Slab Stabilization of Portland Cement Concrete Pavements

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ABSTRACT

The Research and Development Division of the Oklahoma Department of Transportation (ODOT) undertook research to aid in the development of the department's special provisions pertaining to the stabilization of US-69, which had experienced accelerated deterioration and was scheduled for restoration. This effort consisted of determining the proper procedures to effectively stabilize the plain portland cement concrete (PCC) pavement, the quantity of material needed, and a method of verifying the quality of the stabilization operation for this particular highway. A general solution for the stabilization of all pavements is not proposed, but a basis for the method of void identification and the techniques used in pressure grouting is established.

Many of the portland cement concrete (PCC) pavements in Oklahoma are approaching the end of their design life and are showing signs of distress. In the past, a new road would be built or the pavement temporarily upgraded by means of an asphalt overlay; today, there are methods for PCC pavement restoration. This process, known as concrete pavement restoration (CPR), is gaining nationwide attention.

With the advent of the recycling, rehabilitation, resurfacing, and restoration (4R) program, resources were made available to accomplish the much needed work on the Interstate system. The restoration part of the 4R program, particularly the slab stabilization of PCC pavements, is addressed in this paper.

In May 1984, a Demonstration Project 69 (Portland Cement Concrete Pavement Restoration) was done in conjunction with the FHWA Tri-Regional Pavement Rehabilitation Conference in Oklahoma City, Okla. It was observed during the conference that each CPR activity was dependent on the stabilization of the slabs. It was clear that if the slabs were not stabilized properly, other activities such as diamond grinding, joint sealing, and patching may not perform as desired.

Research to develop procedures for slab stabilization began in July 1984. The facts and conclusions drawn from this research are documented in this paper and specifications and procedures are recommended.

BACKGROUND

As a PCC pavement ages, several signs of distress such as faulting and pumping, may be detectable. These symptoms, together, indicate the presence of voids in the supporting layers beneath the pavement. Usually the voids are caused by the infiltration of water, along with the presence of an erodible material in the supporting strata, and the movement of the slabs associated with traffic (1).

The method of slab stabilization, addressed in this paper, is the process by which grouting material is pressure injected beneath a pavement so that it will be laterally distributed into voids that are present to displace either free water, air, or both, that may be located in them. Once the grouting material is injected and allowed to harden, it should provide incompressible, nonsoluble, nonerodible support to the pavement (2).

The Research and Development Division of the Oklahoma Department of Transportation has been investigating the methods of slab stabilization currently in use. A literature review including abstracts revealed that there was little available on the subject of slab stabilization. Requests to several states for specifications on slab stabilization revealed a large variation in requirements from state to state. Therefore, a research effort was outlined and equipment purchased.

The purpose of this study was to determine the pressure grouting criteria needed for the development of the Oklahoma Department of Transportation Special Provisions for Pressure Grouting Portland Cement Concrete Pavement for Federal Aid Project MAF-186(170). The study determined (a) the proper procedures to effectively stabilize the pavement, (b) the quantity of materials needed for a full-scale restoration, and (c) a method of verifying the quality of the stabilization operation. The evaluation and verification of the technique and equipment associated with the Transient Dynamic Response (TDR) method were included, not only for void identification, but also to determine the feasibility of the TDR method as a quality control measure for slab stabilization operations.

LOCATION

The highway selected for the research was a 13-mi section of US-69, south of Muskogee, Oklahoma (Figure 1). This section, constructed in 1978-1979, consists of a 9-in. plain PCC pavement with sawed joints every 15 ft on a 4-in. fine aggregate bituminous base (FABB) with either 0, 6, or 12 in. of lime-treated subgrade. A typical section of the roadway is shown in Figure 2. The road was designed with a predicted average daily traffic (ADT) of 17,300 for 1996. The ADT is currently 9,800 with 21 percent trucks.

