved from the concrete mixture by specified rods. The penetration resistance is calculated as the force required to cause a 1-in. (2.54-cm) depth of penetration of the needle divided by the area of the bearing face of the needle. Time of initial setting is defined as the time elapsed after initial contact of cement and water that is required for the mortar to reach a penetration resistance of 500 psi (3.45 MPa). Time of final setting is the elapsed time required for the mortar to reach a penetration resistance of 4,000 psi (27.6 MPa).

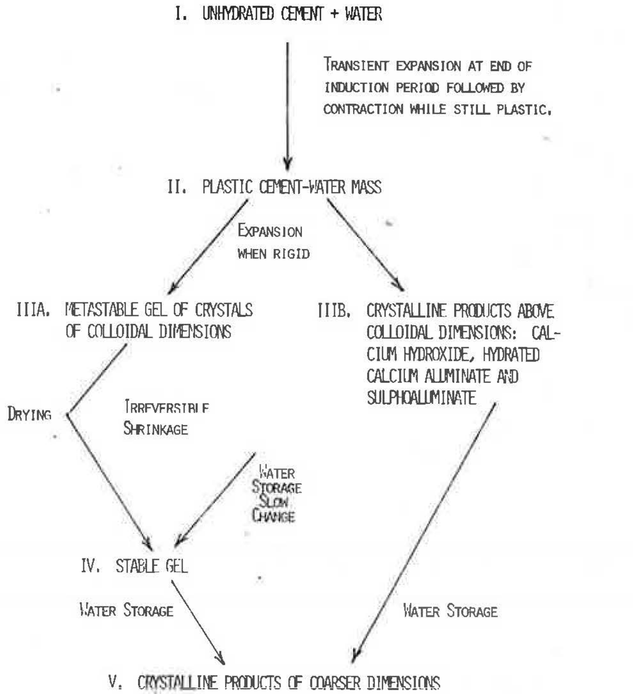
Mechanism of Setting

Lea summarizes the setting mechanism as follows (7). The cement grains are acted on by water to form a supersaturated solution from which the gel-like mass of crystals precipitates. Diffusion of water molecules to the surface, or even into the crystal lattice, to react in situ must also play a part, at least in the later stages of hydration. While still in a plastic condition, the cement paste shrinks slightly because there is a contraction in volume of the system of cement plus water on hydration. Once the mass becomes rigid, a small expansion sets in because the gel mass deposits around the cement grains and causes them to swell and to exert an outward pressure. The quality of the gel mass increases progressively with time, and it spreads into the intergranular spaces.

The cement gel must be regarded as being formed initially in an unstable condition; the fiber crystals of the gel particles, or the gel particles themselves, are farther apart and occupy a greater volume and enclose more water than in their stable state. The gel thus has an inherent tendency to shrink and give off some of the water it contains. In water no measurable contraction occurs, but in any case it would be offset by the effects of continued hydration of previously unattacked cement. During drying, the set cement undergoes an irreversible contraction and reduction in water content as the gel changes into its more stable form. The expansion during subsequent wetting and the amount of water taken up then become reversible and are reproduced in successive cycles of wetting and drying. During prolonged aging under wet conditions, a further slow change may occur by crystal growth. There is, however, a reduction in the surface area with aging, indicating that irreversible changes toward a more stable state are occurring.

The setting, hardening, and aging processes are shown in Figure 5 (7).

Figure 5. Diagram of setting process of portland-cement paste.



The changes from I to IIIA and IIIB (Figure 5) are part of the setting process. The change from stage IIIA to IV occurs when the set cement is allowed to dry in air, but when stored under water, the cement persists longer in stage III although a slow change during aging in water from stage IIIA to IV occurs. The final change from stage IV to V appears to be exceedingly slow and does not appear to occur in mortars or concretes under normal conditions.

Setting of Cement Paste

The most conspicuous change that takes place in the cement-and-water mixture is the gradual stiffening. The stiffening of a cement paste can be demonstrated quantitatively by the decreasing penetration depths of the needle of the standard Vicat apparatus as a function of elapsed time. Typical results obtained with a type III cement are shown in Figure 6 (8).

Setting of Concrete

Results of three series of the standard penetration test (ASTM C 403) obtained on concrete are plotted in the log-log system in Figure 7 (data from Master Builders Laboratory) (8). It can be seen that the three different kinds of concrete provided a common slope with a good approximation within resistance limits of 300 psi (2.07 MPa) and 4,000 psi. The 4,000-psi resistance corresponds to an approximately 100-psi compressive strength (8).

Further analysis of published data (9, pp. 97-117) reveals that the cement brand, curing temperature, water/cement ratio, and certain admixtures do not significantly influence either the straight-line approximation or the slope, whereas other admixtures, particularly certain retarders and water-reducing admixtures, may increase the slope (10, pp. 195-221). When this happens, it indicates that the early stiffening (initial set) is retarded relatively more than the concluding part of the setting (final set); thus the stiffening starts later but

Figure 6. Typical curves of Vicat penetration for cement pastes setting at room temperature.

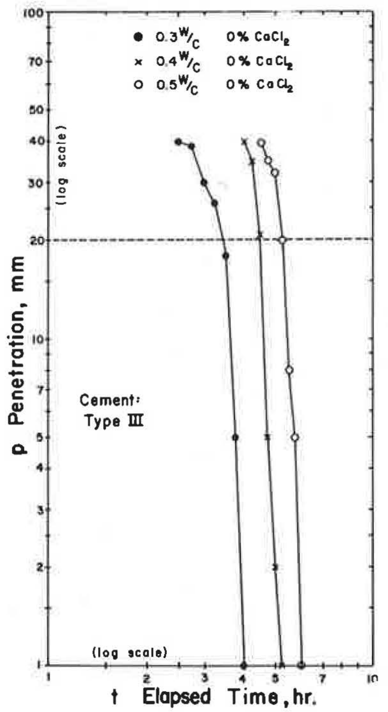
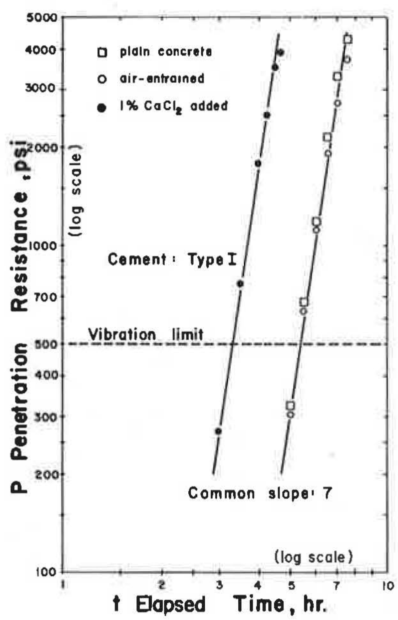


Figure 7. Typical curves of standard penetration for mortars sieved from concretes and setting at room temperature.



the rate of stiffening is higher and the process is more intensive.

Consecutive repetitions of a consistency test on a concrete sample can also be used for the characterization of the stiffening process. An example is shown in Figure 8 (8) in which several typical results of the standard slump test (ASTM C 143) are plotted against the elapsed time [curing temperature, 90 to 95°F (32.2 to 35°C); cement content, 520 lb/yd^3 (310 kg/m^3)]. In these tests the concrete was not remixed after each slump test. As

Figure 8. Slump loss of concrete with and without a given water-reducing admixture as a function of elapsed time.

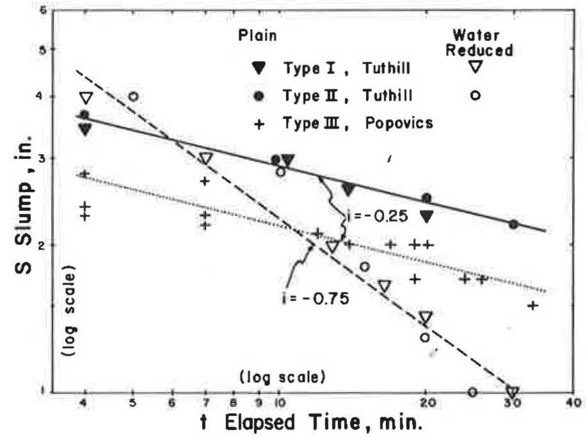
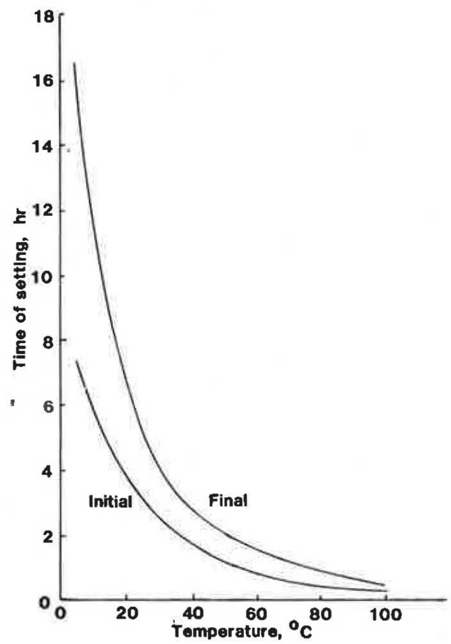


Figure 9. Effect of temperature on initial and final sets of portland-cement mortar (mix proportion, 1:3 by weight).



shown in Figure 8, standard slump results in general form straight lines in the log slump versus log (elapsed-time) system. The degree of approximation shown is fairly good, considering the usual fluctuation of the slump results.

Effect of Temperature on Setting

The standard initial set of commercial portland cements ranges from approximately 2 to 4 hr and the final set from 5 to 8 hr. In concrete mixtures the set usually occurs later because of the higher water/cement ratios in concretes. Under special conditions, however, such as construction at high temperature, a concrete may set early. This accelerating effect of elevated temperatures is demonstrated in Figure 9 (4) not only qualitatively but also quantitatively. In this figure it is shown that an increase in temperature from 59 to 86°F (15 to 30°C) reduces the time of initial set by approximately half. Further details concerning the effects

Table 1. Effects of admixtures on setting at various temperatures: type I cement DI.

Mix	Admixture	Sugars in Admixture	Dosage of Admixture per Sack (lb)	Reduction in Water[a] (%)	Time (hr) to Penetration Resistance of 500 psi	Time (hr) to Penetration Resistance of 4,000 psi	Compressive Strength (psi)	Air Content: Air (%)	Air Content: Air-Entraining Agent (ml/yd³)	Air Content: Air-Detraining Agent (ml/yd³)
Initial Concrete Temperature 50° to 55°F										
R50										
1	None	–	–	–	7.2	12.6	64 at 16 hr	4.0	58	0
2	SO	High	1/4	8.3	10.1	13.8	29 at 16 hr	4.6	0	0
3	DP	Low	1/4	8.3	9.2	13.0	57 at 16 hr	4.8	0	0
4	OP	Low	1/4	9.5	8.5	12.7	57 at 16 hr	4.9	0	0
5	RW	Low	b	4.1	7.2	9.8	115 at 16 hr	3.9	0	0
9	None	–	–	–	8.2	12.5	32 at 12 hr	3.4	37	0
10	QA	High	1/8	4.7	8.5	12.6	20 at 12 hr	3.4	0	0
11	QA	High	1/4	7.1	10.2	15.3	15 at 12 hr	3.8	0	2.0
12	QA	High	3/8	7.9	13.2	20.4	6 at 12 hr	3.5	0	3.0
Initial Concrete Temperature 70° to 75°F										
R										
1	None	–	–	–	4.3	6.9	530 at 16 hr	3.6	80	0
5	SO	High	1/4	6.7	6.4	8.8	527 at 16 hr	3.8	0	0
4	DP	Low	1/4	9.0	5.8	8.0	584 at 16 hr	4.6	0	0
2	OP	Low	1/4	8.6	5.8	7.9	623 at 16 hr	3.6	0	0
3	RW	Low	b	3.5	4.4	6.2	760 at 16 hr	3.6	26	0
13	None	–	–	–	5.3	7.8	172 at 16 hr	3.5	55	0
14	QA	High	1/8	3.5	5.4	7.7	200 at 16 hr	3.5	10	0
15	QA	High	1/4	7.8	6.3	8.8	192 at 16 hr	3.4	0	1.0
16	QA	High	3/8	7.4	7.7	10.5	135 at 16 hr	3.5	0	2.0
Initial Concrete Temperature 90° to 95°F										
R90										
1	None	–	–	–	3.3	4.8	541 at 10 hr	3.4	88	0
2	SO	High	1/4	9.5	5.3	7.0	480 at 10 hr	4.0	21	0
3	DP	Low	1/4	8.3	5.0	6.7	566 at 10 hr	4.7	0	0
4	OP	Low	1/4	7.1	4.9	6.6	462 at 10 hr	3.7	16	0
5	RW	Low	b	3.6	3.7	5.0	595 at 10 hr	3.4	29	0
12	None	–	–	–	4.4	6.0	441 at 12 hr	3.0	60	0
13	QA	High	1/4	5.3	5.7	7.4	520 at 12 hr	3.5	0	0
14	QA	High	3/8	7.9	7.9	9.5	376 at 12 hr	3.3	0	1.0
15	QA	High	1/2	7.6	8.8	10.8	262 at 12 hr	3.8	0	2.0

Notes: 1 lb = 0.45 kg, 1 psi = 145 MPa, $t°F = (t°C \div 0.55) + 32$. DI is the identifying mark of the cement; SO, DP, and so on, are the identifying marks of the admixtures used.
[a]The plain concrete used as a basis of comparison is in all cases air-entrained concrete. The water reduction was computed after adjusting all mixes to the same slump and air content.
[b]Dosage was 6 fluid oz.

of temperature on setting may be found in the literature (1,2).

Cement stored in sealed containers at ordinary temperatures is usually little changed after long periods, but at high temperatures some effect is produced, because although the cement appears normal, it may rapidly develop a flash set if subsequently exposed to air. Trouble with rapid setting is occasionally experienced with cements shipped from temperate to tropical climates. Development of a flash set also occurs occasionally in cement stored in bulk (7).

Special cements may require different criteria for the evaluation of the setting process. For instance, setting of oil-well cements is usually measured in terms of change in viscosity at elevated temperature.

Effects of Retarders and Plasticizers

One way to counteract the rapid setting caused by elevated temperature is to use a set-retarding admixture, or retarder, in concrete. The effects of such an admixture depend on a number of factors, including the temperature. These factors may interfere with one another, changing the effectiveness of the admixture. For instance, use of retarders and water-reducing admixtures, including superplasticizers, may make the slope of the consistency line steeper, the degree of which depends on the type and quantity of the admixture employed as well as on the SO_3, C_3A, alkali, and free lime contents of the cement. This reflects the usual experience that certain admixtures increase the time of initial setting or reduce the amount of water needed for a given initial slump or both, but such concretes may stiffen faster, sometimes too fast even for a cement and an admixture that separately meet all specifications. This happens whenever the composition of the cement and that of the admixture are incompatible either because the readily soluble SO_3 content in the cement is lower or because it is high relative to the C_3A content.

Despite the availability of remedial actions, it is better to check the compatibility of the job cement and the job admixture before the construction starts. A simplified test method is ASTM C 359 for the determination of false set in Ottawa-sand mortar. A more complete test can be made from trial concrete mixes with the job materials in the proper amounts as described in ASTM C 494.

The effects of admixtures on delaying the time of setting are shown by the data in Table 1 (9) and in Figures 10 and 11 (9) (cement type I). The cement content for Figure 11 was 470 lb/yd³ (280 kg/m³).

Expanding or shrinkage-compensating cements may also provide excessive slump loss when combined with certain water-reducing admixtures. In one series of experiments the use of type D lignins of ASTM C 494 proved to be the most effective with type K cement (11). Further details concerning the effects of admixtures on setting may be found in the literature (1,2).

Figure 10. Concretes of similar vibration limit made at various temperatures by varying dosages of set-retarding admixture.

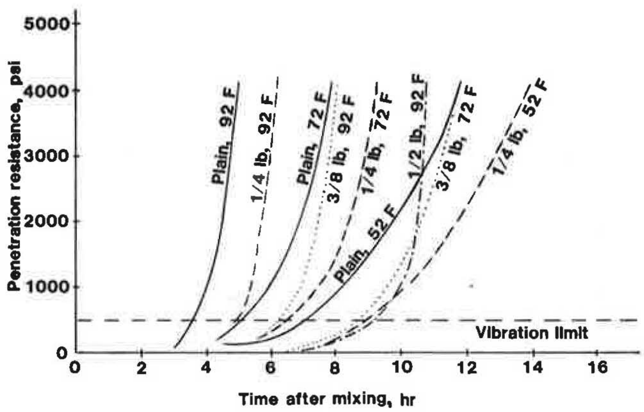
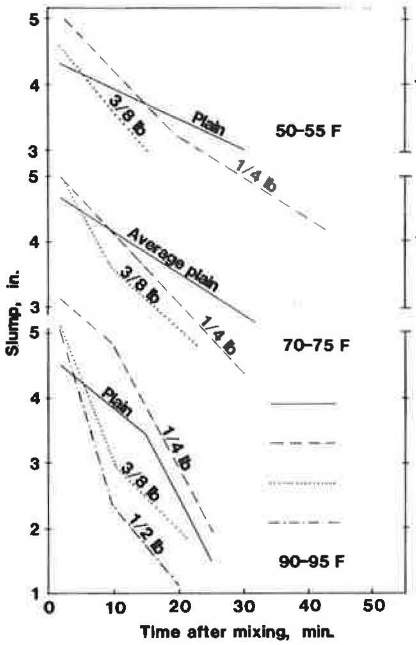


Figure 11. Slump loss as affected by temperature and dosage of retarder.



MECHANISM OF CHEMICAL RETARDATION OF SETTING TIME

Gypsum for Retardation of Setting

Powders of cement clinkers, especially those with a high Fe_2O_3:Al_2O_3 ratio, show a rapid but not a flash set, yet the final set may be unduly slow. This undesirable situation can be eliminated by the addition of gypsum ($CaSO_4 \cdot 2H_2O$). Gypsum retards the intitial set properly and speeds up the final set.

Any system that contains sufficient retarder to be on the plateau for time of set is properly retarded. Such a system is properly retarded as far as control of flash set is concerned, but there are other factors that must be considered. Many physical properties of hardened paste are affected by the quantity of gypsum that is present in the cement. Four of the most important of these are (a) compressive strength, (b) contraction on drying, (c) delayed expansion, and (d) heat liberation.

Lerch used this last property for the definition of properly retarded cement (12). In his opinion, a properly retarded cement can be considered one that contains the minimum quantity of gypsum required to give a curve that shows two cycles of ascending and descending rates of heat liberation and shows no appreciable change with larger additions of gypsum during the first 30 hr of hydration. The quantity of gypsum determined in this way is essentially the same as the quantity of gypsum required for maximum strength and minimum shrinkage, and it avoids abnormal expansion. Thus it can be called optimum.

Unfortunately, it is not clear whether the optimum defined in this way is also optimum from the standpoint of adequate retardation at higher curing temperatures. It has been observed, however, that an excess of gypsum above the traditional optimum amount may produce unfavorable results such as lower strength and delayed expansion (13) on the cement properties when the cement is cured at standard temperature.

The influence of various clinker properties on the optimum gypsum content can readily be seen by examining the rate of heat-liberation curves. The specific surface of the cement and the alkali content of the clinker, as well as the C_3A content of the clinker, affect the amount of gypsum required for proper retardation. In any case, the basic rule appears to be that the maximum (optimum) amount of gypsum permissible in the cement should be such that no or little unreached gypsum remains in the specimen of paste at 24 hr. This rule may help develop an optimum gypsum content specifically for cement to be used for concreting in hot countries.

Action of Set-Retarding Admixtures

Because in many cases the quantity of gypsum in the cement is not enough to assure an adequately long setting time when the concreting takes place at high temperatures, other protective measures are needed. The most commonly used measure is a suitable admixture.

It is important to recognize that admixtures are no substitute for sound concrete-making practices. The proper utilization of admixtures requires increased care, for instance, in batching. The other aspects of the concrete-making procedure should also be kept as constant as possible.

Certain organic compounds or mixtures of organic and inorganic compounds can be used as admixtures to reduce the water requirement of a concrete (ASTM C 494, type A) or to retard the setting (type B) or both (type D).

The new generation of water-reducing admixtures is called superplasticizers because of their improved effectiveness (14-16). These are usually low-molecular-weight polymers; the typical molecular structure is similar to that of lignosulfonates (17). Superplasticizers per se do not markedly affect the setting or the hardening of the cement paste (18).

Set-retarding admixtures are used primarily to offset the accelerating effects of high temperature and to keep concrete workable during the entire placement period. This method is of particular value to maintain the workability of the concrete during a long hauling distance in hot weather and to prevent cracking resulting from form deflection of concrete beams, bridge decks, or composite construction work. Set retarders are also used to keep concrete plastic for a sufficiently long period so that succeeding lifts can be placed without development of discontinuities in the structural unit. Retarders meeting ASTM C 494, type B and type D, delay the setting time of concrete as measured by the standard penetration test (ASTM C 403) but do not retard the slump loss with elapsed time. It has been reported that concrete slump loss was not reduced even when type D admixtures were added to concrete at dosage levels that extended the setting time beyond 24 hr (19).

The amount of retardation obtained is dependent on the specific admixture used, its dosage, the brand and type of cement, temperature, mixing sequence, and other job conditions. For instance, set retarders appear more effective with cements that have lower alkali and C_3A contents. The quantity of admixture added must be accurately determined and measured because a heavy overdosage can seriously damage the setting and hardening of concrete, particularly when an excess of air is entrained in the concrete by the overdosage (1).

There are water-reducing admixtures that also have a set-retarding effect and set-retarding admixtures with a secondary water-reducing effect. For instance, refined white granulated sugar in the quantity of 0.1 percent by weight of the cement not only retards the setting effectively but also provides a more workable mixture and increases the strength at 7 days and later.

The principal role in the mechanism of water reduction and set retardation of the admixtures considered in the classification discussion previously belongs to so-called surface-active agents. These substances are usually composed of long-chain organic molecules that are hydrophobic (no affinity for water) at one end and hydrophilic (strong affinity for water) at the other. Such molecules tend to become concentrated and form a film at the interface between two immiscible phases, such as cement and water, and alter the physiochemical forces acting at this interface. It appears that the group H-C-OH is the active component in the set retardation.

Inadvertent prolonged retardation of cements with high iron contents (types II and V) has been attributed to a water-reducing admixture (lignosulfonate). Publicized instances of retardation apparently are attributable to insufficient sulfate in the cement to retard the deleterious effect of iron on setting and strength. Without the admixture these cements set normally (20). Other incompatibilities of admixtures with some cements have also

Figure 12. Effect of different dosage levels of a retarder on slump loss.

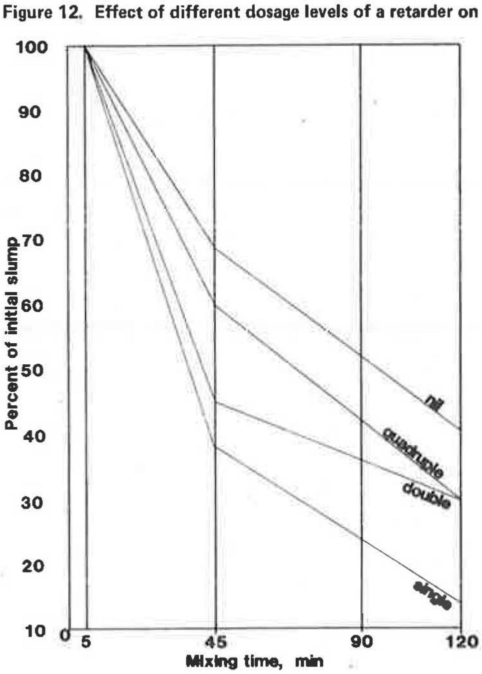
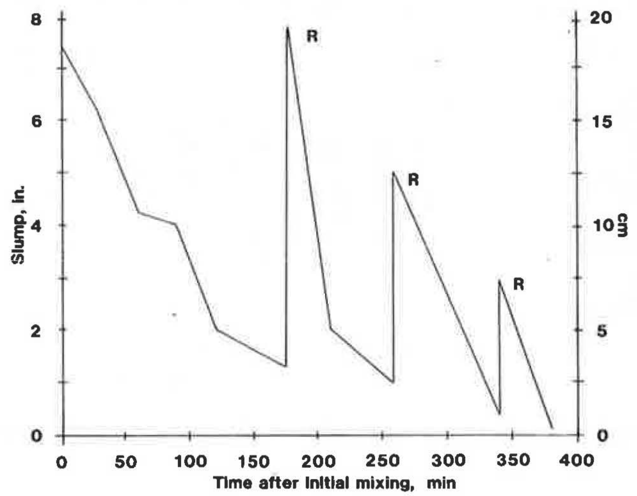


Figure 13. Slump-time retempering study.



been experienced with concrete in laboratory and field.

RETEMPERING

Another method for counterbalancing the stiffening of the cement paste or concrete is to soften its consistency during or after delivery by the addition of a suitable material. Such material can be water, a water-reducing admixture, cement paste, or any combination of these. This procedure is called retempering. It should be noted, however, that the undesirable effect of high temperature starts before retempering because the quantity of mixing water needed for a specified consistency (slump) increases with an increase in temperature (2).

It is not desirable to compensate for expected slump loss by using an initial slump higher than the required final slump when prolonged delays are encountered. Experimental results have shown that the retempering of a concrete will still result in a higher total water/cement ratio when there is initially a higher slump value than when there is a lower slump value (19).

Continuous agitation, such as mixing, does not eliminate the slump loss. Neither does the use of admixtures. Retarders meeting ASTM C 494, type B and type D, delay the setting time of concrete as measured by the standard penetration test (ASTM C 403) but do not retard the slump loss with elapsed time [Figure 12 (21)]. The advantage of the application of water-reducing admixtures is that it allows a reduction in the total water required after retempering.

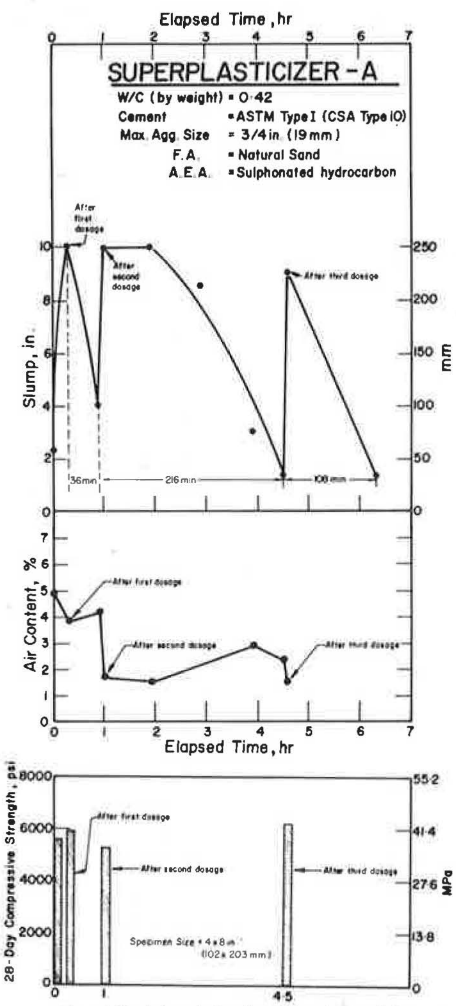
Retempering with water is not the best practice either, because it increases the original water/cement ratio and reduces the quality of concrete. Nevertheless, it may be tolerable because of its simplicity, provided that the increased water/cement ratio is still below the specified limit.

Better results have been obtained by retempering with superplasticizers. It was reported by Ramakrishnan et al. (22) that the slump of retempered concrete both with and without superplasticizer decreased about equally with time; nevertheless, large increases in slumps can be maintained for several hours by repeated retempering with superplasticizer [Figure 13 (R = repeated retempering)], which does not reduce the concrete strength. Ramakrishnan also drew the following conclusions:

1. The slump loss is proportional to the initial slump level for all the mixes; the higher the initial slump, the higher the slump loss. The total time span during which concrete could be kept workable is, however, longer for concrete with higher initial slump.
2. About 60 to 80 percent of the slump of control and retempered control concrete is lost in 60 to 90 min.
3. The ability of the superplasticizer to keep the concrete workable is reduced as the number of retemperings is increased. Higher dosages of superplasticizer lead to segregation. The rate of slump loss is higher for retempered concretes.
4. Repeated dosages of superplasticizer cause a loss in entrained air.
5. An extended period of mixing does not have an adverse effect on the strength of superplasticized concrete; however, extended mixing time does affect the workability of the mix.
6. The properties of hardened retempered concrete such as compressive strength, dry unit weight, and modulus of elasticity are not adversely affected for the mixes both with and without the addition of the superplasticizer.

Recently Malhotra performed a thorough investigation with a standard type I portland cement concerning retempering concrete with superplasticizer (23). His findings support essentially the large slump increases reported by Ramakrishnan. He also pointed out several additional facts, as follows [Figure 14 (superplasticizer used consists of sulfonated naphthalene formaldehyde condensations)].

