TRANSPORTATION RESEARCH RECORD 752

Sealing Joints and Cracks, Thin Resurfacing, and Locating Voids Under Concrete Slabs

TRANSPORTATION RESEARCH BOARD

COMMISSION ON SOCIOTECHNICAL SYSTEMS
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1980

Transportation Research Record 752
Price $4.00
Edited for TRB by Mary McLaughlin

modes
1 highway transportation
4 air transportation

subject areas
32 cement and concrete
33 construction
40 maintenance
41 construction and maintenance equipment

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Sealing joints and cracks, thin resurfacing, and locating voids under concrete slabs.

(Transportation research record; 752)
Reports prepared for the 59th annual meeting of the Transportation Research Board.
1. Pavements, Concrete–Maintenance and repair–Addresses, essays, lectures. I. Title. II. Series.
TE7.H5 no. 752 [TE278] 380.5s [625.8'4] 80-23303
ISBN 0-309-03062-5 ISSN 0361-1981

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Effect of Defective Joint Seals on Pavement Performance

Gordon K. Ray

The reasons for installing and maintaining effective sealants in pavement joints and cracks are discussed. There is some controversy about the need for such seals. Results of studies done in Europe on the performance of pavement with unsealed joints are presented. These studies conclude that in most cases effective joint seals will minimize pavement distress. Various types of distress that develop from joint-seal failures are described.

Joints in concrete pavements are necessary, but they can be the source of many problems and subsequent pavement distress if they are improperly designed, constructed, or maintained. Joints are designed to control cracking, minimize stresses in the pavement caused by volume change, and prevent damage to immovable structures. Joints are expected to provide some load transfer between adjacent slabs and thereby prevent a free-edge condition, reduce pavement deflections and stresses, and prevent faulting at joints. Joints are sometimes designed with a reservoir for a joint sealant that will prevent surface water and incompressible foreign materials from entering the opening. It is this last function that is of concern here.

Joint sealants are designed to bond to the concrete in the joint. They are made to withstand many cycles of tension and compression as the joint opens and closes. Sealants are intended to create a waterproof barrier that will prevent surface water from entering the joint and reaching the subbase and subgrade. To be effective, sealants must also resist the intrusion of incompressible surface material—sand, gravel, stone, and other foreign objects—into the joint reservoir and the crack or joint below the seal.

Since most sealants have a limited service life, joints must be resealed periodically to ensure that they will perform the functions for which they are designed.

What effect do defective joint sealants have on pavement performance? Do sealant defects prevent joints from performing their proper function in the pavement?

PERFORMANCE OF PAVEMENT WITH UNSEALED JOINTS

Unfortunately, there is not complete agreement among paving engineers on the need for sealing all pavement joints. California uses a plain pavement design with short joint spacing [an average of 4.7 m (15.5 ft)] and an erosion-resistant subbase. The joints are sealed only in mountainous areas where there is greater than average precipitation.

In 1979, at the 16th World Congress of the Permanent International Association of Road Congresses (PIARC), the Technical Committee on Concrete Roads presented a report (1) that stated that Spain and Austria build many kilometers of pavement with unsealed joints and that France and Germany have both built substantial test sections with unsealed joints. It was also pointed out that there are hundreds of kilometers of concrete pavement built with sealed joints that did not require any maintenance for many years. The PIARC report concludes that, with joint spacings of 4-6 m (13-20 ft), there is no disadvantage in leaving narrow transverse joints unsealed when (a) traffic is light, (b) traffic is heavy but the climate is dry, and (c) traffic is heavy and the climate is wet but the pavement is doweled.

Most research in the United States on test pavements with sealed and unsealed joints has demonstrated some improvements in performance when joints are kept reasonably well sealed.

PAVEMENT DISTRESS RESULTING FROM JOINT-SEAL FAILURES

Today, many engineers are concerned about water in pavements. Workshops, technical papers, research studies, and even textbooks have focused attention on this problem. Cedergren (2) has called attention to the large volume of water that can reach the subbase or subgrade through the joints in a concrete pavement. Water in pavements or, more importantly, water that reaches the subbase or subgrade under a concrete pavement can result in activity that leads to pavement distress.

Pumping—the ejection of a mixture of soil and water from beneath slabs at joints, cracks, and edges—is one of the first symptoms of pavement distress. Mud pumping can occur when concrete pavements are placed directly on fine-grained, plastic soils. Under certain conditions, fines can be pumped from poorly graded granular materials and even from cement-modified soils. Continued, uncontrolled mud pumping can lead to displacement of enough subsoil to create voids under the slab, destroy the uniformity of support, and leave slab ends unsupported. Cooperative pumping studies (3, p. 281) have shown that three conditions are necessary for mud pumping to occur: (a) a subgrade soil that will go into suspension, (b) frequent passage of heavy wheel loads, and (c) the presence of free water between pavement and subgrade.

Pumping of granular subbase occurred on the structurally underdesigned sections of the AASHO Road Test and led to excessive deflections, numerous cracks, and eventual pavement failure. Water in the subbase was an important factor in the process, since pumping of subbase material was observed only during and after rains (4, p. 171).

Pumping or water action at joints, cracks, and pavement edges can also result in faulting of pavement joints and cracks. Faulting can be caused by voids under the leave slab (the pavement panel on which a vehicle leaves the joint, as opposed to the approach slab) that permit settlement and may eventually lead to transverse or diagonal slab cracks 1.8-3 m (6-10 ft) beyond the faulted joint or crack.

Faulting of joints and cracks on stabilized subbases has been attributed to water action under traffic that results in a migration of fine material from the shoulder or the subbase under the leave slab to the subbase under the approach slab. The deposits that build up under the approach slab lift it above the leave slab, which creates a fault.

Studies in California (5) and Georgia (6) have identified this phenomenon. As in the case of pumping, this research has shown that free water must be present to create the conditions that lead to faulting. In California and Georgia, faulting was shown to lead to pavement cracking ahead of the joint if the deposit of material under the approach slab raised the pavement enough to destroy the uniformity of subbase support.

One other form of distress in concrete pavement can be attributed to water action in joints. Corrosion of embedded steel in concrete slabs is accelerated when the brine solution from deicing salts enters joints that are not effectively sealed. Investigations have shown that such accelerated corrosion can cause serious problems, particularly in northern states where large amounts of deicing salts are used. Tie bars installed in longitudinal joints to hold two slabs together are sometimes found to be ruptured as a result of corrosion. Dowels or other load-transfer devices can become badly corroded after several years of service. This corrosion can result in a reduction in cross section and subsequent rupture, or it can cause the free end of the dowel to become immobilized as a result of expansion.

It is obvious from these problems that free water under a pavement can lead to distress. If properly installed and maintained joint seals will prevent surface water from reaching the subbase and subgrade and from entering the shoulder joint, several major forms of pavement distress can be avoided. Waterproof sealing of transverse joints, longitudinal joints, pavement-edge shoulder joints, and open cracks should be an objective during both construction and subsequent maintenance (7).

Distress caused by infiltration of incompressibles may be of even greater concern than damage from water in pavement. Narrow joints in plain concrete pavements with short slabs are subject to some infiltration, but far greater damage can be caused by the long panels used by some states, which have mesh dowel designs that result in excessive joint openings.

If joints are unsealed or if joint seals are ineffective, foreign materials from the shoulder, surface, or subgrade can enter the joints while they are open in cold weather. When the pavement expands during hot weather, the incompressible materials cause nonuniform pressures on the joint faces. Continued expansion of the pavement can cause stresses great enough to cause joint spalling at the surface, the edge, or even the bottom of the slab. Infiltration at the edges of joints can cause longitudinal restraint cracks that can actually split the slabs.

If joints become filled with foreign material and are then subjected to slab expansion during hot and humid weather, serious compression failures can result. In extreme cases, actual pavement buckling or blowups can develop. If transverse joints are allowed to remain unsealed and to fill with foreign material, the joint openings will increase in size and pavements will tend to grow in length, resulting in closing expansion joints at structures and, eventually, damage to bridges and bridge abutments.

Most joint sealants today are designed to resist the entrance of foreign materials from the surface of the pavement. Many of the older type of sealants, however, held such materials until the joint eventually filled. Simply adding additional joint seal to a joint or crack already filled with incompressibles will not help the situation. All foreign material must be removed before resealing.

Unsealed cracks may also contribute to pavement distress if the joint spacing permits the crack to open or if the reinforcing across the crack fails. At first the crack may be narrow and fairly tight, with no spalling. As the crack opens, however, infiltration begins and spalling results. Water action caused by slab deflection at cracks can cause faulting at cracks as well as at joints.

Finally, there is another effect of defective joint seals that does not receive as much attention: Overfilling joints can have a very detrimental effect on the riding quality of the pavement. A few years ago, it was common practice to pour large quantities of joint sealer on the pavement surface over joints and cracks, but this was ineffective as a joint sealer and actually detrimental to pavement appearance and rideability.

Excessively wide, black joints give the impression of bumps and lack of evenness or continuity in the pavement surface. The resulting wide band of sealant is smooth and even textured, quite different from the adjacent pavement texture. It is common for the tires of vehicles to make a slapping or sucking noise as they cross these oversealed joints or cracks. In many cases, the overfilled sealant with embedded foreign material creates a measurable bump on the pavement surface. Such high spots, which are usually found at regular intervals, make for a rough ride and produce a very objectionable thumping noise. Thus, although overfilled joints may not cause serious distress in the pavement, they certainly have a negative effect on the appearance and ride quality of the pavement. In some cases, excess sealant sticks to tires and is pulled from the joint, destroying its effectiveness.

SUMMARY

Some of the more obvious detrimental effects of unsealed or poorly sealed joints and cracks have been discussed here; more could be mentioned. Several sources (8-13) and other papers in this Record provide more details on the subject.

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Publication of this paper sponsored by Committee on Pavement Maintenance.

Abridgment

Pavement Design Features and Their Effect on Joint-Seal Performance

I. Minkarah and J. P. Cook

The effect of the design parameters of pavement on the long-term performance of joint-sealing materials is studied. The work is a compilation of the work of many researchers in the field of pavement joint sealing. The design features considered are type and size of joint, type of subbase, length of slab, slab thickness, type of load-transfer device, temperature and moisture range, and the material properties of the concrete in the pavement. The effect of each of these features is considered in some detail, and recommendations are made based on the results of past research. No attempt is made to evaluate the merits of the various types of joint-sealing materials.

The joints in rigid pavements are usually sealed to prevent the intrusion of water, incompressible solids, and chemical deicing solutions. The intrusion of these materials could have a detrimental effect on the joint and the pavement system and result in faulting, spalling, blowups, and other distresses common to rigid pavements.

The sealing material must accommodate the repetitive movement between pavement joints while maintaining its integrity. It is the design features of the pavement that determine how much movement the sealant must accommodate.

The movement of a pavement slab is a function of many variables. The most prominent of these factors are type and size of joint, length of slab, slab thickness, type of subgrade, type of load-transfer device, temperature, moisture, material properties, and type and volume of traffic. Joints are introduced into the pavement because the free movement of the pavement is restrained and this induces stresses that cause cracking. The joints are designed and spaced to control cracking.

In order to design the joint properly, one must know the magnitude and direction of movement in the pavement. A slab between joints can move horizontally because of a change in temperature and moisture, or it can curl because of a difference in temperature between the top and bottom of the slab. The horizontal movement of the slab is resisted by the friction between slab and subbase, which induces stresses in the slab. These stresses are shear stresses, accompanied by either tension or compression, depending on the direction of movement. Flexural stresses are induced in the slab under traffic loads. These stresses are greatly increased when the slab lifts off its base as a result of temperature or moisture gradient.

Another type of movement that may occur is differential settlement caused by the subbase or subgrade.

TYPE OF JOINTS

Longitudinal Joints

Longitudinal joints can be located between lanes or between the edge of the pavement and the shoulder. They are used to restrict the lateral and vertical movement of the joint and to relieve the warping stresses induced in the pavement by the temperature and moisture differentials between the top and bottom surfaces of the slab.

Tie bars that are usually 12.7 or 15.9 mm (0.5 or 0.625 in) in diameter are used in all states as well as in Europe (1, 2). They are effective in holding the slabs in contact and in the same vertical plane. Aggregate interlock provides the load transfer across the joint. When the lanes are constructed separately, a longitudinal construction joint is used.

The longitudinal edge-shoulder joint is relatively new in concrete pavements but has been receiving a great deal of attention and study in the past few years. A 1975 highway survey (3) shows a great deal of distress at this joint when asphalt shoulders are used, including a drop-off from the edge of the pavement that can be as great as 5.1 cm (2 in).

Proper sealing of longitudinal joints is not a major problem. If the joints are tied (all center joints and some edge joints), there is very little demand on the performance of the sealant. Consequently, the movement is rather small and the joint can be sealed by using hot-poured materials. Premolded seals can also be used if the joint is sawed. If, however, the joint is not tied—as, for example, when an asphalt shoulder is used—the horizontal and vertical movement becomes large. In such an instance, a low-modulus silicone sealant may be more effective (4).

Transverse Contraction Joints

Contraction joints are designed to control random cracking attributable to warping, frictional stresses, load stresses, shrinkage, and thermal and moisture changes by providing a weakened plane in the concrete slab where the crack occurs. Unfortunately, not all joints crack when they are supposed to, and sometimes they are sealed before cracking. Tension in the sealant results when the joint is finally relieved. The first joints to crack tend to open wide, since they have to accommodate the movement of more than one slab. If they are sealed at this stage, the sealant tends to become highly compressed and even extruded in hot weather.