Within the first year after construction minor faulting was detected at the joints. During the next few years the faulting increased in magnitude and

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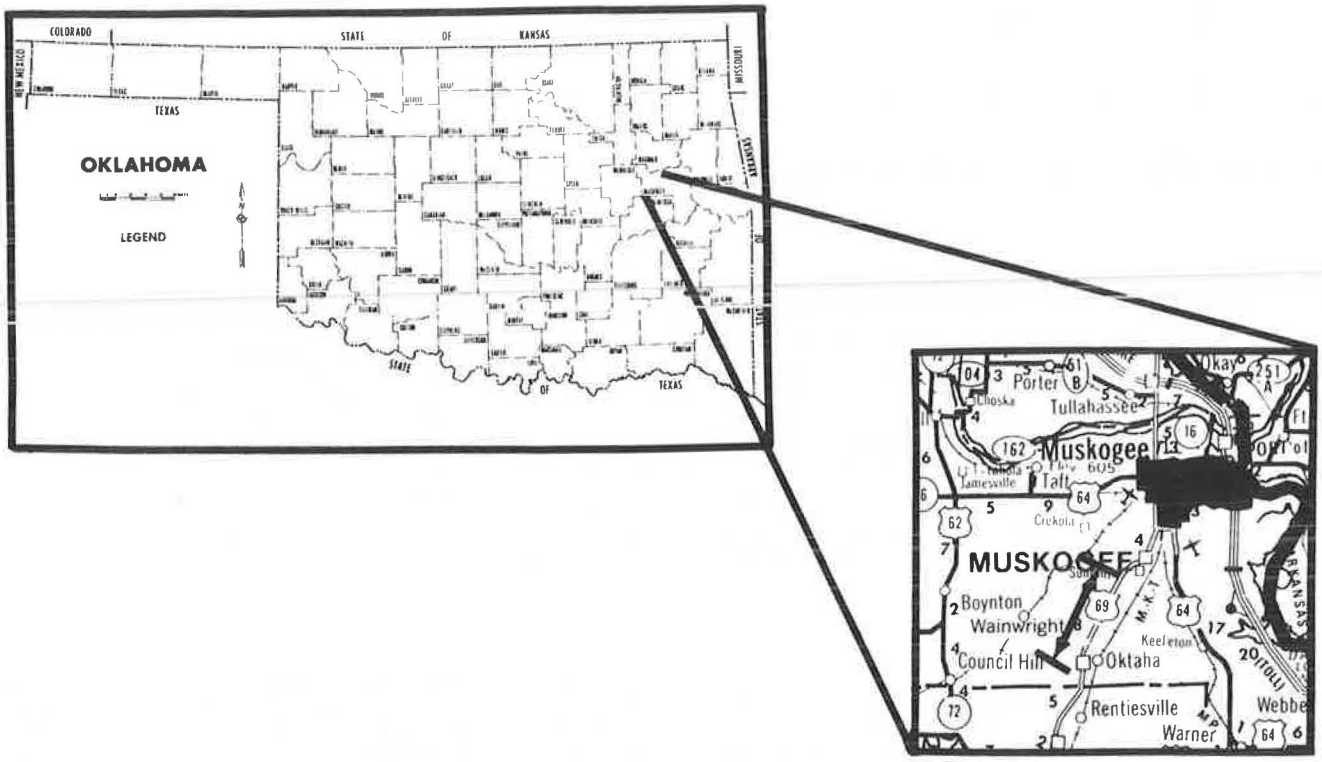

FIGURE 1 Map of location.

pumping became evident. About 4 years after construction diminishing ride quality attracted some concern.

A core investigation was conducted to evaluate the condition of the pavement and the subgrade. The PCC cores showed satisfactory strength, while the subgrade soil condition was defined in the range from poor to very poor. Throughout its life, this segment of highway had been the subject of several investigations. It was scheduled to become Oklahoma's first CPR project to begin in late 1985.

In order to present the findings of the studies, this paper has been organized into six categories: Void Identification, Grouting Material, Injection Holes, Grouting Operation, Quantities, and Quality Control. All slabs tested were in the outside lane. All the voids were identified as being in the base and within 2 in. of the pavement-base interface. For brevity, only the data from Site 1 are included.