Figure 14. Elapsed time from initial mixing versus slump, air content, and compressive strength.



1. The plasticizing effect depends on the type of superplasticizer used.
2. The size and character of the change in air content in the concrete caused by retempering with superplasticizer are a function of the type of superplasticizer used.
3. The size and character of the change in concrete strength caused by such retempering are also dependent on the type of superplasticizer. These strength changes do not always run parallel to the changes in air content.
4. Repeated retemperings produce a diminishing rate of return. Thus, it is usually not worthwhile to retemper the same concrete mixture more than twice.

Another form of retempering is to add cement paste with the appropriate water/cement ratio to the stiffening concrete to regain the specified slump.

Note that the opposite of excessive slump loss--that is, too slow stiffening--can also be a problem because it may result in low strengths, at least in the early ages, and in a delay in the construction.

CONCLUSIONS

Undesirable hot-weather effects on concrete in the plastic state may include

1. Increased water demand;
2. Increased rate of slump loss and corresponding need to add water at the job site;
3. Increased rate of setting resulting in greater difficulty with handling, finishing, and curing and increasing the possibility of cold joints;
4. Increased tendency for plastic cracking; and
5. Increased difficulty in controlling entrained-air content.

Those effects can be mitigated by several methods, including water-reducing admixtures, set-retarding admixtures, or a suitable method of retempering.

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Effects of High Temperatures on the Properties of Hardened Concrete

M. SAMARAI, S. POPOVICS, AND V.M. MALHOTRA

The effects of high temperatures on the properties of hardened concrete and the reactions involved in the hardening process are discussed. The undesirable effects of hot weather on the properties of hardened concrete include decreased strength, increased tendency for drying shrinkage and differential thermal cracking, decreased durability, and increased creep. Recommendations for further research are given.

The effects of a hot climate are not so conspicuous on the properties of hardened concrete as they are on concrete in the fresh state. Nevertheless, such effects are important enough to require attention.

The organized summary presented below emphasizes the engineering aspects of the subject instead of the scientific point of view. Besides this, some of the currently unknown aspects are outlined in the form of recommendations for further research.

EFFECTS OF TEMPERATURE ON HARDENING

Reactions in Hardening Process

After the final set, chemical reactions between cement and water continue at a diminishing rate until one or more of the conditions necessary to the reaction are lacking. This stage of hydration is called the hardening process, during which the predominant reaction is the continuing hydration of the calcium silicates. The decrease in rate is the result of two effects: (a) the surface area of unhydrated cement particles decreases as the smaller particles become completely hydrated and the larger particles become smaller, and (b) a layer of CSH gel forms on the surfaces of the cement particles, slowing down further reaction by forming a protective coating.

Measurements indicate that the kinetics of hydration are influenced by several factors, including the fineness and composition of the cement, temperature, and water/cement ratio of the paste, and admixtures (1, pp. 259-273). It is important to note here that if any of these factors increase the specific rate of hydration, the same change simultaneously intensifies the deceleration of the hydra-

Figure 1. Effect on concrete compressive strength of elevated casting and moist curing temperatures: type I cement up to 1 yr.

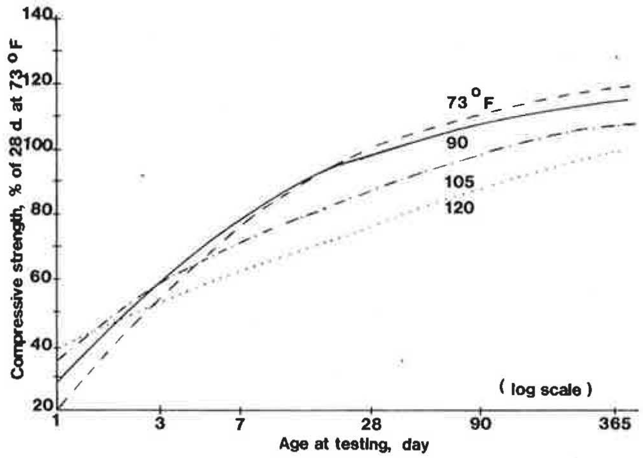
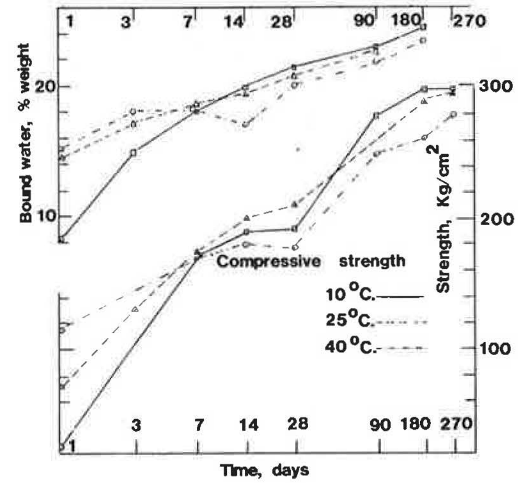


Figure 2. Compressive strength and bound water content of cement containing 12.5 percent C_3A.



tion to a greater degree. Thus, the hydration will begin more strongly but will also level off sooner. The kinetics are important not only because they control the quantity of the hydration products at early ages but also because they influence to a certain extent the quality of hydration (2).

General Description of Effects of Temperature

When concrete specimens are cured at various constant temperatures, the temperature has a double effect on their strengths. On the one hand, a higher curing temperature increases the strengths at early ages, which is expected, but on the other hand, it hinders the strength development later on, which is unexpected. This is illustrated quantitatively in Figure 1 (2,3), which shows the development of compressive strength of an ASTM type I portland cement.

In more general terms, the ultimate strength of a cement paste, mortar, or concrete is frequently lowered by factors that increase the early strengths (4). The term "ultimate strength" is used as the strength obtained after a long duration of moist curing.

There are quite a few analogies in nature for such an inverse relationship: The speed of the sprinter does not carry him as far as the lower speed does the distance runner; a low rate of growth results in larger crystals than a high rate; and so forth. Nevertheless, such cases represent the minority, and perhaps this is why one finds perplexing the existing inverse correlation between early strength and ultimate strength of portland-cement pastes.

From the standpoint of kinetics, the inverse relationship between the early strength and the ultimate strength means that an increase, for instance, in the curing temperature intensifies the rate reduction (deceleration) of hardening. Consequently, if a factor increases linearly the specific rates of the hardening process, it will intensify the specific decelerations more than linearly; that is, the hardening will start out stronger but will also level off sooner.

A possible physical explanation for the unexpected correlation between high early strength and relatively low final strength is the hypothesis that certain strength-affecting properties of the hydration products are modified by a change in the rate of hardening. The essence of this mechanism is that a change in the rate of hardening per se affects inversely the final strength, and the cause of the rate of change is secondary at most. For instance, cementlike CSH gels under the effect of intensifying physical or chemical factors produce a somewhat higher specific surface at early ages (5, pp. 199-219). This means finer texture and, presumably, lower porosity at early ages, which trend may reverse itself at later ages. So the rate of early hardening appears to influence, per se, the strengths at later ages through its influence on the structure of cement gel.

In brief, the inverse relationship between early strengths and final strengths is not quite understood yet. The hydration products formed, say, during curing at 122°F (50°C) do not differ greatly from those formed at 68°F (20°C). The elevated temperature modifies somewhat the morphology of the calcium silicate hydrates, but this effect does not appear to be large enough to have a major effect on the final strength. Thus, the main reason for the relatively low final strengths of the steam-cured and other accelerated concretes seems to be mechanical: The rapid hydration may produce higher final porosity and more microcracks in the gel (6).

EFFECTS OF TEMPERATURE ON STRENGTH

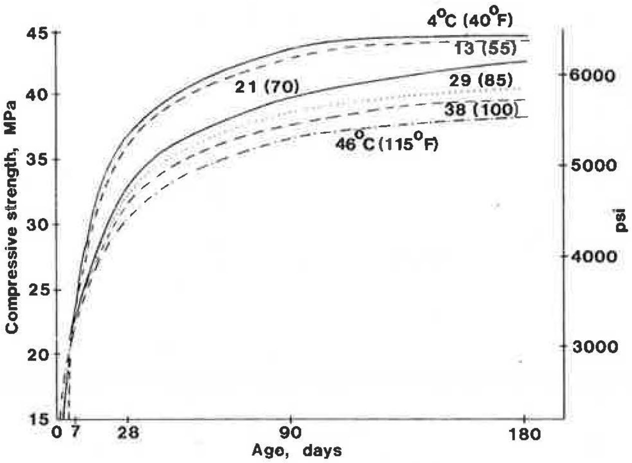
Curing Temperature and Compressive Strength

The extent of the effects of curing temperature on the compressive strength of portland-cement concretes is illustrated quantitatively through several cases.

1. The magnitude of the effect of temperature on the strength development depends not only on the magnitude of temperature change but also on the type of cement used. Note also that when calcium chloride is added to the concrete, the adverse effect of high curing temperature on the late strengths is reduced (7).

2. The primary reason for higher early strengths with increasing temperatures is that the rate of chemical reactions between the cement and water--that is, hydration--increases with increasing temperature. This is demonstrated in Figure 2 (1), where the degree of hydration is characterized by the quantity of the chemically bound water in the hydration products of cement. The strength-reducing effect of higher curing temperatures at later ages is also illustrated.

Figure 3. Effect of temperature on strength development during first 2 hr after casting.



3. When concrete is cast and maintained at a given temperature for several hours and then cured at 70°F (22°C), the higher the initial temperature (within limits), the lower the 28-day strength, as illustrated in Figure 3 (8). The relative strengths at 28 days are maintained at later ages.

4. It may be generalized from the previous paragraph that if the curing temperature is higher than the initial temperature of casting, the resulting 28-day strength will be higher than that for a curing temperature equal to or lower than the initial temperature (9).

5. It appears that there is a curing temperature during the early life of the concrete that may be considered optimum with regard to the strength at later ages, or more strictly, at comparable degrees of hydration. This temperature is influenced somewhat by the cement type. For ASTM types I and II portland cements this temperature is 55°F (13°C); for type III it is 40°F (4°C) (7).

6. The 28-day concrete strength can be increased by using a suitable water-reducing admixture. It is also demonstrated that this beneficial effect is more pronounced when the concrete is mixed at 90 to 95°F (32 to 35°C) than at 70 to 75°F (22 to 25°C) (10).

Maturity Concept

Because strength of concrete depends on both age and temperature, it can be said that strength is a function of Σ (time times temperature), and this summation is called maturity. The temperature is reckoned from an origin found experimentally to be between 11 and 14°F (-12 and -10°C).

When maturity is measured in degrees Fahrenheit times hours or times days, strength plotted against the logarithm of maturity gives a straight line. Experiments by Ramakrishnan also support this finding within practical limits (11, pp. 1-8). It is therefore possible to express strength at any maturity as a percentage of strength of concrete at any other maturity; the latter is often taken as 35,000°F x hr (19,800°C x hr, which is the maturity of concrete cured at 64°F (18°C) for 28 days. The ratio of strengths--that is, relative strength (f_{rel})--can then be written as follows:

$$f_{rel} = A + B \log_{10} (\text{maturity} \times 10^{-3}) \tag{1}$$

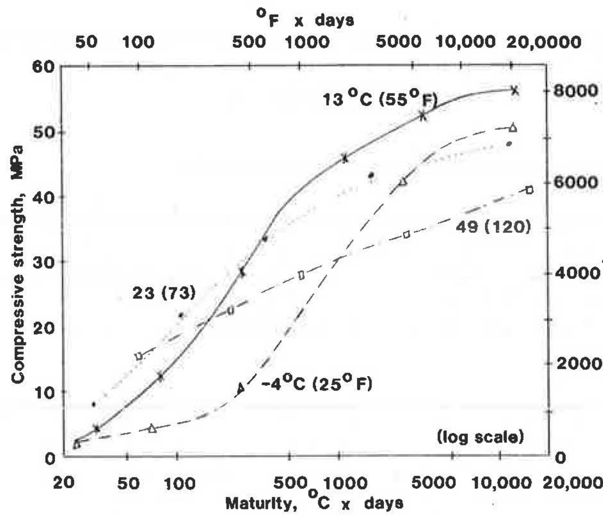
The value of the coefficients A and B depends on the strength level of concrete; those suggested by Plowman are given in Table 1. It can be seen that the strength-maturity relation depends on the properties of the cement and on the general quality of the concrete and is valid only within a range of temperatures. The severe limitation of the temperature ranges is apparent, for instance, from Figure 4 (7), obtained by Klieger, who tested ordinary portland-cement concrete with a water/cement ratio of 0.43 and an air content of 4.5 percent and that was cured at 73°F (23°C) from the age of 28 days. The separate relationships for each initial curing temperature indicate poor correlation of strength with maturity. A further complication arises because the effects of a period of exposure to a higher temperature are not the same when this occurs immediately after casting or later in the life of the concrete. Specifically, early high temperature leads to a lower strength for a given total maturity than when heating is delayed for at least a week or is absent.

Table 1. Plowman's coefficients for maturity equation.

Strength After 28 Days at 64°F[a] (psi)	Coefficient A	Coefficient B
2,500	-7	68
2,500-5,000	6	61
5,000-7,500	18	54
7,500-10,000	30	46.5

Note: $t°F = (t°C \div 0.55) + 32$; 1 psi = 145 MPa.
[a]Maturity of 35,600°F x hr.

Figure 4. Influence of temperature on strength versus maturity relationship of type I cement during first 28 days after casting.



Malhotra points out in this connection that the maturity rule is applicable only if

1. The relation between the logarithm of maturity and strength is linear, and this is so only within the range of maturity represented by about 3 to 28 days at normal temperatures;
2. The initial temperature of the concrete is between 60 and 80°F (15.5 and 26.6°C); and
3. No loss of moisture by drying occurs during the curing period, which is a difficult condition to fulfill in the field (12).

Flexural Strength of Concrete

The preceding discussion of compressive-strength data applies equally to the flexural-strength data. Flexural strengths at early ages increased with increase in temperature. At later ages, the effect of temperature was reversed. These concretes made and cured at the lower temperatures showed highest flexural strengths at 1 yr. The optimum temperatures for flexural-strength development appear to be the same as those for compressive strength (7).

The use of calcium chloride frequently resulted in flexural strengths at later ages somewhat lower than for comparable concretes at the same temperature but without calcium chloride.

EFFECTS OF CURING

Effects of Wetness

The strength of a hardening cement paste is the consequence of the hydration process. Therefore, the strength development stops when the paste in the concrete dries out, that is, when the amount of free water in the paste decreases below a level critical for the hydration. Because this has a great significance in concrete construction, especially in a hot climate, it should be discussed here.

The loss of water in concrete is the result of two actions: evaporation and the gradual using up of the mixing water by the hydration of cement, which is called self-desiccation. Thus, a long-enough strength development in mortars and concretes requires two countermeasures: (a) the elimination or at least reduction of the early evaporation of water from the concrete and (b) replacement of the water lost by self-desiccation and evaporation with water from outside. This double countermeasure is called curing; the (b) portion is the wet or moist curing. Self-desiccation has an important role in stopping the strength development when the water/cement ratio is below about 0.50 by weight. Therefore, moist curing is particularly important for such concretes.

It should be stressed that for a satisfactory strength development it is not necessary for all cement to hydrate; indeed, this is only rarely achieved in practice. If, however, the water-filled or empty porosity in the fresh cement paste of the concrete is greater than the volume that can be filled by the hydration products, greater hydration leads to a higher strength and lower permeability.

The rate and extent of drying through evaporation depend on a number of factors, such as the area of the exposed concrete surface relative to its volume, the humidity of the surrounding air [Figure 5 (13)], the temperatures of the air and concrete [Figure 6 (13)], the difference between the temperatures of concrete and air, and the wind velocity [Figure 7 (13)]. [In Figures 5-7 air temperature is 70°F (21°C), wind velocity is 10 mph (4.5 m/sec), and relative humidity is 70 percent.]

Curing Methods

A means of reducing the drying is to use an impermeable membrane or waterproof paper. A membrane, provided it is not punctured or damaged, will effectively prevent evaporation of water from the concrete but will not allow ingress of water to replenish that lost by self-desiccation. The membrane is formed by sealing compounds, which may be clear, white, or black. The opaque compounds have the effect of shading the concrete, and a light color leads to a lower absorption of the heat from the sun and hence to a smaller rise in the temperature of the concrete. ASTM C 156 prescribes tests for the efficiency of curing compounds.

Except when used on concrete with a high water/cement ratio, sealing compounds reduce the degree and rate of hydration compared with efficient wet curing. Wet curing is often applied only intermittently, however, so that in practice sealing may lead to better results.

The other usual way of curing, the wet curing, can be provided by keeping the concrete in contact with a source of water. This may be achieved by spraying or flooding (ponding) or by covering the concrete with wet sand or earth, sawdust, or straw.

Figure 5. Influence of relative humidity on loss of water from concrete in early stages after placement.

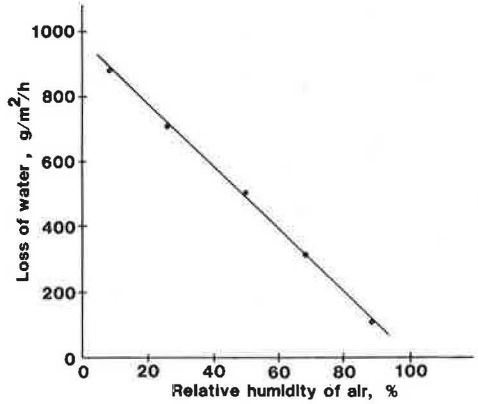


Figure 6. Influence of air and concrete temperature on loss of water from concrete in early stages after placement.

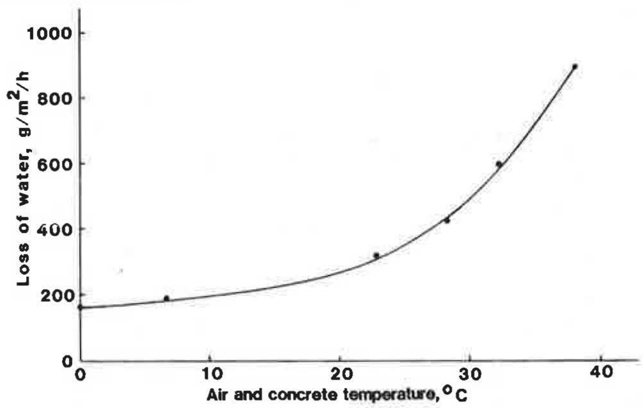


Figure 7. Influence of wind velocity on loss of water from concrete in early stages after placement.

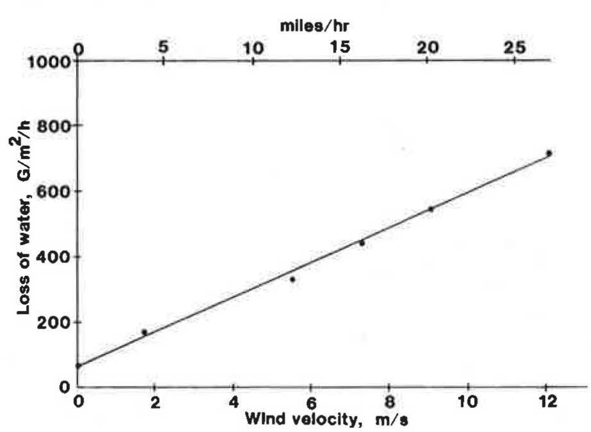
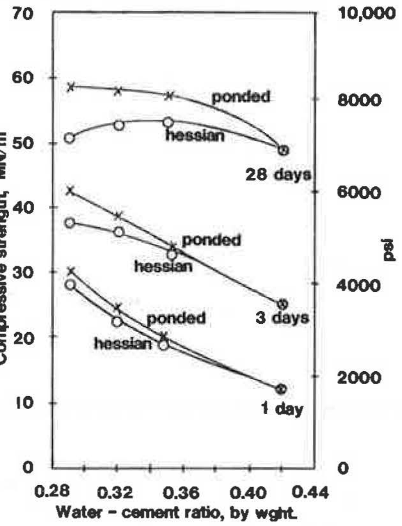


Figure 8. Influence of curing conditions on strength of test cylinders.



Periodically wetted hessian or cotton mats may be used, or alternatively an absorbent covering with access to water may be placed over the concrete. A continuous supply of water is naturally more efficient than an intermittent one, and Figure 8 compares the strength development of concrete cylinders whose top surface was flooded during the first 24 hr with that of cylinders covered with wet hessian. The difference is greatest at low water/cement ratios where self-desiccation operates rapidly. The influence of curing conditions on strength is lower in the case of air-entrained than non-air-entrained concrete (14).

The magnitude of the influence of wet curing on the strength has also been demonstrated. It has been shown clearly that the development of strength stops at early ages if the concrete specimen is exposed to dry air with no previous wet curing. Concrete exposed to dry air from the time of its placement is about 50 percent as strong at 3 months as concrete that has been moist cured continuously. Resumption of moist curing after a period of air drying results in resumption of hydration, although at a slower rate than that in progress when drying was begun. Note also that the measured concrete strength is influenced by the moisture content of the concrete at the time of testing. Specimens exposed to air and tested in the air-dry condition are one-quarter to one-third stronger than corresponding specimens exposed to air for the same period but saturated just before being tested. In general, the more dense and strong the concrete, the greater is this influence.

Large surfaces of concrete, such as road slabs, present a serious curing problem. In order to prevent crazing of the surface on drying out, loss of water must be prevented even before setting. Because the concrete is at that time mechanically weak, it is necessary to suspend a covering above the concrete surface. This protection is required only in dry weather but may also be useful in preventing rain from marring the surface of fresh concrete.

The period of curing cannot be prescribed simply, but it is usual to specify a minimum of 7 days for type I portland-cement concrete. With slower-hardening cements a longer curing period is desirable. The temperature also affects the length of the required period of curing, and the British Code of Practice for the Structural Use of Concrete (CP 110, 1972) lays down the normal curing periods for different cements and exposure conditions in terms of maturity of concrete (13).

High-strength concrete should be cured at an early age because partial hydration may make porosity discontinuous; on renewal of curing, water would not be able to enter the interior of the concrete and no further hydration would result. Nevertheless, mixes with a high water/cement ratio always retain a large volume of capillaries so that curing can be effectively resumed at any time.

SHRINKAGE AND CREEP

It is worthwhile to mention shrinkage and creep of the hardened concrete here because both of these deformation mechanisms are based on drying and other moisture movements and also because they play an important role in the cracking of concrete.

Cracks in concrete structures can indicate major structural problems and can mar the appearance of monolithic construction. They can expose reinforcing steel to oxygen and moisture and make the steel more susceptible to corrosion.

When concrete dries, it contracts or shrinks, and when it is wetted again, it expands. These volume changes, with changes in moisture content, are inherent characteristics of hydraulic cement concretes. It is the change in moisture content of the cement paste that causes the shrinkage or swelling of concrete, whereas the aggregates provide an internal restraint that significantly reduces the magnitude of these volume changes.

Why does concrete crack due to shrinkage? If the shrinkage of concrete caused by drying could take place without any restraint, the concrete would not crack. In a structure, however, the concrete is always subject to some degree of restraint by either the foundation or another part of the structure or by the reinforcing steel embedded in the concrete. This combination of shrinkage and restraint develops tensile stress. When this tensile stress reaches the tensile strength, the concrete will crack.

Another type of restraint is developed by the difference in the shrinkage at the surface and that in the interior of a concrete member, especially at early ages. Because the drying shrinkage is always larger at the exposed surface, the interior portion of the member restrains the shrinkage of the surface concrete, thus developing tensile stresses. This may cause surface cracking, in which cracks do not penetrate deep into the concrete. These surface cracks may with time penetrate deeper into the concrete member as the interior portion of the concrete is subject to additional drying.

The influence of temperature on creep has become of increased interest in connection with the use of concrete in the construction of prestressed concrete nuclear pressure vessels, but the problem is of significance also in other types of structures, e.g., bridges. The rate of creep increases with temperature up to about 160°F (70°C) when, at least for a 1:7 mixture with a water/cement ratio of 0.60, it is approximately 3.5 times higher than at 70°F (21°C). Between 160°F and 205°F (96°C) the rate drops off to 1.7 times the rate at 70°F. These differences in rate persist at least for 15 months under load. Figure 9 (13) illustrates the progress of creep (ratio of stress to strength is 0.70). This behavior is believed to be due to desorption of water from the surface of the gel so that gradually the gel itself becomes the sole phase subject to molecular diffusion shear flow; consequently the rate of creep decreases. The behavior over a wide range of temperatures is shown in Figure 10 (15).

Figure 9. Relationship between creep and time under load for concretes stored at different temperatures.

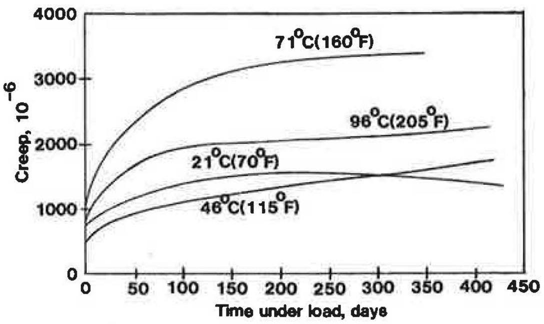
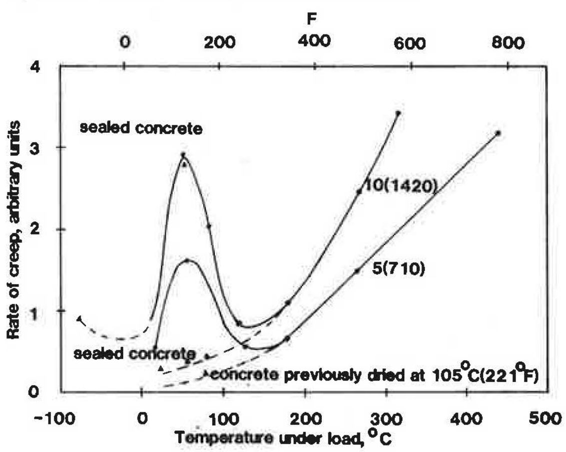


Figure 10. Influence of temperature on rate of creep.



RECOMMENDATIONS FOR FURTHER RESEARCH

The development of a numerical method is desirable that provides the strength development of portland cement in terms of compound composition, fineness, age, and curing temperature. Although certain efforts have been made in this direction (4), much more work is needed for a general, reliable theory and formulas for the prediction of concrete strength cured at elevated temperatures.

A recent committee report by the Réunion Internationale des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions (RILEM) recommended the following topics for further investigation concerning hardened concrete (16):

1. Studies of the influence of hot and dry and hot and humid environments on strength, shrinkage, and creep of portland-cement concretes in relation to the composition of the cement;
2. Long-term study of shrinkage and creep of concrete exposed to elevated temperature combined with different amounts of relative humidity, intermittently or permanently;
3. Study of durability of building materials in a hot and humid environment;
4. Long-term field observations of corrosion of reinforcement in structures exposed to elevated temperatures and different amounts of relative humidity;
5. Systematic field measurements of thermal stresses in structures in a hot environment (supported by appropriate analysis and tests);
6. Study of thermal stresses in concrete structures in relation to cracking and deformation; and
7. Field observations of structures with a view to systematic information relating climatic conditions (hot and dry and hot and humid) to satisfactory performance.

CONCLUSIONS

Undesirable hot-weather effects on concrete in the hardened state may include

1. Decreased strength resulting from higher water demand and increased temperature level,
2. Increased tendency for drying shrinkage and differential thermal cracking,
3. Decreased durability,
4. Decreased uniformity of surface appearance, and
5. Increased creep.

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Effects of High Temperatures on the Properties of Fresh and Hardened Concrete: A Bibliography (1915-1983)

M. SAMARAI, S. POPOVICS, AND V.M. MALHOTRA

This bibliography covers the international literature up to the early part of 1983 on the effects of high temperature [up to 140°F (60°C)] on the properties of fresh and hardened concrete. The following topics are covered: the workability of concrete, including setting, slump loss, and admixtures; curing and the cracking tendency of fresh concrete in hot climates; strength development of concrete, including early strengths, later-age strengths, and maturity of concrete; other properties of hardened concrete, such as shrinkage and creep; and construction practices in hot climates, including selection of materials, especially admixtures; protective measures; construction methods; and pertinent specifications. Steam and autoclave curing and other accelerating treatments of concrete are not covered.

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Development of a Bayesian Acceptance Approach for Bituminous Pavements

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Traditional approaches for estimating the percentage of a lot that is within specification limits (PWL) are based on random samples taken from the lot being evaluated. These approaches suffer from the small sample sizes necessitated by the destructive and time-consuming tests that are usually used in determining the quality of the materials. The development of a Bayesian approach for estimating PWL is presented that incorporates information concerning the contractor's past performance on the project along with the current sample results in determining the estimate for the PWL of the current lot. The procedure assumes that the daily population mean is a random variable that follows a normal distribution, that the production process is also normally distributed, and that the process variance is constant. These assumptions are confirmed by using goodness-of-fit tests on data collected from 13 bituminous runway-paving projects. Computer simulation shows that the Bayesian PWL estimators are slightly biased as compared with the unbiased traditional quality-index method but that the PWL estimators exhibit smaller variances than the traditional method.