Besides horizontal movement caused by a change in temperature and moisture, the slab tends to curl because of differences in temperature and moisture between the top and bottom of the slab. The ends tend to lift up when the surface of the pavement is cooler than the bottom. The slab assumes a reverse curl when the surface is warmer than the bottom. The pavement is constantly under fatigue loading caused by traffic. When the top surface of the pavement is cooler than the bottom and the pavement ends are curled upward, traffic loads depressing the slab ends accentuate the fatigue-loading situation. Consequently, the sealant in a transverse joint is subjected to high adhesive and cohesive tensions as well as shearing stresses. The magnitudes of these stresses are dependent on the environment, the length of span, and other factors.

The type of sealant and the dimensions of the joint are designed to correspond to the expected movement in the pavement. Shearing stresses should be considered in the design of the seals. Faulting induces shear-type stresses in the sealant section. In field-molded sealants, this displacement increases the tensile stresses and in some instances doubles the strain on a joint sealant. According to Thornton (4),

> Preformed open cell seals cannot normally accommodate faulting in excess of 3.2 mm [0.125 in] without slipping on the joint face. They are not designed for a shearing type movement and the greater the degree of compression, the less the shearing movement that can be accommodated.

Sealant failures can generally be attributed not to deficiencies in the seal material but to poor joint designs (too narrow, too deep, or with too large a movement), which subject the sealants to excessive stresses (1).

SIZE OF JOINT

In discussing the size of the joint, we should differentiate between field-molded and preformed seals. Tons (5), in his theoretical study of rectangular field-molded seals, showed that the greater the minimum width and the shallower the sealant in a joint, the less is the strain in the sealant.

Preformed seals are precompressed and inserted into the joint in the compressed state. As they attempt to return to normal shape, they exert a force against the joint wall, thus forming an effective seal. The seal must be exactly sized for its joint opening. The recommended working range of the preformed seal was suggested to be 30 percent of its initial width with a minimum 20 percent compression (1, 6).

According to the 1975 survey of practice (3), joints in concrete pavements are formed mainly by sawing. The width of the joint in reinforced and doweled pavements varies from 3.2 to 19.1 mm (0.125-0.75 in) with no correlation between width and slab length; the depth varies from 5.1 to 8.3 cm (2-3.25 in). The depth of the joint in plain concrete pavements is D/4, and the width varies from 3.2 to 9.6 mm (0.125-0.375 in). Obviously, more attention should be paid to the dimension of the joint.

SLAB LENGTH

The opening and closing of a joint, and thus the stresses in the seal, are a direct function of the length of the slab between the joints. Long slabs result in larger openings than short slabs. It is prudent to use shorter slabs for the following reasons:

1. Because the joint will be narrower, incompressibles will slide more easily over it rather than getting embedded in the sealant.
2. There will be less movement and, therefore, smaller stresses in the sealant.
3. Continuity between adjacent spans will be improved, and this will result in an increase in load transfer through aggregate interlock.
4. Joint performance will be improved and spalling and blowups reduced (3).
5. Intermediate transverse cracks will be minimized or even eliminated.

Joint spacing in plain pavements in different states varies from 4.6 to 9.2 m (15-30 ft). Plain pavements with a joint spacing greater than 6.1 m (20 ft) show a marked loss in aggregate interlock, which increases the risk of faulting. These longer slabs also tend to develop midslab cracks. Since the cracks are not restrained by reinforcement, they tend to widen and spall more easily. Several states use a random repeated spacing of skewed joints: 4, 5.8, 5.5, 3.7 m (13, 19, 18, 12 ft).

For reinforced pavements, joint spacing varies from 9.1 to 24.4 m (30-80 ft). The percentage of reinforcement increases with an increase in joint spacing, but there is a net saving in the costs associated with the elimination of some of the joints. The optimal spacing, based on a 1975 survey of average cost of mesh reinforcement, dowels, and sawing and sealing the joints, is 12.1-15.2 m (40-50 ft). Although 12.1-15.2 m is the optimal spacing from a first-cost point of view, it might not be so in the long run. Slabs longer than 6.1 m (20 ft) crack. The cracks, while generally held tight by the reinforcement, tend to spall earlier than the joints. Pavements that have properly designed thicknesses and 4.6- to 6.1-m (15- to 20-ft) long slabs do not crack transversely between joints (3).

SLAB THICKNESS

The design for the thickness of a concrete pavement is based either on serviceability criteria or on allowable stresses. For primary roads, a slab thickness of 22.9 or 25.4 cm (9 or 10 in) is generally required for reinforced and doweled pavements; 22.9 cm or more for doweled, plain pavements; and 20.3-33 cm (8-13 in) for plain, undoweled pavements. Increased slab thickness reduces deflections and improves the performance of the pavement.

When the surface of the pavement is cooler than the bottom, the slab tends to dish upward at the end to a degree determined by slab length. Truck traffic that passes over a typical contraction joint when the slabs are dished upward causes a repetitive vertical movement that creates a great potential for fatigue failure of a sealant. The vertical movement in a 22.9-cm undoweled slab with short joint spacing is small. It is at maximum when the truck is moving at a slow speed close to the edge of the pavement (7-9). A thicker slab would reduce deflections, but dowel bars on a stabilized base would achieve the same result more cost-effectively.

TYPE OF SUBBASE

Three types of subbase are now generally used in highway construction: granular, cement stabilized, and asphalt stabilized. Stabilized subbases are, of course, more expensive, but they are stronger and more erosion resistant. They reduce pavement deflection and the migration of fines under the pavement. No matter what type of subbase is used, a well-drained subgrade must be included as part of the overall system.

Besides a reduction in the vertical deflection of the edge of the pavement, cement-treated subbases help to maintain more uniform joint openings as a result of high friction values between the subbase and the slab (10-12).

The erosion of the subbase is a factor that contributes to most of the distresses that occur at a joint. The top of the joint is normally sealed to prevent the intrusion of water and foreign material from the surface of the pavement. The bottom of the joint and the vertical edge are not sealed and therefore provide access for intrusion of material from the subbase and the shoulder. Gravity keeps the material intruding from the subbase at the bottom of the joint. This material prevents the joint from closing and thus induces shear stresses that cause spalling of the bottom and edge of the pavement. Blowup of the pavement could result if the spalling at the bottom of the joint becomes excessive.

A seven-year study of a pavement in Ohio indicates that bottom spalling is not a function of the type of subbase but of joint spacing. Compared with pavement sections that have 12.2-m (40-ft) spacing of joints, sections of pavement with 6.4-m (21-ft) spacing stand out as a group because of the mildness of spalling that occurs at the bottom irrespective of whether the subbase is granular or stabilized.

Spalling at the bottom of the contraction joints is a much more serious problem than surface spalling. The manner of construction of the normal contraction joint leaves a jagged edge at the bottom of the pavement that is much more conducive to spalling than the straight, sawed edges at the top of the pavement.

There may be a lesson to be learned from history. Submerged plane contraction joints were tried more than 25 years ago but were discontinued because they left a jagged crack in the pavement surface that spalled easily and was difficult to seal. The present method of contraction-joint construction may well be creating exactly the same problem in reverse—i.e., simply putting the spalling at the bottom of the pavement where it can't be seen. It is well worth considering using both the submerged plane contraction joint and the sawed joint at the surface. This design is used in the United Kingdom.

Faulting is another distress common in pavements. It is a function of repetitive heavy-truck loading and free water under the slab as well as the type of subbase. As NCHRP Report 56 (10) states, "Unless the conditions causing faulting are corrected, elevation differences between adjacent slabs usually become progressively greater. This contributes to the failure of joint seals through shearing action as joint faces move vertically."

Faulting is more common in undoweled pavements because dowels reduce live-load deflections. Stabilized bases provide more protection against faulting than granular bases because of less deflection of the pavement and less loose material that could be pumped under the approach slab.

TYPE OF LOAD-TRANSFER DEVICE

Although various types of load-transfer devices have been used throughout the world, the round steel dowel has become the predominant method of load transfer. Two major problems remain, however: misalignment and corrosion.

Bryden (13) showed that a 12.7-mm (0.5-in) misalignment of one dowel caused cracking of the test slab.

Dowel corrosion causes a swelling of the bar and can be severe enough to freeze the joint. Frozen joints can be identified because the adjacent slabs usually develop one midslab crack. This will be a working crack, and evidence of corroded reinforcement will usually be noted.

Various corrosion-resistant coatings, such as Monel, nickel, and stainless steel, have been used with different degrees of success. Metallic coatings, however, are expensive. Most recent experience has been with plastic-coated dowels (8, 13, 14). Plastic coatings naturally vary on different experimental projects. One such coating is a two-layer coating of 0.1 mm (4 mils) of asphalt covered by 0.4 mm (17 mils) of polyethylene. The plastic-coated dowels show promise of excellent long-term performance.

TEMPERATURE AND MOISTURE EFFECTS

Temperature is widely believed to be the primary factor that affects joint movement. There are actually four separate temperatures to be considered: air, pavement surface, midslab, and subgrade. However, since the moisture content of the pavement also affects slab movement, both temperature and moisture effects should be considered. Both are believed to have an effect on the curling of pavements as well as on longitudinal movements.

Several studies (6-8, 13, 15-18) have been conducted on the relation between temperature and longitudinal movement. Only a few studies take into account the effects of both temperature and moisture. Lang (15) has given an excellent summary of these factors. He recorded temperatures at six places in a 17.8-cm (7-in) slab and at five places in the subgrade and made moisture measurements at midslab and at three places in the subgrade.

Allen (19), in his work on pavement curl, concluded that the primary factor affecting curl was swell or shrinkage of the subgrade rather than temperature or moisture gradients throughout the slab.

However, the curl of pavements as measured by the deflection of slab ends has been reported by several states (8, 15). South Carolina has reported that, in the morning when the top of the slab was cool, deflections were five to six times as great as deflections on the same joints in the afternoon. New York and Ohio have reported the same conclusion.

In summary, however, even though engineers in every state are aware of moisture effects and the existence of pavement curl, the majority of states use the temperature range only as the design factor in determining slab length and the size of joint openings.

PROPERTIES OF PAVEMENT MATERIALS

The thermal-expansion characteristics of concrete are important in anticipating changes in joint width. Concrete made with aggregates that are high in quartz or chert content exhibits more movement than concrete made of most limestones.

However, in joint resealing, joint spalling may be the major consideration. Joint spalling is affected not only by the coefficient of expansion but also by the tensile strength of the concrete. Tensile strength depends on the type of aggregate, the permeability and strength of the paste, and the pore and void characteristics of the mix.

Pavement growth also causes difficulties with resealing. Relief joints have to be cut in the pavement to prevent blowups or excessive pavement translation. Louisiana investigators (18) have concluded that, in their area, pavement growth is related mainly to incompressibles in the joint. However, expansive aggregates are used in some parts of the country. In New York State, it was found many years ago that combining certain aggregates with high-alkali cements resulted in the formation of a gel around the aggregate particles, which caused the pavement to expand.

CONCLUSIONS

There is a wealth of information available on the various pavement design features that affect joint-seal performance. Because different designs are used in various states, it becomes difficult to draw concise conclusions. Several facts, however, do stand out.

1. Most states have had the best performance from pavement in which only contraction joints are used. Skewed joints are being used successfully in many areas.
2. Joint size can be related to slab length and temperature range.
3. Slab lengths are progressively shortening because of the presence of midslab cracks in the longer slabs. Midslab cracks are usually not seen in slabs 6.4 m (21 ft) or less in length.
4. Although many different types of load-transfer devices have been tried, the standard dowel is still the most commonly used and is quite successful. Plastic-coated dowels have performed well in preventing dowel corrosion.
5. Treated subbases have been effective in reducing pavement curl and midslab cracking in longer slabs.
6. Although the effect of moisture is acknowledged, it is generally ignored in design. The design of slab length is based on temperature range.
7. Material properties have a marked effect on the service life of a pavement. Angular aggregates give better aggregate interlock. The tensile and shear strength of the aggregate and paste affect the amount of spalling in the pavement.

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Publication of this paper sponsored by Committee on Sealants and Fillers for Joints and Cracks.

Pavement Restoration Measures to Precede Joint Resealing

J. B. Thornton and Wouter Gulden

Various methods of rehabilitating jointed concrete pavement are discussed, based on the experience of the Georgia Department of Transportation. Special emphasis is given to techniques that may be required before joints are resealed. The measures discussed are stabilizing moving

slabs, spall patching, slab replacement, edge drains, and grinding. Joint sealing is briefly described, and the use of low-modulus silicone sealant in resealing joints is emphasized. There is a need for adequate condition surveys of existing pavements so that the rehabilitation needs of an individual project can be established. A condition survey is also useful in establishing priorities among projects and in determining deterioration rates and performance of rehabilitated pavements. Attention is given to rehabilitation strategies, since it is only rarely that any one treatment is successful. For each individual treatment to be effective, a combination of treatments is required, such as slab stabilization and joint resealing.

Many state transportation departments are faced with the need to rehabilitate older sections of the Interstate system. Increasing interest is being shown in rehabilitation methods and repair techniques for concrete pavements. At one time, it was thought that concrete roads would last for a long time without the need for any kind of maintenance. The effects of the combination of water and heavy loads on the pavement system were either unknown to or ignored by designers, and so little or no provision was made for drainage. Typical designs showed the concrete pavement surrounded by impervious materials. Either transverse joints were left unsealed, or the sealant used was inadequate for long-term performance. No attempt was generally made to seal longitudinal shoulder joints.