VOID IDENTIFICATION

To fill a void in the supporting strata beneath a pavement, it is necessary to know the characteristics of the void. The dimensions, location, and boundaries of the void are important variables that must be investigated to determine what procedures should be taken. The dimensions (area and thickness) of the void determine the quantities of material required for filling the void. The location (void pattern and depth) of the void determines the frequency, location, and depth of the injection holes. These, along with the boundaries (confined or vented, material above, below or in the void, and moisture) of the void, will determine the pumping requirements for the proper injection of the grout.

Many tests are currently available to the engineer for obtaining required information. These tests are classified as either destructive or nondestructive. The destructive methods include coring, lifting of slabs, and excavation of the shoulder. The nondestructive testing (NDT) methods include visual surveys, roughness surveys, fault measurements, deflection measurements, and other sophisticated techniques.

For the current investigation, a new, nondestructive state-of-the-art method of void identification, known as TDR, was used along with other conventional

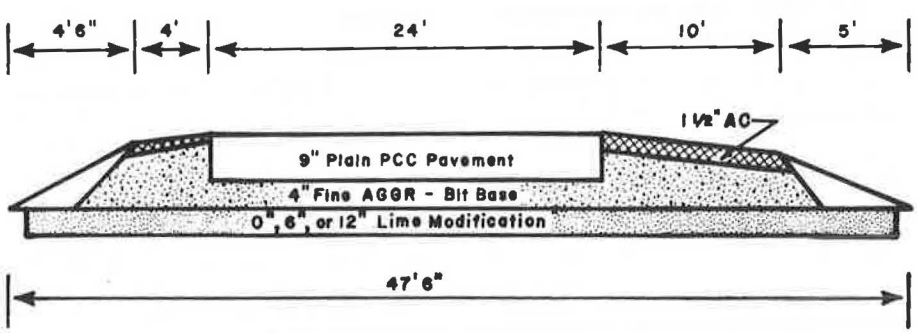

FIGURE 2 Typical section of US-69.

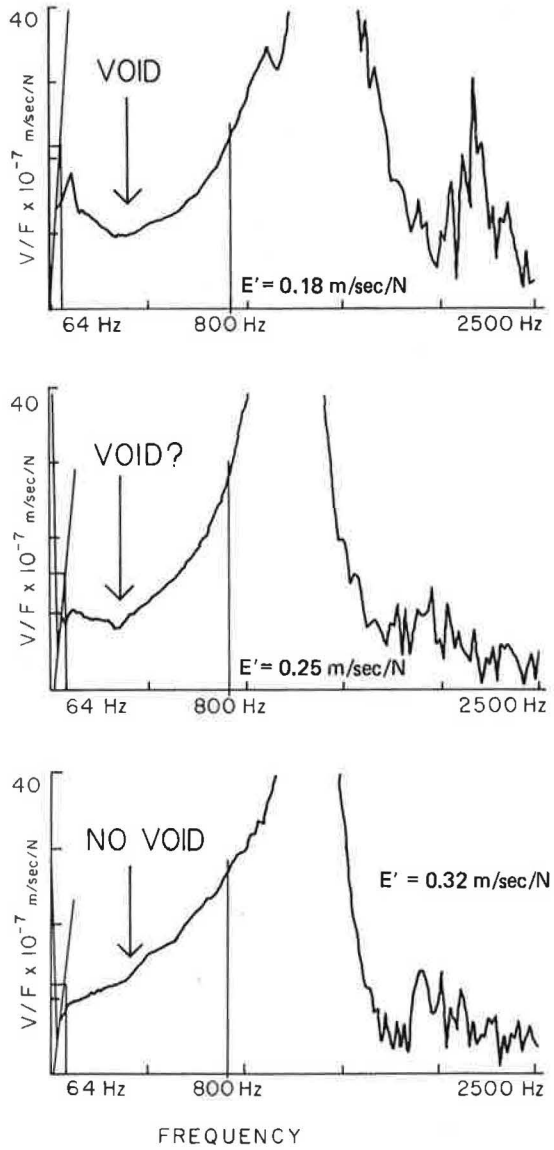

FIGURE 3 Transient dynamic response output.

tools. The basis of the TDR method is the analysis of steady-state vibration by the measurement of mechanical impedance. This method has been proved in the testing of piles in Europe (3).