To determine the acceptability of and the ultimate payment for bituminous pavements, a procedure is necessary for estimating the quality of those materials. In this paper a method for estimating the quality of construction materials is presented that incorporates empirical Bayes (EB) techniques into the estimate. It is believed that such a procedure will be an improvement over current procedures based solely on classical techniques because it incorporates information concerning the contractor's past production record into the estimate for the current lot of material. The method, which should be applicable for many construction materials, will be developed from data collected on bituminous concrete pavement construction projects.

TYPES OF ACCEPTANCE PLANS

Several different types of acceptance plans have been developed and recommended for bituminous concrete pavement materials. Some acceptance plans (1-4) determine the acceptability of the material from the average, or mean, of the test results from a sample. These plans are based on the assumption that the standard deviation is known (or assumed). This known standard deviation is used to determine the acceptance limits within which the sample means must fall.

Other types of acceptance plans (1,2,5,6) are based on the fact that the standard deviation is not known and must be estimated from the sample results. In one type of plan the sample mean must fall a specified number (k) of standard deviations from the acceptance limit (e.g., $\bar{X} - L \geq ks$). The specified number (k), in essence, determines the percentage of the material that must exceed the acceptance limit before the material is accepted. In an extension of this method (1,2,7-9), the calculated percentage of the material that is within the acceptance limits (percentage within limits, or PWL) is used for acceptance purposes. This method provides a natural measure for the relative quality of the material (presumably 90 PWL is superior to 80 PWL). It can therefore be used for developing a price-adjustment schedule that relates the quality of the material (as measured by PWL) to the payment to be received for the material.

The PWL method has the advantage that it considers both the mean and the variability of the material and then incorporates these into one value, PWL, which can then be related to payment level. The PWL approach is a well-accepted method that has been adopted by state (8,9) and federal (10) agencies. The PWL approach to acceptance is the one for which an EB estimator will be developed in this paper.

PROBLEMS WITH EXISTING PLANS

Many acceptance plans currently in use that attempt to account for material variability use the sample range to estimate this variability. When the range is employed in a PWL acceptance plan, it is actually used to estimate the sample standard deviation. It has been pointed out (11) that the range method provides a biased estimate of PWL. The sample standard deviation, which provides an unbiased estimate for PWL, will be used in this paper to provide a better method for estimating PWL.

One major problem common to all types of construction-material acceptance plans is the relatively high costs and destructive nature of many of the tests commonly used to measure the quality and acceptability of the material. The luxury of using a sample size of 100 for each lot may be possible in industrial and manufacturing applications but is totally impractical for most construction situations. Sample sizes in construction-material acceptance plans are typically about four or five samples per lot. The objective in this paper is to develop an acceptance procedure that addresses this problem of a limited sample size.

The method used will be to employ an EB procedure to estimate the quality of a given lot of material. This procedure will incorporate the preceding information from the contractor's production history on the project into the estimate for the quality of the material placed during the day for which the quality is being estimated. In other words, the preceding information about the contractor's process capabilities will be pooled with the test results from the current sample to estimate the quality of the material in question. As pointed out by Martz (12), this pooling of data tends to have the effect of increasing the sample size.

BAYESIAN ESTIMATOR FOR PWL

The case to be considered in the development of the Bayesian estimator is that of basing the acceptance of a lot of material on an estimate of the PWL value for the lot. It is common practice to estimate the PWL value for a given lot of material from the mean ($\bar{X}$) and standard deviation (s) of a number of tests performed on samples randomly selected from the lot. In this way, each lot is considered totally independent of preceding or subsequent lots. The method to be developed will employ Bayesian concepts to pool information from previous lots with the results from the current lot to estimate the PWL for the current lot.

In the development of the Bayesian estimator it will be assumed that the sampling is from a normal process, i.e., that the daily test results are normally distributed. It will also be assumed that the process has a constant variance. The final assumption to be made is that the daily population means are also normally distributed. The Bayesian approach considers this daily population mean as a random variable and assigns some distribution to it. The appropriateness of these assumptions will be verified against data collected from 13 field construction projects, and this analysis will be presented in a later section.

The normal-normal assumption is quite convenient and is frequently used when the data are only approximately normally distributed because it forms what is known as a conjugate family. Conjugate families make updating of the preceding distribution with current data to form the subsequent distribution relatively easy. That is, if one samples from a normal process with a fixed variance (σ^2) and the preceding distribution for the mean is normal, the subsequent distribution for the mean will also be normally distributed. This allows for a fairly convenient estimator for the mean of the subsequent distribution.

Bayes Estimator for Daily Population Mean

Parameters Known

The underlying assumptions on which the estimator will be developed are presented in Figure 1 and may be summarized as follows:

$$X_{ij} \sim N(\mu_i, \sigma^2) \tag{1}$$

$$\mu_i \sim N(\mu_p, \sigma_p^2) \tag{2}$$

where

X_{ij} = j daily test results for day i;
μ_i = population mean for day i;
σ^2 = process variance (assumed equal for all days, i.e., constant variance);
μ_p = preceding mean of the distribution of daily population means (μ_i); i.e., μ_i is a random variable with mean equal to μ_p;
σ_p^2 = preceding variance of the distribution of daily population means (μ_i); i.e., μ_i is a random variable with variance equal to σ_p^2;
i = 1, 2, 3, . . ., N = number of days; and
j = 1, 2, . . ., n = number of tests per day.

If σ^2, μ_p, and σ_p^2 were known, then with a squared error loss function, the Bayes estimator for daily population mean (call it $\hat{\mu}$) can be shown to be given by

$$\hat{\mu}_i = [\sigma_p^2\, \overline{X}_i + \mu_p(\sigma^2/n)]/[\sigma_p^2 + (\sigma^2/n)] \tag{3}$$

The derivation of this equation has been discussed by Burati (13).

Parameters Unknown

The estimator in Equation 3 was based on the assumption that the process variance and preceding distribution were known. Because σ^2, μ_p, and σ_p^2 are not known in the typical construction situation, their values must be estimated from the sample results. Based on the results of N previous days, natural estimators for μ_p and σ^2 are

$$\hat{\mu}_p = (1/N) \sum_{i=1}^{N} \overline{X}_i \tag{4}$$

$$\hat{\sigma}^2 = (1/N) \sum_{i=1}^{N} s_i^2 \tag{5}$$

where

N = number of previous days,
$\overline{X}_i$ = daily mean for day i, and
s_i^2 = daily variance for day i.

Equation 5 is the pooled estimate for variance for the case of an equal number of tests each day. This is typically the case for density test results in asphalt pavement construction. If the number of tests per day is not constant, which may be the case for certain test results, such as the Marshall and extraction tests, the following formula must be used to determine the pooled estimate for process variance:

$$\hat{\sigma}^2 = [(n_1 - 1)s_1^2 + (n_2 - 1)s_2^2 + \ldots + (n_k - 1)s_k^2]/(n_1 + n_2 + \ldots + n_k - k) \tag{6}$$

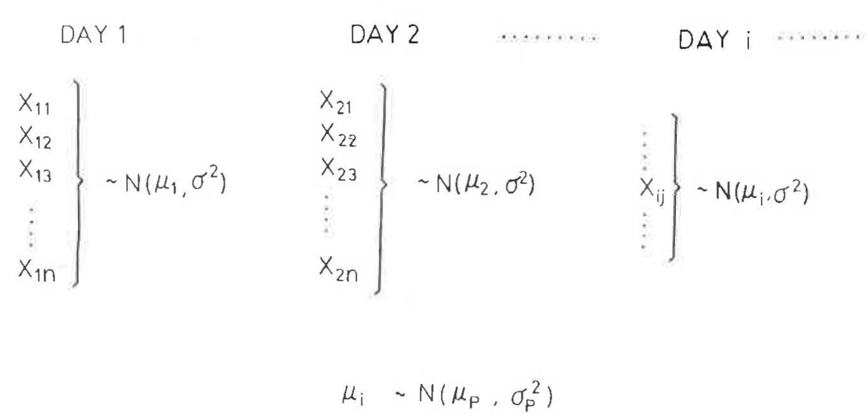
where s_1^2, s_2^2, . . ., s_k^2 are the daily sample variances for day 1, 2, . . ., k and n_1, n_2, . . ., n_k are the number of tests per day for day 1, 2, . . ., k.

The development of an estimate of σ_p^2 is not so obvious as the case of μ_p and σ^2, because there is no natural estimator that is readily apparent. The daily means can be thought of as being equal to the daily population mean (μ_i) plus some average error term ($\bar{\epsilon}$); i.e.,

$$\overline{X}_i = \mu_i + \bar{\epsilon} \tag{7}$$

This can be illustrated as follows:

Figure 1. Assumptions made in development of Bayesian estimator for daily population mean.



$$X_{i1} = \mu_i + \epsilon_1$$
$$X_{i2} = \mu_i + \epsilon_2$$
$$X_{i3} = \mu_i + \epsilon_3$$
$$\vdots$$
$$X_{in} = \mu_i + \epsilon_n$$

and

$$\bar{X}_i = (1/n) \sum_{j=1}^{n} X_{ij} \tag{8}$$

but this can also be written as follows:

$$\bar{X}_i = (1/n)\left(n\mu_i + \sum_{j=1}^{n} \epsilon_j\right) \tag{9}$$

or

$$\bar{X}_i = \mu_i + (1/n) \sum_{j=1}^{n} \epsilon_j \tag{10}$$

but

$$(1/n) \sum_{j=1}^{n} \epsilon_j$$

is merely the average error ($\bar{\epsilon}$), so

$$\bar{X}_i = \mu_i + \bar{\epsilon} \tag{11}$$

If it is assumed that the daily population mean (μ_i) and the average error term for the day ($\bar{\epsilon}$) are independent of each other, because

$$\bar{X}_i = \mu_i + \bar{\epsilon} \tag{12}$$

then

$$\mathrm{Var}(\bar{X}_i) = \mathrm{Var}(\mu_i) + \mathrm{Var}(\bar{\epsilon}) \tag{13}$$

It may be recalled that $\mu_i \sim N(\mu_p, \sigma_p^2)$; then $\mathrm{Var}(\mu_i) = \sigma_p^2$. Also, it may be noted that

$$X_{ij} \sim N(\mu_i, \sigma^2) \tag{14}$$

Then

$$\epsilon_{ij} \sim N(0, \sigma^2) \tag{15}$$

so

$$\bar{\epsilon} \sim N(0, \sigma^2/n) \tag{16}$$

Therefore,

$$\mathrm{Var}(\bar{\epsilon}) = \sigma^2/n \tag{17}$$

If this information is combined, it can be stated

$$\mathrm{Var}(\bar{X}_i) = \sigma_p^2 + (\sigma^2/n) \tag{18}$$

The variance of the daily means can also be stated as follows:

$$\mathrm{Var}(\bar{X}_i) = [1/(N-1)] \sum_{i=1}^{N} (\bar{X}_i - \bar{\bar{X}})^2 \tag{19}$$

where

- N = number of previous days;
- $\bar{X}_i$ = daily mean for day i; and
- $\bar{\bar{X}}$ = grand mean, i.e., mean of daily means for N previous days.

If the right side of Equation 19 is referred to as S_N^2, then the estimate for σ_p^2 is therefore as follows:

$$\hat{\sigma}_p^2 = S_N^2 - (\sigma^2/n) \tag{20}$$

Because all these values are estimates, it may be possible on some occasions for σ^2/n to exceed S_N^2. Because it is not possible to have a variance less than zero, the estimate of σ_p^2 will be as follows:

$$\hat{\sigma}_p^2 = S_N^2 - (\sigma^2/n) \quad \text{if positive,}$$
$$= 0 \quad \text{if } S_N^2 - (\sigma^2/n) \text{ is negative}$$

To summarize, an EB estimate for the daily population mean ($\hat{\mu}_i$) may be calculated from the sample results as follows:

$$\hat{\mu}_i = [\hat{\sigma}_p^2 \bar{X}_i + \hat{\mu}_p (\hat{\sigma}^2/n)] / [\hat{\sigma}_p^2 + (\hat{\sigma}^2/n)]$$

where $\hat{\sigma}_p^2$, $\hat{\mu}_p$, and $\hat{\sigma}^2$ are as defined above.

ESTIMATING PERCENTAGE WITHIN LIMITS

Because the acceptance procedure under consideration is based on PWL, it is necessary to develop a method for estimating PWL by using the EB estimator for daily population mean (EB mean estimator) that was developed above. A number of possible estimators for PWL that are based on the EB mean estimator can be developed.

Traditional Quality-Index Approach

The traditional approach uses the daily sample mean and standard deviation results to calculate a quality index (Q_L or Q_U). Once the quality index has been calculated for a given lot, the estimated PWL can be determined from tabled values. The quality index can be calculated as follows:

$$Q_L = (\bar{X} - L)/s \tag{21}$$

or

$$Q_U = (U - \bar{X})/s \tag{22}$$

where

- Q_L = lower quality index,
- Q_U = upper quality index,
- L = lower specification limit,
- U = upper specification limit,
- $\bar{X}$ = sample mean, and
- s = sample standard deviation.

Table 1 is used to estimate PWL values based on Q_L and Q_U values. The procedure for deriving this table was developed by Willenbrock and Kopac (11).

An example will help to describe this approach, referred to as method 1, for estimating PWL. The following values are known for the case of acceptance of a lot based on mat density: $\bar{X}$ = 97.6 percent, s = 1.05 percent, L = 96.7 percent, and n = 4. The quality index can then be calculated as follows:

$$Q_L = (\bar{X} - L)/s = (97.6 - 96.7)/1.05 = +0.857.$$

From Table 1, the estimated PWL for the lot is 78.6 percent.

Bayes Quality-Index Approach

Logically, the first step in incorporating the EB

Table 1. Standard deviation method for estimating percentage of lot within limits.

Percentage Within Limits	Negative Values of Q_U or Q_L					Percentage Within Limits	Positive Values of Q_U or Q_L				
	n = 3	n = 4	n = 5	n = 6	n = 7		n = 3	n = 4	n = 5	n = 6	n = 7
50	0.0000	0.0000	0.0000	0.0000	0.0000	99	1.1510	1.4700	1.6719	1.8016	1.8893
45	0.1806	0.1500	0.1406	0.1364	0.1338	98	1.1476	1.4400	1.6018	1.6990	1.7615
40	0.3568	0.3000	0.2823	0.2740	0.2689	97	1.1439	1.4100	1.5428	1.6190	1.6662
39	0.3912	0.3300	0.3106	0.3018	0.2966	96	1.1402	1.3800	1.4898	1.5500	1.5868
38	0.4252	0.3600	0.3392	0.3295	0.3238	95	1.1367	1.3500	1.4408	1.4892	1.5184
37	0.4587	0.3900	0.3678	0.3577	0.3515	94	1.1330	1.3200	1.3946	1.4332	1.4562
36	0.4917	0.4200	0.3968	0.3859	0.3791	93	1.1263	1.2900	1.3510	1.3813	1.3990
35	0.5242	0.4500	0.4254	0.4140	0.4073	92	1.1170	1.2600	1.3091	1.3328	1.3465
34	0.5564	0.4800	0.4544	0.4426	0.4354	91	1.1087	1.2300	1.2683	1.2866	1.2966
33	0.5878	0.5100	0.4837	0.4712	0.4639	90	1.0977	1.2000	1.2293	1.2421	1.2494
32	0.6187	0.5400	0.5131	0.5002	0.4952	89	1.0864	1.1700	1.1911	1.2001	1.2045
31	0.6490	0.5700	0.5424	0.5292	0.5211	88	1.0732	1.1400	1.1538	1.1592	1.1615
30	0.6788	0.6000	0.5717	0.5586	0.5506	87	1.0596	1.1100	1.1174	1.1196	1.1202
29	0.7076	0.6300	0.6018	0.5880	0.5846	86	1.0446	1.0800	1.0819	1.0813	1.0793
28	0.7360	0.6600	0.6315	0.6178	0.6095	85	1.0286	1.0500	1.0469	1.0437	1.0413
27	0.7635	0.6900	0.6619	0.6480	0.6395	84	1.0118	1.0200	1.0125	1.0073	1.0032
26	0.7905	0.7200	0.6919	0.6782	0.6703	83	0.9940	0.9900	0.9782	0.9718	0.9673
25	0.8164	0.7500	0.7227	0.7093	0.7011	82	0.9748	0.9600	0.9453	0.9367	0.9315
24	0.8416	0.7800	0.7535	0.7403	0.7320	81	0.9555	0.9000	0.9123	0.9028	0.8966
23	0.8661	0.8100	0.7846	0.7717	0.7642	80	0.9342	0.9000	0.8798	0.8693	0.8626
22	0.8896	0.8400	0.8161	0.8040	0.7964	79	0.9122	0.8700	0.8479	0.8363	0.8290
21	0.9122	0.8700	0.8479	0.8363	0.8290	78	0.8896	0.8400	0.8161	0.8040	0.7964
20	0.9342	0.9000	0.8798	0.8693	0.8626	77	0.8661	0.8100	0.7846	0.7717	0.7642
19	0.9555	0.9300	0.9123	0.9028	0.8966	76	0.8416	0.7800	0.7535	0.7403	0.7320
18	0.9748	0.9600	0.9453	0.9367	0.9315	75	0.8164	0.7500	0.7227	0.7093	0.7011
17	0.9940	0.9900	0.9782	0.9718	0.9673	74	0.7905	0.7200	0.6919	0.6782	0.6703
16	1.0118	1.0200	1.0125	1.0073	1.0032	73	0.7635	0.6900	0.6619	0.6480	0.6395
15	1.0286	1.0500	1.0469	1.0437	1.0413	72	0.7360	0.6600	0.6315	0.6178	0.6095
14	1.0446	1.0800	1.0819	1.0813	1.0798	71	0.7076	0.6300	0.6018	0.5880	0.5846
13	1.0597	1.1100	1.1174	1.1196	1.1202	70	0.6788	0.6000	0.5717	0.5586	0.5506
12	1.0732	1.1400	1.1538	1.1592	1.1615	69	0.6490	0.5700	0.5424	0.5292	0.5211
11	1.0864	1.1700	1.1911	1.2001	1.2045	68	0.6187	0.5400	0.5131	0.5002	0.4925
10	1.0977	1.2000	1.2293	1.2421	1.2494	67	0.5878	0.5100	0.4837	0.4712	0.4639
9	1.1087	1.2300	1.2683	1.2866	1.2966	66	0.5564	0.4800	0.4544	0.4426	0.4354
8	1.1170	1.2600	1.3091	1.3328	1.3465	65	0.5242	0.4500	0.4254	0.4140	0.4073
7	1.1263	1.2900	1.3510	1.3813	1.3990	64	0.4917	0.4200	0.3968	0.3859	0.3791
6	1.1330	1.3200	1.3946	1.4332	1.4562	63	0.4587	0.3900	0.3678	0.3577	0.3515
5	1.1367	1.3500	1.4408	1.4892	1.5184	62	0.4252	0.3600	0.3392	0.3295	0.3238
4	1.1402	1.3800	1.4898	1.5500	1.5868	61	0.3912	0.3300	0.3106	0.3018	0.2966
3	1.1439	1.4100	1.5428	1.6190	1.6662	60	0.3568	0.3000	0.2823	0.2740	0.2689
2	1.1476	1.4400	1.6018	1.6990	1.7615	55	0.1806	0.1500	0.1406	0.1364	0.1338
1	1.1510	1.4700	1.6719	1.8016	1.8893	50	0.0000	0.0000	0.0000	0.0000	0.0000

mean estimator into the PWL estimate is simply to substitute it for the sample mean in the calculation of the quality index. This approach, referred to as method 2, can be written as follows:

$$Q_L = (\hat{\mu}_{EB} - L)/s \tag{23}$$

or

$$Q_U = (U - \hat{\mu}_{EB})/s \tag{24}$$

where $\hat{\mu}_{EB}$ is the EB estimator for daily population mean and Q_L, Q_U, L, U, and s are as described before.

The next logical step in developing a Bayes quality-index approach is to extend the concept of pooling preceding information with current sample results to the estimate of the variability of the material. Because in the development of the EB mean estimator it was assumed that the process had a constant variance, the estimate for the process variance ($\hat{\sigma}^2$), as defined in Equation 5, should be an improvement on the use of the sample standard deviation (s) in the quality-index calculation. This approach, method 3, can be written as follows:

$$Q_L = (\hat{\mu}_{EB} - L)/(\hat{\sigma}^2)^{1/2} \tag{25}$$

or

$$Q_U = (U - \hat{\mu}_{EB})/(\hat{\sigma}^2)^{1/2} \tag{26}$$

where $(\sigma^2)^{1/2}$ is the estimated process variance as defined in Equation 5 and Q_L, Q_U, $\hat{\mu}_{EB}$, L, and U are as described before. Although the use of $\hat{\sigma}^2$ may not be strictly Bayesian, it is based on the same principle of pooling preceding information because it is defined as the average of all daily sample variances on the project. As long as the constant-process variance assumption is appropriate, this third method should provide a good estimate for PWL.

Normal-Distribution Approach

Another approach to estimating the PWL for the lot of material is simply to consider the process to be normally distributed and to use the daily estimates for mean and standard deviation as the parameters for the normal distribution that describes the population for the day. In this way, the PWL can be estimated by using a standardized statistic (z) and tables of the standard normal distribution to estimate the proportion of the daily population that is within the specification limits.

Two Bayes approaches can be developed for calculating the z-value to be used for estimating PWL. These approaches, methods 4 and 5, differ in the estimator that is used for the daily population standard deviation. These two methods parallel the way that variability was estimated in methods 2 and 3, which have been described previously.

Method 4 uses the daily sample standard deviation to estimate the population standard deviation along with the EB mean estimator. The calculation for z can then be described by the following:

$$Z_L = (L - \hat{\mu}_{EB})/s \quad (27)$$

or

$$Z_U = (U - \hat{\mu}_{EB})/s \quad (28)$$

where Z_L is the standardized variate for the lower specification limit; Z_U is the standardized variate for the upper specification limit; and L, U, $\hat{\mu}_{EB}$, and s are as described before.

Method 5, on the other hand, uses the pooled estimate of process variance as the estimate for the daily population variability. This can be described as follows:

$$Z_L = (L - \hat{\mu}_{EB})/(\hat{\sigma}^2)^{1/2} \quad (29)$$

or

$$Z_U = (U - \hat{\mu}_{EB})/(\hat{\sigma}^2)^{1/2} \quad (30)$$

where all terms are as described before.

VERIFICATION OF ASSUMPTIONS

In the previous section, several semiempirical Bayes estimators for PWL (EB PWL estimators) were developed. These estimators were based on three assumptions concerning the production process and the distribution of the daily population means. It was assumed that the process was normally distributed with a constant variance from day to day and that the daily population means were normally distributed. Before the EB PWL estimators can be evaluated, it is necessary to determine whether the assumptions on which they were developed are appropriate for the case of a bituminous concrete surface course. In order to verify or refute these assumptions, a number of goodness-of-fit (GOF) tests were conducted on density, asphalt content, and aggregate gradation data collected on actual paving projects. Although GOF tests were conducted on all of these properties, only density is discussed in this paper.

Data were collected from 13 bituminous concrete runway-paving projects in four states. The projects, designated A through M, and their locations and approximate tonnages placed are given in Table 2. More than 200,000 tons of bituminous concrete were placed on these projects.

Table 2. Projects from which data were collected.

Project	State	Concrete Placed (tons)
A	New York	8,210
B	Pennsylvania	2,500
C	Virginia	25,300
D	New York	10,660
E	New York	60,000
F	Pennsylvania	6,850
G	New York	3,250
H	New Jersey	6,850
I	Virginia	2,230
J	Virginia	3,050
K	Virginia	8,000
L	Virginia	33,000
M	New York	36,000
Total		205,900

Daily Population Means

The first assumption to be considered is that the daily population means are normally distributed. This assumption can readily be checked by applying a GOF test to the daily sample means from the 13 projects for which data are available to see whether they can be assumed to follow a normal distribution. Three commonly employed GOF tests that were considered for the analysis included the chi-square (χ^2), Kolmogorov-Smirnov (K-S), and Cramer-von Mises (CvM) tests. The χ^2-test requires a relatively large sample size and was therefore not appropriate in this instance because the largest number of days on any project was only 26. The K-S test is a widely accepted general-purpose GOF test. The CvM test is particularly appropriate for small sample sizes, which was the case in this analysis.

The K-S and CvM tests were implemented by means of a computer program, GOF, written by Don T. Phillips of Purdue University (14). The program, which had to be modified to run on the compiler that was being used, is capable of conducting χ^2, K-S, and CvM GOF tests on up to 500 sample observations.

Table 3. K-S GOF results for assumption of normal distribution for daily population density.

Project	Degrees of Freedom	K-S Test Statistic	Critical Value ($\alpha = 0.05$)	Decision
A	15	0.1375	0.220	@
B	6	0.2062	0.319	@
C	19	0.1127	0.195	@
D	17	0.1530	0.206	@
E	12	0.1809	0.242	@
F	10	0.1972	0.258	@
G	23	0.1060	0.184	@
H	10	0.2092	0.258	@
I	13	0.1003	0.234	@
J	9	0.1690	0.271	@
K	21	0.0959	0.188	@
L	26	0.1146	0.176	@
M	18	0.1572	0.200	@

Note: @ = do not reject normality assumption.

The results of the K-S tests on daily population density are given in Table 3. The critical values for a 5 percent level of significance ($\alpha = 0.05$) given in the table are taken from a paper by Lilliefors (15). These values, which were determined by Monte Carlo methods, are appropriate for the case where the mean and standard deviation for the theoretical distribution to be tested are determined from the sample observations. As can be seen from an examination of Table 3, there are no instances in which the normality assumption can be rejected at the $\alpha = 0.05$ level. This provides a solid argument in favor of the validity of the normal distribution assumption for daily population means.

The results of the CvM tests on daily population density are given in Table 4. The critical values for $\alpha = 0.05$ shown in the table are taken from the monograph by Phillips (14). Once again, there are no instances in which the normality assumption can be rejected.

Distribution of Production Process

The next assumption to be considered is that the process from which the daily samples are drawn is normally distributed. The logical way to test this assumption is to conduct GOF tests on the individual daily test results to determine whether they can be assumed to follow a normal distribution. This approach is not feasible because of the small number

Table 4. CvM GOF results for assumption of normal distribution for daily population density.

Project	Project Days	CVM Test Statistic	Critical Value (α = 0.05)	Decision
A	15	0.1009	0.461	@
B	6	0.0602	0.461	@
C	19	0.0782	0.461	@
D	17	0.1003	0.461	@
E	12	0.0620	0.461	@
F	10	0.0792	0.461	@
G	23	0.2029	0.461	@
H	10	0.2700	0.461	@
I	13	0.0698	0.461	@
J	9	0.0491	0.461	@
K	21	0.2974	0.461	@
L	26	0.0674	0.461	@
M	18	0.1306	0.461	@

Note: @ = do not reject normality assumption.

Table 5. K-S GOF results for assumption of normal distribution for density residuals.

Project	Degrees of Freedom	K-S Test Statistic	Critical Value (α = 0.05)	Decision
A	60	0.0996	0.114	@
B	24	0.0746	0.182	@
C	73	0.0843	0.104	@
D	65	0.0690	0.110	@
E	36	0.0623	0.148	@
F	30	0.1093	0.161	@
G	69	0.1072	0.1067	X
H	40	0.0807	0.140	@
I	91	0.0850	0.093	@
J	34	0.0760	0.152	@
K	84	0.0894	0.097	@
L	91	0.0473	0.093	@
M	68	0.0823	0.107	@

Note: @ = do not reject normality assumption; X = reject normality assumption.

of tests (usually four) per day. This sample size is too small for even the CvM test to provide meaningful results. To conduct a GOF test it was therefore necessary to pool, or group, the individual daily test results for all days on the project.

When the data were pooled from day to day, it was not possible simply to combine the daily tests, because each day had a different sample mean. To eliminate the day-to-day variation in sample means, instead of pooling the actual test values, the daily residuals were combined to form one large sample for each project. The residuals were defined as the difference between the individual test results for each day and the daily sample mean for the respective day. The procedure for determining the residuals can be illustrated in equation form as follows:

$$r_{ij} = (x_{ij} - \bar{X}_i) \tag{31}$$

where

x_{ij} = test result number j for day i,
$\bar{X}_i$ = daily sample mean for day i, and
r_{ij} = residual number j for day i.

If the production process from which the individual daily test samples were drawn is normal, these i x j residuals should be normally distributed with mean equal to zero.