Studies done by such states as California and Georgia have shown that the entrance of free surface water through these joints and cracks, in combination with heavy loads and the presence of erodible materials, causes the types of pavement distress commonly found on heavily traveled jointed concrete pavements—e.g., faulting and slab cracking.

Much research has been done to improve joint-seal materials, joint-shape factors, and the design of base and pavement sections. Concrete pavement designs in Georgia now call for econocrete base, 6.1-m (20-ft) joint spacing, silicone sealants, dowel bars, and concrete shoulders. These improvements for newly designed pavements, however, did not address the deterioration of existing pavements. Yearly condition surveys showed jointed concrete pavements in Georgia to be deteriorating at an accelerated rate as a result of increasing volumes of truck traffic.

Increasing emphasis has been placed recently on restoration and rehabilitation techniques for jointed concrete pavements. Several major research studies in this area are currently planned or under way, such as NCHRP Project I-21, Repair of Joint-Related Distress in Portland Cement Concrete Pavements. This paper describes the Georgia experience with various methods of pavement restoration.

REHABILITATION CONCEPTS

Rehabilitation or restoration of concrete pavements can mean different things to different people. The restoration measures used in Georgia are undersealing, drainage, slab replacement, spall patching, joint sealing, and grinding. Overlays of concrete pavements, although a valid restoration method, are not considered to be within the scope of this paper and will not be discussed.

In pavement restoration, decisions must be made on when to rehabilitate and how much restoration is needed. It is important to conduct condition surveys to establish the relative amount of distress that is present in comparison with other projects. Condition surveys will also provide the basic data required to make the initial decisions on the type of treatment that is needed. Annual surveys will also establish the rate of deterioration of the pavement, which should be taken into consideration when restoration priorities are determined for various projects.

Many levels of restoration are available, ranging from sealing joints as a preventative measure to full restoration, which would include all six measures mentioned above. By use of a condition survey, all of the projects involved can be rated and the type of treatment that is needed can be established.

A restoration program should not necessarily concentrate on the worst-distressed pavements only but should perhaps be balanced between preventive joint sealing, to prevent future and larger expenses, and complete restoration. A project that shows distress but still has acceptable serviceability might not be given as high a priority as a project that shows less distress but has a higher rate of deterioration and a larger projected truck volume.

Once it has been decided to restore or rehabilitate a project, decisions must be made on the degree of restoration needed. Undersealing of unstable slabs will be of little value if it is not immediately followed by resealing of the joints because the pumping process will continue if no provisions are made to prevent water infiltration. In the same manner, the addition of edge drains will not be sufficient if moving slabs are not stabilized. Joint sealing can be done without other activities if the project is in good condition and the slabs are stable. A project will generally need several different restoration treatments to get the maximum benefit out of each individual treatment.

CONDITION SURVEYS

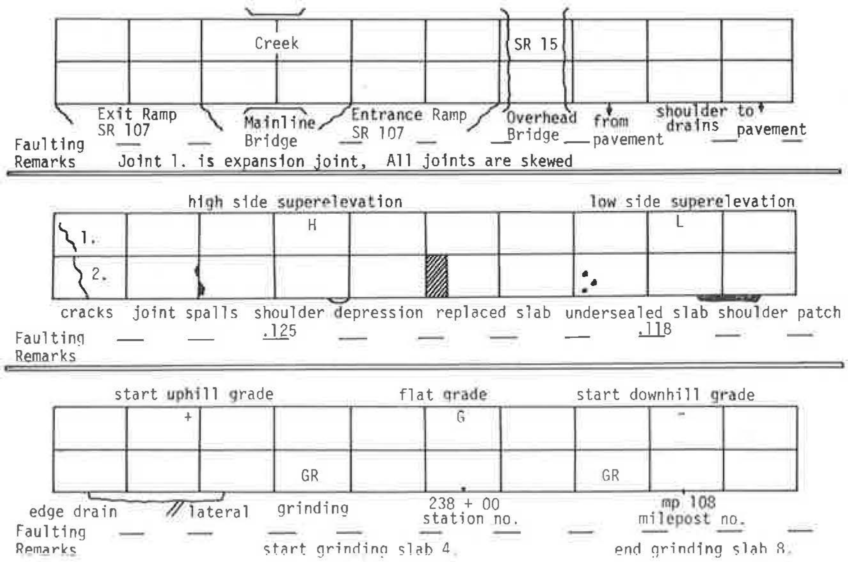
The Georgia Department of Transportation (DOT) conducts annual condition surveys of the jointed concrete pavements on its Interstate system, which totals approximately 1200 km (750 miles). The surveys were started in 1971 and initially used a sampling method in which approximately 10 percent of each mile was measured for faulting and observed for distress. This method was sufficient to determine the general condition of a project and the rate of deterioration. In 1977, the procedure was modified so that observations and a record of distress are now made for every slab and faulting measurements are made on every fourth joint. This change in procedure was required so that the data could be used for detailed planning of the restoration requirements of individual projects and for evaluation of the performance of the rehabilitated projects.

In the condition survey, all distress on each slab is recorded on strip maps along with other pertinent information such as ramp locations, station numbers, mileposts, curvature, grade, and previously performed maintenance activities. In addition, the faulting measurements are recorded on the strip map at the appropriate joint. An example of such a strip map is shown in Figure 1.

The data are reduced to give the condition of the pavement on a mile-to-mile basis and to show the faulting index, roughness, number of cracked slabs, and number of replaced slabs. This information is used in priority ranking the restoration needs for the various areas.

As a means of determining rate of deterioration and as a tool for follow-up evaluation of the effectiveness of restoration treatments, condition surveys are an important aspect of a rehabilitation program for planning purposes.

Figure 1. Example of pavement condition survey sheet.



STABILIZING CONCRETE PAVEMENT SLABS

Reasons for Stabilizing Slabs

One of the three major contributing factors in faulting and pumping of concrete pavement slabs is excessive movement of the slab corners under loads. Some movement should always be expected. The magnitude of movement varies with the type of subbase, slab interlock, and amount of curling. Only when an erodible base course is present in combination with a void space filled with water will the movement of the slabs become detrimental to the performance of the pavement.

One of the important steps in rehabilitating concrete pavement, either by using an overlay or by grinding, is to identify slabs that show excessive movement and attempt to stabilize them by undersealing. Undersealing appears at first to be a simple process that consists of drilling a hole in the concrete and forcing grout material into the cavity under the slab. But several basic problems are associated with proper undersealing methods:

1. Determination of slabs to be undersealed,
2. Flow characteristics of the grout material,
3. Proper distribution of the grout material,
4. Determination of when to stop undersealing a slab, and
5. Determination of the effectiveness of the undersealing effort.

Determination of Slabs to Be Undersealed

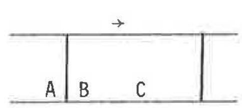
Undersealing slabs that do not really need it can do more harm than good by causing an uneven lifting of the slabs and thus loss of support at the slab edges and center. The primary reason for undersealing a slab is to reduce its movement under load. The criterion for determining whether a slab needs undersealing, therefore, should logically be the magnitude of the movement of the slab when it is subjected to a specific load. In many instances, grout can be forced under a slab that shows little movement under a load. This is especially true when there is a slight separation between the slab and the base that allows the slab to be raised easily so that grout can be forced under it. A slight separation between a concrete pavement slab and the base course will not be detrimental if slab movement is minimal and water is kept out from under it. As stated earlier, damage can be done to a slab when an attempt is made to fill small separations by creating larger voids elsewhere under the slab.

Determination of Excessive Movement

Movement of concrete pavement slabs can be determined by static or dynamic means. Static measurements are obtained by using a dial gauge to measure the movement of a slab when an 80-kN [18 000-lbf (18-kip)] axle load is placed on the slab corner. Dynamic deflections are obtained in Georgia by using the Dynaflect, which vibrates the pavement through an oscillating 4.4-kN (1000-lbf) load at 8 cycles/s.

Slab movement varies during the day as well as during the year because of daily curling effects and expansion and contraction of joints. Slabs can have a high rate of deflection at night or early in the morning and show no movement in the afternoon. An example of this change of deflection during the day is shown in Figure 2.

Movement is also dependent on the stiffness of the subbase and the subgrade. Slabs should be measured when the largest movement is expected, since this time period would be most critical to the performance of a slab. The curling movement of slabs varies from day to day and cannot be controlled. An example of the amount of curling that can take place is given in Table 1 for several Georgia locations. The following diagram shows the points in the slabs at which the measurements were taken:



Curling and temperature measurements for I-85 in Fulton County made in November 1977 are given below (1°C = 1.8°F; 1 mm = 0.039 in):

Item	11/9/77	11/15/77
Maximum temperature difference between slab sections (°C)		
Top slab	-3.3	-6.1
Bottom slab	+6.7	+9.4
Total movement (mm)		
Leave slab, joint 1	0.70	1.68
Approach slab, joint 2	0.25	1.17

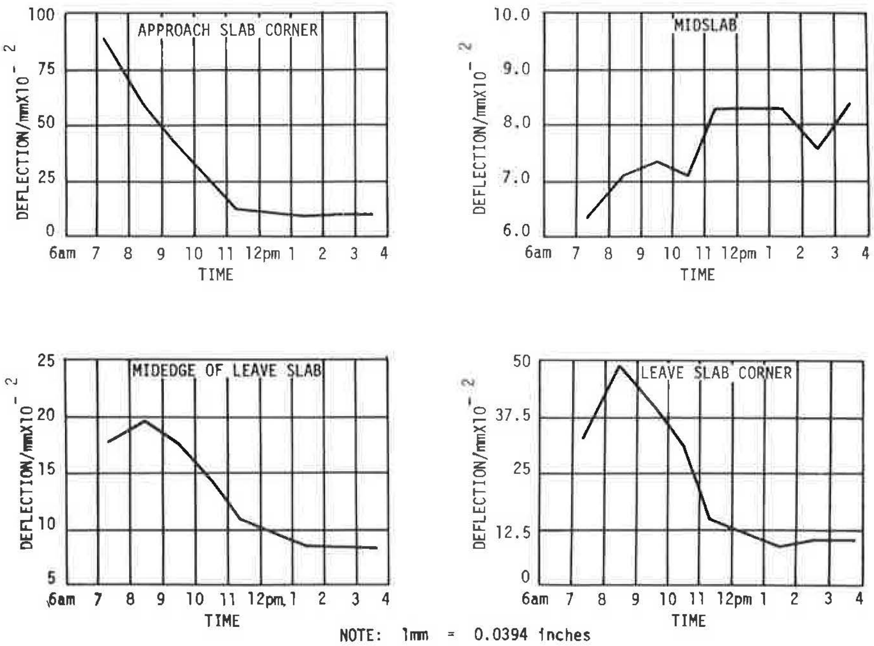


Figure 2. Changes in deflection: northbound lane of I-85 in Coweta County on June 21, 1977.

Table 1. Corner curl measurements.

Location	Type of Base Course	Date	Time	Surface Temperature (°C)	Curl[a] (mm) A	B	C
I-85, Troup County, under construction, no shoulders	Econocrete	7/26/77	7:00 a.m.-3:00 p.m.	20-29	0.914	0.813	0.051
I-85, Coweta County	Cement-stabilized graded aggregate	7/22/77	7:00 a.m.-3:00 p.m.	24-41	1.295	1.880	-
I-75, Monroe County, southbound lane	Bituminous-stabilized soil aggregate	7/19/77	7:20 a.m.-1:05 p.m.	19-35	0.203	0.533	-
I-85, near airport, abandoned section	Soil aggregate	7/16/77	7:00 a.m.-5:00 p.m.	19-28	0.406	0.432	0.254
		8/19/77	7:00 a.m.-3:00 p.m.	23-37	0.686	0.584	0.254
		8/23/77	7:00 a.m. 4:00 p.m.	26-46	0.686	0.635	0.381
I-85, Troup County, under construction, no shoulders	Cement-stabilized graded aggregate and 25.4 mm asphaltic concrete	8/24/77	7:00 a.m.-2:30 p.m.	25-43	0.381	0.279	0.584

Note: $t^{\circ} = (t^{\circ}F - 32)/1.8$; 1 mm = 0.039 in.
[a]See diagram in text.

The substantial difference in the total amount of curl measured on two different days can be accounted for by the differences in temperature ranges. In addition, on November 15, the slab temperature was much colder early in the morning and the joint opening was probably larger than it was on November 9. The wider joint opening would put less restriction on the vertical movement of the slab corner. The curl movement indicated in the table above relates to the total upward and downward movement but does not distinguish between the amount of upward curl and downward curl. The question as to what is excessive movement is, therefore, difficult to answer when so many factors influence the deflections that are measured at any given time. In addition, excessive movement of a slab does not necessarily indicate the existence of a void beneath the pavement slab, since other factors such as a weak base or subgrade can cause high deflections.