The TDR method utilizes a twin-channel Spectrum Analyzer, an HP-85 computer, a plotter, a geophone, and a hammer with a built-in force transducer. The test is performed by placing the geophone firmly on the slab surface, and striking the pavement with the instrumented hammer. The distance between the geophone and the point of impact is kept constant throughout the testing. Force and velocity signals are recorded and plotted out as the mechanical admittance versus frequency (v/F) via Fast Fourier Transformation Analysis. The slope of the low frequency (up to 100 Hz) portion of the response curve is the apparent slab flexibility, S. The apparent slab stiffness, E' = 1/S, is expressed in units of meter per second per Newton (m/sec/N). The apparent slab stiffnesses can also be found on the plot. Three plots, as shown in Figure 3, are labeled VOID, VOID?, and NO VOID.

The existence of support for a particular point can be determined by the continuous increase of the curve in the 0 to 800 Hz range, as illustrated by NO VOID. A void is identified by any down slope in this region, as illustrated by VOID. The magnitude of the void, that is the thickness, can be correlated to the peak-to-valley difference. The larger the difference the thicker the void. A small difference, as illustrated in VOID? generally denotes a delamination.

Through a series of tests on a given slab, information can be assembled to yield a map of the void pattern. The more points tested, the more accurate the void pattern. A map of a typical void pattern is shown in Figure 4.

In order to establish the reliability of the TDR method of void investigation, three sites (good, average, poor) were chosen. Selection was based on previous work in which it was determined that the level of faulting correlated with the Mays Ride Meter data on US-69. Each site consisted of 90 consecutive slabs. Within each site, nine slabs were selected for repeat testing to verify the consistency of the TDR method, and 15 slabs were selected to be stabilized and retested. Also, a slab was removed within each site, based on the data that was provided, and several cores were taken to verify the results.

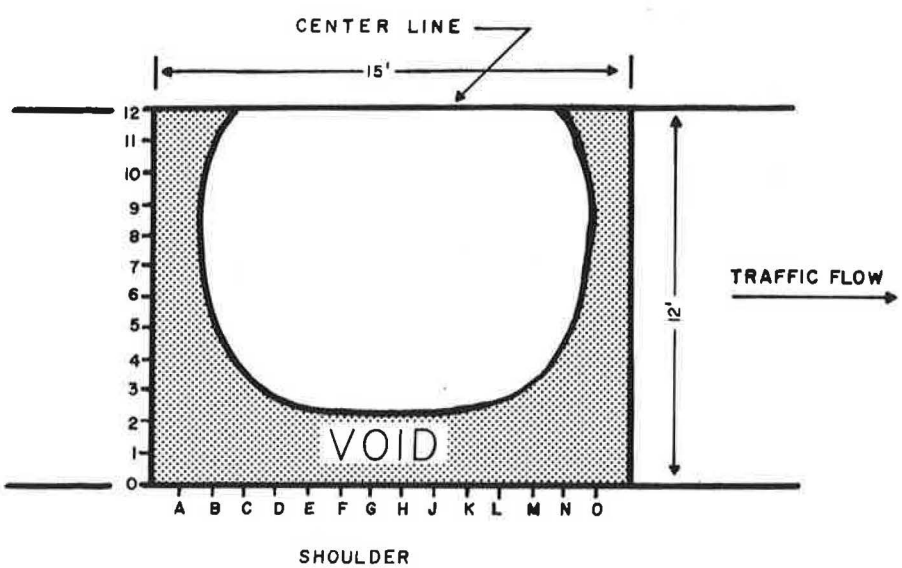

FIGURE 4 Map of void pattern with grid system.