K-S tests were conducted on the residuals for each project to determine whether they could be assumed to follow a normal distribution. Because of the larger sample sizes for the residuals, CvM tests were not conducted. The results of the K-S test for normality on the individual test residuals for density are shown in Table 5. Once again, the critical values for α = 0.05 are from Lilliefors (15). With the exception of one project, the normality assumption cannot be rejected at the α = 0.05 level. This is convincing evidence of the appropriateness of the assumption that the production process is normally distributed.

Assumption of Constant Process Variance

The final assumption to be addressed is that of a constant process variance from day to day throughout the project. To determine the appropriateness of this assumption it is necessary to test the equality of the sample variances from each of the project days. Bartlett's test is most often used (16) to test homogeneity of variances for random samples drawn from several populations. This test is based on a statistic whose sampling distribution approximates a chi-square when the random samples are drawn from independent normal populations. The normality assumption has already been addressed in the preceding paragraphs.

The form of the Bartlett test used to evaluate the assumption of constant process variance is that presented by Neter and Wasserman (17). The procedure consists of determining a pooled estimate for process variance (s_p) by using the following:

$$s_p^2 = [1/(n_T - k)] \sum_{i=1}^{k} (n_i - 1)s_i^2 \tag{32}$$

where

s_p^2 = pooled estimate for process variance,
s_i^2 = k-sample variances,
n_i = sample size for k-project days,
k = number of sample variances, and

$$n_T = \sum_{i=1}^{k} n_i$$

Once the pooled variance estimate has been determined, the test statistic (B) can be determined from the following:

$$B = (2.302585/C)\left[(n_T - k)\log s_p^2 - \sum_{i=1}^{k} (n_i - 1)\log s_i^2\right] \tag{33}$$

where

$$C = 1 + [1/3(k-1)]\left(\left\{\sum_{i=1}^{k} [1/(n_i - 1)]\right\} - [1/(n_T - k)]\right) \tag{34}$$

The test statistic is a value of a random variable that approximately follows a chi-square distribution with k - 1 degrees of freedom.

A simple FORTRAN program was written to calculate the values of the Bartlett test statistics given in Table 6 for density. The critical values given in the table were determined from the appropriate chi-square distribution for the 5 percent level of significance (α = 0.05).

The constant-variance assumption was rejected on 4 of 13 projects for density. Although the constant-variance assumption did not fare as well as the two normality assumptions tested by the K-S and CvM procedures, based on the results given in Table 6 it still appears to be a reasonable assumption.

COMPUTER SIMULATION

The performance of the EB estimator for PWL can be evaluated against the traditional method by means of computer simulation. An extensive computer simula-

Table 6. Results of Bartlett test for constant variance on density.

Project	Pooled Variance	Test Statistic	Degrees of Freedom	Critical Value ($\alpha = 0.05$)	Decision
A	0.8278	18.8862	14	23.68	@
B	2.2600	3.1491	5	11.07	@
C	2.4903	25.3461	18	28.87	@
D	1.3883	11.0024	16	26.30	@
E	0.333	9.9718	11	19.68	@
F	0.3107	20.3424	9	16.92	X
G	0.5200	34.0504	22	33.92	X
H	0.7799	6.3637	9	16.92	@
I	0.1255	34.3288	12	21.03	X
J	0.5570	13.3109	8	15.51	@
K	1.4526	37.9786	20	31.41	X
L	0.5676	19.9066	25	37.64	@
M	0.4250	12.2958	17	27.59	@

Note: @ = do not reject normality assumption; X = reject normality assumption.

tion analysis was conducted with the five methods for estimating PWL that were described previously. A detailed description and discussion of this simulation analysis have been made (13), and a paper detailing this analysis is also currently in preparation. A brief description of the basic simulation procedure used is presented in this paper along with some of the results of the analysis.

Simulation Design

Each simulation run consisted of 3,000 project days. Because paving projects are typically of relatively short duration, the 3,000 project days were made up of 100 projects of 30 days' duration each. In the simulation the values for the daily population means (μ_i) were generated from a (μ_p, σ_p^2) normal distribution. Values of σ^2 were held constant for a given project but allowed to vary among projects. For each simulation, however, the ratio of the variance of the daily sample means (σ^2/n) to the preceding variance (σ_p^2) was held constant for all projects. The ratio [$(\sigma^2/n)/\sigma_p^2$] was chosen to be 2.0, 1.0, or 0.5. A value of 2.0 would favor the EB estimator because it meant that the preceding distribution had a smaller variance than did the daily sample means. Similarly, a value of 0.5 for this ratio favored the traditional method for estimating PWL because it meant that the preceding distribution was more variable than were the daily sample means.

In each simulation conducted, three approaches, designated approaches A, B, and C, were used to establish the initial estimate for the preceding distribution to be used. The approaches were

1. The use of 20 earlier production days to establish a preceding production history for the contractor;
2. The use of the first five project production days to establish the precedent for the project in question; in this approach, $\bar{X}_i$ and s_i were used to estimate PWL in the traditional manner for the first five project days, and then the EB PWL estimators were used for days 6 through 30; and
3. No knowledge concerning the contractor's preceding production was assumed; $\bar{X}_i$ and s_i were used on project days 1 and 2, and then EB PWL estimators were used.

Evaluation Procedure

A number of different measures for evaluating the performance of the EB PWL estimators (methods 2-5)

Table 7. Summary of results from typical computer simulation for density.

$(\sigma^2/n)/\sigma_p^2$	Approach for Establishing Preceding Distribution	Method of Estimating PWL	Avg Error for Each Project: Avg Daily PWL Error (PWL)	Avg Daily Squared PWL Error (PWL^2)
0.5	A	1	0.020	88.19
		2	1.814	76.45
		3	1.645	52.30
		4	1.094	66.51
		5	0.792	44.20
	B	1	0.020	88.19
		2	1.550	82.21
		3	1.396	62.30
		4	0.997	74.86
		5	0.737	55.68
	C	1	0.020	88.19
		2	1.787	82.69
		3	1.633	61.60
		4	1.169	74.56
		5	0.900	54.12
1.0	A	1	-0.011	85.50
		2	1.688	64.20
		3	1.374	40.88
		4	0.944	54.42
		5	0.581	33.22
	B	1	-0.011	85.50
		2	1.357	73.92
		3	1.106	53.04
		4	0.770	65.16
		5	0.454	45.67
	C	1	-0.011	85.50
		2	1.585	72.79
		3	1.323	51.02
		4	0.928	63.23
		5	0.596	42.86
2.0	A	1	-0.007	84.40
		2	1.511	54.93
		3	1.086	30.59
		4	0.789	44.88
		5	0.380	23.54
	B	1	-0.007	84.40
		2	1.135	61.16
		3	0.827	42.06
		4	0.548	52.92
		5	0.214	34.59
	C	1	-0.007	84.40
		2	1.321	58.72
		3	0.998	39.06
		4	0.670	49.70
		5	0.313	30.73

with respect to the traditional approach (method 1) were employed in the analysis. Two will be considered here.

For each project day, the correct PWL value for the daily population--call this p_i--was calculated from the known μ_i- and σ^2-values. By using computer simulation, it was possible to obtain the correct value of p_i, a luxury that is not possible in an actual construction situation. Also, based on the daily sample, the estimated PWL--call this $\hat{p}_i$--was determined for each approach (A, B, C) and each method (1-5). The performance of each PWL estimator was then evaluated by determining the difference ($\delta_i = \hat{p}_i - p_i$) between the estimated and correct PWL values. Average values of δ_i were determined for each project and pooled for the 100 projects to determine a mean PWL error for each approach and method used in each simulation. These daily PWL errors should be distributed about zero, i.e., have a mean error equal to zero. A positive or negative mean error is indicative of some bias in the method of estimating p_i. To evaluate the variability associated with each method for estimating p_i, the squared PWL error [$\delta_i^2 = (\hat{p}_i - p_i)^2$] was also determined for each project day, and then an average squared PWL error was determined for each project and for the entire simulation for each method of estimating p_i.

Analysis of Results

The results of a typical computer simulation for density are given in Table 7 (n = 4). In this table, method 1 is the traditional quality-index approach (based on $\overline{X}_i$ and s_i) and can be used as the control against which to measure the performance of the EB PWL estimators.

A review of Table 7 indicates that the traditional estimator produces a better estimate in terms of the average PWL error (δ) because the average error is nearly zero; i.e., it is an unbiased estimator. The EB estimators, on the other hand, produce average errors that are slightly biased toward the high side. Nevertheless, the EB estimators always produce a lower average squared error than the traditional method. This means that the EB estimate for PWL has a higher likelihood of being close to the true PWL value because it has less variability associated with it. Space does not allow a detailed analysis of the results in Table 7 here. A thorough discussion of the results of the computer simulation analyses may be found elsewhere (13).

CONCLUSIONS

In this paper the steps involved in the development of an acceptance approach for bituminous pavements based on EB techniques are presented. The approach developed uses a Bayesian estimator for the percentage of the lot of material within specification limits (PWL) for determining the level of quality for the pavement. The estimator was developed on the basis of three assumptions:

1. The production process follows a normal distribution,
2. The daily population means follow a normal distribution, and
3. There is a constant process variance.

GOF tests conducted on data collected from 13 bituminous runway-paving projects verified the reasonableness of the three assumptions. Computer simulation indicated that the Bayesian estimators for PWL were slightly biased toward higher estimated PWL values but that they had a lower variance for the estimated PWL value than that obtained with the traditional quality-index method.

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Statistical Evaluation of Random Versus Stratified Random Sampling for Pavement Test Sections

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In bituminous highway and runway pavement construction, specification procedures sometimes call for the selection of test specimens from a relatively small, rectangular section of pavement. These small sections are known as control strips, test strips, or test sections. The trial pavement sections are used to verify that the mix design, plant operations, and laydown procedures can meet specification requirements or to establish target values against which to evaluate actual production paving or both. An analysis and evaluation of two sampling schemes, random sampling and stratified random sampling, for use with pavement sections are presented. In both sampling schemes, the sample mean obtained is used as the estimator of the population parameter of interest, e.g., pavement density. Because of the small size of the test section it is assumed that the sample results are correlated and that the correlation between two sampling locations decreases exponentially with distance. Equations are developed to determine the theoretical variance of the sample mean by using the two sampling schemes. It is shown that as determined by the lower sampling variance, stratified sampling is preferred over random sampling when target characteristics of pavements are obtained.

In bituminous highway and runway pavement construction, specification procedures sometimes call for the selection of test specimens from a relatively small, rectangular section of pavement. These small sections are known as control strips, test strips, or test sections. There are typically two reasons for the use of these small trial pavement sections before actual pavement construction.

The first is to establish a target value against which to measure the results achieved during the actual pavement construction. This approach is referred to as the control-strip method and is employed by several state highway agencies (1) to establish a target value for the in-place pavement density. In this method, measurements are made on a small stretch of pavement, the control strip, to determine whether specification requirements for minimum in-place density have been achieved. The results of these measurements on the control strip establish a target density against which the densities achieved during paving operations can be evaluated.

The second use for a preliminary trial pavement section is to verify that the mix design, plant operations, and laydown procedures can satisfactorily meet the requirements and tolerances set forth in the specifications before the actual paving operations begin. In this application, the trial pavement section is usually referred to as a test strip or test section. The Federal Aviation Administration Eastern Region specification (2) for bituminous surface course is an example of a specification requiring a test section before approval of the commencement of paving. The reason for the test section is presented quite clearly in the note to the engineer that accompanies the specification (2): "The test section affords the Contractor and the Engineer an opportunity to determine the quality of the mixture in place, as well as performance of the plant and laydown equipment."

OBJECTIVES

Typical practice in deciding where to take measurements on the test section is to use a random-sampling procedure to determine the sampling location. This procedure differs from that usually employed for the actual pavement construction. The sampling procedure usually employed for the in-place pavement is to take a stratified random sample. In this approach the lot of the pavement to be evaluated is divided into a number of sublots, commonly four or five, and a sample is randomly selected from within each sublot. The reason for this sampling plan is to assure that all samples are not taken from a relatively small section of the total lot but are spread over the entire pavement length. It is assumed that stratified random sampling is not required on the test section because of its relatively small size and because the material for the test section is produced at the plant during a short time frame, thereby eliminating any chance of changes in the production process that may occur through the course of a day.

Nevertheless, the small size of the test sections assures that they will be located much closer together than those during the actual pavement construction. This proximity of testing locations yields a higher likelihood of correlation between the test results. It seems reasonable to assume that two sampling locations that are close together are more likely to yield similar test results than two locations that are far apart. One procedure to reduce the possibility of correlation effects is to use a stratified random sample on the test section similar to that employed on the actual production paving.

The objective of this paper is to evaluate the performance of two sampling plans, random and stratified random, in establishing a target value from a control strip or test section.

ANALYSIS PROCEDURE

The case to be considered in the analysis is the development of a target density as the average of a number of measurements taken on a control strip or test section. The test section requirements that will be used in the analysis are those from the FAA P-401 specification (section 401-3.3, Test Section) (2):

> Prior to full production, the Contractor shall prepare a quantity of bituminous mixture according to the job mix formula. The amount of mixture should be sufficient to construct a test section at least 100 feet (30.5 m) long and two spreader widths wide. . . . Four (4) samples of finished pavement, and four (4) samples that span the longitudinal joint, shall be randomly taken and tested to determine conformance to acceptance criteria.

In practice, in accordance with specification requirements, the sample mean based on a random sample is used to develop the target density. In this paper, the sample mean, well known to be an unbiased estimator for the population mean, is used as the estimator for the target density for stratified as well as random sampling. Expressions for the variance of the estimator for each of the two sampling plans are derived and compared.

In the derivation of the expressions for the variance of the estimated mean, it is assumed that a relationship exists between the distance separating two sampling points and the correlation between the measurements taken at those points. Intuitively, it is reasonable to assume that two points that are closer together will produce measurements that are more closely related. This is because the material is more likely to have come from the same truckload and to have been compacted under similar localized subgrade and temperature conditions. Specifically, the following relationship is assumed to apply between the correlation coefficient (ρ) and the distance between two points:

$$\rho(d_{ij}) = \exp(-\alpha d_{ij}^2) \quad (1)$$

where

α = some constant,
d_{ij} = distance between specimens i and j, and
$\rho(d_{ij})$ = correlation between specimens i and j.

Figure 1. Plots of correlation coefficient between samples as a function of distance between those samples for various α-values.

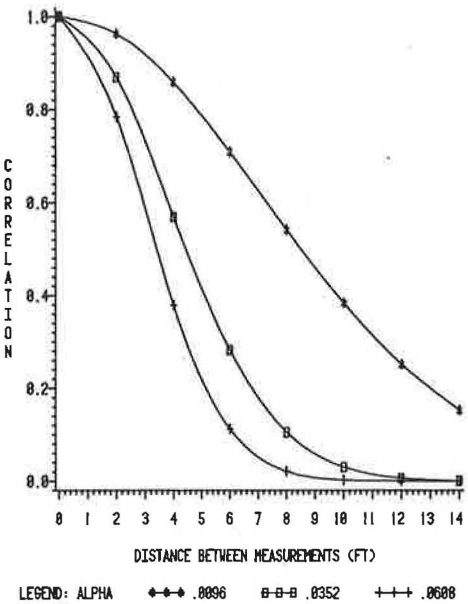
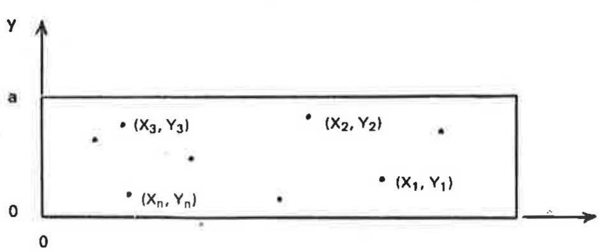


Figure 2. Random sampling of n points in n x a section of pavement.



This relationship is selected to provide a derivation for the expected value of the correlation coefficient between two points that is analytically tractable. This relationship does, however, conform to what would be expected intuitively. Equation 1 yields high correlation values at close spacings between points that decrease exponentially with increasing distance between the points. As shown in Figure 1, the decrease in the correlation coefficient with distance is established by the selection of α.

Once expressions have been derived for the variance of the estimated mean for each sampling plan, these will be applied to several test section sampling situations that may develop under the FAA P-401 specification to determine which sampling scheme provides the mean estimate with the lower sampling variance.

THEORY

Random sampling is performed by choosing n measurement locations independently over the entire area of pavement. This means that the location of the ith measurement (X_i, Y_i) can be considered a random vector with X_i and Y_i having a joint uniform distribution over the range $0 < X_i < n$ and $0 < Y_i < a$ (see Figure 2). [Note: Without loss of generality the upper bound on X_i can be scaled to be n.]

Stratified random sampling is performed by choosing n measurement locations, one location in each of n 1 x a rectangles into which the strip of pavement has been partitioned. Here each pair of X_i and Y_i has a joint uniform distribution over the range $(i - 1) \leq X_i < i$ and $0 < Y_i < a$ (see Figure 3).

The distance (D_{ij}) between any two sampling locations i and j can be expressed as a function of the random variables U_{ij} and V_{ij}, defined as follows:

$$D_{ij} = [(X_i - X_j)^2 + (Y_i - Y_j)^2]^{1/2}$$
$$= (U_{ij}^2 + V_{ij}^2)^{1/2}$$

For random sampling U_{ij} and V_{ij} have, respectively, the following probability density functions:

$$f(u) = (u + n)/n^2 \quad -n \leq u \leq 0$$
$$= (n - u)/n^2 \quad 0 \leq u \leq n$$

and

$$f(v) = (v + a)/a^2 \quad -a \leq v \leq 0$$
$$= (a - v)/a^2 \quad 0 \leq v \leq a$$

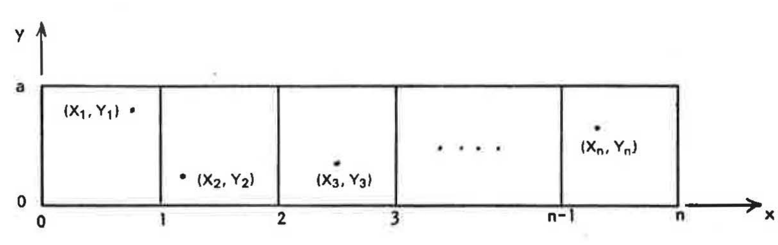
For stratified random sampling for two points separated by p blocks, U_{ij} has the following probability density:

$$h(u) = u - p \quad p \leq u \leq p + 1$$
$$= p + 2 - u \quad p + 1 \leq u \leq p + 2$$

V_{ij} has probability density g(v) as defined above.

At each location (X_i, Y_i) some measurement of pavement quality (S_i) is taken. Assume that the S_i

Figure 3. Stratified sampling of n points in n partitions of n x a section of pavement.



are distributed with mean μ and variance σ^2. For both sampling plans the sample mean $(\bar{S})$ is to be used to estimate μ. Because $\bar{S}$ is an unbiased estimator of μ, the variance of $\bar{S}$ using random sampling $[\mathrm{Var}_r(\bar{S})]$ must be compared with the variance of $\bar{S}$ using stratified random sampling $[\mathrm{Var}_s(\bar{S})]$.

As discussed earlier, it is reasonable to assume that the S_i, $i = 1, \ldots, n$, are not independent. Furthermore, the correlation between S_i and S_j depends on the distance (D_{ij}) between (X_i, Y_i) and (X_j, Y_j). Let $\underline{D}$ represent the random vector of the $\binom{n}{2}$ distances between n locations. Thus

$$\begin{aligned}\mathrm{Var}(\bar{S}) &= E_{\underline{D}}\,[\mathrm{Var}(\bar{S} \mid \underline{D})] + \mathrm{Var}_{\underline{D}}\,[E(\bar{S} \mid \underline{D})] \\ &= E_{\underline{D}}\left\{(1/n^2)\left[\sum_{i=1}^{n} \mathrm{Var}(S_i|\underline{D}) + 2\sum_{j=2}^{n}\sum_{i=1}^{j-1} \mathrm{Cov}(S_i, S_j|\underline{D})\right]\right\} + 0 \\ &= E_{\underline{D}}\left\{(1/n^2)\left[n\sigma^2 + 2\sum_{j=2}^{n}\sum_{i=1}^{j-1} \mathrm{Cov}(S_i, S_j|\underline{D})\right]\right\} \\ &= (1/n^2)\left[n\sigma^2 + 2\sum_{j=2}^{n}\sum_{i=1}^{j-1} E_{D_{ij}}\mathrm{Cov}(S_i, S_j|D_{ij})\right] \qquad (2)\end{aligned}$$

because the covariance between S_i and S_j depends only on the distance between their sampling locations. In the random-sampling case the D_{ij} have the same distribution for all i and j, $i \neq j$. Denoting $\mathrm{Cov}(S_i, S_j|D_{ij}) = \sigma^2\rho(D)$, Equation 2 becomes

$$\mathrm{Var}_r(\bar{S}) = (\sigma^2/n)\left\{1 + (n-1)E[\rho(D)]\right\} \qquad (3)$$

In the stratified case, let $\mathrm{Cov}(S_i, S_j|D_{ij}) = \sigma^2\rho(D_p)$, where $\rho(D_p)$ is the correlation between measurements in blocks that are separated by p other blocks. In this case, Equation 2 becomes

$$\mathrm{Var}_s(\bar{S}) = (\sigma^2/n)\left\{1 + (2/n)\sum_{i=0}^{n-2}(n-1-i)E[\rho(D_i)]\right\} \qquad (4)$$

As discussed earlier (Equation 1), it is assumed that the correlation between measurement S_i and S_j is

$$\rho(d_{ij}) = \exp(-\alpha d_{ij}^2)$$

For a random distance D_{ij}

$$\begin{aligned}E[\rho(D_{ij})] &= E[\exp(-\alpha D_{ij}^2)] \\ &= E\{\exp[-\alpha(U_{ij}^2 + V_{ij}^2)]\} \\ &= E[\exp(-\alpha U_{ij}^2)\exp(-\alpha V_{ij}^2)] \qquad (5)\end{aligned}$$

For random sampling Equation 5 becomes

$$\begin{aligned}E[\rho(D)] &= \int_{-a}^{a}\int_{-n}^{n} \exp(-\alpha u^2)\exp(-\alpha v^2)f(u)g(v)\,du\,dv \\ &= (4/a^2n^2)\{n(\pi/\alpha)^{1/2}\,\Phi[n(2\alpha)^{1/2}] + (1/2\alpha)[\exp(-\alpha n^2) - 1]\} \\ &\quad \times \{a(\pi/\alpha)^{1/2}\Phi[a(2\alpha)^{1/2}] + (1/2\alpha)[\exp(-\alpha a^2) - 1]\} \qquad (6)\end{aligned}$$

where

$$\Phi(x) = \int_0^x [1/(2\pi)^{1/2}]\,\exp[-(t^2/2)]\,dt$$

For the stratified case Equation 5 becomes

$$\begin{aligned}E[\rho(D_p)] &= \int_{-a}^{a}\int_{p}^{p+2} \exp(-\alpha u^2)\exp(-\alpha v^2)h(u)g(v)\,du\,dv \\ &= (2/a^2)\{a(\pi/\alpha)^{1/2}\,\Phi[a(2\alpha)^{1/2}] + (1/2\alpha)[\exp(-\alpha a^2) - 1]\} \\ &\quad \times \{(1/2\alpha)\exp[-\alpha(p+2)^2] - (1/\alpha)\exp[-\alpha(p+1)^2] \\ &\quad + (1/2\alpha)\exp(-\alpha p^2) + (p+2)(\pi/\alpha)^{1/2}\,\Phi[(p+2)(2\alpha)^{1/2}] \\ &\quad - (2p+2)(\pi/\alpha)^{1/2}\,\Phi[(p+1)(2\alpha)^{1/2}] + p(\pi/\alpha)^{1/2}\,\Phi[p(2\alpha)^{1/2}]\} \qquad (7)\end{aligned}$$

If the limit is taken as $a \to 0$ in Equations 6 and 7, the expected correlation between measurements is obtained when sampling is done along a straight line. Because

$$\lim_{a\to 0}(2/a^2)\left\{a(\pi/\alpha)^{1/2}\,\Phi[a(2\alpha)^{1/2}] + (1/2\alpha)[\exp(-\alpha a^2) - 1]\right\} = 1$$

the expected correlations for random and stratified sampling along a straight line are, respectively,

$$E[\rho(D)] = (2/n^2)\left\{n(\pi/\alpha)^{1/2}\,\Phi[n(2\alpha)^{1/2}] + (1/2\alpha)[\exp(-\alpha n^2) - 1]\right\} \qquad (8)$$

and

$$\begin{aligned}E[\rho(D_p)] &= (1/2\alpha)\exp[-\alpha(p+2)^2] - (1/\alpha)\exp[-\alpha(p+1)^2] \\ &\quad + (1/2\alpha)\exp(-\alpha p^2) + (p+2)(\pi/\alpha)^{1/2}\,\Phi[(p+2)(2\alpha)^{1/2}] \\ &\quad - (2p+2)(\pi/\alpha)^{1/2}\,\Phi[(p+1)(2\alpha)^{1/2}] + p(\pi/\alpha)^{1/2}\,\Phi[p(2\alpha)^{1/2}] \qquad (9)\end{aligned}$$

Using Equations 6, 7, 8, and 9 appropriately in Equations 3 and 4 yields the necessary variances for comparing the effectiveness of the sample mean as an estimator for stratified sampling as opposed to random sampling. Such comparisons are discussed by example in the remaining sections.

APPLICATION TO TYPICAL TEST SECTIONS

Equations 3 and 4 are general in nature. In this section, these equations are applied to several test sections that are typical of those that may be encountered under the FAA specification. Two test sections, one 25 ft wide and 100 ft long (section 1) and one 25 ft wide and 200 ft long (section 2), are considered. These correspond to ratios of n to a of 4:1 and 8:1, respectively. In addition, the joint-sampling requirement for the two test sections is also considered by taking the case where a goes to zero (Equation 8). This provides the solution for a line that corresponds to the joint in the test section. A discussion follows of the results for each case considered.

Figure 4 shows the geometry of test section 1. For the specification sample size of 4, the test section is divided into four 25 x 25-ft segments for stratified sampling (Figure 5). The results of the analysis are presented in Table 1 for α-values of 0.0096, 0.0352, and 0.0608. Without loss of generality, assuming $\sigma^2 = 1$, the data in Table 1 show that the variance of the sample mean $(\sigma_{\bar{x}}^2)$ is always smaller for stratified sampling than for random sampling. Thus, statistically, stratified sampling yields a better estimator than random sampling.

Table 1 also includes the results of the analysis on test section 2. Section 2 is 25 x 200 ft (see Figure 6) and is subdivided into four 50-ft segments for stratified sampling (Figure 7). As with test

Figure 4. Geometry of test section 1.

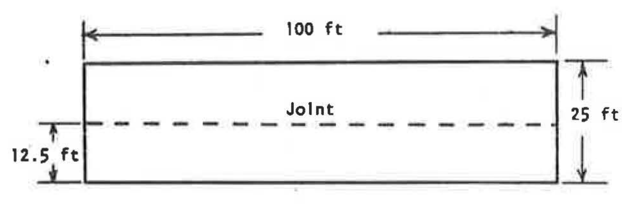


Figure 5. Stratified sampling subsections for test section 1.

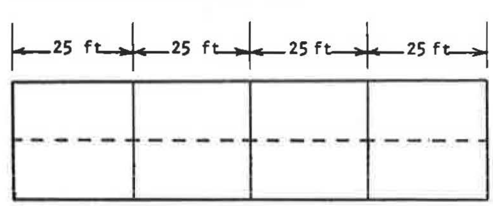


Table 1. Variance of sample mean for selected pavement sections and longitudinal joints.

	Test Sections		Longitudinal Joints	
α	Random Sampling	Stratified Sampling	Random Sampling	Stratified Sampling
Test Section 1				
0.0096	0.3212	0.2674	0.3779	0.2812
0.0352	0.2728	0.2528	0.3187	0.2585
0.0608	0.2638	0.2519	0.3027	0.2549
Test Section 2				
0.0096	0.2867	0.2544	0.3159	0.2578
0.0352	0.2616	0.2507	0.2849	0.2521
0.0608	0.2570	0.2503	0.2766	0.2512
Roadway Section				
0.0096	0.252512	0.250019		
0.0352	0.250784	0.250003		
0.0608	0.250469	0.250001		

Figure 6. Geometry of test section 2.

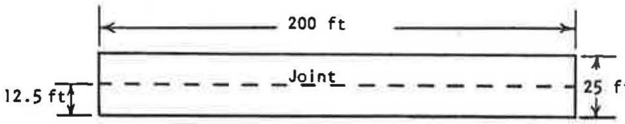
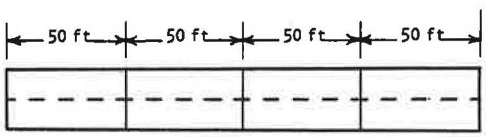


Figure 7. Stratified sampling subsections for test section 2.



section 1, stratified sampling produces a smaller variance of the sample mean for all three α-values considered.