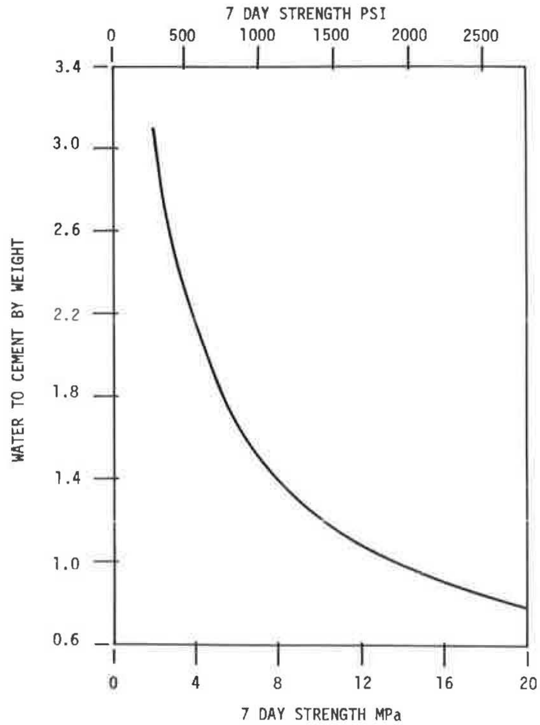
Through experience and observation of grout take versus slab deflection, a limiting criterion of 0.64 mm (0.025 in) of movement under an 80-kN single-axle load is currently used in Georgia. It is assumed that any slab that shows movement of 0.64 mm or less does not show sufficient movement to warrant stabilizing. It does not mean that these slabs will not take grout because almost any slab can be made to take grout. The potential of damaging a slab that exhibits small deflections outweighs any advantage that may be gained by reducing the movement of the slab through undersealing.

Grout Mixes for Undersealing

The type of grout used in undersealing is an important factor in obtaining a good, stable slab. If the grout is too stiff, there will be "stooling" of the material around the grout hole and little distribution of the material under the slab. If the material is too soupy, there is a loss of strength and a large amount of shrinkage can be expected. The type of materials that are selected to make up the grout also greatly affects the consistency and strength of the grout mix.

Many grout mixes were prepared and tested by the Georgia DOT in an attempt to determine the most practical mixes for field trials and to establish the properties of mixes already being used. Cement, limestone dust, and fly ash were the main ingredients used in the grout mixes tested in the laboratory. Hydrated lime has not been used in grout mixes in Georgia because it makes

Figure 3. Strength of grout mixes.



the mix so costly. Among the various mixes tested, the best strength was obtained by using cement and limestone dust only. The relation between the seven-day strength and the water-cement ratio of the various mixes tested is shown in Figure 3. The results showed that good correlation existed between strength and water-cement ratio, which was independent of any other added mix ingredients. The mixes that included added ingredients such as fly ash showed less strength because of the additional water required for fly ash to obtain a good flow consistency.

The grout mixes that are approved for use in Georgia are given below:

Grout Type	Material (percentage by weight of dry materials) Cement	Limestone Dust	Fly Ash	Fine Aggregate
1	25		25	50
2	25	25		50
3	25	75		
4	25	50	25	
5	100			

Although this table contains five mixes, in reality only mix 3, which consists of one part cement and three parts limestone dust, is used. A mix with cement, fly ash, and limestone dust has also been used. But there is no obvious advantage in using this mix rather than the cement-limestone mix and, in addition, it requires one more component, which makes batching slightly more complicated. In Georgia, it has been found that a sand or soil mix is not suitable for undersealing pavement slabs. Grout mixes that contain sand are still in the specifications because they may be needed for purposes such as filling large voids at bridge approaches.

The flow of the mix is also important. The specifications require a flow of 16-22 s when measured with a flow cone. It is preferred that the mix used on the job have a flow of 16 s to facilitate the distribution of the grout under the slab without excessively lifting the slab. Although limited laboratory work has been done in Georgia on the use of additives in the grout, such as water-reducing agents and calcium chloride, no additives have been used in actual practice and further investigation is needed.

Undersealing a Slab

Close inspection by the contractor and the state inspector is required during the undersealing process. The purpose of undersealing is merely to stabilize the slab by filling existing voids with grout and not to raise the slab back to grade.

Excessive lifting of slabs can be very detrimental to the pavement and can cause the creation of voids elsewhere as well as overstress the slab and eventually cause cracking of the pavement. The Georgia specifications allow the slab to be lifted up to 3.2 mm (0.125 in). A device to monitor lift is therefore a necessity on a project. Georgia uses a modified Benkelmen beam device that indicates total movement as well as the differential lift between adjacent slabs at the corner. This device, shown in Figure 4, can be used to control the amount of pumping that is done in each hole. Other factors that are used to determine when to cease pumping in a hole are the appearance of grout in adjacent holes and joints or cracks and the displacement of water from under the slab. Another indication that grouting should cease is the pumping time on a hole. When no evidence of grout appears in joints or a hole, and no lift is being recorded on the gauge after a reasonable amount of time, grouting should cease. This is especially true when grout is being pumped next to the centerline, where the grout is liable to be pumped under the adjacent lane. In some instances, when the outside lane was being stabilized, the grout broke through the inside shoulder.

There is no set procedure that can guarantee that a slab has been properly undersealed. Experience is a key factor in the undersealing operation.

A variety of hole patterns have also been used in Georgia; frequently, some experimentation at the start of a project is necessary to determine the hole pattern that gives the best results. A typical hole pattern that works well on many projects is shown in Figure 5. This hole pattern is designed to fill voids that exist under the corner of the slab, since experience has shown that the void generally does not extend 3 m (10 ft) beyond the joint.

Effectiveness of Undersealing

The primary indication of the effectiveness of undersealing is the performance of the rehabilitated pavement. Undersealing is not effective if no provision is made to seal out surface water or rapidly remove infiltrated surface water. Undersealing alone does not stop slab pumping and faulting.

The effectiveness of undersealing in stabilizing the slab can be determined by measuring the movement of the slab corners again. If the movement is still greater than 3.2 mm (0.125 in), the slab is regrouted in case voids were formed during the initial undersealing attempt or the existing voids were not entirely filled. No additional grouting attempt should be made after the second attempt.

SPALL REPAIR AND SMALL PATCHES

Concrete pavements have been overlaid with asphalt because basically sound concrete developed spalling at slab

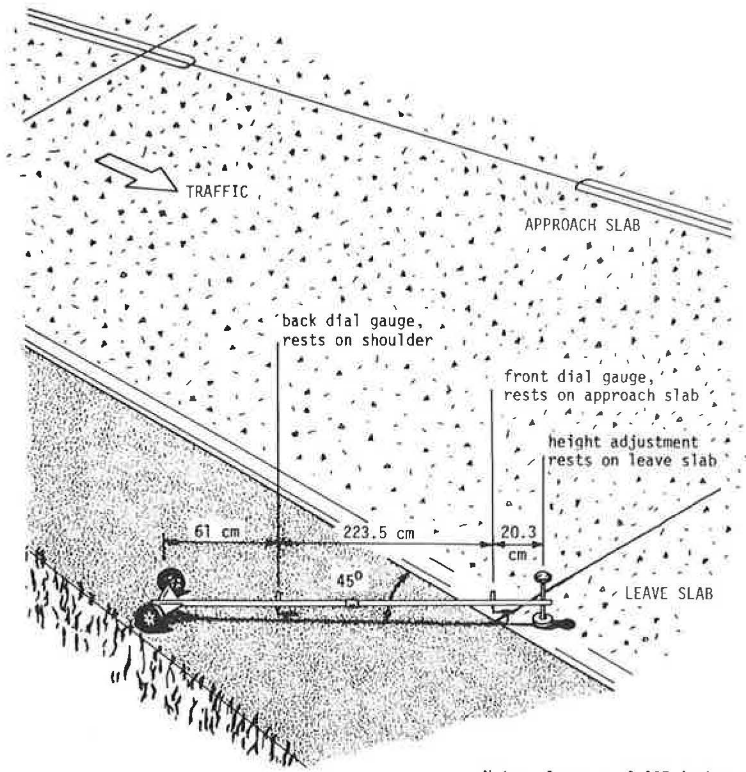


Figure 4. Lift-measuring device.

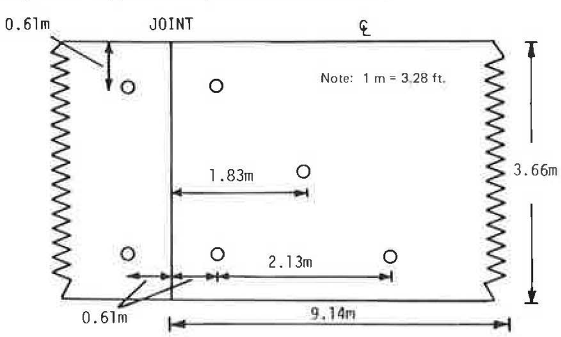


Figure 5. Typical hole pattern for underseal.

corners and at joints. The alternative would have been very costly, labor-intensive patching and joint sealing and questionable performance. The value of a patch cannot be predicted because of a lack of historical data on which to base empirical equations for comparisons with other alternatives. The results are more predictable for an overlay, which can be done in stages with specialized equipment, in contrast to tedious patch preparation and placement, which require much labor and skill if the desired results are to be constantly obtained.

Patching may most often be the most feasible solution to the spalled pavement problem. Patch preparation is the most important phase of spall patching. Surfaces must be free of oil, dust, dirt, traces of asphaltic concrete, and other contaminants, and some methods and materials require neatly sawed vertical edges to prevent feather edges. The materials must have the necessary properties, and they must be properly used.

Georgia has had some experience with three types of patching:

1. Twenty-four-hour accelerated-strength concrete bonded with an epoxy adhesive for bonding plastic concrete to hardened concrete,
2. Rapid-setting patching materials of cement-mortar-like consistency, and
3. Epoxy mortar or epoxy concrete.

Repair Method 1

Use of 24-h accelerated-strength concrete bonded with type 2 epoxy requires that the edges of the patch be vertical and not feathered. The practice in this type of patching has been to prepare the spalled area by sawing around the periphery of the deteriorated area and removing the deteriorated material, leaving vertical edges not less than 3.8 cm (1.5 in). The concrete does not gain strength as fast as some of the patching grouts, and traffic control must be extended for longer periods. Normally, concrete with 2 percent calcium chloride is used for the patching, and the time it takes this mix to develop strength is well established.

These patches have been used with reasonable success, but consideration must be given to the following characteristics:

1. Time required for the concrete mix to gain sufficient strength to support traffic,
2. Coefficient of thermal expansion of the epoxy mortar, and
3. Hydrostatic pressure.

The coefficient of thermal expansion is in the range of 36-54 × 10^{-6}/°C (20-30 × 10^{-6}/°F) compared with 5-11 × 10^{-6}/°C (3-6 × 10^{-6}/°F) for concrete. Experience has shown that, for the 0.25- to 0.50-mm (10- to 20-mil) thickness, no bond failure occurs. Apparently, the very thin epoxy section is restrained by the concrete, and the epoxy is strong enough to withstand the induced stress.

Patches of some permeable materials have been known to unbond for no apparent reason. Some of this problem has been attributed to hydrostatic pressure on an impermeable face. The success of an epoxy-bonded patch may well depend on the ratio of permeability of the substrate to the bonding agent. If an area was not "wet" with epoxy, low pressure distributed over a large area may exert enough force to unbond the epoxy. Pressure from water confined under tires may cause high pressure in concrete voids and unbond patches at the periphery.

Repair Method 2

Most extensive patching has involved the use of rapid-setting patching materials. Magnesium phosphate patching grouts meet this specification. When the area to be patched is properly prepared, these patches perform well.

Repair Method 3

Epoxy mortar and epoxy concrete mixes have been used with good success. Most experience in Georgia with this method is in the patching of small spalls on new construction. Sometimes concrete ravels at joints when it is necessary to saw early to prevent cracking. Small spalls that cannot be corrected by a final saw cut must be repaired if preformed seals are to be used or the seal will not be watertight. Good results can be achieved by patching these spalls, which are usually small, with an epoxy-concrete mix. Accurate forming of the joint through these patches is not always possible, and it is sometimes necessary or desirable to place the patch so that excess material can be cut from the patch adjacent to the joint by running the saw through the joint. If there are spalls, they should be patched before the final joint saw cut.

In summary, the following observations are made concerning patches:

1. Filler stone used in conjunction with repair method 2 sometimes contains too many fines, which kills the ability of the patching material to wet the surface that is to be bonded. This problem can usually be corrected by reducing the quantity of stone used in the mix.
2. Inadequate consolidation sometimes leaves too many large voids in the patch.
3. Many patches are installed in such a way that they span an initial fracture when the relief crack was not vertical. This practice should not be permitted because the patch is sure to fail. When such a condition exists, two patches should be made and a joint formed between them, as shown in Figure 6.
4. Retempering of the mix in an attempt to use old batches should not be permitted. Set occurs very fast, and care must be taken not to mix too much material at one time.

SLAB REPLACEMENT

Most of the slab cracking on plain concrete pavement originates because of loss of support under the slab corners. When faulting progresses to a certain degree, the slab becomes cantilever and a crack develops. This crack represents a structural failure that requires repair. Usually the crack separates a relatively large portion of slab from the original slab. Since partial support is regained when the slab cracks and resets on the base, deflection under load is often reduced when the slab cracks. Some advantage can be gained by removing and repouring the slab:

1. The soft base can be removed and good support restored.
2. Dowels can be added by drilling and epoxied in place.
3. The surface can be corrected so that the impact from loads over a faulted joint is reduced.
4. The replacement unit is somewhat larger than that removed and therefore more stable.
5. The resulting joints are in better condition and easier to seal.