The data presented is limited to Site 1. This site was characterized, from previous work, as average. All 90 slabs were tested and mapped. The following day nine slabs were retested. The same slabs were tested again the third day. The stiffness values of the repeat testing for Points B3 and 03 on the slabs are shown in Figure 5. The area of the voids did not change significantly during the repeat testing.

Fifteen stabilized slabs were also checked. The area of the voids before grouting and after grouting is plotted in Figure 6. The results indicated that the area of voids decreased with stabilization. However, subsequent testing indicated that the voids reappeared when traffic resumed. This may be attributed to the pounding action of traffic across the faulted joints; the pavement was not subjected to diamond grinding.

Before and after stabilization, the slabs were subjected to deflection measurements according to industry standards (1,4-6). A truck with a single-axle load of 18 kip was used for both static and dynamic loading and two dial gages were used to measure the deflections. All deflections recorded with the 18-kip loading method indicated that voids were not present. In fact, most of the readings were comparable to readings taken on a new pavement before traffic exposure (in the range of 5-thousandths to 10-thousandths of an inch). However, the TDR method identified voids under these slabs. The voids accepted grout, and cores confirmed that the voids existed as mapped.

Another approach was to remove one unstabilized slab from each of the three sites. It was found that the TDR method was not only useful in finding and mapping the voids, but delamination in the base as small as 1/16-in. thick could also be located. Although the TDR method identified the area of the void, the thickness of the void could only be determined through destructive means, and, therefore, the volume of the void could not be determined. Also, multiple layers of voids were not identified.

Slab testing became a nighttime operation when the daytime slab temperature exceeded 75°F causing slab lockup.

GROUTING MATERIAL

The material used in the investigation was the standard mix (2,4-7), that is one part (by volume) Portland Cement Type 1, three parts (by volume) Class C fly ash, and sufficient water to achieve fluidity. The material was purchased premixed in 80-lb sacks. No additives were used.

The fluidity of the grout was monitored with the use of a flow cone in accordance with ASTM C 939-81. Generally, the efflux time of the grout ranged from 10 to 12 sec, compared to water at 8 sec.