The joint-sampling results (a = 0) for both test sections are also given in Table 1. Once again, in all instances stratified sampling yielded a lower variance of the sample mean than did random sampling.

APPLICATION TO PRODUCTION PAVING

Equations 3 and 4 can also be used to evaluate the mean estimate obtained from stratified random-sampling measurements on the in-place pavement against that obtained from a simple random-sampling plan. Because the width of the pavement, usually up to 24 ft, is much less than the length of paving in the lot, often in the thousands of feet, the production paving case approaches the case where a equals zero that was presented previously.

To evaluate the mean estimate, the variance of the estimate using random and stratified random sampling on a pavement section 25 ft wide and 3,000 ft long will be considered. The results of this analysis are given in Table 1. For this case, the values are much closer together, but stratified sampling still produces a lower sampling variance for all three α-values. The stratified sampling in this case produces sampling variances that closely approach the value for $\sigma_{\bar{x}}^2/\sigma^2$ of 2.25 that is the value when there is no correlation between the sample results. [Note: $\sigma_{\bar{x}}^2 = \sigma^2/n$; therefore $\sigma_{\bar{x}}^2/\sigma^2 = 0.25$ when n = 4.]

CONCLUSION

It has been shown by using an intuitively appealing relationship between correlation and distance that the sample mean is statistically a better estimator when stratified sampling is used than when random sampling is used. It is reasonable that this would be true for any such relationship between correlation and distance that damps exponentially. Practically, because stratified sampling does not result in extra cost, it is preferred over random sampling when target characteristics of pavement are obtained.

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Precision of the Maximum-Density Estimate in Control-Strip Specifications

RICARDO T. BARROS

One method of controlling embankment densification refers to the maximum density observed in a local control strip. The local maximum is the standard against which all mainline densities are compared, yet research to date has not addressed the precision of what is truly an *estimated* maximum density. The precision of the maximum-density estimate is quantified by simulating typical rolling procedures and density-growth curves with pertinent parameters, such as the decision rule and sample size, controlled in a factorial design. Analysis of the simulation results leads to the following general findings: (a) The sampling plan with the same locations and correlated comparisons is the most efficient of the four plans investigated, and (b) by employing this plan, the true relative density will be greater than 90 percent about 95 percent of the time.

The embankment-densification process is sufficiently indeterminate to warrant flexible requirements in compaction specifications. These specifications require that maximum density be achieved during the construction process, not under operational loads. The precise value of the maximum density, however, is an elusive quantity. Maximum-density values vary between soils, as does the compactive effort necessary to achieve them.

Control-strip specifications afford the flexibility required by variable soil response characteristics. In these specifications, a pilot section, or control strip, is constructed and closely monitored during its densification. Successive passes with a roller are made, and when the incremental density change between any two passes drops to some predetermined small difference, compaction stops. The resultant density is declared the maximum. Other density measurements taken elsewhere in the project are evaluated in light of this relative maximum until conditions change, in which case a new control strip is created.

Another consideration, essential to the performance of control-strip specifications, has apparently been overlooked. The reference density accepted as the maximum is really an estimate. If this estimate were to be low, all other relative comparisons could be adversely affected. The principal objective of this analysis was to determine the precision of these maximum-density estimates. This was done primarily through computer simulation.

A second objective was to determine the optimum sampling strategy. Several specific questions addressed include the following:

1. Should the same locations be repeatedly sampled between successive passes of a roller or should different locations be randomly selected?
2. What effect would reuse of a sampled density observation have on the sampling plan's overall effectiveness? This situation would occur if a density value used to gauge the effect of a roller pass were to be recycled into the assessment of the effect of the next pass.
3. To what degree does the sample size affect the precision of the maximum-density estimate?

A final objective was to investigate the impact of various decision rules. The decision rule establishes the largest density change for which rolling can stop and is not necessarily zero. Previous field applications have been ambiguous in this regard, leaving the actual decision rule to the inspector's discretion.

DENSITY-GROWTH CURVE AND ROLLING DISTRIBUTION

Field experience indicates that the sampled density increases are large at first and then gradually become smaller. In actual practice, the final density change may be negative. This apparent density decrease may be attributable to two causes: (a) a true density increase did occur but was not detected due to sampling and testing error, or (b) tightly interlocked particles were actually loosened by the rolling process, which increased the volume and decreased the density. Additional passes would reconsolidate the material in the latter case. Points on an actual density-growth curve (1) have been plotted in Figure 1.

For this curve, the maximum average density of 138.5 pcf would be achieved after eight roller passes. Of course, this would not be known to field engineers; they would have to infer true points on the growth curve from sample estimates. One of the possible sampling plans used to estimate maximum density might read as follows:

Select three random locations within the control-strip boundaries. By using a nuclear gauge, measure the density at each location before and after each pass of the roller. If the average density after the roller's pass is greater than the average density before, select another three random locations and repeat this procedure. Rolling should stop only when a decrease in the average density is observed for the current three locations. The maximum average density is then defined as the largest average density achieved by this procedure.

The effectiveness of this sampling plan may be assessed through computer simulation. With the growth curve shown in Figure 1, 1,000 applications of this procedure were simulated. The number of passes required to estimate maximum density was recorded for each simulation, which produced the frequency histogram shown in Figure 2. The average number of passes made was approximately 7 in this simulation. The overall average density when compaction stopped was 137.0 pcf, which yielded a relative density of approximately 98 percent.

There was some dispersion about this average value, as can be inferred from Figure 2. For example, 13 percent of the simulations stopped on or before the fourth pass, which corresponds to a relative density of about 93 percent. At the other extreme, 11 percent of the applications required nine or more passes. Thus, for this combination of growth curve and sampling plan, there is a 13 percent risk of stopping compaction at 93 percent relative density or less and a similar risk of requiring an excessive amount of compaction.

Although this type of information would be of great value to specification designers, this specific information is meaningful only to those dealing with similar growth curves and sampling plans. Density-growth curves are highly variable between embankment materials, which raises the question of this sampling plan's more general operating characteristics. A sensitivity analysis of these characteristics will be presented after the simulation procedure and its underlying theory have been developed.

SIMULATION MODEL

An exponential equation was selected to model the density-growth curve. Although the simulation is relatively insensitive to the precise mathematical function used, provided its shape is reasonably correct, the exponential curve affords certain conveniences. Density increases behave similarly to those observed in the field, the curve is easily fitted to specific points, and the average maximum-density plateau is reflected in the theoretical asymptote.

Three sources of variability contribute to any measured density value. These are the variability of the virgin material, the testing variability, and the variability introduced by the rolling process. Of these, only the first two have been experimentally identified; the roller variability must be inferred from empirical observations. Fortunately, this is easily accomplished by a soil-variance anal-

Figure 1. Typical density-growth curve.

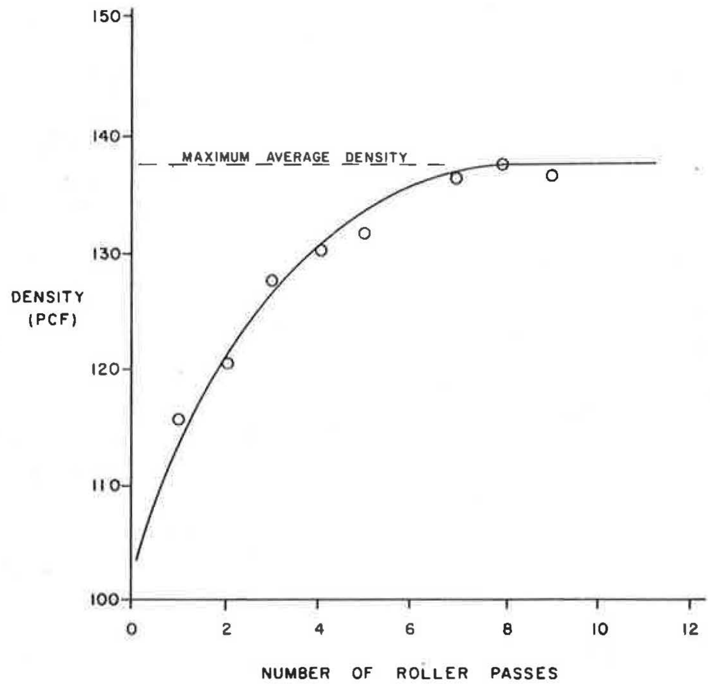
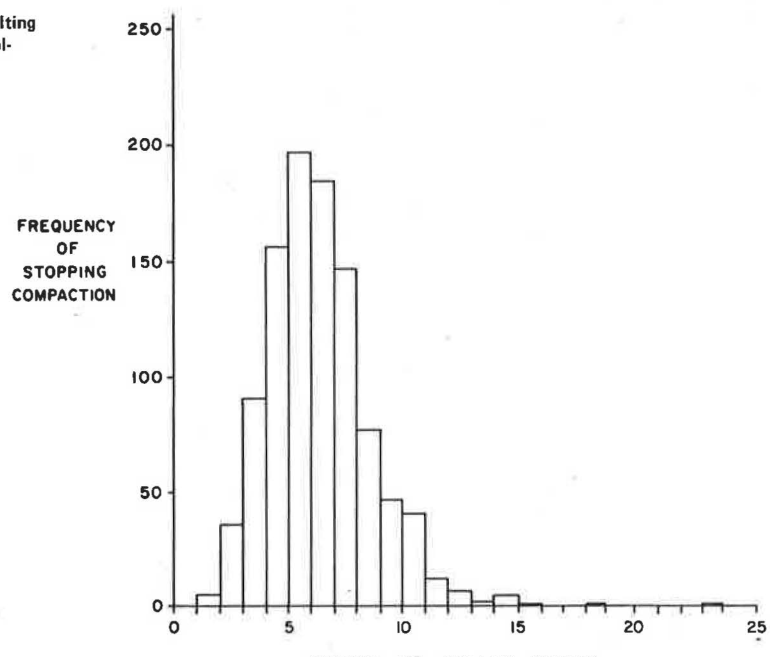


Figure 2. Frequency histogram resulting from computer simulation of control-strip procedure.

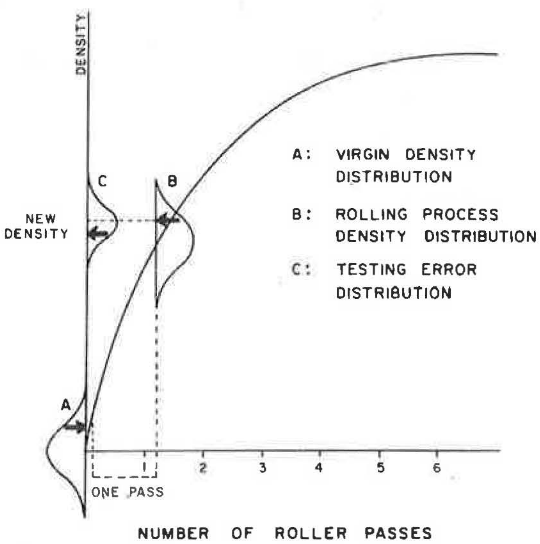


ysis. The total variability after maximum density has been reached is known, as is the shape of the growth curve. A trial-and-error process may be used to determine the magnitude of roller variability that must be used in the model to match the final dispersion observed in the field. Typical standard deviations found for the virgin material, roller variability, and testing error are 3.0, 2.5, and 2.0 pcf, respectively.

Figure 3 shows the algorithm used to simulate each successive roller pass. Individual density observations were randomly selected from a normal distribution on the Y-axis and used to find the corresponding X-values. Each X-value, representing the theoretical number of roller passes, was incremented by one unit to simulate the pass of a roller. New Y-values were then computed to determine the resultant densities. These new densities were then perturbed with a scaled random normal deviate to model the variability of the rolling procedure. The resultant densities now represent the true distribution after X passes but must be perturbed once more to model the testing error incurred in their interpretation. Finally, these transformed densities are averaged and either stored for later review or disposed of in accordance with the procedure under test.

The above procedure was repeated 1,000 times for each X-value, and the entire process was repeated for as many as 20 roller passes. Figure 4 shows several typical density distributions as the control strip is transformed along the growth curve. The distributions remain essentially normal although there is a subtle negative skewness. This skewness parallels the real-world tendency in which distributions are skewed away from a natural boundary (maximum density in this case), but the magnitude of this skewness is negligible for practical purposes. Note that the total dispersion decreases slightly with additional passes, a result of the nonlinear transformation in which low density values are increased at a faster rate than high values.

Figure 3. Simulated compaction process.



POSSIBLE SAMPLING SCHEMES

Maximum-density estimation using the control-strip technique is essentially a form of a statistical hypothesis test. Two hypotheses are made, one that maximum density has been reached and the other that further densification is possible. Only if a small density increase is observed can it be inferred that the soil is at or near its maximum density.

Three important distinctions make this particular hypothesis test unlike most of its statistical counterparts. First, the test itself ignores dispersion, because only mean values (i.e., the average densities) are computed. Second, the test is intended to be performed iteratively. Thus, due simply to chance, it is unlikely that even an unrealistically long growth curve will survive many of the

Figure 4. Typical density distributions obtained by simulation procedure.

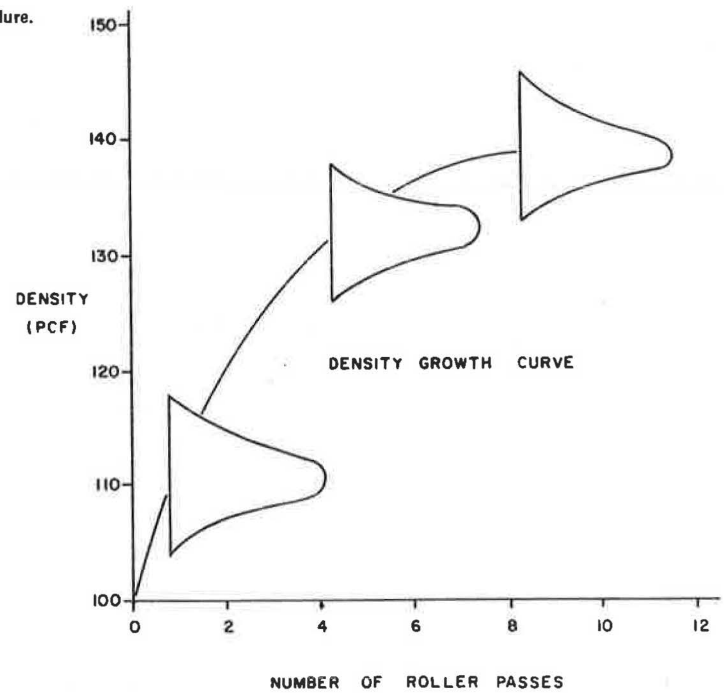
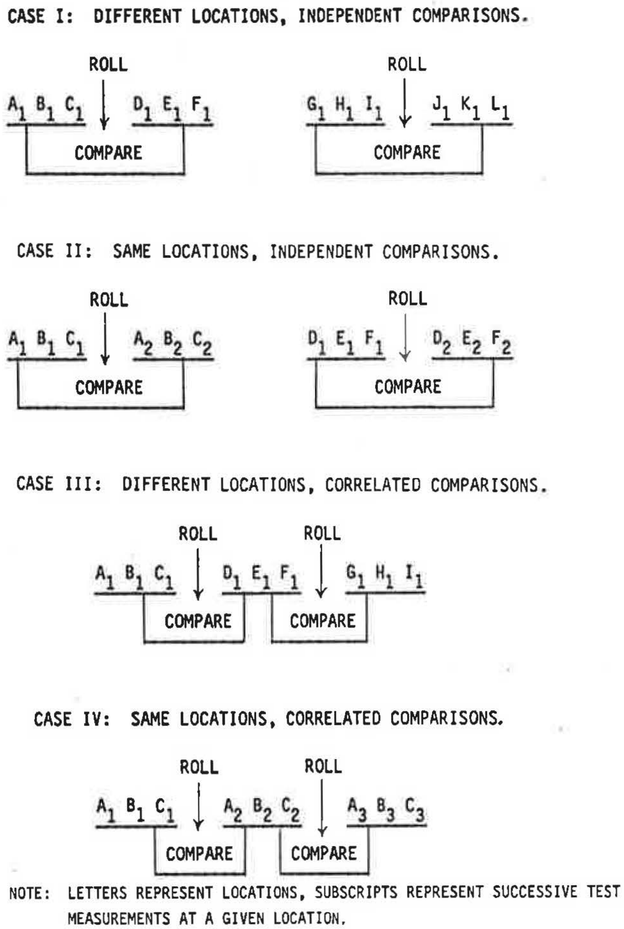


Figure 5. Four alternative sampling plans.



repeated stop-continue decisions. Finally, the sampling procedure itself may inadvertently influence the outcome. This would occur if independence were lost between any two successive comparisons. This last point is subtle and will be explained below in the analysis of alternative sampling strategies.

Three decisions must be made by an observer who wants to make sequential inferences about a density-growth curve. The first is simply the number of density measurements that will be averaged together. The second is whether the measured densities are to be paired by location before and after a pass or whether entirely different locations are to be measured. Finally, a decision must be made whether the sample averages will be used more than once in successive comparisons.

For a sample size of 3, Figure 5 shows the four possible sampling plans that result from these decisions. Each of these plans may be distinguished by the manner in which the test locations are selected and the comparison process is repeated.

Consider case I. Densities are measured at random locations A, B, and C before a pass and at random locations D, E, and F afterwards. Thus, the locations are independent from each other before and after the pass. A new set of random measurements is then made, and the entire test procedure is repeated for the next pass.

In case II the same random locations are monitored before and after each pass. The comparisons remain independent, however, because the entire procedure is replicated for each rolling sequence.

A subtle variation is introduced by the sampling plan in case III. Here locations A, B, and C are measured before the first pass, and locations D, E, and F are measured afterwards. Then, without an additional three density measurements as in case I, the second roller pass is made. Densities at new locations G, H, and I are subsequently compared with densities at previous locations D, E, and F. Although both the first and the second comparisons are individually independent, they result in a correlated test procedure because the measurements at locations D, E, and F were used twice. Thus, the outcome of the first comparison may have some influence on the outcome of the second.

The final sampling plan, case IV, simply remeasures the densities at the same locations after every pass. Although this may be a practical alternative, it most certainly compounds the correlation problem cited in case III.

One criterion by which the relative merits of these four plans may be evaluated is the sampling effort required. Note that 12 density measurements are required for two roller passes in cases I and II, but only 9 measurements are required for cases III and IV. Thus, the latter plans require a lesser sampling effort.

OPTIMUM SAMPLING STRATEGY

Figure 6 shows the operating characteristics of four distinct sampling strategies. Density increases are plotted on the X-axis and the probability of stopping compaction is plotted on the Y-axis. Note that the probability of stopping compaction, which is equivalent to the risk of a false maximum-density indication whenever a true density increase does occur, becomes larger with progressively smaller density changes. In other words, it becomes more difficult to detect density increases as the true density approaches the maximum.

It is desirable to minimize the risk of false maximum-density indications. This is done if for any given density increase, a particular sampling plan is associated with the smallest probability of stopping compaction. Figure 6 indicates that of the four plans investigated, the one with the same locations and independent comparisons is most powerful because it has the lowest operating-characteristic (OC) curve. The plan with different locations and independent comparisons is the next most powerful for small density increases.

Note the distinct impact of intercomparison correlation: The risk of prematurely stopping compaction is substantially increased near the point of maximum density. Within the subclassification of comparison type, however, the same-location sampling plan is still more powerful. (Curve IV is lower than curve III, as curve II is lower than curve I.)

A trade-off must be considered in deciding which same-location sampling plan is most efficient, the one with independent comparisons or another in which the comparisons are correlated. Plans with independent comparisons are clearly more discriminating, but they also require a larger effective sample size. For the same effective sample size, i.e., the same number of total measurements between two passes, the independent-comparison OC curve and the correlated-comparison OC curve cross so that neither is consistently lower. (The effect is similar to that of curve I crossing curve IV.) The independent-comparison OC curve is lower for small density changes, and the correlated-comparison OC curve is lower for large density changes. Under these circumstances, the net effect of the two sampling plans must be evaluated directly from the density distributions when rolling stops.

Figure 6. Comparison of sampling plans.

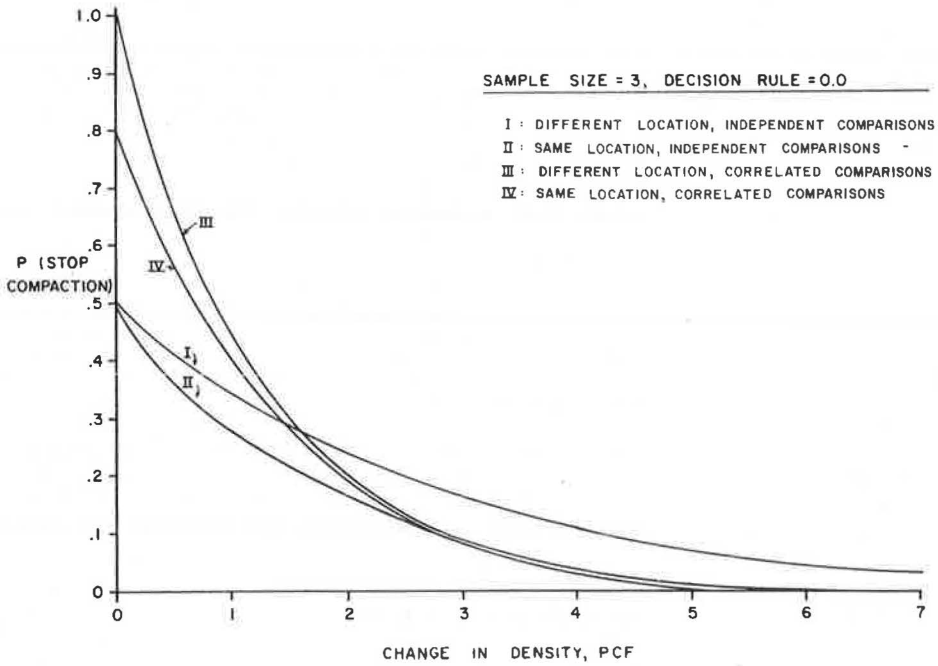
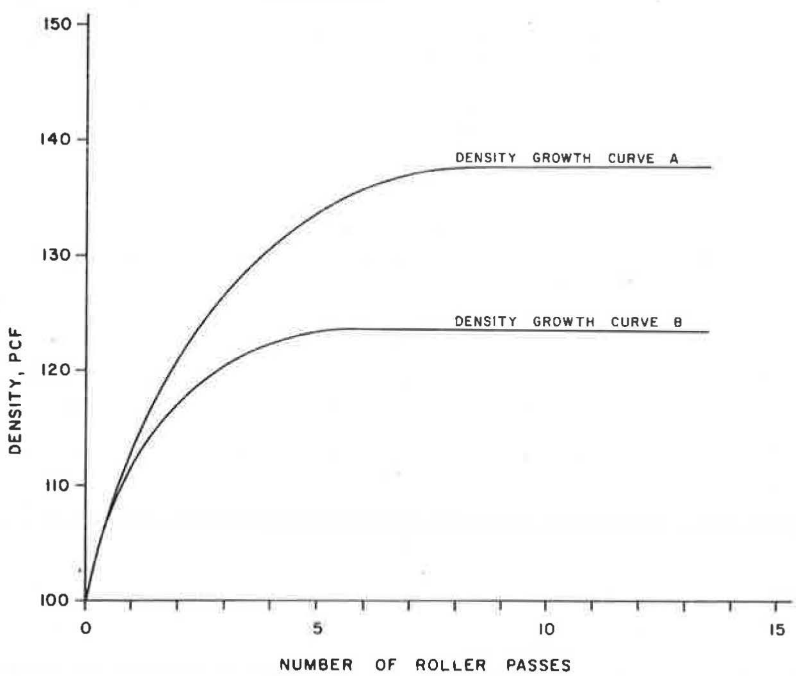


Figure 7. Two possible density-growth curves.



Simulation results indicate that for the same effective sample size, plans with correlated comparisons produce higher average relative densities. They also tend to produce density distributions with a smaller degree of dispersion. Consider two plans with an effective sample size of 4, for example. A same-location, independent-comparison plan would have a sample size of 2, and a same-location, correlated-comparison plan would have a sample size of 4. One simulation analysis revealed that the average relative densities were 97.4 percent for the correlated-comparison plan and 96.4 percent for the independent-comparison plan. The corresponding threshold densities at the 5 percent level of risk, i.e., the lower relative-density limits that are exceeded by 95 percent of the observations, were 94.0 and 91.6 percent, respectively. Clearly the correlated-comparison plan is more powerful. Further discussion will concentrate on the same-location, correlated-comparison plan because it is more efficient for the small effective sample sizes commonly used (case IV in Figure 5).

SENSITIVITY ANALYSIS

Alternative control-strip simulations were investigated in which the effect of the growth curve, the variability of individual density values about the growth curve, the sample size, and the decision rule were all controlled parameters. For each combination of these parameters, 1,000 replications were simulated. Although the individual final density distributions were fairly sensitive to these parameters, reflecting the variable nature of soil compaction characteristics, the density-change OC curves were not. This sensitivity analysis focuses on these OC curves because they are most general, but control-strip-specific results are also presented in a summary format.

Two distinct density-growth curves were considered. Figure 7 shows that growth curve A reaches a higher average maximum density than growth curve B but requires additional roller passes to do so. Although individual density values frequently exceeded these maximum average values, neither growth curve

Figure 8. Effect of density-growth function on density-change OC curve.

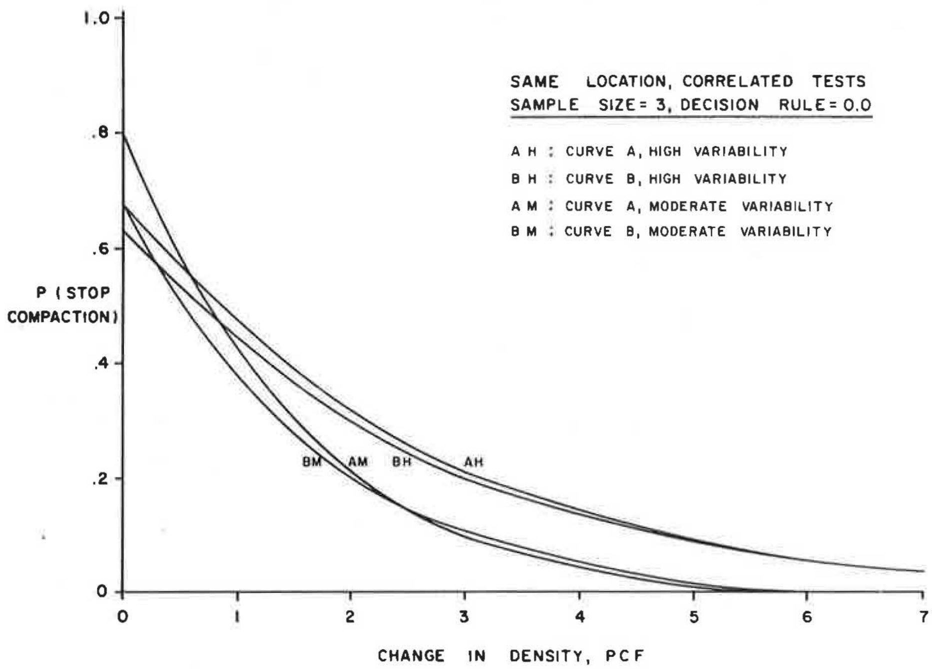
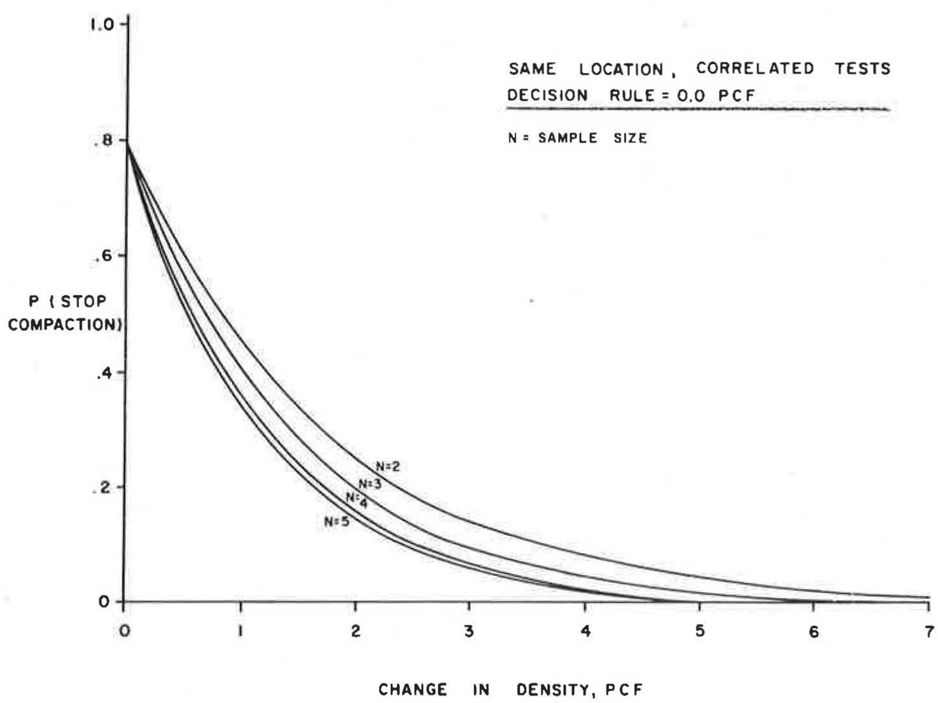


Figure 9. Effect of sample size on density-change OC curve.



exhibited a substantial increase in the average density once the theoretical asymptote was reached.