Some minimum slab size exists below which performance is jeopardized. The minimum slab length used in Georgia is 2.9 m (9.5 ft). A single transverse crack in the middle third of a 9.1-m (30-ft) slab and nearly normal to the centerline should be grouted and sealed unless evidence of excessive slab movement occurs under load. In case of excessive movement that requires reconstruction of the base and/or the subgrade, the repair is made so that no slab portion left in place or repoured is shorter than 2.9 m (9.5 ft). It may be necessary to remove the whole slab. Portions of slabs that have interconnected cracks are removed. The base and/or subgrade is reconstructed as necessary or replaced to a minimum depth of 35.6 cm (14 in) with concrete unless the base is cement stabilized. If the base is cement stabilized, only the loose material that may be present is removed.

A device has been developed in Georgia that permits drilling of three holes at one time to insert dowels into the existing slab. A single drill, however, can sometimes drill as many holes as the three drills together, since a large volume of air is required to operate the drills. The diameter of the dowels is 3.2 cm (1.25 in), and the spacing is 40.6 cm (16 in). The hole size should be no larger than is necessary to accommodate the dowel, and a high-viscosity epoxy should be applied in such a way that it is forced to fill the void and be extruded slightly from the joint when the bar is inserted. A detail of the slab replacement and dowel spacing is shown in Figure 7.

Figure 6. Proper patching over relief crack.

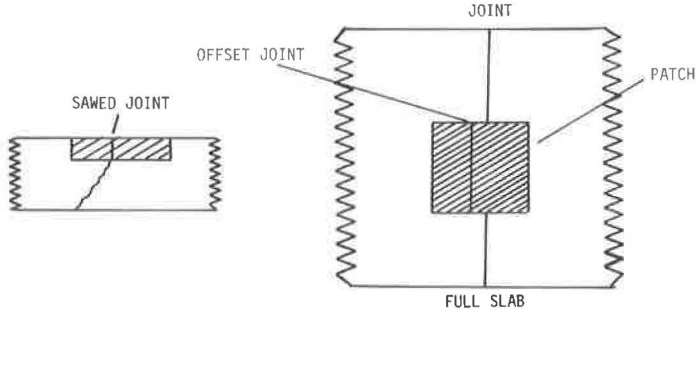


Figure 7. Details of dowel placement in repair of concrete slabs.

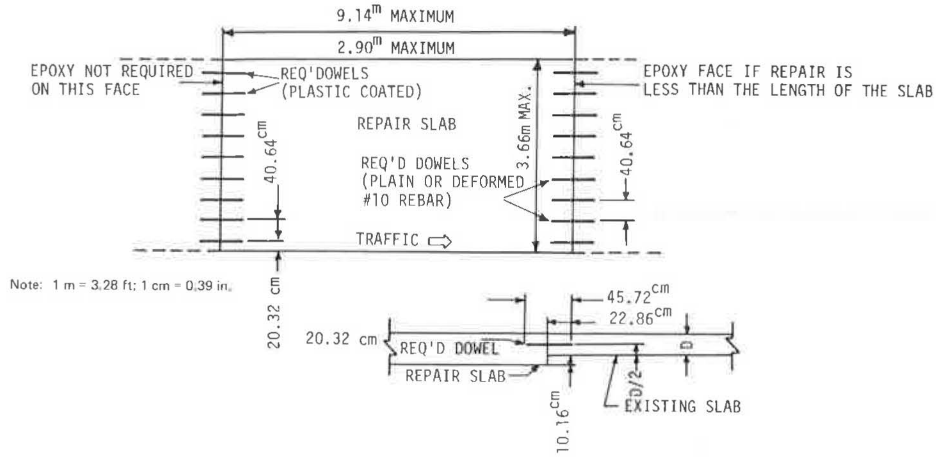
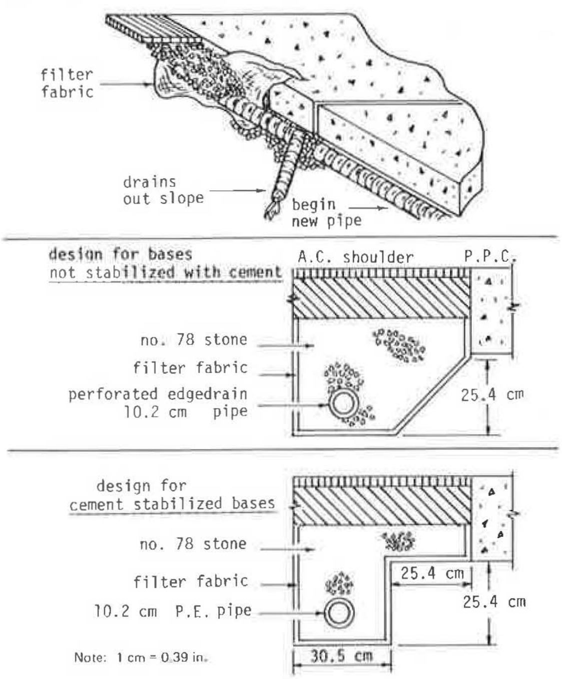


Figure 8. Section of edge-drain system.



EDGE DRAINS

One of the factors that contributes to the deterioration of jointed concrete pavements is the entrance of surface water into the pavement system. The removal of free surface water or the prevention of the entrance of water into the system in the first place plays an important part in the rehabilitation process.

In 1974, the maintenance department of the Georgia DOT began installing longitudinal edge drains with corrugated plastic pipe for rapid removal of water from under slabs. Previous efforts had been concentrated on placing lateral stone drains at pumping joints. This type of drainage system was generally ineffective because water entered the pavement system at places other than the transverse joint and dirt and grass would eventually clog the daylighted lateral drain.

A complete review of existing edge-drain designs, including field observations, was performed in 1978 after it became apparent that there were potential problems in some areas with the performance of the edge-drain systems.

Sections were excavated on several projects to determine the contamination of the drainage stone and to observe the general condition of the system. It was evident that contamination of the stone was severe around the pipe and that some material was still being pumped from under the slab into the drainage system. Faulting data from some of the first projects where edge drains had been placed also showed that the presence of the drains did not necessarily stop the faulting process. One of the key factors in the contamination of the drainage stone and continued pumping was the presence of moving slabs. It was found in some areas that the slabs were moving considerably after they had been stabilized.

The investigation produced several recommendations that were implemented immediately. A typical section of the edge-drain system after the modifications were made is shown in Figure 8. The major changes included adding filter fabric, increasing the asphalt cap thickness from 7.6 to 15.2 cm (3-6 in), and making each outlet drain its own section of the roadway rather than placing a continuous longitudinal pipe with lateral drains tied in to the main pipe.

During the investigation, a test section that contained various filter fabrics was also removed for examination. It was evident that the fabrics were effective in preventing contamination of the drainage stone. It was also evident that a caking of soil about 3.2 mm (0.125 in) thick was formed behind the filter fabric along the vertical face of the pavement edge. Samples of the fabric were tested in the laboratory to determine the flow rate of clean sections and caked sections. The container used in the test measured 51 mm (2 in) in diameter and 79 mm (3.1 in) in height. A significant reduction in flow rate was found, as indicated below:

Material	Drain Time (s) Clean Fabric	Drain Time (s) Caked Fabric
Stabilenka T-100	20	31
Bidim C-22	23	120
Mirafi 140	20	36
Typar 3401	20	360 (for 37 percent of volume)
Supac	23	78

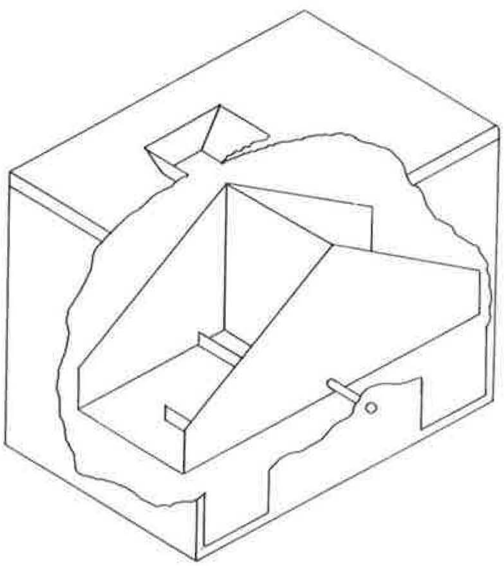
Filter fabrics are designed to aid in the formation of a natural filter layer behind the fabric. The use of filter fabric may therefore defeat the purpose for which edge drains are installed in the first place—the rapid removal of infiltrated surface water. Our investigation has shown, however, that without filter fabric there is a possibility of continuing or even accelerating loss of material from under the slab. Experience with some of the earlier installations and with the filter-fabric test sections indicates that the use of edge drains along concrete pavement may not be effective on a long-term basis. The Georgia DOT has imposed a moratorium on any additional edge-drain installations until the performance of the existing 454 km (282 miles) of edge drains, especially those that use the current design, can be evaluated.

This evaluation consists of condition surveys and faulting measurements. In addition, flow measurements are being made at selected sites that have various design features to determine the volume of water that enters the drainage system. Five sites have been instrumented to determine flow through the edge drains in correlation with rainfall. Other sites at which various joint-sealing treatments and edge-drain designs have been used will also be instrumented.

The measurements are made by using a tipping bucket similar to one used by the University of Illinois in a previous study (see Figure 9). Rainfall is measured by using a smaller-scale tipping bucket at each site.

The evaluation is in the early stages, and a large amount of data on the flow characteristics is not yet available. A typical graph obtained at one site is shown in Figure 10. This graph indicates the capability of the edge-drain system to remove large volumes of water in a relatively short time after the commencement of rainfall. It also points out the need for proper sealing of all joints and cracks to prevent water from infiltrating the area under the pavement in the first place.

Figure 9. Flow-gauge tipping bucket.



GRINDING

Although grinding of the pavement is not required on projects when joints are to be sealed or resealed, it is an essential part of some projects to restore rideability and skid resistance and can be a part of a complete rehabilitation project. The type of grinding machine used and whether grinding is done before or after joint sealing have a bearing on the joint-sealing procedure.

Georgia has experimented with several types of texture-restoring machines, including diamond-blade grinding, Klarcrete, CMI Rotomill, and Barber-Greene Dynaplane. Diamond-blade grinding was found to be the only method that did not damage the joints if the joints were not prepared before grinding. The other texturing and grinding devices cause severe joint spalling, which would make it extremely difficult to seal the joints properly.

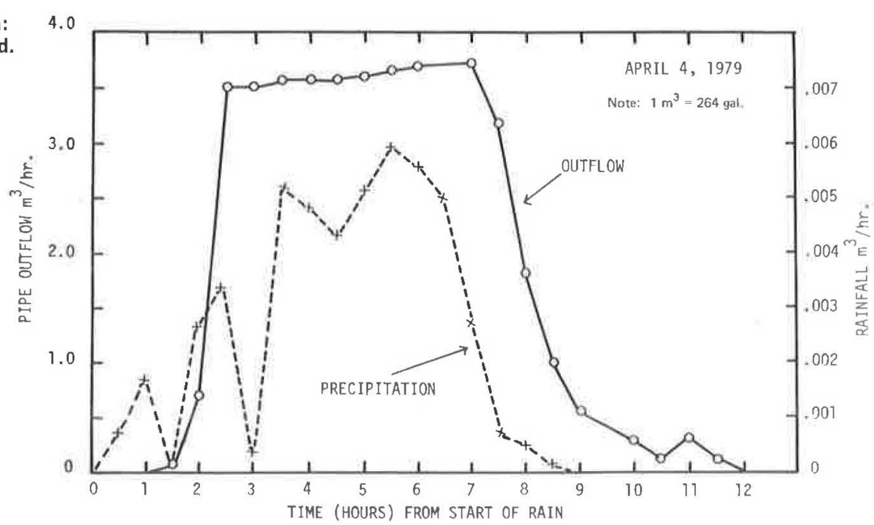
Some experimentation was done with packing the joints with grout, epoxy, and other material, but only moderate success was obtained with Set-45. The texture produced by the Rotomill, Dynaplane, and Klarcrete was also noisier and less uniform than that produced by diamond-blade grinding.

It is preferred that grinding be done before joint sealing. Grinding, however, is the slowest of the rehabilitation operations, and it is imperative that the joints be sealed within a short time after the slabs have been stabilized to minimize the entrance of surface water under the stabilized slabs. Grinding, therefore, is generally done after the joints are sealed, and this must be taken into account in determining the depth at which the joint sealant is placed.

JOINT SEALING

A complete discussion on joint sealing is beyond the scope of this paper, but justice cannot be done to the topic of rehabilitation without some mention of joint sealing. In the opinion of many highway engineers, effective sealing of joints in concrete pavement is still not possible or at least not practical. There are at least

Figure 10. Edge-drain outflow and precipitation: transverse joints sealed, shoulder joints not sealed.



six different philosophies concerning joint sealing. These are listed below:

1. Watertight sealing of joints is not possible and may as well not be attempted.
2. Joints should be sealed with relatively inexpensive sealant. Although adhesive or cohesive failure occurs in cold weather, the sealant is very effective in reducing the quantity of water that can infiltrate a joint.
3. Joints should be filled with a material that will keep incompressibles from restricting closing movement. The sealant probably will not be watertight but will keep water infiltration through joints to a minimum.
4. Sealing of transverse contraction joints is of no value unless the longitudinal shoulder joint is sealed.
5. Sealing of transverse contraction joints in conjunction with edge drains located near the longitudinal shoulder joint is effective.
6. A pavement system can be effectively sealed with sealants now available. An attempt should be made to keep transverse and longitudinal joints sealed. Edge drains will not be needed to remove infiltrated water. Underdrains should be used as required to remove subsurface water.