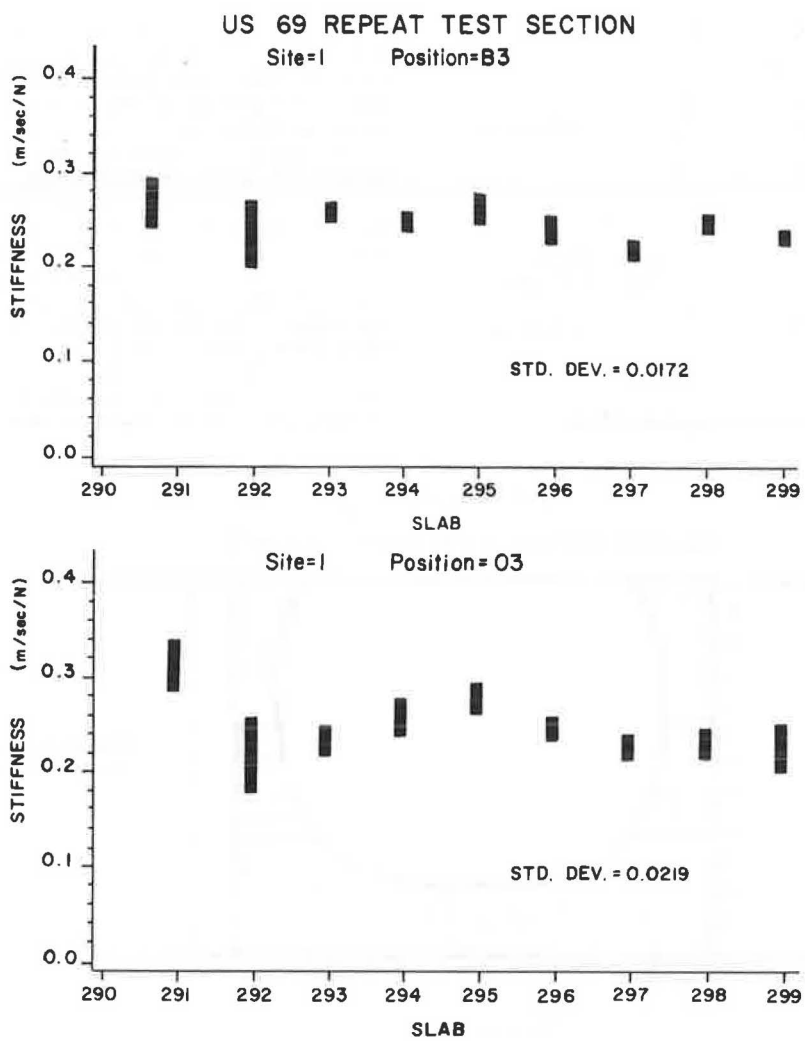

FIGURE 5 Plots of repeat testing.

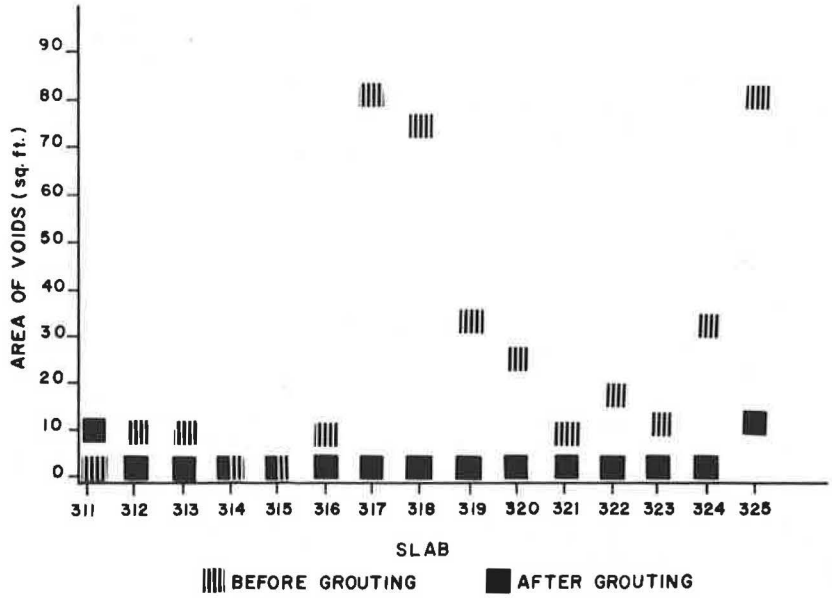

FIGURE 6 Area of voids (before and after grouting).

The mix design used in the field was 13.2 gal of water for a four-sack mix (w/c = 0.34). A laboratory report on this design is shown as follows:

- Mix Design Based on Absolute Volume
  1 part cement (AASHTO-M-85) + 3 parts Class C fly ash (ASTM-C-618 C), 51 percent
  Water, 49 percent
- Compressive Strength Averages for Three Molded 2-in. Cubes (AASHTO-T-106)
  1-day testing 141 psi
  3-day testing 203 psi
  7-day testing 811 psi

  Flow of mixture, 20.2 sec (ASTM C-939)
  Shrinkage or expansion, -7.9 percent (ASTM D-472)
  Water retentively 60 ml, 172 sec (ASTM-C-941)
  Initial set, 180 min (AASHTO-T-131)

The set time in the field did not correspond to that in the laboratory. In many cases the grout appeared not to have hardened for several days. This was evident in cores that were taken, as well as from the slow hardening of discarded grout that was piled off the shoulder.

INJECTION HOLES

To fill a void supporting strata beneath a pavement, it is necessary to provide injection holes to access the void. The diameter, frequency, location, and depth of the injection holes are important variables that must be considered for effective PCC pavement stabilization.