Moderate- and high-variability components were incorporated into each of the growth curves. The total initial standard deviation of 4.4 pcf, which has been used in the preceding examples, was increased to 7.3 pcf for the high-variability simulations. (The contributory standard deviations were 2.0, 5.0, and 5.0 pcf for the testing error, virgin material, and roller variability.)

Figure 8 shows the OC curves for growth curves A and B at both moderate and high levels of variability. The OC curves appear to be relatively insensitive to the shape of the density-growth curve but not to its level of variability. This will be used to advantage in an analytical approximation that is briefly discussed in the following section. Note here that growth curves with high variability tend to have a greater risk of false maximum-density estimates.

The impact of varying the sample size for a single combination of growth curve and acceptance plan

Figure 10. Effect of decision rule on density-change OC curve.

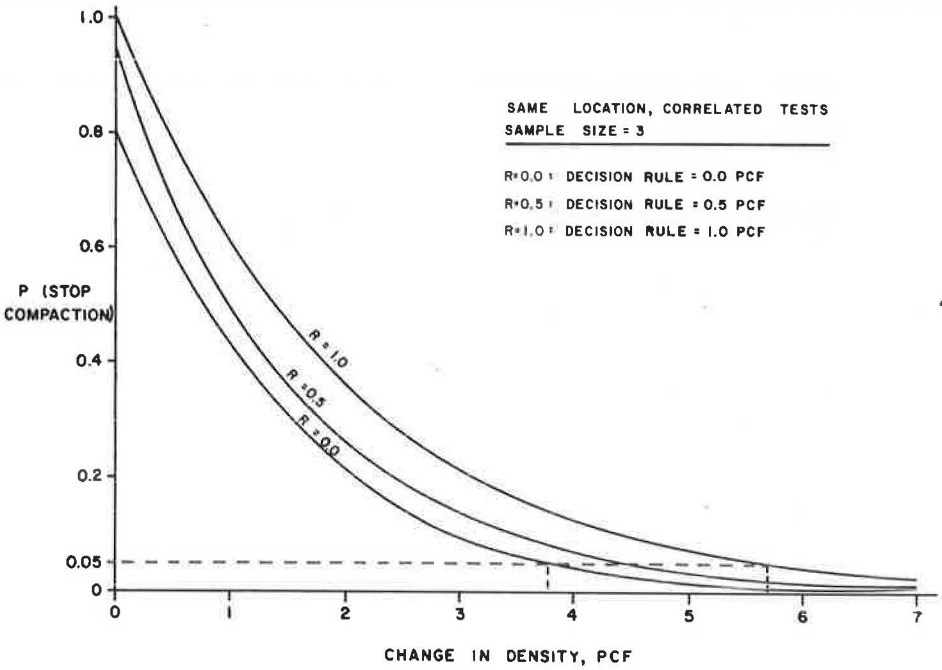


Table 1. Simulation results: paired-location, correlated-test sampling plan.

			Moderate Variability[b]			High Variability[c]		
Curve[a]	Sample Size	Decision Rule (pcf)	Expected Relative Density (%)	Relative Density at 95 Percent Confidence Limit (%)	Maximum No. of Passes at 95 Percent Confidence Limit	Expected Relative Density (%)	Relative Density at 95 Percent Confidence Limit (%)	Maximum No. of Passes at 95 Percent Confidence Limit
A	2	0.0	96.63	91.09	11	97.11	89.37	10
A	2	0.5	96.62	90.54	10	96.81	88.81	9
A	2	1.0	95.72	90.14	10	96.44	88.80	9
A	3	0.0	97.03	93.09	11	97.78	92.49	10
A	3	0.5	96.71	92.13	11	97.52	91.18	10
A	3	1.0	96.27	91.44	10	97.23	90.86	9
A	4	0.0	97.41	93.97	12	98.16	92.91	10
A	4	0.5	96.97	93.19	11	97.78	92.49	10
A	4	1.0	96.54	92.50	10	97.39	90.95	9
A	5	0.0	97.48	94.04	12	98.34	93.58	10
A	5	0.5	97.17	93.56	11	98.09	93.18	10
A	5	1.0	96.66	92.90	10	97.61	92.51	9
B	2	0.0	99.09	95.47	8	98.92	93.31	7
B	2	0.5	98.88	95.10	8	98.78	92.58	7
B	2	1.0	98.74	94.97	7	98.60	92.00	7
B	3	0.0	99.26	96.19	9	99.10	95.62	8
B	3	0.5	99.05	95.55	8	98.99	94.55	7
B	3	1.0	98.84	95.39	8	98.85	93.60	7
B	4	0.0	99.35	96.86	9	99.28	96.21	8
B	4	0.5	99.20	96.23	8	99.16	95.98	7
B	4	1.0	99.04	95.64	7	99.01	94.89	7

[a] Curves A and B in Figure 7. [b] Total initial standard deviation = 4.4 pcf. [c] Total initial standard deviation = 7.3 pcf.

Table 2. Regression coefficients for empirical OC curves.

Sample Size	Decision Rule (pcf)	Moderate Variability[a] B_1	Moderate Variability[a] B_2	High Variability[b] B_1	High Variability[b] B_2
2	0.0	0.80	-0.57	0.70	-0.32
	0.5	0.90	-0.51	0.77	-0.31
	1.0	1.00	-0.45	0.85	-0.29
3	0.0	0.80	-0.68	0.69	-0.39
	0.5	0.95	-0.62	0.77	-0.38
	1.0	1.00	-0.51	0.89	-0.38
4	0.0	0.83	-0.81	0.69	-0.48
	0.5	0.91	-0.64	0.77	-0.43
	1.0	1.00	-0.55	0.88	-0.40
5	0.0	0.76	-0.82	0.73	-0.53
	0.5	0.99	-0.74	0.80	-0.47
	1.0	1.00	-0.63	0.90	-0.42

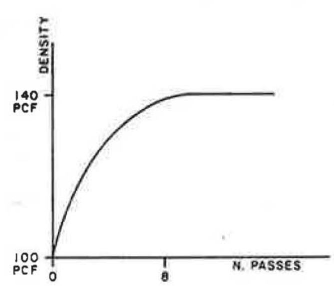
[a]Total initial standard deviation = 4.4 pcf.
[b]Total initial standard deviation = 7.3 pcf.

is shown in Figure 9. Increasing the sample size does effect an improvement, although a point of diminishing returns is soon reached. It is thought unlikely that a sample size greater than 5 would be considered practical in this application.

Nonzero decision rules tend to degrade the precision of maximum-density estimates. The three OC curves shown in Figure 10 indicate that at the 5 percent level of risk, the threshold density change increased by about 2.0 pcf as a result of increasing the decision rule from zero to 1.0 pcf. The risk increase is intuitively logical: To state it simply, rolling stops at a lower point on the density-growth curve.

These same results are reflected in Table 1, where curve-specific results are listed for several of the combinations investigated. It can be seen that the expected relative densities were generally high and generally unaffected by the sample size. Although in some cases growth curves with high variability had slightly higher expected relative densities, they also had a greater dispersion about

Figure 11. Worksheet for empirical OC curve construction.



Sample Size 3
Decision Rule 0.5 PCF
Variability Moderate
B_1 0.95
B_2 -0.62

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7
PASS NUMBER (i)	ABSOLUTE DENSITY (PCF)	RELATIVE DENSITY (%)	DELTA, CHANGE IN ABSOLUTE DENSITY (PCF)	P(STOP) ON THIS PASS $P(STOP)=B_1e^{B_2 DELTA}$	REPLICATIONS REACHING THIS PASS	REPLICATIONS STOPPING ON THIS PASS
0	100.00	71.43	-	-	1000	0
1	112.00	80.00	12.00	0.00	1000	0
2	120.00	85.71	8.00	0.01	1000	10
3	126.00	90.00	6.00	0.02	990	20
4	130.00	92.86	4.00	0.08	970	78
5	133.00	95.00	3.00	0.15	892	134
6	135.00	96.43	2.00	0.27	758	205
7	136.50	97.50	1.50	0.37	553	205
8	137.50	98.21	1.00	0.51	348	177
9	138.50	98.93	1.00	0.51	171	87
10	139.25	99.46	0.75	0.60	84	50
11	139.75	99.82	0.50	0.70	34	24
12	140.00	100.00	0.25	0.81	10	8
13	140.00	100.00	0	0.95	2	2
					Check: Column 7 Total=1000	

EXPLANATION

Columns 1 and 2: From assumed density growth curve.

Column 3 : $(\text{Col. }3)_i = (\text{Col. }2)_i/(\text{Col. }2)_{i\,=\,\text{final}}$

Column 4 : $(\text{Col. }4)_i = (\text{Col. }2)_i - (\text{Col. }2)_{i-1}$

Column 5 : B_1, B_2 from Table 2, Delta = $(\text{Col. }4)_i$

Column 6 : 1000 initially, then $(\text{Col. }6)_i = (\text{Col. }6)_{i-1} - (\text{Col. }7)_{i-1}$

Column 7 : 0 initially, then $(\text{Col. }7)_i = (\text{Col. }5)_i \times (\text{Col. }6)_i$

Expected Relative Density = 96.7%
Average of (Col. 3) weighted by (Col. 7)

Threshold Relative Density, 95% confidence limit = 93.4%
(Interpolate value in Column 3 for an observation of 950 in Column 6.)

Maximum Number of Passes, 95% confidence limit = 11
((Col. 1) value for which (Col. 6) ≈ 50.)

this average. Their threshold relative densities at the 95 percent level of confidence for the high-variability growth curves were uniformly lower than those of their moderate-variability counterparts. These threshold densities are sensitive to the sample size and may be influential in determining the required sampling effort. Finally the maximum number of required passes at the 95 percent confidence limit is also listed. This enables the user to estimate the maximum number of passes that can be expected for any one control strip.

ANALYTICAL TECHNIQUES

Specification designers fortunate enough to recognize in Table 1 a growth curve modeling their regional soils may directly implement the results of these findings. The sample size and decision rule would be selected with reference to the relative densities, the anticipated measuring-time delay, and the maximum number of passes. [Of course, the specification that subsequently uses the maximum-density estimate would also be considered. These specifications typically allow a certain percentage of the density distribution to fall below the estimated maximum (see paper by Barros, Weed, and Willenbrock in this Record).] Other designers may generate tables similar to Table 1 by using the density-change OC curve.

The density-change OC curve underlies all relative-density estimates and is not sensitive to the shape of the density-growth curve. If this OC curve can be constructed and paired with a particular growth function, then specific probabilities may be computed. One nonlinear regression model that closely matches the form of the simulated density-change OC curve is given as follows:

$$P(STOP) = B_1 \exp(B_2 DELTA)$$

where P(STOP) is the probability of stopping compaction given a DELTA density increase and DELTA is the observed density increase in pounds per cubic foot. The regression coefficients that best estimate the simulated same-location, correlated-test OC curves are given in Table 2. The coefficients were derived specifically from growth curve A, but they are similar to and conservatively represent the coefficients associated with lower growth functions. Plots of these regression models are similar to those shown in Figures 6 and 8-10.

An analytical technique that empirically reconstructs the results of computer simulation analyses will now be discussed briefly. This technique uses the density-change OC curve to determine the shape of the rolling-frequency distribution for a given sample size, such as the one shown in Figure 2. As indicated by the worksheet shown in Figure 11, a rough approximation of the density-growth curve must be assumed. The relative densities at incremental stages of compaction are then computed, as is the incremental density increase after each pass. These incremental density increases are then substituted into the appropriate regression model of Table 2, and the probability of stopping after any pass may be estimated. The product of this probability and the number of replications surviving previous stop or continue decisions gives the number of replications that stop after the current pass.

The average relative density and the average number of passes made, both weighted by their frequency of occurrence, represent the expected relative density and expected number of passes, respectively. Threshold values are determined simply by proceeding from either tail of the frequency distribution until a sufficient tail area has been accumulated. An example of this procedure is presented in worksheet form in Figure 11.

This is a practical, if empirical, analytical technique. It provides a means by which specification provisions may be linked to previously unquantified conditions. Iteration of the analysis for a range of possible growth functions should provide a reasonable estimate of the maximum-density estimate obtainable in control strips under field conditions.

SUMMARY AND CONCLUSIONS

Control-strip specifications monitor the magnitude of successive density changes to estimate relative densification. Small density changes, which occur with increased height on the density-growth curve, signal the approach of maximum density. The density-monitoring procedure is therefore critical to the precision of the maximum-density estimate.

Same-location sampling plans are more effective than their different-location counterparts. Apparently some location-to-location variability is screened from the inference-making process by these plans, thereby increasing the plan's efficiency.

Correlation of successive comparisons in a sampling plan adversely affects the maximum-density estimate. This consideration must be weighed against the sampling effort itself: Correlated-comparison sampling plans require a smaller effective sample size. In practice, it is anticipated that the same-location, correlated-comparison sampling plan will be most useful.

Two density-growth curves were investigated by using both moderate- and high-variability components. Although moderate variability should more realistically reflect true field conditions, high variability was included as part of a sensitivity analysis. Aspects of the sampling plan, such as the decision rule and sample size, were investigated at both levels of variability for the two density-growth curves.

Although nonzero decision rules tend to degrade the ability to achieve maximum density, two factors are in their favor. The marginal loss in precision is not great, and a nonzero decision rule may be more easily implemented. Both agency inspectors and the contractor's personnel may be more easily persuaded that a small density deficiency is critical if the decision rule is 0.5 pcf rather than 0.0.

Efficiency of the estimation procedure does improve with increased sample sizes, but a sample size of 3 may be sufficient in practice. In any event, these and other subjective decisions must be made by the specification designer.

A precision-gauging technique was presented that quantifies key aspects of the decision-making procedure. This technique led to three application-specific parameters: the expected relative density, the threshold relative density, and the maximum number of passes.

Finally, although maximum-density estimates may be influenced by the growth curve and its inherent variability, these are not within the designer's control. Specification provisions have been identified that will control the density-change OC curve, and the expected relative densities are consistently high. When these densities are evaluated in light of the relatively small sampling effort, it is evident that control-strip maximum density estimates may be exceptionally precise.

REFERENCE

1. M.C. Anday and C.S. Hughes. Compaction Control of Granular Base Course Materials by Use of Nuclear Devices and a Control Strip Technique. HRB, Highway Research Record 177, 1967, pp. 136-143.

Publication of this paper sponsored by Committee on Quality Assurance and Acceptance Procedures

Software Package for Design and Analysis of Acceptance Procedures Based on Percent Defective

RICARDO T. BARROS, RICHARD M. WEED, AND JACK H. WILLENBROCK

The trend toward statistical end-result specifications has led to the development of construction specifications based on the concept of percent defective. To analyze the risks and determine the effectiveness of the acceptance procedures associated with these specifications, operating-characteristic curves must be constructed. However, many potential users do not have a working knowledge of the noncentral t and beta distributions necessary for this development. The underlying theory, several useful references, and a conversational computer program that greatly simplifies the design and analysis of specifications of this type are presented.

The current trend toward statistical end-result specifications has been a natural step in the evolution of the highway quality-assurance system. Whereas the earlier method-type specifications outlined in detail precisely how the work was to be accomplished, the more modern approach has been to define the characteristics and quality requirements of the finished product. Contractors are allowed considerable flexibility in meeting these requirements and the specifying agency is responsible primarily for the evaluation of the finished work.

The end-result approach offers several advantages over the earlier method-type specifications. First, by recognizing the existence of both inherent and testing variability, it deals with construction parameters in a more realistic manner. Highway engineers have begun to realize that it is not unusual, nor necessarily undesirable, for a small percentage of test values to fall outside realistic specification limits. Second, by defining the control of the construction process as the contractor's responsibility and the acceptance of the work (end result) as the agency's responsibility, the likelihood of contractual disputes can be reduced. Third, by clearly defining acceptance criteria and random-sampling procedures, the risks to both the contractor and the highway agency can be controlled and known in advance. Under the earlier method-type specifications, a contractor's bid was often influenced by the reputation of the highway inspector assigned to the project. Fourth, the development of adjusted-payment schedules provides a practical means to deal with work that is substandard but not so deficient that it warrants removal and replacement. Finally, because the random-sampling plans avoid the biases that are likely to occur when an inspector attempts to select a representative sample, reliable estimates of the as-built construction quality can be made. This information can also be used as feedback to determine whether further modifications of the specifications are desirable.

One of the most important steps in the design of an end-result specification is the development of the operating-characteristic (OC) curve describing its capabilities. Although most of the necessary theory is available in one form or another, much of it is not familiar or easily accessible to highway engineers. In this paper this theory is outlined, appropriate references are cited, and a conversational computer program that greatly simplifies the design or analysis of the type of statistical acceptance procedure normally used with end-result specifications is presented.

PERCENT DEFECTIVE AS A MEASURE OF QUALITY

Although several statistical measures of quality are available, highway engineers have exhibited a strong preference for the concept of percent defective, the estimated percentage of the work falling outside specification limits (or its complement, the percent within limits). This measure is particularly appealing, not only because the amount of material falling within limits is believed to be strongly related to actual performance, but because it can be applied to virtually any construction quality characteristic. This general philosophy is promulgated in Standard 214 (1) of the American Concrete Institute (ACI), for example, although the ACI acceptance criteria do not use a purely percent defective approach.

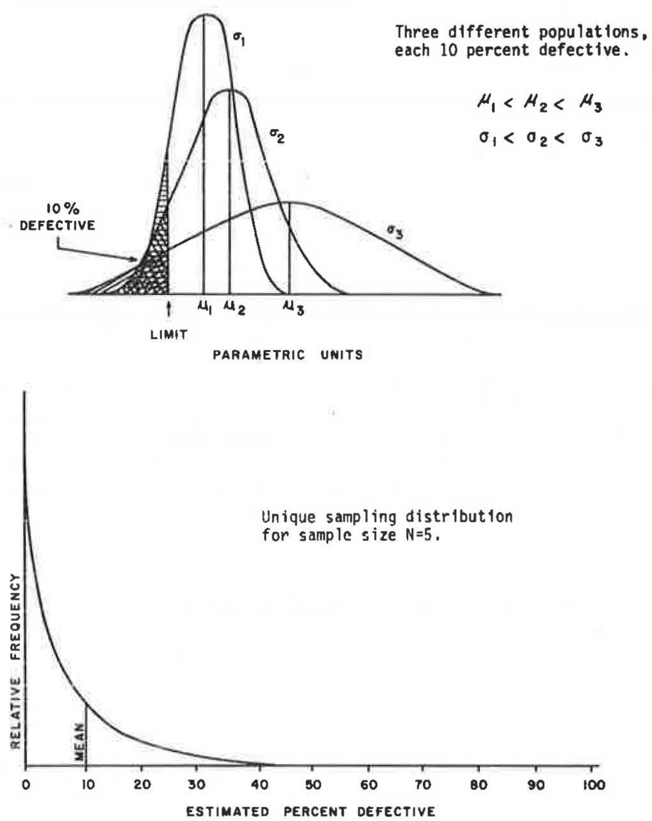
Two statistical parameters commonly used with these procedures are the process mean and standard deviation. In this paper the situation is addressed in which the values of these parameters are not known and must be estimated from sample observations. This development is appropriate for those situations in which these values may change during the course of a project.

Figure 1 illustrates three possible parent populations having identical percent defective levels and the sampling distribution associated with a sample size of 5. The sampling distribution is strongly skewed, but because the technique for estimating percent defective is unbiased, its mean is exactly at the true population percent defective. The significance of this is that although the quality of any single lot may be overestimated or underestimated, the long-term average of these estimates will be exactly equal to the true lot quality. This is of particular importance in developing fair and equitable construction specifications.

The theory associated with the development of specifications based on percent defective is somewhat involved and uses frequency distributions seldom encountered in introductory statistics courses.

Estimates of percent defective are derived from the symmetrical beta distribution (2), and the associated OC curves are computed with the aid of the noncentral t distribution (3). The literature on these two distributions is cited for the sake of completeness, but for practical purposes a conversational computer program (4) will be presented that tremendously simplifies the application of this theory. Acceptance plans based on percent defective can also be developed by use of Military Standard 414 (5), although the flexibility is quite limited.

Figure 1. Distinction between distribution of population parameter and distribution of percent defective estimates.



ESTIMATING PERCENT DEFECTIVE

The mechanics of estimating the percent defective of any construction parameter are exceedingly simple but require that the basic assumptions of a normal population and random sampling be satisfied. Once the sample has been taken and the test values have been obtained, the mean ($\bar{X}$) and standard deviation (S) are computed. Then, in order to estimate the percent defective below some lower limit (L), a quality index (Q) is calculated as follows:

$$Q = (\bar{X} - L)/S \tag{1}$$

All that remains is to determine the level of percent defective associated with the computed value of Q. This is accomplished by means of special tables such as that shown in Figure 2. (A different table is used for each sample size. Due to certain limitations of the underlying beta distribution, no tables exist for sample sizes smaller than 3.) For example, for a sample size of 5 and a quality index of 1.25, the estimated percent defective read from Figure 2 is 9.46.

If it were desired to estimate the percentage of material falling above an upper limit (U), the Q statistic would be computed by Equation 2 and the same procedure would be employed with the appropriate Q value table.

$$Q = (U - \bar{X})/S \tag{2}$$

For acceptance procedures with both lower and upper limits, the percent defective estimate is the sum of the results obtained by using Equations 1 and 2. The analysis is much more complicated in this case, however, and is beyond the scope of this paper.

Figure 2. Typical Q table for estimating percent defective of normal population.

SAMPLE SIZE = 5

Q	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	50.00	49.64	49.29	48.93	48.58	48.22	47.86	47.51	47.15	46.80
0.1	46.44	46.09	45.73	45.38	45.02	44.67	44.31	43.96	43.60	43.25
0.2	42.90	42.54	42.19	41.84	41.48	41.13	40.78	40.43	40.08	39.72
0.3	39.37	39.02	38.67	38.32	37.97	37.62	37.28	36.93	36.58	36.23
0.4	35.88	35.54	35.19	34.85	34.50	34.16	33.81	33.47	33.12	32.78
0.5	32.44	32.10	31.76	31.42	31.08	30.74	30.40	30.06	29.73	29.39
0.6	29.05	28.72	28.39	28.05	27.72	27.39	27.06	26.73	26.40	26.07
0.7	25.74	25.41	25.09	24.76	24.44	24.11	23.79	23.47	23.15	22.83
0.8	22.51	22.19	21.87	21.56	21.24	20.93	20.62	20.31	20.00	19.69
0.9	19.38	19.07	18.77	18.46	18.16	17.86	17.55	17.25	16.96	16.66
1.0	16.36	16.07	15.78	15.48	15.19	14.91	14.62	14.33	14.05	13.77
1.1	13.48	13.20	12.93	12.65	12.37	12.10	11.83	11.56	11.29	11.02
1.2	10.76	10.50	10.23	9.98	9.72	9.46	9.21	8.96	8.71	8.46
1.3	8.21	7.97	7.73	7.49	7.25	7.02	6.79	6.56	6.33	6.10
1.4	5.88	5.66	5.44	5.23	5.02	4.81	4.60	4.39	4.19	3.99
1.5	3.80	3.61	3.42	3.23	3.05	2.87	2.69	2.52	2.35	2.19
1.6	2.03	1.87	1.72	1.57	1.42	1.28	1.15	1.02	0.89	0.77
1.7	0.66	0.55	0.45	0.36	0.27	0.19	0.12	0.07	0.02	0.0

(Q VALUES ARE COMPUTED BY THE STANDARD DEVIATION METHOD. FOR POSITIVE Q VALUES, THE PERCENT DEFECTIVE ESTIMATE IS READ DIRECTLY FROM THE TABLE. FOR NEGATIVE Q VALUES, THE TABLE VALUE MUST BE SUBTRACTED FROM 100.00.)

DEFINITION OF QUALITY LEVELS

In the development of statistical specifications, two quality levels are of particular significance. These are the acceptable quality level (AQL) and the rejectable quality level (RQL), defined as follows: AQL is the maximum percent defective that (for the purposes of the acceptance specification) can be considered satisfactory as a process average. RQL is the percent defective value that if equaled or exceeded represents a seriously defective or potentially dangerous level of quality.

A common setting for the AQL is 10 percent defective. The RQL is usually set at a point at which the specifying agency reserves the option to require removal and replacement of the work at the contractor's expense. Typical values might be in the range of 40 to 60 percent defective. (It should be noted that it is possible to develop an acceptance procedure without explicitly defining an RQL.)

STATISTICAL QUALITY INFERENCES

In making an inference about the quality of any particular lot, two types of error are possible. AQL lots may be rejected or RQL lots may be accepted. The risks of making these errors are known as the producer's and consumer's risks, respectively, and are defined as follows: Alpha (α) is the producer's risk that AQL material will be rejected. Beta (β) is the consumer's risk that RQL material will be accepted.

Obviously, it is desirable that both risks be as small as possible. However, the cost of sampling, the consequences of accepting defective work, and other factors tend to dictate the levels of risk that are considered acceptable.

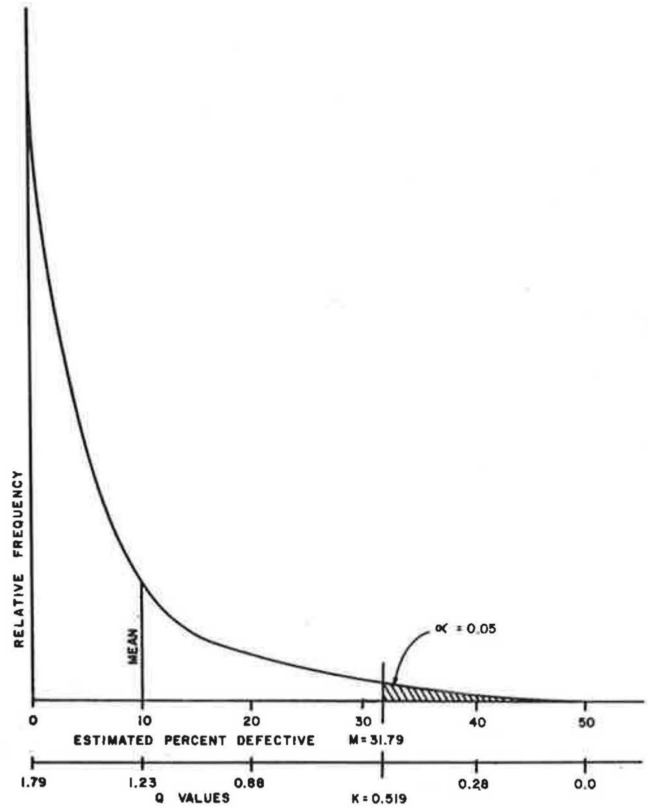
The percent defective estimate sampling distribution for a 10 percent defective quality level and a sample size of 5 is shown in Figure 3. As with any frequency distribution, there exists some limit on the estimated percent defective axis that cuts off 5 percent of the area in the upper tail. As shown in Figure 3, this limit occurs at 31.79 percent defective. If a lot is inferred to be AQL whenever a sample size of 5 estimates the quality as 31.79 percent defective or less, truly AQL material will be rejected only 5 percent of the time. In other words, if a sample size of 5 is used and a producer's risk of 0.05 is desired, the tolerable percent defective (M) estimated by a sample is 31.79 percent. For practical purposes, a value of M = 32 percent would probably be used.

The Q values used to estimate percent defective may also be scaled on the abscissa of the sampling distribution for percent defective as shown in Figure 3. For a given sample size, there is a unique correspondence between any Q value and a percent defective estimate. It is as meaningful to say that 5 percent of the sampling distribution lies beyond the Q value of k = 0.519 as it is to say that 5 percent of the percent defective estimates exceed M = 31.79 percent. The limit k of the Q scale, which corresponds to the limiting percent defective estimate (M), is defined as the acceptability constant.

Specification of either M or k along with a sample size and a lower or upper limit uniquely identifies an acceptance plan for a single-limit statistical specification that uses the percent defective approach. There are three ways in which the acceptance plan developed above could be stated:

1. Accept a lot as AQL (10 percent defective) if the estimated percent defective based on a sample size of 5 is less than or equal to M = 32 percent.