Advances in joint-sealing materials have been made in recent years to the point that joints can be sealed effectively for extended periods of time. Georgia has test installations of low-modulus-silicone sealant that have been in place for almost five years and have given excellent performance. The few failures on these joints can be explained as follows: (a) The sealant was in contact with traffic, (b) old rubber-asphalt contaminated the joint face, and/or (c) fractured concrete was present at the time of sealing. The joints or portions of joints that were properly prepared are well sealed after almost five years, and the only deterioration is in the surface skin that has been contaminated by oil, dirt, tire rubber, etc.

It appeared evident from these small test installations that low-modulus silicone was capable of sealing transverse contraction joints in concrete pavement. Maintenance crews in Georgia were equipped to install this material in order to provide a larger amount of seal for an evaluation and to develop installation techniques. The results of these installations were excellent, and silicone joint sealant is now used on rehabilitation projects as well as on new construction.

Some failures have occurred on completed projects, but virtually all of the failures with low-modulus-silicone joint sealant can be traced to faulty joint preparation. The joint should be uniform, clean, and dry, and the concrete in the joint area should be sound. Guidelines established in Georgia for sealing joints with silicone cover joint preparation from sawing to sealing as well as proper inspection procedures.

SUMMARY

The successful performance of a rehabilitated jointed concrete pavement depends on choosing the correct treatments for the condition of the pavement. In order for joint sealing to be effective (assuming proper installation techniques and the use of proper materials), other corrective measures must generally be used. Restoration techniques that should be considered include the use of undersealing, spall repair, edge drains, slab replacement, and grinding. Each of these techniques is important to the overall success of a rehabilitated pavement and to the performance of each individual treatment.

ACKNOWLEDGMENT

The material, concepts, and data presented in this paper are the result of a research study funded by the Federal Highway Administration and the Georgia Department of Transportation.

The contents of this paper reflect our views, and we are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Georgia Department of Transportation or the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

Publication of this paper sponsored by Committee on Pavement Maintenance.

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Conditions and Operations for Joint and Crack Resealing of Airfield Pavement

Charles B. McKerall, Jr.

Joint-sealing techniques used by the Air Training Command are summarized. Methods of ensuring maximum sealer life, the treatment of reflective cracking, and the use of compression seals are briefly discussed.

Joint resealing or joint maintenance is probably one of the most neglected facets of pavement maintenance. Resealing joints at the proper time and using the proper materials and methods will reduce the overall maintenance cost and prolong the life of the pavement.

To establish a good joint-maintenance program, it is first necessary to be able to evaluate the existing sealer. It must be kept in mind that the joint sealer performs three functions: (a) It maintains a water barrier, (b) it keeps incompressibles out of the joint, and (c) it reduces the potential of damage from fallen objects.

In evaluating the condition of the sealer, it must be determined which of the above functions is paramount in a particular location. For example, in the Air Training Command (ATC), we have one base with a moisture-susceptible subgrade that is located where the annual rainfall exceeds 52 in. Clearly, in this case we would be interested in maintaining the integrity of the seal as a water barrier. On the other end of the scale, we have another base located where the annual rainfall is about 7 in and the subgrade is a sandy material that is little affected by changes in moisture content. In that case, as long as the joint seal remains in the joint and keeps incompressibles out, we consider it to be doing its job.

When we decide something has to be done, the next decision is what. Several cases must be considered.

1. If the top of the sealer is still "alive" and the bond is good, capping may be all that is necessary. This involves cleaning out the joint with compressed air and applying additional sealer. In this and the following cases, the top of the new sealer should be about 0.125 in below the pavement surface.
2. The top of the sealer may have oxidized while the lower part is still good. In this instance, it is possible to plow off the oxidized material and cap the sealer, as described above.
3. When the sealer has completely failed and the entire depth has oxidized, all sealer must, of course, be removed.

This brings up another area in which proper techniques are not always used nor maximum results achieved. There are several operations that must be done correctly to ensure maximum sealer life:

1. Removal of old sealant—Plow, wire brush, and compressed air should be used. The plow should be free to move laterally and be controlled vertically. Any binding will spall the joint.
2. Refacing of joints—Sides should be as near vertical and parallel as possible. Reface with two spaced blades on a common arbor.
3. Place bond-breaker/separating medium—Bond-breaker/separating material keeps the joint sealer from migrating down into the joint, separates incompatible materials, and prevents the sealer from bonding to the bottom of the joint. It should be compatible with the new sealer but should not bond to it. It should also be compressible, nonshrink, and nonabsorptive.
4. Place new joint-seal material—The joint has been plowed, brushed, and blasted and is now dry. The air and pavement temperature are within specification limits. The joint sealer has been heated to the right temperature (in accordance with the manufacturer's recommendations) and is ready to be applied. The joint should be cleaned one last time, immediately in front of the point of placement of the separating-medium insertion. The sealant should be pumped into the joint from the bottom up. Care should be taken not to entrap air. The joint should be filled to within 0.125 in of the surface. A check should be made that the seal is tack free before the section is open to traffic, and curing should take place within a specified time.

If all of the above steps have been done properly, a good long-lasting joint seal will result.

Cracks are sealed and resealed in the same way as joints except for the initial preparation. Unsealed cracks must be prepared by forming a reservoir for the sealer. This is done by the use of a vertical spindle router. There are other types of routers that are faster but, in my opinion, they are not as effective. Most of them "beat" out the concrete. A vertical spindle router chews it out, leaving smooth, vertical sides and a uniform width. Here, too, a separating medium should be used. In addition, all loose particles along "working" cracks must be removed. Of course, if a crack is not working and has not opened more than 0.062-0.125 in, it is not sealed at all.

At the other end of the spectrum is the crack or joint that has opened excessively—more than 1 in. This is caused either by pavement movement or many cycles of refacing during resealing operations. In this case, we rebuild the joint by sawing 2 in deep, 6 in either side of the joint; breaking out; filling void; and sawing new joint. This procedure is used on aprons on which the sealer must be resistant to jet fuel.

Two other procedures should be mentioned. The first is the treatment of reflective cracks in asphalt concrete overlays over portland cement concrete (PCC) pavement. It has become ATC policy to saw and seal directly above the PCC joint. We use a smaller joint, about 0.25 in wide, sawed to T/4 of the overlay and packed to within 0.5 in of the surface and sealed. Some of the pavements on which we have used this technique are 15 years old. Perhaps 10 percent of the joints have developed adjacent cracks, and the original sealer is still in place. Our philosophy is that reflective cracks are going to develop and they should be controlled by sawing.

The other procedure is the use of compression seals. The compression seal is one of the best things to happen to pavement maintenance engineers in a long time. The Air Training Command is using this material wherever possible. There is no problem on new work and, if the joints to be resealed are fairly uniform in width, with vertical sides, it can be used in resealing as well. For those who may not be familiar with this product, it is an extruded neoprene that comes in rolls. The uncompressed width is twice the width of the joint. The sides of the joint are coated with a lubricant adhesive, and the seal is squeezed into the joint by using a simple little machine. At one installation at Randolph Air Force Base, this material has been in the joints for more than 15 years and looks as good today as it did when it was installed. My recommendation would be, if compression seals can be used, use them. The first cost is a little higher—about 25 percent—but the life-cycle cost is much lower.

Publication of this paper sponsored by Committee on Pavement Maintenance.

Use of a Very Thin Overlay to Reestablish the Skid Resistance of a Concrete Pavement

Charles F. Scholer and Ryan R. Forrestel

An experimental project conducted on an old concrete pavement to evaluate two methods of reestablishing pavement skid resistance is described. Both methods involve using very thin overlays to enhance skid resistance rather than improving the structural capacity of the pavement. Good skid resistance was ensured by selecting aggregates that had proved to give superior skid performance when subjected to polishing action. Adhesion of the very thin overlay to the existing pavement surface was enhanced by the use of admixtures. Both latex and acrylic emulsions were found to be effective. Placement methods were by screeding on a 0.6-cm (0.25-in) thickness or by brooming on a 0.3-cm (0.125-in) thickness. The brooming method was successful, but the screeding was not because the material lost adhesion in many areas. Placement details and skid-test data are presented. The excellent results, in regard to both skid resistance and adhesion, warrant further applications of the technique in order to evaluate its performance under conditions of heavier traffic.

The surface of many kilometers of structurally sound concrete pavement has become worn from traffic, and this has resulted in a reduction in pavement skid resistance. This paper describes an experimental project conducted to reestablish pavement skid resistance by using a relatively economical, very thin portland cement mortar overlay (1). The structural capacity of the pavement section is neither reduced nor increased. Several techniques were evaluated in a field installation; one technique was exceptionally successful.

In the context of this investigation, a very thin overlay was defined as having a depth of 0.9 cm (0.375 in) or less. The goals were to evaluate application techniques and the adhesion of the resulting very thin overlay to the original pavement.

Good long-term skid resistance was ensured by using selected aggregate for the overlay mixture. In Indiana, two materials that exhibit outstanding resistance to polishing and also give a good macrotexture are blast-furnace slag and crushed lightweight aggregate (expanded shale) (2). Natural sand was used in some early test strips to evaluate the application techniques.

Adhesion was enhanced by using either liquid latex or acrylic emulsion in the overlay mixtures. These materials proved to be effective, and, because of the shallow depth of the most successful technique, their cost is reasonable for these applications.

SITE DESCRIPTION

The site of the field test installation was on a straight section of an old 5.5-m (18-ft) wide concrete pavement (formerly US-52) that is currently part of the Tippecanoe County, Indiana, highway system (see Figure 1). The average daily traffic is approximately 200 vehicles. The pavement is badly cracked and faulted, and there is old joint sealer adjacent to many cracks and joints. Some areas of bituminous patching exist (see Figure 2). The pavement is not in structurally sound condition. It was selected because it was convenient and available long enough to allow evaluation by a skid trailer and had some traffic and exposure to winter snow-removal operations.

FIELD TEST STRIPS

The layout of the 14 test strips is shown in Figure 3. Two application techniques, screeding and brooming, were used with different mortar mixes. This resulted

Figure 1. View of test site showing test strips in place on left side of pavement.

Figure 2. Bituminous patch on existing pavement before overlay.

Figure 3. Layout of test strips.

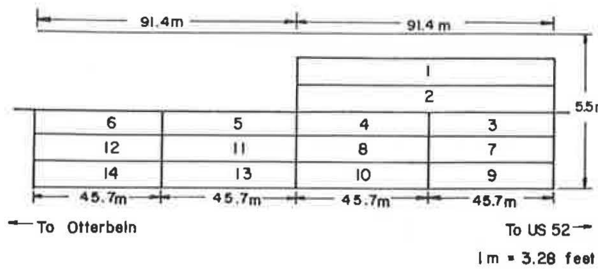


Table 1. Mix designs for test strips.

Strip	Aggregate	Absorption (%)	m[a]	Aggregate[b] (kg)	Latex[c] Percent	Latex[c] Amount (kg)	Antifoam B[d]	Rhoplex[e] Amount (kg)	Rhoplex[e] Percent	Dow Antifoam
1	Natural sand	2.5	2.75	79.43	-	-	-	-	-	-
2	Natural sand	2.5	2.5	72.63	-	-	-	-	-	-
3	Slag	4	2.5	69.29	10.66	40	0.43	-	-	-
4	Slag	4	2.75	76.20	10.66	40	0.43	-	-	-
5	Lightweight	15	1.5	45.97	12.00	45	0.48	-	-	-
6	Lightweight	15	2.0	61.29	12.00	45	0.48	-	-	-
7	Lightweight	15	1.0	30.65	12.00	45	0.48	-	-	-
8	Lightweight	15	1.5	43.97	12.00	45	0.48	-	-	-
9	Slag	4	2.25	62.36	10.66	40	0.43	-	-	-
10	Slag	4	2.0	55.43	-	-	-	32	8.50	0.43
11	Slag	4	2.5	69.29	10.66	40	0.43	-	-	-
12	Slag	4	2.0	55.43	12.00	45	0.48	-	-	-
13	Slag	4	2.5	69.29	10.66	40	0.43	-	-	-
14	Lightweight	15	1.5	43.97	-	-	-	32	8.50	0.43

Note: 1 kg = 2.2 lb; 1 m³ = 264 gal.
[a]Sand-cement ratio, by weight.
[b]Aggregate weight = (weight of cement x m) + (weight of cement x m x absorption).
[c]Liquid latex, by weight of cement.
[d]Antifoam B is 1.82 percent of latex, by weight.
[e]Liquid Rhoplex, by weight of cement.
[f]Antifoam B is 2.29 percent of Rhoplex MC-76, by weight.
[g]Total amount of water added = (water from latex + acrylic) + extra water.

Figure 4. Texture of screeded overlay.

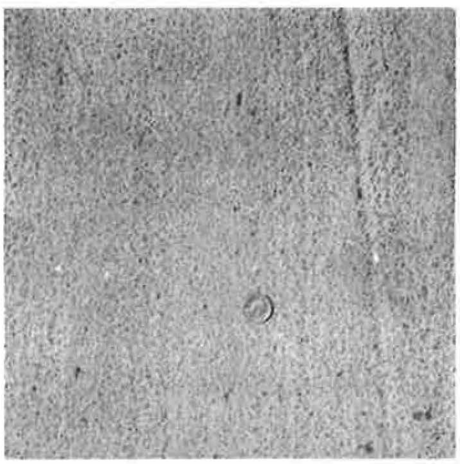


Figure 5. Brooming mortar onto old pavement surface.

in 12 overlay test strips (strips 1 and 2 were placed in October 1977 as part of the preliminary evaluation and are not considered in this paper except to say that they did not adhere well; the remaining 12 strips were placed in May 1978).