The diameter of the injection holes and the thickness of the void determines the cross-sectional area of the void that is exposed to the infiltration of the grouting material. If the diameter of the injection hole is reduced by one-half, then the cross-sectional area of the void is also reduced by one-half and vice versa.

The frequency and location of the injection holes are subject to the ability of the grout to disperse through the voids. If the voids are large (thickness at the injection hole greater than 0.20 in.), then the grout is more likely to be distributed far from the injection hole, and perhaps communicate up through other injection holes or up through the joints. In this case, a few injection holes might be all that are needed. However, if the voids are in the early stages, there may be a need for many holes. It is important to keep the number of injection holes at a minimum while providing adequate filling of the voids.

The location is also dependent on the borders of the slab. The injection holes are usually located at least 18 in. from any edge (1,5). The depth of the injection holes determines which voids are subject to the infiltration of grouting material. If a void is lower than the bottom of the injection hole, it is unlikely that the grout will penetrate to that level. If there are several layers of voids in contact with the injection hole, the grout may only flow in the channel of least resistance. Therefore, repeated grouting of a particular location may be necessary to ensure that all the voids are filled.

Ideally, to stabilize a PCC pavement the injection hole should be designed to expose the maximum cross-sectional area of the void to the infiltration of grouting material; the frequency, depth, and location of the injection holes should provide for proper distribution of the grouting material.

The method of drilling injection holes has been improved since the days of mud jacking. However, limiting the weight of the hammer and the down pressure applied, has not prevented the occurrence of the bottom breakout of the slab known as cone failure. This failure is the conical shattering of the bottom 2 in. of the slab within a 6-in radius of the injection hole. Cone failure still occurs to a large degree, although less frequently. To avoid this phenomenon on the US-69 project, the injection holes were cored. It takes more time to core holes than to drill them with a rock bit. However, coring equipment is available that can compete in terms of expedience as well as economy.

GROUTING OPERATION

Several factors lead to the success of the pressure grouting phase. Assuming the voids have been identi-

fied, the injection holes properly selected and placed, and the proper grouting material available, the pressure grouting sequence begins.

As mentioned earlier, the cross-sectional area of the void exposed to the infiltration of grout must be maximized. Some states have already implemented the flushing of injection holes with either air, water, or both. The US-69 investigation supports this practice. No problems were encountered when injecting grout into holes that were flushed with water. Flushing the holes with water removes debris that could potentially block the grouting channel. Also, by coring the injection holes, fragments of concrete associated with the blocking of the grouting channel were eliminated, thereby eliminating the high pressures associated with the initial injection of the grout.

The pressure at which the grout is injected plays an important role in slab stabilization because the uplift of the slab should be held to a minimum. Theoretically, for a 12- by 15-ft free-standing slab of 9-in. PCC, an applied pressure of 1 psi is all that is needed to lift it. Additional pressure in the field is necessary only to overcome the frictional and mechanical, or both, restraints. For this investigation, there was no need to exceed 60 psi.

The flow rate at which the grouting material is injected controls not only the rate of distribution but also the pressure buildup. The grouting material must be pumped in a manner that will allow time for continuous distribution throughout the void. If the grouting material is pumped at too high a flow rate, it may not have enough time to penetrate and the pressure will begin to build up. If the grouting material is pumped at too low a flow rate, the flow may not be continuous causing blockage and pressure buildup. A flow rate of 7 gal/min produced the desired distribution of grouting material during the US-69 investigation.

Another important aspect of pressure grouting is the displacement of the free water and air. If the void is confined, water or air cannot escape. Because water is incompressible, there is the potential that an air pocket will be formed in the grout matrix when the water dissipates. However, if seepage of grout is noted in the joints or in adjacent holes, the possibility of pockets being formed is minimal because the specific gravity of grout is greater than that of water or air.

Pumping at the injection hole was stopped if (a) the pumping pressure exceeded 60 psi (b) excessive grout was observed seeping up through the joints, or (c) if the slab was raised more than 0.035 in. The low pressure and moderate flow rate allowed for greater control, and good distribution of the grout, thereby preventing excessive raising of the slab.