Figure 3. Sampling distribution for true population percent defective of 10 percent and sample size of 5.



2. Accept a lot as AQL if the Q statistic, based on a sample size of 5, is greater than or equal to k = 0.519.

3. Accept a lot if

$$\bar{X} \geq L + kS \qquad (3)$$

where

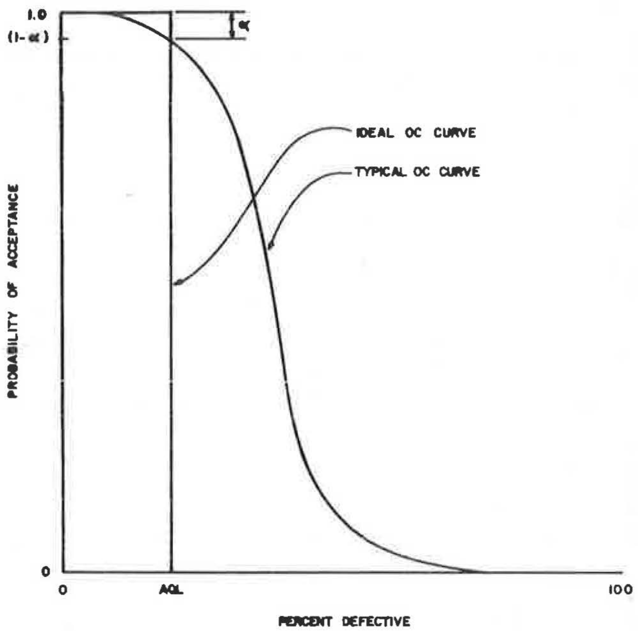
$\bar{X}$ = average value of N = 5 tests,
S = standard deviation of N = 5 tests,
L = a lower specification limit, and
k = the acceptability constant, 0.519.

If exactly AQL material were submitted under any one of these acceptance procedures, it would be accepted approximately 95 percent of the time.

The OC curve is a graphical representation of the manner in which an acceptance plan actually works and is uniquely identified by two parameters: the sample size and either k or M. It relates the probability of acceptance to the entire range of the percent defective quality measure. It will, at a glance, indicate the error risks that are incurred and it permits the meaningful comparison of alternative acceptance procedures.

An ideal OC curve is shown in Figure 4. It consists of two horizontal tails and a vertical line directly above the AQL. This curve indicates that AQL material (or better) will always be accepted. This ideal OC curve also implies that a wrong inference will never be made. With a sample size of 5, however, it is indicated in Figure 3 that the likely range of percent defective estimates extends from 0.0 to nearly 50.0 percent when the true quality is 10 percent defective. For example, if the allowable percent defective is 32 percent, it can be seen in Figure 3 that about 5 percent of the estimates will

Figure 4. Ideal and typical OC curves.



exceed M. Thus, real-world OC curves pass through the point (AQL, $1 - \alpha$) and have a more gradual slope. This more gradual slope reflects the element of risk associated with statistical acceptance procedures. It is through the construction and analysis of curves such as this that fair and effective specifications are developed.

NONCENTT PROGRAM

An interactive software package, named NONCENTT after the noncentral t distribution, has been developed to facilitate the design and evaluation of acceptance plans based on percent defective. It is written in the standard FORTRAN language and should be compatible with most computer installations. All the necessary subroutines have been incorporated into the coding so that the program is completely self-contained. Once the program has been loaded and compiled onto a computer system, it may be executed without further assistance from systems-level personnel.

NONCENTT is typically accessed by the instructions "run noncentt." As shown in Figure 5, the computer will respond with the program's title, a brief description of the program's purpose, instructions concerning the interaction procedure, and a request for information. Note the convention, which will be followed in all subsequent examples, of printing all input information in lowercase letters against the left-hand margin and all output information indented at least 10 spaces and formatted in uppercase letters. Note also the conversational nature of the expected interaction. Program requirements, as well as all diagnostic error messages, are always expressed in an easily understood conversational manner.

Detailed explanations of the input requirements and the calculations performed are available at any input stage of a NONCENTT session. In response to the instruction SELECT THE OPTION OF INTEREST, suppose the word "help" were entered. This would cause a more detailed explanation of the available options to be printed out, which is also shown in Figure 5.

On review of the available options, suppose it is

Figure 5. Initial portion of interactive session with NONCENTT.

```
run noncentt
EXECUTION BEGINS...
          NONCENTT:
          ---------

          APPLICATIONS OF THE NONCENTRAL T AND
          SYMMETRICAL BETA DISTRIBUTIONS TO THE
          DESIGN OF STATISTICAL SPECIFICATIONS


               AT SELECTED POINTS IN THE EXECUTION OF THIS PROGRAM YOU
          WILL BE ASKED FOR SPECIFIC INFORMATION.  YOU WILL BE EXPECTED TO
          RESPOND IN ONE OF THREE WAYS:

                    1) INPUT THE REQUESTED INFORMATION,
                    2) TYPE 'HELP' FOR A MORE DETAILED EXPLANATION, OR
                    3) TYPE 'QUIT' TO  EXIT ANY OPTION OR TERMINATE
                       NONCENTT SESSION.


          SELECT THE OPTION OF INTEREST.

help
               THE NONCENTRAL T AND SYMMETRICAL BETA DISTRIBUTIONS
          UNDERLIE ONE-SIDED VARIABLES SPECIFICATIONS WITH THE STANDARD
          DEVIATION UNKNOWN, SUCH AS THOSE FOUND IN MILITARY STANDARD
          414.  DESIGNERS OF DOUBLE LIMIT SPECIFICATIONS ARE REFERRED TO
          MILITARY STANDARD 414, AND TO 'QUALITY CONTROL AND INDUSTRIAL
          STATISTICS', BY A.J. DUNCAN.

               THE FOLLOWING OPTIONS ARE AVAILABLE IN THIS SOFTWARE
          PACKAGE.  ADDITIONAL EXPLANATORY COMMENTS WILL BE AVAILABLE
          AS EACH OPTION IS SELECTED.

                    1) ESTIMATION OF LOT PERCENT DEFECTIVE.

                    2) PASSING AN OC CURVE THROUGH ONE POINT.

                    3) PASSING AN OC CURVE THROUGH TWO POINTS.

                    4) ESTABLISHING POINTS ON AN OC CURVE.

                    5) CONFIDENCE LIMITS FOR TRUE PERCENT DEFECTIVE.

                    6) PROBABILITY OF EXCEEDING CRITICAL PERCENT DEFECTIVE
                       LIMITS.

                    7) THE EXPECTED PAYMENT CURVE FOR A STATISTICAL
                       SPECIFICATION.

                    *) TYPE 'QUIT' TO TERMINATE NONCENTT SESSION.



          SELECT THE OPTION OF INTEREST.

two
ERROR ------- IMPROPER INPUT OR INPUT FORMAT. RETYPE LAST LINE.
```

decided to run option 2 first. If the word "two" is typed instead of the numeral 2, it will not be accepted by the computer. NONCENTT will perform an error check on all data entered for compatibility with the requested information and for logical consistency. In this example, the selected option was not properly identified. The numeral 2 should have been entered to correctly access the desired option.

The NONCENTT session illustrated by the following examples has been streamlined for conciseness. No further input errors will be made nor will help be requested. The direct interaction that follows demonstrates the efficiency available when this software package is accessible to an experienced user. A summary of the NONCENTT options currently available is given in Table 1.

EVALUATION AND MODIFICATION OF EXISTING ACCEPTANCE PROCEDURE

For the purposes of this paper, assume that an agency is currently using a specification in which the AQL is 10 percent and the sample size is 5. Further assume that the RQL has been identified as 50 percent defective and that both alpha and beta are intended to be at the 0.05 level. Option 2 of the NONCENTT program can be used to determine the ac-

Table 1. Capabilities of NONCENTT program.

Option No.	Program Function: Option Title	Possible Application
1	Estimation of lot percent defective	Converting Q statistic into percent defective estimates Converting acceptability constant (k) into maximum allowable percent defective in a sample (M)
2	Passing an OC curve through single predetermined point	Identifying (N, k) combination that results in a specified producer's risk (α) that AQL material will be rejected Identifying (N, M) combination that results in a specified producer's risk (α) that AQL material will be rejected Placing a one-tailed confidence limit on a percent defective estimate provided the true population percent defective is known
3	Passing OC curve through two predetermined points	Identifying sample size and acceptance parameter required to pass an OC curve through both (AQL, $1 - \alpha$) and (RQL, β)
4	Establishing points on OC curve	Performing a detailed investigation of probability of accepting material whose true quality may vary over a range of possible values
5	Confidence limits for true percent defective	Determining two extreme percent defective estimate distributions that could have produced, with a level of risk equal to $\alpha/2$, the observed sample estimate
6	Probability of exceeding critical percent defective limits	Determining probability of misinterpreting AQL quality to be RQL, or vice versa Determining likelihood of achieving a particular pay factor
7	Expected-payment curve for statistical specification	Determining expected payment associated with stepped, continuous linear, or continuous curvilinear adjusted-payment schedules

Figure 6. Option 2 of NONCENTT: passing OC curve through single predetermined point.

```
        SELECT THE OPTION OF INTEREST;

2

        PASSING AN OC CURVE THROUGH ONE POINT
        -------------------------------------

        ENTER:    1) THE SAMPLE SIZE                      (AN INTEGER ≥ 3)
                  2) THE PERCENT DEFECTIVE AT THE AQL        (A PERCENT)
                  3) THE PRODUCER'S RISK, ALPHA         (0.0< ALPHA <1.0)

5 10 0.05

          EITHER OF THE FOLLOWING CRITERIA MAY BE USED:
          ---------------------------------------------

                                     MAXIMUM ALLOWABLE
          ACCEPTABILITY              PERCENT DEFECTIVE
          CONSTANT                   IN A SAMPLE
          -------------              -----------------

            K = 0.519                   M = 31.79
```

ceptability constant associated with N = 5, AQL = 10 percent, and $\alpha = 0.05$.

The printout in Figure 6 shows that for a sample size of 5, an AQL of 10 percent, and an alpha risk of 0.05, the acceptability constant (k) is 0.519. Alternatively, the tolerable percent defective as estimated by a sample of 5 tests is 31.79 percent, or approximately 32.0 percent.

The OC curve associated with the above acceptance procedure can be calculated with option 4 of NONCENTT. This option will accept either the k or the M parameter and it will compute the acceptance probabilities over a range of true percent defective values selected by the user. Assume that the OC curve is to be identified over the quality range of 10 to 90 percent in steps of 10 percent defective. The appropriate entries and the resulting output are shown in Figure 7.

The nine points on the OC curve computed by option 4 have been plotted in Figure 8. Note that at the RQL (i.e., the 50.0 percent defective quality level) the probability of acceptance is approximately 0.16, or 16 percent. Thus the consumer's risk of accepting RQL material is considerably larger than the intended value of $\beta = 0.05$. It is apparent that a sample size of 5 is simply too small to correctly recognize both the AQL and the RQL 95 percent of the time.

Figure 7. Option 4 of NONCENTT: establishing points on OC curve.

```
        SELECT THE OPTION OF INTEREST.

4

        ESTABLISHING POINTS ON AN OC CURVE
        ----------------------------------

        ENTER:    1) THE SAMPLE SIZE                       (AN INTEGER ≥ 3)
                  2) EITHER 'K= *** ' OR 'M= *** '        (A LETTER, EITHER
                                                           K OR M, FOLLOWED
                                                           BY VALUE OF K OR
                                                           M.)
                  3) THE PERCENT DEFECTIVE RANGE OF          (TWO PERCENT
                     INTEREST                                 VALUES)
                  4) THE OC CURVE PLOT INCREMENT              (A PERCENT)

5 K=0.519 10 90 10

                    POINTS ON OC CURVE
                    ------------------

                        N =    5
                        K =  0.519
                ---------------------------------
                PERCENT           PROBABILITY
                DEFECTIVE         OF ACCEPTANCE
                ---------         -------------

                 10.00              0.949983
                 20.00              0.769359
                 30.00              0.531302
                 40.00              0.313540
                 50.00              0.155191
                 60.00              0.061190
                 70.00              0.017200
                 80.00              0.002668
                 90.00              0.000099
```

One may be tempted to pass the OC curve through the (RQL, β) point rather than the (AQL, $1 - \alpha$). This could be done, but it would increase the producer's risk to approximately 0.24. If the α and β risks are to be balanced near the intended level of 0.05, the sample size must be increased. To pass an OC curve through both (AQL, $1 - \alpha$) and (RQL, β), option 3 is selected as shown in Figure 9.

Properties of the OC curve for k = 0.686 indicate that this plan produces nearly the desired risks at both the AQL and the RQL and the required sample size is 9. This OC curve has also been plotted in Figure 8, and provided that the required sample size of 9 is reasonable, the acceptance procedure development process would be complete. Otherwise, if the sample size is reduced, some increase in acceptable risk levels would have to be tolerated. Further runs of option 3 could then be made to arrive at a suitable compromise.

Figure 8. Two OC curves, each passing through (AQL, 1 - α).

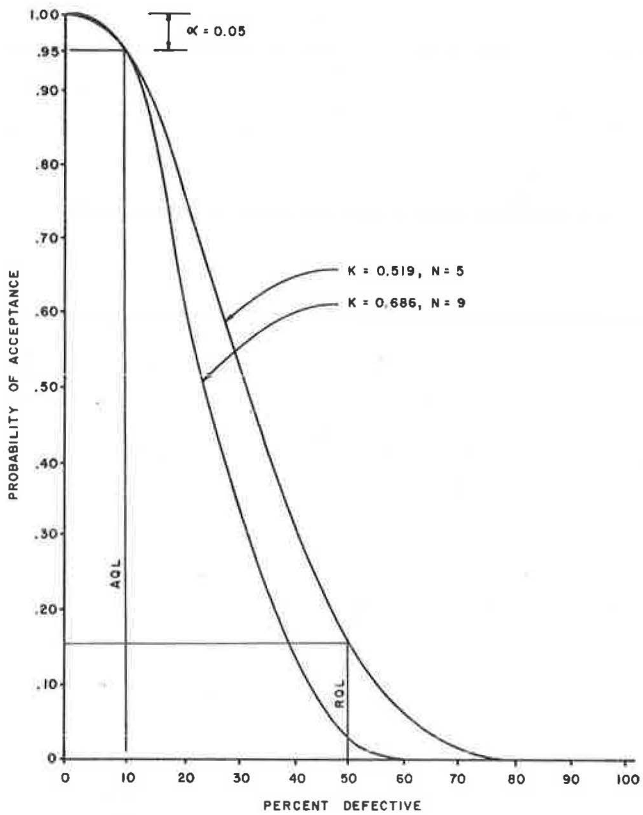


Figure 9. Option 3 of NONCENTT: passing OC curve through two predetermined points.

```
        SELECT THE OPTION OF INTEREST.

?
3

        PASSING AN OC CURVE THROUGH TWO POINTS
        --------------------------------------

        ENTER:   1) THE PERCENT DEFECTIVE AT THE AQL          (A PERCENT)
                 2) THE PRODUCER'S RISK, ALPHA          (0.0 < ALPHA < 1.0)
                 3) THE PERCENT DEFECTIVE AT THE RQL          (A PERCENT)
                 4) THE BUYER'S RISK, BETA              (0.0 < BETA < 1.0)

?
10 0.05 50 0.05

            AQL   = 10.00%   |   RQL  = 50.00%
            ALPHA = 0.0500   |   BETA = 0.0360
            -----------------|-----------------

            SAMPLE SIZE               N =     9
            ACCEPTABILITY CONSTANT    K =  0.686
            MAX. ALLOW. PCT. DEF.     M = 25.21%
```

APPLICATION TO PAY SCHEDULES

It has become common practice for highway agencies to employ adjusted-payment provisions and a recent paper succinctly states the purpose and justification for such an approach (6, p. 18):

> A construction item that falls just short of the specified quality level does not warrant rejection but neither does it deserve 100 percent payment. Accordingly, statistical specifications usually employ some form of adjusted pay schedule to award payment in proportion to the level of quality actually achieved.... Ordinarily, a pavement is designed to sustain a specified number of load applications before major repair (overlaying with bituminous concrete) is required. If, due to construction deficiencies, the pavement is not capable of withstanding the design loading, it will fail prematurely. The necessity of repairing this pavement at an earlier date results in an additional expense that, since it usually occurs long after any contractual obligations have expired, must be borne by the highway agency. It is the purpose of the adjusted pay schedule to withhold sufficient payment at the time of construction to cover the extra cost anticipated in the future as the result of deficient-quality work.

Table 2. Sample stepped pay schedule.

Step	Range of percent defective	Pay Factor (%)
1	0.0-10.00	100
2	10.01-20.00	90
3	20.01-30.00	80
4	30.01-40.00	70
5	40.01-50.00	60
6	50.01-100.00	50[a]

[a]The agency reserves the right to require removal and replacement at the contractor's expense of any lot the percent defective of which exceeds 50.00 percent. If for practical reasons this option is not invoked, the lot receives the minimum pay factor of 50 percent.

There are two basic types of adjusted-payment schedules--stepped and continuous. Stepped pay schedules define discrete intervals of quality and award a single pay factor for each. Continuous pay schedules express the pay factor in equation form as a function of the selected quality measure. Although stepped pay schedules are more common, continuous pay schedules do offer certain advantages. Besides being more concise, they more precisely match the appropriate pay factor with the estimated quality for any given lot. This tends to minimize the harshness of having just missed the next higher pay level. Nevertheless, stepped and continuous pay schedules can be constructed that will have essentially the same long-term performance.

For demonstration purposes, suppose that a highway agency has developed the stepped pay schedule shown in Table 2 for use with a particular acceptance procedure. The first step of this pay schedule indicates that a pay factor of 100 percent will be awarded if the percent defective quality measure is less than or equal to 10 percent, the AQL. If the estimated percent defective is greater than 10 percent but less than or equal to 20 percent defective, 90 percent of the contract amount will be awarded, and so on. Note that for practical purposes this stepped pay schedule can be briefly summarized by listing only the upper limits of the quality intervals along with the associated pay factors.

It would be misleading, however, to compare alternative pay schedules purely on the basis of their indicated pay factors. That a pay factor is associated with some level of quality does not guarantee that material of that quality will, on the average, receive that pay factor. Seldom is that the case. True quality levels are estimated by the quality levels of samples, and these sample estimates are used in the pay-factor determinations. The distribution of pay factors, therefore, is influenced both by the sample-estimate distribution and by the adjusted-payment schedule. In most cases, some degree of distortion is found to occur between the respective distributions.

Expected pay factors are computed as the sum of the products of all pay factors multiplied by the probability of obtaining each pay factor (7). This computation will numerically identify the mean value of the pay-factor distribution. The expectedpayment (EP) curve relates probable payment to the true level of quality. This allows one to read the average pay factor directly from the Y-axis for any level of true percent defective, analogous to the OC curves already discussed.

The EP curve may be computed by option 7 of the NONCENTT program. This first example will produce the EP curve associated with the stepped pay schedule just presented. In order to perform the necessary computations throughout the entire range of percent defective, it will be assumed that all RQL lots receive the minimum pay factor of 50 percent. The input and output are shown in Figure 10.

The EP curve has been plotted in Figure 11 and provides the means to judge the probable payment from the perspectives of both the highway agency and the contractor. The underlying goals are (a) to provide sufficient incentive for the contractor to produce good-quality work and (b) to pay a fair reduced price when the work is substandard. To determine whether the first objective has been met, the highway agency must judge whether it is in the contractor's best interest to produce the desired level of quality. To judge whether the second objective has been met, various methods have been proposed (6-9). In some cases, when little information has been available relating quality measures to performance, these methods have necessarily been quite arbitrary. In other cases, for which the quality-performance relationship can be established, more logical and rational procedures can be employed (6).

Nevertheless, there is one obvious problem apparent in Figure 11. A producer who consistently supplies the AQL of 10 percent defective will not, on the average, receive 100 percent payment. Instead, the expected pay factor for AQL work is approximately 93 percent. As demonstrated in an earlier paper (10), an inequitable condition such as this imposes a severe hardship on the producer.

To correct this problem, the EP curve must be raised so that the expected pay factor is 100 percent when the quality is exactly at the AQL. To do this, it is necessary to use a pay schedule that is capable of awarding pay factors greater than 100 percent. Either a crediting provision (9), in which pay factors greater than 100 percent are used to

Figure 10. Option 7 of NONCENTT: establishing points on EP curve.

```
        SELECT THE OPTION OF INTEREST.

7

        EXPECTED PAYMENT CURVE
        ----------------------

            THIS OPTION COMPUTES THE EXPECTED PAY FACTOR BASED ON A
        GIVEN PAY SCHEDULE AND SAMPLE SIZE.

            ENTER '1' IF A CONTINUOUS PAY SCHEDULE IS TO BE USED,
        OR ENTER THE NUMBER OF STEPS IF A STEPPED PAY SCHEDULE IS
        PREFERRED.

6

        ENTER:     THE UPPER PERCENT DEFECTIVE       ( 6 PERCENT DEFEC-
                   LIMIT FOR EACH STEP                TIVE VALUES)

10 20 30 40 50 100

        ENTER:   THE  6 PAY FACTORS ASSOCIATED
                 WITH THE  6 PAY STEPS.              ( 6 PERCENT VALUES)

100 90 80 70 60 50

        ENTER:   THE SAMPLE SIZE                     (AN INTEGER ≥ 3)

5

        ENTER:   1) THE PERCENT DEFECTIVE RANGE OF    (TWO PERCENT
                    INTEREST                           VALUES)
                 2) THE EP CURVE PLOT INCREMENT        (A PERCENT)

10 90 10

                 POINTS ON THE EXPECTED PAYMENT CURVE
                 ------------------------------------

                 SAMPLE SIZE        =    5
```

STEP	QUALITY INTERVAL	PAY FACTOR
1	0.0% ≤ PCT. DEF. ≤ 10.0%	100.0%
2	10.0% < PCT. DEF. ≤ 20.0%	90.0%
3	20.0% < PCT. DEF. ≤ 30.0%	80.0%
4	30.0% < PCT. DEF. ≤ 40.0%	70.0%
5	40.0% < PCT. DEF. ≤ 50.0%	60.0%
6	50.0% < PCT. DEF. ≤ 100.0%	50.0%

PERCENT DEFECTIVE	EXPECTED PAY FACTOR, %
10.00	93.156
20.00	84.453
30.00	75.560
40.00	67.289
50.00	60.334
60.00	55.169
70.00	51.952
80.00	50.442
90.00	50.025

Figure 11. EP curve for six-step adjusted-pay schedule shown in Figure 10.

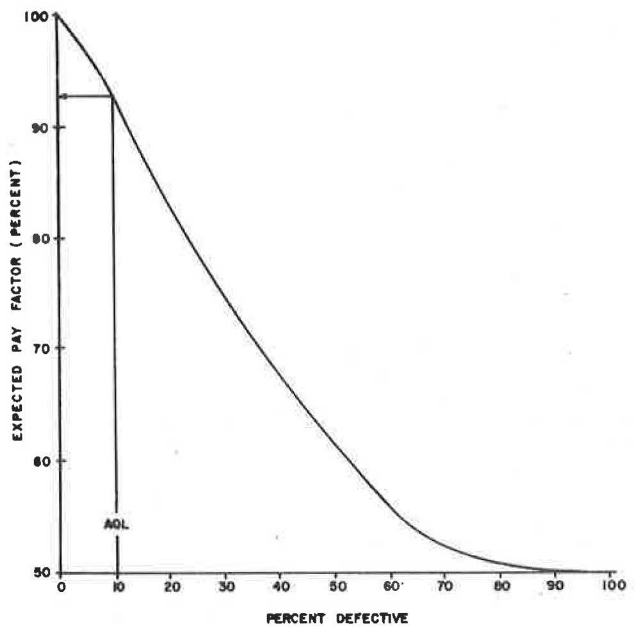


Figure 12. Points on EP curve for four-step adjusted-pay schedule.

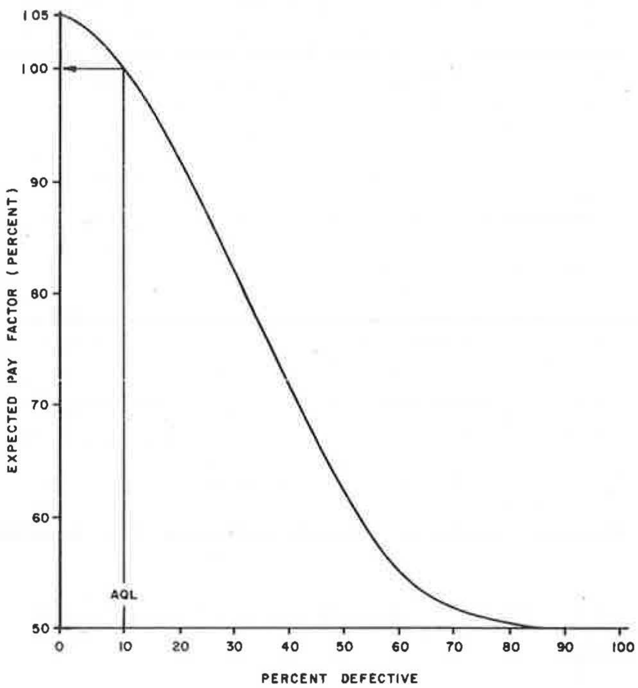
POINTS ON THE EXPECTED PAYMENT CURVE

SAMPLE SIZE = 5

STEP	QUALITY INTERVAL	PAY FACTOR
1	0.0% ≤ PCT. DEF. ≤ 10.0%	105.0%
2	10.0% < PCT. DEF. ≤ 20.0%	97.0%
3	20.0% < PCT. DEF. ≤ 40.0%	90.0%
4	40.0% < PCT. DEF. ≤ 100.0%	50.0%

PERCENT DEFECTIVE	EXPECTED PAY FACTOR, %
10.00	97.826
20.00	92.143
30.00	82.207
40.00	71.571
50.00	62.253
60.00	55.584
70.00	51.831
80.00	50.337
90.00	50.013

Figure 13. EP curve for four-step pay schedule shown in Figure 12.



offset pay factors below 100 percent, or a true bonus provision (11) may be used. It will be assumed in the next example that a bonus provision is in effect.

Option 7 of the NONCENTT program was run once again to produce the output shown in Figure 12. As before, it is assumed that the minimum pay factor of 50 percent is assigned when the lot is estimated to be at or below the RQL. The associated EP curve has been plotted in Figure 13.

Three points are worthy of note concerning the EP curve in Figure 13. First, the AQL (10 percent defective) now receives an expected payment of virtually 100 percent, whereas large percent defective values retain their previous expected payment. Second, the quality intervals identified in the pay schedule in Figure 12 need not be directly associated with the AQL and RQL definitions. This emphasizes that the adjusted-payment schedule addresses the level-of-quality estimates, whereas the AQL and RQL definitions pertain to true levels of quality. Consequently, it is the EP curve that should be analyzed, not the pay schedule itself. Finally, the pay schedule has been simplified. Four pay levels are now specified rather than six.

At this point, the highway agency must judge whether the acceptance procedure is suitable. Recent publications (6-8,11) provide guidance in the development of equitable and effective specifications, but the ultimate decision must rest with the agency itself. The NONCENTT program has served its purpose by providing the information on which this decision can be based.

A continuous (equation-form) pay schedule could also be used. Again by using option 7, it was found by trial and error that Equation 4 is essentially equivalent to the stepped pay schedule shown in Figure 12. Here again, it is assumed that the minimum pay factor of 50 percent is assigned whenever the percent defective estimate is greater than or equal to 40 percent.

$$PF = 105.0 - 0.5PD \tag{4}$$

where PF is the pay factor in percent and PD is the estimated percent defective.

For this particular example, the linear function used in Equation 4 produces essentially the same EP curve as that shown in Figure 13. In other cases, it may be necessary to include a quadratic term in the pay equation. Accordingly, option 7 of NONCENTT provides this capability.

SUMMARY AND CONCLUSIONS

Statistical acceptance procedures based on percent defective are now in common use by many highway agencies. In order to develop effective specifications in an expeditious manner and minimize costly and time-consuming field trials, it is necessary to develop and compare the OC curves for the various plans under consideration. This requires the use of statistical theory and special frequency distributions unfamiliar to many potential users. With the aid of the conversational computer program presented in this paper, however, these steps can easily be performed by individuals who have only a basic theoretical background.

This new capability should have several effects. First, it will greatly simplify the work of agencies planning to develop additional statistical specifications. Second, it will make it possible to more formally check existing specifications the risk levels of which may be far from optimal. Finally, this added convenience may serve to overcome the reluctance of the relatively few agencies who have yet to realize the advantages of statistical quality assurance.