Strip 1 and strips 3-6 were all placed by screeding at an overlay depth of 0.9 cm (0.375 in) (strip 2 was broomed and had the same depth). Strips 7-14 were applied by brooming, and the overlay depth was 0.3 cm (0.125 in).

VERY THIN OVERLAY MIXTURES

A type 1 portland cement was used. Aggregates were either blast-furnace slag or lightweight (expanded shale) of sand size in which approximately 22 percent was retained on the 2.36-mm (No. 8) sieve and 11 percent on the 0.3-mm (No. 50) sieve.

The two admixtures used to enhance adhesion were Dow Latex 464, a Saran-base latex emulsion with 50 percent solids that is produced by Dow Chemical Company, and Rhoplex MC-76, an acrylic emulsion with 46-48 percent solids that is produced by Rohm and Haas Company. An appropriate antifoamer, recommended by the manufacturer, was used with both admixtures. Calcium chloride was used as an accelerator on five of the test strips.

Mixing and placement were done on site under a variety of temperatures [10°C-35°C (50°F-95°F)] and wind conditions. The water-cement ratio was varied for each mix to obtain the desired consistency. The target was a water-cement ratio of 0.45, but actual values varied from 0.42 to 0.57.

Details of each mix are given in Table 1. The variations that occurred did not have a detectable influence on either the adhesion or the skid resistance of the test strips.

APPLICATION TECHNIQUES

The overlay for each test strip was applied by hand after mixing in a small portable mortar mixer. The methods were relatively primitive because of the hand labor required. The two techniques evaluated were screeding and brooming. Both are capable of being mechanized on a larger application. Two strips were applied to dry pavement, which required more mix water for the needed consistency. All others were applied to moist pavement. Because dry pavement quickly absorbs water and liquid admixtures, stronger adhesion may result, but no difference is yet apparent.

Although the existing pavement surface was not in good condition, it was free of oil. All overlays were applied to the existing pavement without treating the

$CaCl_2$	Aggregate Moisture Content (%)	Actual Water Added (kg)	Total Water per Batch[a] (kg)	Actual Water-Cement Ratio	Batch Volume (m^3)
-	8.3	-	-	-	-
-	9.0	-	-	-	-
-	7.0	1.6	6.92	0.44	0.051
0.80	8.1	1.1	6.46	0.47	0.055
0.80	20.0	0.0	6.00	0.57	0.055
0.80	12.9	2.5	8.49	0.52	0.064
-	6.0	3.6	9.63	0.43	0.050
-	5.8	4.1	10.08	0.47	0.051
0.80	7.0	2.3	7.60	0.45	0.049
-	6.0	3.6	8.14	0.43	0.045
-	5.2	4.5	9.87	0.51	0.053
-	5.2	2.3	3.26	0.42	0.046
-	5.2	4.5	9.87	0.51	0.053
0.80	9.5	3.6	8.14	0.46	0.049

surface or repairing the joints. Needless to say, cracks reflected through, and the overlay did not adhere to the bituminous patches.

Screeding

The screeding procedure was used on strips 1, 3, 4, 5, and 6. After the pavement was swept, the screeding technique involved the following steps:

1. The 0.9x3.5-cm (0.375x1.375-in) doorstop molding was placed on the pavement to form a 0.9-m (3-ft) wide strip.
2. The mortar was dumped in several piles within the strip from a buggy.
3. As the mortar was spread to its approximate depth, it was brushed into the wetted pavement with a stiff broom in an attempt to improve bonding.
4. The mortar was screeded by using an aluminum channel.
5. While a batch was being periodically screeded, stops were made to patch any irregularities in the fresh overlay. This was usually done with a trowel but, if the problem area was large, the overlay was rescreeded.
6. After a batch was placed, a curing compound was sprayed on the strip if necessary. A curing compound was used only on days when a high rate of water evaporation could occur—i.e., on sunny days or warm, windy days. If curing compound was used, it was sprayed approximately 1 h after placement of the batch. When the compound was not used, it was felt that the skin formed by the latex was enough to prevent excess evaporation.
7. The forms were removed any time after the screeding was finished and before the mortar set.

The largest drawback of the screeding operation was the slow rate of overlay placement. Since the screeding was done by hand, it proved to be very laborious, and it was difficult to complete a strip in one day. Both problems could be partly alleviated by installing a vibrator on the channel. This would cause the mortar to flow into place and reduce the amount of physical work while increasing production.

A very serious problem with this technique is that of finishing. This was caused by the latex, not the application procedure. Seconds after a batch is exposed to the air, the latex forms a thin film all over the surface. This film is easily broken by vigorously working the mortar, as in brooming or screeding. When tining was attempted, however, this surface did not break and the mortar tended to tear. Since this made tining impractical, it was discontinued. The texture produced by the screeding (see Figure 4) was thought to have adequate skid-resistance qualities and was used as the final texture.

Another problem with the screeding technique was that of spreading the mortar to approximately the required depth. If this was not done properly, large amounts of mortar would build up in front of the channel and make screeding by hand virtually impossible.

The principal advantage of screeding lies in the anticipated life of the overlay. Because of its thickness, the overlay should last for many years under normal traffic conditions.

Brooming

The brooming technique was used on strip 2 and strips 7-14. After the pavement was swept clean, the brooming method involved the following steps:

1. Form molding was placed on one side of the strip 0.9 m (3 ft) from the edge of the adjacent strip (the forms were used only to mark the proper strip width).
2. The mortar was dumped in several piles within the strip from a buggy.
3. As the mortar was spread out over the strip to its approximate depth, it was scrubbed into the pavement surface with a stiff broom.
4. After the mortar was spread, a stiff broom was dragged longitudinally over it (see Figure 5) so that excess mortar was pulled forward, and the amount of pressure applied on the broom depended on the depth of the mortar and the harshness of the material (for a given area covered by one broom width, one to four passes were normally made to obtain the desired thickness).
5. The overlay was then patted with a second broom to force any loose aggregate into the overlay.
6. Curing membrane was applied after placement by use of a hand sprayer.
7. The forms were removed at a convenient time after the brooming of a batch was finished but before the mortar set.
8. After the mortar set, water was sprinkled onto the overlay two to four times for each fresh strip to keep the strip from drying.

The use of the brooming technique has three major advantages:

1. To complete a strip took a crew of five about 1.5 h, or 7.5 person-h. This would allow a crew with the proper experience and equipment to overlay a large area each working day.
2. Because of the overlay thickness, 0.3 cm (0.125 in), the yield per batch is very high.
3. No finishing was required because the brooming produces a naturally harsh surface (see Figure 6).

The primary problems in brooming were obtaining a uniform thickness and curing. When the broom was dragged forward to remove all excess mortar, the depth obtained fluctuated depending on the harshness of the material and the pressure applied on the broom. Because the overlay is so thin, these fluctuations are very difficult to see and are not detectable when one drives over the overlay. The real problem with the thinness of the broomed overlay lies in the fact that the service life of the overlay is significantly reduced.

On hot, sunny days, the overlays tended to dry very quickly if they were not cured with curing compound shortly after being placed. On the hottest days, even prompt curing was not enough to prevent detrimental water evaporation. Under these circumstances, water

Figure 6. Texture of broomed overlay.

had to be sprinkled over the overlay to prevent excessive evaporation.

Brooming is definitely the more efficient technique to use: It was four times faster to place a strip by brooming than by screeding. On the other hand, the screeded overlays are presumed to have a much longer life than the broomed overlays, not only because of thickness but also because it is anticipated that the individual aggregate particles will become dislodged from the broomed overlays. In the screeding technique, each aggregate particle is forced into the mortar by the action of the channel. In the brooming method, however, the particles tend to sit on top of the mortar so that more of each particle is exposed and the particles are more susceptible to being broken away from the mortar.

Generally, the brooming technique would be easier to use in the field because of its fast application rate. Once a local area of low skid resistance (a curve or intersection) was discovered and improvement was determined to be necessary, the area could feasibly be repaired between the morning and afternoon peak traffic hours, which would minimize traffic delays and inconvenience to drivers.

RESULTS OF FIELD EVALUATIONS

All field test strips functioned well for several months. Then, the strips placed by screeding were found to be adhering poorly to the original pavement. All delaminated material was removed before the onset of winter weather.

In contrast, the broomed overlay adhered extremely well. This has continued to be true into the second winter. Three salt applications were made by highway commission personnel. It is this method of application along with the admixtured mortar that has been successful and warrants further consideration.

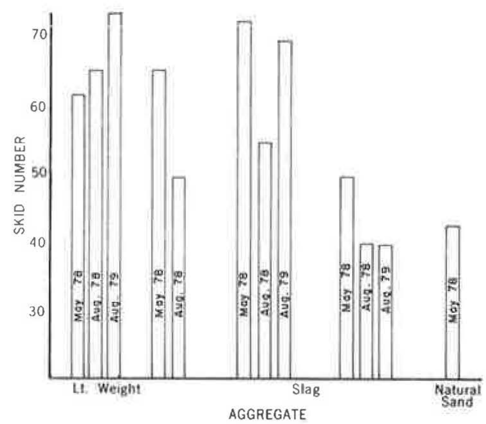
The primary reason for the success of the broomed application method is believed to be its thinness. In general, it is no more than 0.3 cm (0.125 in) thick, except for the larger aggregate particles that protrude above the nominal thickness. This thin section is fragile and many cracks can occur—for a variety of thermal, shrinkage, or flexural reasons—without destroying the

Table 2. Skid numbers determined from skid-trailer measurements at 66 km/h (40 miles/h).

Test Overlay Number	Skid Number			Standard Deviation[a]		
	5/31/78	8/31/78	8/11/79	5/31/78	8/31/78	8/11/79
1[b]	40.6			-		
2[b]	43.2			2.66		
3	38.2	34.2		2.57	2.27	
4	34.2	32.9		3.78	0.83	
5	39.4	39.2		3.46	1.10	
6	40.8	44.4		-	1.50	
7	72.8	66.6		2.06	6.65	
8	61.8	65.0	73.4	1.32	-	2.67
9	50.9	42.7	50.0	2.10	1.44	2.95
10	49.1	39.8	39.8	1.66	-	3.14
11	72.6	54.8	69.3	2.27	4.17	3.11
12	59.0	55.2		1.71		
13	62.8	49.4	62.4	3.40	7.74	2.81
14	65.2	49.5		1.96	-	-

[a]Based on three measurements.
[b]Preliminary evaluation data recorded one week after completion of overlay 2.

Figure 7. Skid-trailer measurements.



adhesion of the very thin overlay to the underlying surface.

Local traffic was allowed to start passing over the thin overlays on the evening after the overlay placement. Sometimes this was only 4 h later. Obviously, a low strength exists in the overlay materials, but there is no apparent damage in the test strips, which are under the outer wheel path of the traffic.

Skid Resistance

Skid-trailer tests were conducted on each section by the Indiana State Highway Commission. The results are given in Table 2 and illustrated in Figure 7. Initially, the broomed sections had extremely high skid numbers because of the coarse texture and sharp character of the aggregate, which sometimes protruded from the surface. Traffic was sufficient to reduce that extreme, by breaking off protruding particles, to levels that are consistent with those of a new concrete pavement. Both slag and lightweight aggregate are producing good results, as are both the latex and acrylic admixtures.

FINDINGS AND RECOMMENDATIONS

A very thin, 0.3-cm (0.125-in) mortar containing selected aggregate for skid resistance and admixtures for good adhesion gave excellent results when it was applied by brooming onto an old existing concrete pavement. The results are explained as follows:

1. The brooming technique gives a high production rate and could be used with mechanical equipment to place a small amount of material per unit of pavement area. A reasonable yield would be 273 m^2 of very thin overlay per cubic meter of mortar (250 yd^2/yd^3).
2. Adhesion has been excellent. The only delamination occurred over bituminous patches and broke away, but the layer is so thin that it breaks up without damage to traffic and without causing objectionable roughness.
3. Skid resistance equal to or better than the original good skid resistance of the pavement was established.

It is recommended that this technique be further evaluated on concrete pavement surfaces that have less than desirable skid resistance. Any application should, in our opinion, ensure the following:

1. Mortar should contain polish-resisting aggregate.
2. Admixtures for adhesion, such as latex or acrylic emulsions, should be included in the mortar.
3. Pavement surfaces should be reasonably free of oil and loose dirt. High-pressure application of water may be sufficient to remove road oil, or sand blasting may be required.
4. Vigorous brooming is required to ensure adhesion.
5. The thickness of the mortar should be kept to a minimum. It is recommended that mortar depth not exceed 0.4 cm (0.187 in). This is not intended to include protruding aggregate particles.
6. Early traffic applications should be at speeds that are either uniform or less than 66 km/h (40 miles/h) to reduce the problem of shear forces on the young overlay.

The Indiana State Highway Commission is developing plans for applying a very thin broomed overlay to a section of state highway in the summer of 1980.

ACKNOWLEDGMENT

The contents of this paper reflect our views and not necessarily the views or policies of the sponsoring agencies.

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Publication of this paper sponsored by Committee on Pavement Maintenance.

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Applicability of Radar Subsurface Profiling in Estimating Sidewalk Undermining

G. G. Clemeña and K. H. McGhee

The results of an evaluation of the applicability of the geophysical technique of radar subsurface profiling to estimating the extent of sidewalk undermining are reported. It was found that there is a distinct difference between the observed radar-echo patterns from a nonundermined sidewalk and those from an undermined sidewalk. Therefore, it is feasible to determine from a radar scan the length of sidewalk that is undermined. It is also feasible to determine the approximate depth of voids beneath an undermined sidewalk, although this may sometimes be difficult to achieve.