QUANTITIES

The quantities of grouting material required for a full-scale restoration project on US-69 were based on the volume of grout used to stabilize various slabs. It was found that the average slab required less than 2 ft^3 of material for stabilization.

QUALITY CONTROL

Verification of the grouting operation has been a most difficult task. Following the guidelines relating to proof rolling, it was found that the 18-kip loading method can be meaningless because of the low magnitude of the deflections. On the other hand, the TDR method proved successful.

The TDR method yields two quantitative results, stiffness and void area. When an unstable slab has been stabilized, stiffness increases and the void area decreases. However, the TDR method is labor intensive, costly, and subject to environmental conditions.

The most critical factors in a successful stabilization project are the inspector and the contractor who should have a thorough working knowledge of the entire process, because their level of understanding determines the quality of the job (8).

SUMMARY

The effectiveness of slab stabilization depends on the identification of the voids, the grouting material, the design of the injection holes, the techniques of grouting, and the competence of the workers; the TDR method can map void patterns and measure the relative stiffness of PCC slabs; and finally, note that the 18-kip loading method of void detection can be meaningless.

RECOMMENDATIONS

The following recommendations are associated with the special provisions relating to the pressure grouting of US-69.

1. Core the injection holes,
2. Flush the injection holes with water before grouting,
3. Limit maximum pressure to 60 psi,
4. Specify the flow rate at 7 gal/min,
5. Do not specify repeat grouting,
6. Estimate the quantities based on 2 ft^3 of material per slab, and
7. Do not specify proof rolling.

Other general recommendations:

1. Further investigation is needed to verify and refine the foregoing recommendations on a project-by-project basis,
2. The TDR method should be further investigated as a quality control method for slab stabilization, and
3. The effects and timing of each CPR activity on the stabilization of PCC slabs should be investigated.

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Portland Cement Concrete Pavement Performance as Influenced by Sealed and Unsealed Contraction Joints

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ABSTRACT

In the 1950s and 1960s, the contraction joints in portions of several Wisconsin pavements were purposely left unfilled in an effort to determine the effect joint filling (and routine refilling) had on subsequent pavement performance. After 11 to 19 years of observation it was determined that the initial filling or refilling of contraction joints (40- to 100-ft spacings) had no beneficial effect on overall pavement performance. In 1974 a carefully designed joint and sealant study began with the objectives of evaluating the effect of joint spacing and joint sealing or nonsealing on total pavement performance, and evaluating joint sealants. This study was conducted on a new 9-in. jointed reinforced concrete pavement on a well-drained subgrade, employed five joint sealants, and considered four joint spacings (20, 40, 60, and 80 in.). A total of 22 test sections were evaluated, including eight control sections in which the joints were left unsealed and 14 test sections in which the joints were sealed (the joints in these test sections were resealed to maintain a sealed system for 10 years). Based on 10 years of monitoring total pavement performance (considering summer and winter ride, pavement distress and material integrity), it was found that some sealants served well for 10 years, short joint spacings gave the best pavement performance, and the pavement with unsealed joints had better performance than the pavement with sealed joints. It was concluded that there may be conditions and circumstances that do not justify the cost of sealing PCC pavement joints.

In order to understand the origin of the subject of this paper, it is helpful to review past experience in Wisconsin relating to sealing of joints in portland cement concrete (PCC) pavements. In 1953 a jointed plain concrete pavement (JPCP), with a 40-ft contraction joint spacing of 0.25-in. wide joints, was built on US-151 in two contiguous counties (Lafayette and Iowa) in the southern part of the state. In Wisconsin the counties perform the maintenance work, and, at that time, PCC pavement joint resealing was, or was not, routinely performed in each county based on a number of factors. In both counties the joints and cracks were sealed (actually filled) with an asphalt-based sealant at the time of construction; but in Iowa County they were routinely resealed (refilled) to prevent the intrusion of incompressibles and water, whereas there was no reseal-

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