ACKNOWLEDGMENT

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Correlation of Quality-Control Data and Performance of PCC Pavements

KAMRAN MAJIDZADEH, GEORGE J. ILVES, MICHAEL LUTHER, AND PETER KOPAC

The interrelationship between concrete pavement quality indicators and pavement performance is presented. In the study reported here, a literature review was conducted to help identify pavement quality indicators, such as water/cement ratio, strength, slump, air content, and so forth. A detailed field investigation was carried out in five states to collect quality-indicator data. A pavement-condition-rating (PCR) procedure was developed to collect PCR data for various pavement sections. Linear and nonlinear statistical analyses were conducted to develop models interrelating quality-control data with PCR data. The results of the statistical analyses and the nature of the models developed are discussed in detail.

The development of statistically based performance specifications as part of quality assurance programs in highway construction and maintenance is geared toward establishing construction and material quality levels based on expected performance. Payment adjustment schedules can then be adopted by which contractors are paid according to the performance of the final product. Payment penalties are based on failure to meet performance specifications rather than on material specifications. Such programs reduce the need for materials testing as well as the necessity for revising or creating materials-based specifications, and contractors have more latitude in their choice of materials and construction methods as long as the final product performs as expected. Nevertheless, the development and implementation of such specifications for pavement quality-control variables have raised two questions: How do material variables relate to pavement performance, and are these variables adequate indicators of pavement performance and quality?

Establishing interrelationships between pavement performance and quality-control criteria requires a basic understanding of the parameters affecting performance, an identification of those parameters indicative of quality, and a knowledge of their statistical variations. These parameters are usually classified into several categories--environmental, geometrical, boundary, material, construction, traffic loading, and design variables. The degree to which each variable influences performance is often affected by the interaction of numerous parameters, which requires sophisticated statistical analyses of the data in order to establish the relative significance of each variable.

The reliability of such interrelationships is highly dependent on the nature of the data collected, the statistical significance, and the validity. Many sources of material and construction quality data can prove to be biased or inaccurate. This is particularly true when subjective judgments are used to reject on site some materials suspected of not meeting specifications whereas other materials deemed to be in compliance are accepted and used without actual testing to verify whether they meet specifications. To establish accurate relationships between material quality indicators and performance, truly unbiased estimates of those parameters that affect pavement quality must be obtained.

The validity of these relationships also depends on having a reliable method for estimating pavement performance. Ideally, performance should be evaluated through detailed measurements, both destructive and nondestructive, to determine remaining life. Because this is a time-consuming and expensive process, a rapid, cost-effective, reliable pavement condition evaluation system that reflects actual conditions is needed.

In this paper the results of a recent study (1) of the interrelationships between quality indicators and performance of concrete pavements are reported. In that study, historical and construction data on selected quality variables were collected for 104 concrete pavement projects in five states. In addition to these data, the 104 projects were subjected to pavement condition evaluations to establish current performance levels. Statistical analyses were performed to establish relationships between performance rating and quality-indicator data, and 30 models were developed and tested.

A general model and representative data from the Ohio projects in that study are presented here to illustrate the types of performance and quality data required to develop statistically reliable relationships, the types of results that can be obtained from such analyses, and the impact of missing data on model development and reliability. A brief description of quality indicators known to affect concrete pavement quality and performance is presented in the next section. In the third section the pavement condition evaluation system used to rate performance of the pavement projects is discussed. Data collection is outlined in the fourth section, and in the last section the statistical analyses performed and results obtained are summarized.

QUALITY INDICATORS IN PCC PAVEMENTS

When quality-assurance programs are carried out that use statistically based quality-indicator specifications to meet performance requirements, it is necessary to establish whether specific quality indicators, tests, measurements, or observations are truly related to future rigid pavement performance. Not all parameters that influence distress modes are indicative of concrete quality. Rather, external factors such as load, traffic patterns, soil conditions, and climate may significantly influence rigid pavement performance. The relative importance of these factors depends on the type of distress and the performance parameters being considered.

The following primary quality indicators have been identified as essential for rigid pavement construction, long-lasting performance, and least-cost maintenance requirements:

1. Pavement thickness,
2. Concrete strength (tensile, flexural, and compressive),
3. Concrete consistency (slump),
4. Concrete density,
5. Water/cement ratio,
6. Air content,
7. Mix temperature, and
8. Aggregate quality and durability.

These indicators are frequently cited as those parameters that, in response to load, traffic, environment, and so forth, affect rigid pavement performance. These indicators are briefly discussed in the following paragraphs.

Pavement Thickness

The AASHO Road Test studies of rigid pavements provided substantial data on the inverse relationship between pavement thickness and deflection, an expression of structural integrity and response to load, environment, and so on. One of the most significant factors in the extended AASHO rigid pavement design equation is pavement thickness. The expected change in service life due to overestimated or underestimated thickness was more than 20 percent in most cases, which supports the conclusion that a negative variation in thickness significantly affects pavement life (2). Numerous studies confirm that pavement thickness is a primary factor influencing performance.

Concrete Strength

Flexural concrete strength, in addition to thickness, is one of the most significant factors in the extended AASHO rigid pavement design equation. Concrete strength is known to have a high probability of variation under construction conditions (2). It has also been widely reported to be related to pavement deflection and crack spacing in continuously reinforced concrete (CRC) pavements. Concrete with low tensile strength has been reported to exhibit higher deflections than stronger concrete (3,4).

Concrete strength is also affected by air content and water/cement ratio. Weed (5) reported that concrete compressive strength decreased by 10 percent (400 psi) for each 1 percent increase in air entrainment. He also reported the inverse relationship between compressive strength and water/cement ratio, a finding reported in numerous other studies.

Concrete Consistency (Slump)

Consistency is a practical consideration in obtaining workable concrete. It usually denotes the fluidity or wetness as indicated by slump or corresponding tests. Dry concretes with low slump values tend to crumble unless carefully handled. Although dry concrete can be consolidated into a rigid mass under vigorous vibration, it will exhibit voids or honeycombing unless special care is taken.

Concrete Density

In-place relative density is often cited as a primary parameter in rigid pavement performance. Low density resulting from poorly controlled vibration at construction leads to honeycombing and other failure problems. Walker (6) reported that density decreases with faster vibrations and greater spacings. It has also been reported that higher densities are obtained in mixes with higher slump values (1-2 in.).

Water/Cement Ratio

In hardened concrete, properties such as strength are functions of the density, which in turn is controlled by the ratio of water to cement in the original mix. Thus there are practical limits to the proportions of cement, water, and aggregate in normal mixtures. In the hardening process, a unit quantity of a particular cement can potentially combine with a specific quantity of water. A mix having a high water/cement ratio will have a larger volume of potentially uncombined water. Because capillary-pore space derives from uncombined water, leaner mixes have a more porous structure. It has been well documented that the strength and porosity of concrete paste structures depend almost entirely on the water/cement ratio.

Air Content

The general effects of air entrainment include increased workability, decreased unit weight, decreased strength, reduced bleeding and segregation, and increased durability. Air entrainment permits a lower sand content in the mix and a reduction in mixing water of about 3 percent for each 1 percent entrained air. It has been used to reduce frost damage, with a necessary trade-off in concrete strength. Air content also affects mixture workability.

Mix Temperature

Temperature effects for moist-cured concrete strength depend on the time-temperature history. When concrete is cast and maintained at a given constant temperature, the higher that temperature, the more rapid the hydration and resulting gain in strength at ages up to 28 days. At later ages, the strengths are not greatly different but the higher the curing temperature, the lower the strength. When concrete is cast and maintained at a given temperature for several hours and then cured at 70°F, the higher the initial temperature, the lower the 28-day strength. In general, if the curing temperature is higher than the initial casting temperature, the resulting 28-day strength will be higher than that for a curing temperature equal to or lower than the initial temperature.

Aggregate Quality and Durability

As in asphaltic mixes, the quality and performance of concrete mixtures are substantially influenced by aggregate type, quality, and durability. For example, the porosity or absorption rate of an aggregate affects the water/cement ratio, strength, and so on, and will significantly influence pavement susceptibility to saturation and freeze-thaw damage.

DEVELOPMENT OF PAVEMENT CONDITION EVALUATION SYSTEM

Concrete pavement performance must be evaluated in a rational, concise, and descriptive manner that permits statistical correlation with quality-control (QC) parameters. Performance can be qualified in terms of structural capacity, physical deterioration or distress, or rider quality and other user-related factors, which constitute the serviceability of the pavement. Recent advances in pavement design have dealt with quantifying pavement life by predicting the occurrence of different types of distress. It is generally believed that pavement distress precedes loss of structural capacity and reduction in ride quality.

QC criteria, which define acceptable limits on material properties, should be related to the occurrence of different types of material failures or distress. The literature provides ample evidence that these two factors are indeed related. Because pavement distress results from complex interactions among design, construction, materials, environment, traffic, and maintenance in which one distress type can lead to another, those performance parameters identified as quality indicators can often influence more than one type of distress.

Where pavement performance is qualified by distress manifestations, the rating procedures used to assess pavement condition must include a uniform method for identifying and quantifying distress severity and extent. The pavement-condition-rating (PCR) system developed for this study involved rating the pavement based on the presence of visible distress and is a modification of the method developed by the Ohio Department of Transportation (ODOT) (7), which provides a procedure for uniformly identifying and describing pavement distress in terms of extent and severity. The mathematical expression for PCR provides an index reflecting the composite effects of various distress types, their severity, and the extent of the effects on overall pavement condition.

The ODOT method for computing PCR is based on the summation of points deducted for each type of visible distress. Total points deducted is subtracted from 100 to yield the PCR. The distress types identified by the ODOT PCR procedure for concrete and CRC pavements are given below. Distress types have been grouped under three main categories: surface defects, pavement support, and cracking; if applicable, a category for joint deficiencies may also be used. The PCR field manual contains standard descriptions of each distress type as well as assistance in defining distress severity and extent.

1. Jointed concrete pavement
   a. Surface defects
      (1) Surface deterioration
      (2) Patching
      (3) Popouts
   b. Pavement support
      (1) Pumping
      (2) Faulting
      (3) Settlement
   c. Cracking
      (1) Transverse
      (2) Longitudinal
      (3) Corner breaks
   d. Joint distress
      (1) Joint spalling
      (2) Joint sealant damage
      (3) Pressure damage
2. CRC pavement
   a. Surface defects
      (1) Surface deterioration
      (2) Patching
      (3) Pressure damage
      (4) Popouts
   b. Pavement support
      (1) Pumping
      (2) Settlement or waves
   c. Cracking
      (1) Transverse spacing
      (2) Longitudinal
      (3) Punchouts or edge breaks
      (4) Spalling

The subtotal for structural deductions is the total of points deducted for those distress types believed to be related to pavement structural integrity. ODOT plans to use the structural deduction subtotal to identify pavements for further evaluation by using nondestructive methods.

The modified system used in this study was called the Concrete Pavement-Condition-Rating System (CPCR). CPCR uses the same distress types as those used in the ODOT system except that reactive aggregate durability and swell have been added. Mathematical calculation of PCR is the same as that in the ODOT system.

Separate ratings are made for each QC section within a project. QC sections are continuous subdivisions of the project length; each QC section is characterized by its own set of QC data (compressive strength, slump, percentage of air, and so on). For each QC section, the QC data are the independent variables, whereas the CPCR performance data (along with subgroupings) are the dependent variables. QC section length is established by the frequency of available QC data values. For Ohio, each QC parameter is available at a frequency of about six values per directional mile of roadway. At least three parameter values are desirable for characterizing the QC section with adequate reliability. For states or individual projects with greater frequency of QC data values, smaller sections can be used. Short QC sections are desirable because the range of QC data and sensitivity of the CPCR procedure are enhanced. QC section length is set before field performance ratings are conducted.

When the 104 projects included in this study were evaluated, two research teams were used to minimize perceptual bias during data collection and field rating. Thus, the membership of the pavement-rating team differed from that of the team that collected the concrete QC data from historical and construction records and that also selected the projects for field rating and established the QC section lengths for those projects.

PROJECT SITE SELECTION AND DATA COLLECTION

Site Selection

For this study project sites were needed for which unbiased QC test results were available. After a list of desirable quality indicators had been established, as well as design, traffic, and environmental variables for which data would need to be collected, preliminary interviews were held with various state departments of transportation to determine the types and amounts of QC data available.

Two significant factors affected a state's ability to provide needed data for a project. The first was the amount of concrete pavement that is still exposed. In Ohio, for instance, most concrete pavements were originally built during the late 1950s and 1960s. Much of that mileage has been overlaid, and overlaid pavements could not be used in the study because their condition had been altered. The second factor was time constraints on record keeping. Most states dispose of QC data after a given time period. Because much of the concrete pavement in the United States is more than 10 yr old, the records have been destroyed in many instances.

Five states were selected for inclusion in this study: Florida, Louisiana, Maryland, New York, and Ohio. Selections were based on availability of QC data, pavement accessibility, willingness to participate, and geographic location (for variation in environment). A total of 104 projects was selected. Of these, 25 were in Florida, 8 in Louisiana, 11 in Maryland, 10 in New York, and 50 in Ohio. The projects included plain jointed, dowelled jointed, and CRC pavements with thicknesses ranging from 8 to 10 in. The Florida pavements were relatively new (4 to 9 yr old), whereas the Louisiana projects were 16 to 20 yr old, Maryland pavements were 10 to 13 yr, New York pavements ranged from 12 to 15 yr, and Ohio projects from 5 to 14 yr. Traffic also varied from a cumulative equivalent axle load of 0.2 million to 15.5 million lb.

Data Collection

The CPCR procedure discussed in the preceding section was used to rate each of the 734 PCR sections established in this study. A PCR section consisted of approximately 0.5 mile of roadway, except in New York where 0.25 mile was used; one lane in each traffic direction was rated. It was originally planned to consider each direction (lane) separately, but because only 8 percent of the QC data was identified by lane, the PCR values and distress measurements were averaged over both lanes.

Along with PCR values, a riding comfort index (RCI) was assigned to each lane section rated, and direct distress measurements were made. Two 200-ft sections per lane were evaluated for distress measurements of the 0.5-mile section lengths, and one 200-ft section per lane was used for the 0.25-mile section lengths. Distress measurements consisted of estimating the lineal feet of medium- or high-severity transverse crack spalling, the area of cracking, and number of punchouts for CRC pavements.

STATISTICAL ANALYSIS OF DATA

Statistical Techniques Used

The ultimate objective of this study was to determine which of the quality indicators influence pavement performance and to what extent. The techniques available are linear and nonlinear multiple stepwise regression methods, such as those of the Society for Automation in the Social Sciences (SASS) and the Statistical Package for the Social Sciences (SPSS).

Although it is somewhat simplistic to expect that a subject as complex as pavement performance can be analyzed with linear models, the nonlinear models are difficult to use if the form of the nonlinear model is not known beforehand and are further complicated when several independent variables are being considered. Thus in this study linear stepwise regression was employed. Selection of a linear regression model does not, however, restrict analysis to linear forms of the independent variable; it is possible to define dummy variables in terms of nonlinear forms of the independent variables as well as interaction terms between some forms of the independent variables. This approach was selected for analysis.

Quantifying Qualitative Variables

Qualitative variables are those to which numerical values cannot be assigned, such as climate (wet or wet and freezing) or subgrade type (good, fair, poor). Stepwise regression methods include qualitative variables as part of the variable list by assigning arbitrary numerical values for different variable levels, such as 0 for wet and 1 for wet and freezing. Because the net effect of qualitative variables modifies the intercept, however, the influence of such variables in these models is generally rather poor. In this study, various techniques such as BMDP, SASS, and SPSS were tried by using qualitative variables for climate, base type, and subgrade type. Such variables cannot be used to determine interactive effects, however, and because their use resulted in poor models in this study (the best R^2 was 0.28), it was decided to quantify climate and subgrade. Climate was subdivided into three variables--mean annual rainfall, frost penetration, and freezing index. The subgrade type was quantified by assigning an average California bearing ratio (CBR) value based on soil classification. Poor soils were assigned a CBR of 5; fair soils, a CBR of 8; and good soils, a CBR of 12.

Missing-Value Problem

One of the most frustrating aspects of the analysis was the problem of missing values for the QC data. State transportation departments do not always agree on the type of QC tests to be done, and the types of tests conducted often vary from project to project within a state. Also, some QC data have been lost over the years. Thus, values were not available for many of the QC variables for a significant number of the sections under investigation. Table 1 lists the QC variables used in this analysis and the number of PCR sections having nonzero values for these variables. Although it appears that many of these variables had reasonable populations, it was rare that sections had data for most of these variables at the same time. The first, most obvious approach was to substitute project average values for section variables with missing data by using the following justification:

1. If project average values were not substituted, the models would have to be developed on significantly smaller populations [252 sections instead of 529 for models involving temperature, and 260 instead of 558 sections if temperature (N30 and N31) is ignored].
2. In most instances, the projects where substitution could be made already showed reasonable uniformity in data.

In the second approach, the correlation matrix was examined to see whether any relationship existed between the dependent variables.

Numerous attempts were made to overcome the missing-data problem through models involving the other variables, with notable lack of success. The only exception was a model relating the water/cement ratio (N36) to slump, air content, and core compressive strength.

Another approach was tried with a technique developed at Ohio State University (8) for handling missing-value problems. In this technique, the missing values are replaced with zeros; a regression relationship is developed from all data, which forces the intercept through near zero. Then the data are shifted by an amount equaling the derived intercept, a regression relationship is redeveloped, and the process is iterated until a stable intercept is obtained. This intercept represents the most

Table 1. Variables used in analysis.

Variable	Description	No. of Sections[a]
N9	Annual rainfall	734
N10	Frost penetration	734
N11	Freezing index	734
N13	Subgrade CBR	734
N15	Joint/crack spacing	734
N16	Cumulative traffic	734
N17	Age	734
N18	Design thickness	734
	Pour temperature	
N30	Minimum	460 (564)
N31	Maximum	460 (564)
N32	Slump	561 (638)
N33	Percentage of air entrained	515 (610)
N34	Air content	6 (6)
N35	Unit weight	170 (255)
N36	W/C ratio	262 (276)
N37	Yield	155 (158)
	Flexible strength	
N38	0 to 3 days	36 (115)
N39	4 to 7 days	179 (275)
N40	8 to 14 days	86 (185)
N41	>14 days	89 (93)
	Cylinder strength	
N42	0 to 3 days	6 (6)
N43	4 to 7 days	336 (462)
N44	8 to 14 days	200 (367)
N45	>14 days	269 (377)
N46	Core strength	488 (661)
N47	Core age	394 (456)
N48	Thickness deviation	564 (611)

[a]Numbers in parentheses indicate number of sections having nonzero values after project average values were assigned.

probable value of the missing variable. Because of the complex nonlinear relationships among variables, however, this effort did not yield reasonable results in this study.

Stepwise Regression Analysis

Because it was expected that the performance prediction model would be quite complex, with nonlinear interaction between independent variables, a large number of dummy variables were defined in terms of the elementary forms of the independent variables. The dummy variables were defined in terms of 1/x, x*x, 1/x*x, log x, 1/log x, SQRT(x), and x**0.75. In addition, a number of two-level interactions (interaction of variable i with variable j) and three-level interactions (interaction of variable i with variables j and k) were defined.

Table 1 gives the list of independent variables used in the predictive models. Two other variables were defined along with their nonlinear forms discussed above: H = design thickness (N18) + thickness deviation (N48) and TEMP = minimum pour temperature (N30) + maximum pour temperature (N31). Using the variables described in Table 1 resulted in 529 sections having nonzero values for all variables. Of these, 307 sections had granular bases and 222 had stabilized bases. The inclusion of any other variables, it was found, would have drastically reduced the population.

Three models were developed to predict pavement performance as measured by PCR from the variables listed in Table 1. These models were

1. General model, 529 sections;
2. Granular-base model, 307 sections; and
3. Stabilized-base model, 222 sections.

As will be discussed later, temperature did not always enter the models; when N30 or N31 or both were involved, their effect was relatively small. Thus, in order to include New York data and to broaden the data base, the same three models were also developed and N30 and N31 were omitted from the variable list. Exclusion of N30 and N31 resulted in 558 sections for the general model, 335 sections for the granular-base model, and 223 sections for the stabilized-base model.

An attempt was made to model jointed and CRC pavements, without great success. All Ohio CRC pavements were built on stabilized bases, all Maryland pavements were CRC on granular bases, and Florida and New York projects were all jointed pavements. Thus, separating by both pavement and base type not only leads to models with small populations but also restricts each model to only one state. Because such a restriction would make the models less useful, this approach was not pursued.

In addition to six PCR models, models were also developed for the PCR subgroups [structural deduction (ST.D.), surface deduction (SU.D.), support deduction (SP.D.), and cracking deduction], defined as follows for jointed pavements:

1. Structural = pumping + faulting + transverse cracking + longitudinal cracking + corner breaks,
2. Surface = surface deterioration + popouts + patching,
3. Joint = joint spalling + pressure damage + sealant damage,
4. Support = pumping + settlement + faulting, and
5. Cracking = transverse cracking + longitudinal cracking + corner breaks.

For CRC pavements, deduction totals were defined as follows:

1. Structural = patching + pumping + settlement + transverse cracking + longitudinal cracking + punchouts,
2. Surface = surface deterioration + popouts + patching + pressure damage,
3. Support = pumping + settlements, and
4. Cracking = transverse cracking + longitudinal cracking + punchouts + crack spalling.

Table 2 shows a summary of the 30 models developed for this study. As can be seen, the equations are all relatively long, ranging from 9 to 34 terms. Except for the cracking-deduction model with temperature, the granular-base models have the highest correlation coefficients (R^2) and the lowest standard error of estimate (comparing within a dependent variable group), and the stabilized-base models generally have the lowest correlations and the highest standard errors. The stabilized-base models have the lowest populations, which partly explains the poorer relationship, but by this argument, the general models should have the best correlation, which was not the case. It is more likely that the quality of stabilized bases is quite variable by design. Some of the bases are asphalt treated, whereas others are cement treated, and the asphalt/cement content most probably varies for both. Ideally, bases should be described by modulus and thickness, but such information was not available for this study.

CONCLUSIONS AND RECOMMENDATIONS

It is interesting to note that the term H*H/log N16 entered most of the 30 models developed. This term describes the effect of pavement thickness in the AASHTO rigid pavement design equation. Also included in many models is the term log N16/N46*N46, which describes the effect of concrete strength

Table 2. Correlation coefficients.

Model	R^2	Standard Error	No. of Terms
PCR			
GEN-T[a]	0.634	4.62	25
GRN-T	0.780	3.39	25
STB-T	0.529	5.59	10
GEN	0.653	4.49	29
GRN	0.742	3.69	20
STB	0.567	5.36	15
Structural deduction			
GEN-T	0.692	3.52	30
GRN-T	0.750	2.53	23
STB-T	0.710	4.07	19
GEN	0.663	3.75	27
GRN	0.733	2.76	18
STB	0.651	4.44	15
Surface deduction			
GEN-T	0.737	1.10	26
GRN-T	0.797	0.91	19
STB-T	0.613	1.15	16
GEN	0.728	1.12	27
GRN	0.739	1.03	20
STB	0.590	1.20	16
Support deduction			
GEN-T	0.791	2.22	34
GRN-T	0.842	1.83	21
STB-T	0.716	2.57	18
GEN	0.779	2.039	33
GRN	0.837	1.92	20
STB	0.695	2.66	17
Cracking deduction			
GEN-I	0.662	2.38	24
GRN-T	0.692	1.54	21
STB-T	0.721	2.87	12
GEN	0.683	2.30	31
GRN	0.711	1.53	23
STB	0.661	3.16	9

Note: GEN, general model; GRN, granular-base model; STB, stabilized-base model.

[a]T indicates that N30 and N31 were included in the model.

(assuming that flexural strength is related to compressive strength) in the AASHTO equation.

The sensitivity analysis of the AASHTO rigid pavement equation is shown below. Similar analyses were conducted for the predictive models.

Variable	Effect
N13	6.4
N16	-34.7
N18	214.0
N46	59.2

The AASHTO design equation predicts the design life (N16) to a particular serviceability index (Pt) as a function of subgrade reaction, concrete flexural strength, and pavement thickness. It is, however, possible to predict the terminal serviceability index as a function of traffic, subgrade reaction, concrete flexural strength, and pavement thickness. A partial comparison of this equation with the predictive models for PCR is thus possible if modulus of subgrade reaction (k) is related to CBR and if concrete flexural strength is related to compressive strength. The following transformation equations were used:

$$k = 55.85 N13^{0.552} \quad (1)$$

and

$$\text{Flex} = 9 \cdot \text{SQRT}(N46) \quad (2)$$

Both the k relationship and the Flex equation were derived from published relationships (9,10).

As shown in the sensitivity analysis, the AASHTO equation is sensitive to pavement thickness (N18) and rather insensitive to subgrade support (N13).

Table 3. Model trends.

Variable	No. of Models Entered	No. Affected Beneficially	No. Affected Detrimentally
N9	25	5	20
N10	25	20	5
N11	24	15	9
N13	30	28	2
N15	11	5	6
N16	30	5	25
N17	30	10	20
N18	29	28	1
N30	8	4	4
N31	7	3	4
N32	20	12	8
N33	28	3	25
N36	6	2	4
N46	30	29	1
N48	29	26	3

Sensitivity analyses of the predictive models showed that thickness was an important variable but its effect was some 10 times lower than in the AASHTO equation, whereas subgrade played a somewhat larger role than in the AASHTO model.

One of the major difficulties in comparing with the AASHTO model is that it predicts terminal serviceability rather than decline of serviceability. The relationship shown in the sensitivity analysis covers a range in Pt from 2.5 to 1.5 (the limits of validity for this equation), i.e., for pavements that have nearly failed. Nevertheless, the pavements evaluated in this study were still in relatively good condition, as reflected by the PCR, which ranges from 50 to 98 with a mean of 75. Thus, the comparisons are not strictly valid. Another difficulty in comparing the predictive models with the AASHTO equation is that the latter is dependent on only four variables, whereas the predictive models incorporate the effects of 8 to 12 variables. Because variation in performance is distributed among a much larger list of variables, the effect of each would be smaller than in a model with fewer variables.

Sensitivity analysis results showed that the models are generally consistent with one another and that the effect of the variables is generally consistent with expectations; there are some exceptions. It is generally accepted that increased rainfall is detrimental to performance; however, the stabilized-base model without temperature shows a positive effect. Also, increased age and traffic should decrease performance, but the granular-base model with temperature shows the opposite trend. It is, however, encouraging that these inverted effects were confined to the stabilized-base models, which had rather poor correlation coefficients.

Most trends shown in Table 3 were more or less consistent with experience. The effects of rainfall (N9), subgrade support (N13), design thickness (N18), thickness deviation (N48), and compressive strength (N46) were according to expectation in a majority of cases, and the reversal of trends was primarily limited to the stabilized-base models with low R^2-values.

The analytical results make it apparent that variation in climate, subgrade support, and pavement thickness have by far the greatest effect on pavement performance and that the effect of concrete quality indicators is secondary. One of the primary reasons for the lack of effect of QC variables is that the values of most QC indicators are controlled via concrete mix design to fall within an acceptable range; i.e., concrete is generally designed to have a slump between 2 and 4 in. and an air content

Table 4. Range of values for variables used in study.

Variable	Mean	Standard Deviation	Minimum	Maximum
N9	45.3	9.8	32.0	65.0
N10	12.0	9.6	0.0	35.0
N11	147.7	177.5	0.0	400.0
N13	8.1	1.8	5.0	12.0
N15	44.0	22.1	0.0	60.0
N16	4.4	3.5	0.2	17.7
N17	11.3	3.8	4.0	20.0
N18	9.1	0.5	8.0	10.0
N30	58.6	10.3	25.8	81.0
N31	78.4	10.2	38.3	95.3
N32	2.1	0.6	0.7	4.4
N33	5.6	1.0	2.0	8.6
N36	0.44	0.17	0.26	0.76
N46	5,350.0	1,076.0	2,753.0	7,833.0
N48	0.20	0.24	-0.97	0.94

around 4 percent. The acceptable limits have been determined from years of experience and when they are exceeded, job-site inspections are expected to reject unacceptable batches. Table 4 presents the means and standard deviations of the QC variables in this study and shows that the allowable limits were rarely exceeded.

Also, a minimum concrete strength is required for satisfactory performance, but once the minimum has been exceeded, performance is not significantly affected. This is confirmed by most of the models, which show that compressive strength does not have a large effect. As shown in Table 4, the mean value for N46 was 5,300 psi with a standard deviation of 1,080 psi; i.e., most concrete exceeded the minimum specification of 4,000 psi.

Of the concrete quality indicators, compressive strength (N46) is generally the most important variable, but the relative ranking depends on the particular model selected. Air content (N33) was also fairly significant, both statistically in that it entered 28 of the 30 models and also because the magnitude of effect is generally larger than that for the other QC variables. Temperature parameters (N30 and N31), when they entered the models, exhibited rather large effects at times, but the statistical significance of these variables is questionable, as previously discussed.

It might be tempting to conclude from this analysis that measuring concrete quality is not justified, but such a conclusion is probably erroneous. Without any control, these variables would undoubtedly have had a significantly larger variation than was found in this study, with a significantly greater effect on performance.

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