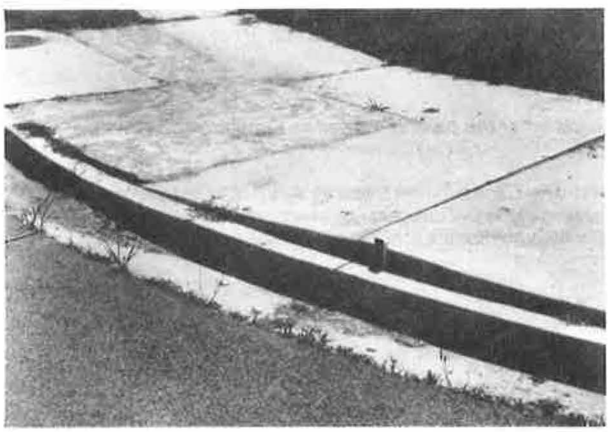
Severe undermining of sidewalks as a result of the erodibility of certain soils is a widespread problem in Fairfax County, Virginia (1). As Figures 1 and 2 show, undermining removes the support from under the sidewalks and results in faulting of the joints, thus creating hazards for pedestrians and peripheral problems in drainage and siltation. The maintenance costs associated with the problem of undermining amount to several million dollars per year. Maintenance personnel believe that as much as $50 million would currently be needed to correct the sidewalk problems that exist throughout Fairfax County.

Most major sidewalk maintenance is contracted, and it is desirable to require reliable estimates of materials and work quantities before the contract is drawn. However, the nature of sidewalk undermining is such that only in the most extreme cases is it possible to accurately estimate repair quantities before beginning the work. The result is that quantities typically run over the limit provided for in the contract. Then, in order to complete the necessary work in a given area, a new contract must be let at additional administrative costs and often at prices less favorable than those in the original contract.

Clearly, a reliable method of estimating the quantity of work required in the repair of an undermined sidewalk would be of great benefit to field engineers charged with the responsibility for maintaining sidewalks. The research reported in this paper was per-

Figure 1. Severe sidewalk undermining.

Figure 2. Sidewalk distortion caused by undermining.



formed in recognition of this problem. A quick survey was made of the geophysical techniques available, and it was found that the newly developed technique of short-pulse radar subsurface profiling could most likely be used to obtain a solution. Therefore, a study was undertaken to assess the applicability of this technique to estimating the extent of voids under concrete sidewalks.

PRINCIPLE OF RADAR SUBSURFACE PROFILING

In radar subsurface profiling, a transducer unit transmits a radar pulse of fixed width at some constant time interval into the earth or other media to be tested. When the pulse strikes an interface between two subsurface materials of differing electrical properties, some of the pulse energy is reflected; the remainder continues on through the interface into the second material until it strikes another interface, and so on. The reflected pulse energy E_r from a given interface is related to the incident energy E_o by the following relation (2):

$$\rho_{12} = (E_r/E_o) = [(\epsilon_1)^{1/2} - (\epsilon_2)^{1/2}]/[(\epsilon_1)^{1/2} + (\epsilon_2)^{1/2}] \quad (1)$$

where ρ_{12} = the reflection coefficient at the interface between materials 1 and 2 and ϵ_1, ϵ_2 = the relative dielectric constants for materials 1 and 2, respectively. (If material 2 has a relative dielectric constant greater than that of material 1, then the reflection coefficient has a negative value. Thus, the reflected pulse will have polarity opposite to the transmitted pulse.) The reflected pulses, or echoes, are then received by the transducer, where replicas of the transmitted pulse, followed by the reflected pulses, are generated and sent to a control unit to be monitored on an oscilloscope and printed on a facsimile graphic recorder.

After it is determined which radar echo corresponds to the reflection from which interface, the thickness D_m of a given layer of subsurface material m through which the pulses have passed can be estimated from the following relation (3):

$$D_m = t_m V_m/2 \quad (2)$$

where t_m = elapsed time between the reflected pulses from the top and the bottom of the material and V_m = velocity of radar-pulse propagation through the material for normal incidence. V_m is related to ϵ_m, the relative dielectric constant of the material, by

$$V_m = C/(\epsilon_m)^{1/2} \quad (3)$$

where C is the velocity of radar-pulse propagation through air [1 ft/ns (0.3 m/ns)].

EXPERIMENTAL PROCEDURES

Location of Sidewalks

To determine whether radar subsurface profiling could be applied in estimating the extent of sidewalk undermining, two sidewalks with apparent undermining were selected and surveyed. Both were located in subdivisions bordering Braddock Road in Fairfax County. One sidewalk was located along Ponderosa Drive and the other along Thames Street. The sidewalks ranged in length from 200 to 300 ft (67-100 m).

Instrumentation

The radar system used (see Figure 3) was manufactured by Geophysical Survey Systems, Inc., of Hudson, New Hampshire. The major components of the system included a model 4400 radar control unit, a model 101C transducer (900 MHz or 1.1 ns), a model EPC 2208 graphic recorder, and a magnetic tape recorder. Power was supplied by a 12-V car battery through a model 02 power distribution unit. Before the survey of the sidewalks, the chart was calibrated by using a 10-ns calibrator so that the elapsed time for any radar echo could be estimated from the echo pattern recorded on the chart.

Survey Procedure

Four simulated sidewalk underminings of different depths—4, 11, 26, and 36 in (100, 280, 660, and 910 mm), respectively—were prepared by digging trenches about 3 ft (0.9 m) wide and covering them with old concrete slabs previously removed from undermined sidewalks. Then the radar transducer was placed successively over each simulated undermined sidewalk, and the corresponding stationary radar-echo pattern was recorded. A similar radar measurement was also made by using a slab placed over solid ground with no void at all.

Each of the two selected sidewalks was marked with curb ticks at 4-ft (1.2-m) intervals. Then the transducer was placed at approximately midwidth of the sidewalk, which was approximately 4 ft wide, and was pulled

Figure 3. Instrumentation for radar subsurface profiling: radar transducer at top and, at bottom, clockwise from lower left, (a) graphic recorder, (b) control unit, and (c) magnetic tape recorder.

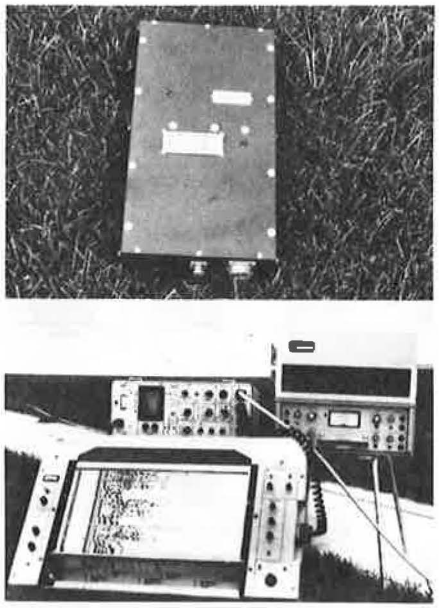
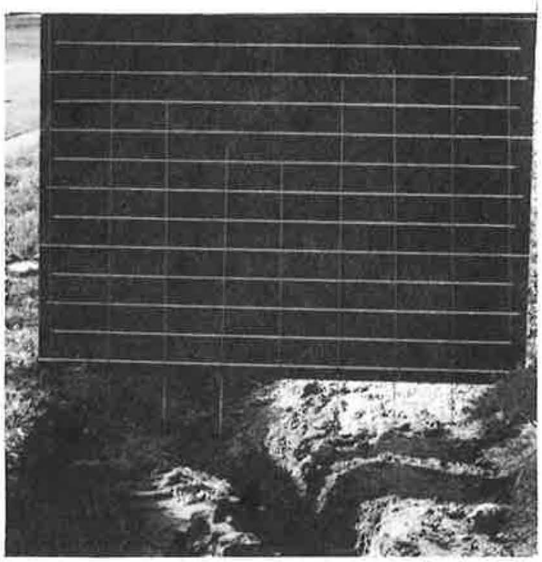


Figure 4. Physical measurement of undermining with a microtopographic profile gauge.



by an operator along the length of the sidewalk at a pace of approximately 1 ft/s (0.3 m/s). As the transducer passed by a benchmark on the sidewalk, the operator activated a hand-held remote marker that provided an electrical signal for the graphic recorder to automatically mark the chart for locational purposes. Thus, the chart recorder provided a linear scan that, as later discussion will indicate, reflects variations in the extent of undermining beneath the middle portion of the sidewalk. From the recorded profiles, numerous spots of varied undermining on each sidewalk were selected for a more detailed lateral survey. This survey was performed by recording the radar-echo patterns as the radar transducer was placed and left stationary first on the left third, then in the middle, and finally on the right third of each selected sidewalk spot.

Physical Measurement of Undermining

Shortly after the two sidewalks were surveyed by radar subsurface profiling, the slabs were carefully removed to permit the actual extent of undermining to be measured. This was done by using a photographic technique that incorporates a microtopographic profile gauge (4). A slightly modified version of the gauge was constructed and used in this study. The device, shown in use in Figure 4, is a wooden frame 3 ft (0.9 m) high and 4 ft (1.2 m) wide (the standard sidewalk width) that holds nine 3-ft metal pins spaced 0.5 ft (0.15 m) apart. The pins are arranged so that they can freely move vertically before a scale formed by horizontal lines spaced 3 in (75 mm) apart on a black backboard. When the gauge is placed over the undermined sidewalk, the pins drop and rest at the bottom of the undermining, and a profile is graphed by the tops of the pins against the backboard. The undermining profile can then be photographed. Profiles of the undermining beneath the two sidewalks were made at 4-ft intervals to provide a means by which the applicability of radar subsurface profiling could be evaluated.

RESULTS AND DISCUSSION

Correlation of Void Depth and Radar-Echo Elapsed Time

The radar-echo waveforms observed for the five simulated undermined sidewalks are shown in Figure 5 as they appeared on the oscilloscope of the control unit. One can simplify interpretation by having only the negative peaks in each waveform printed on the facsimile chart recorder to obtain the echo patterns shown in Figure 6. The intensity of each band in the echo patterns corresponds directly to the amplitude of the peaks in the waveforms.

At the top of Figure 5 and at the left of Figure 6 are the waveform and the echo pattern, respectively, for the composite radar echo detected by the radar transducer where there was no void between the concrete slab and the sandy soil base. The waveform and echo pattern for the zero-void case are distinctly different from those for the cases that correspond to simulated undermining or voids of various extents in that the third and succeeding bands are relatively much weaker for the zero-void case.

In order to understand each echo pattern, one can start with the left pattern and consider the simplistic case of an air-concrete-soil system in which there is no void under the concrete sidewalk slab, as shown in Figure 7. When a transmitted radar pulse strikes the air-concrete interface, part of the pulse energy, as determined by the reflection coefficient of this interface, is reflected (R1). The remainder penetrates into the concrete slab, eventually striking the concrete-soil interface, where part of it, too, is reflected, the amount depending on the reflection coefficient at this interface. The reflected pulse will travel back through the concrete until it strikes the concrete-air interface. Part of the pulse (R21) will penetrate and reach the radar transducer, while the remainder may reverberate within the concrete slab. After from one to four or more reverberations, this remaining pulse will eventually penetrate the concrete-air interface as pulses R22, R23, R24, and so on. Since each succeeding reflected pulse is weaker than the preceding one, as

Figure 5. Radar-echo waveforms observed for simulated sidewalks with different degrees of voids under the concrete slab.

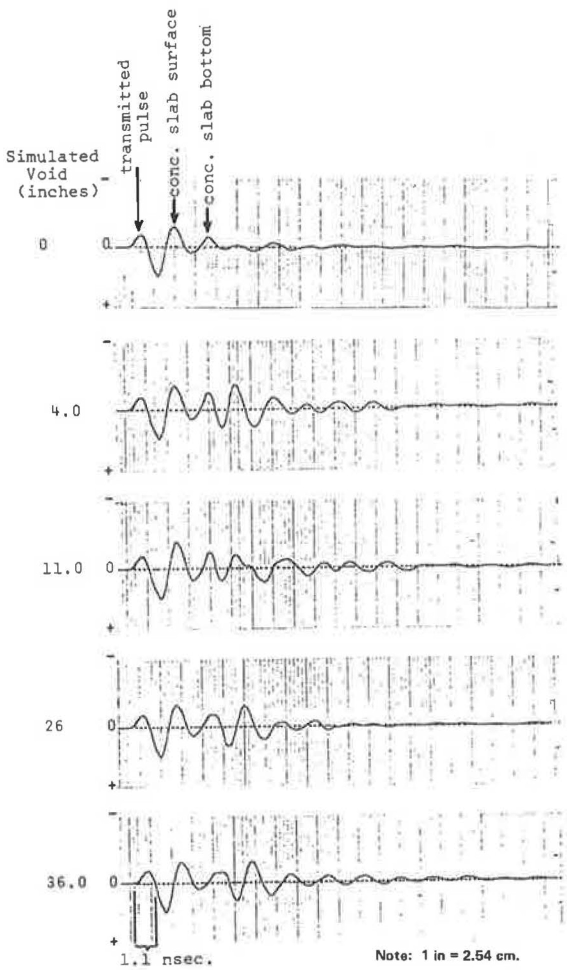


Figure 6. Radar-echo patterns observed for different simulated sidewalk underminings (measured elapsed times in nanoseconds noted beside each band).

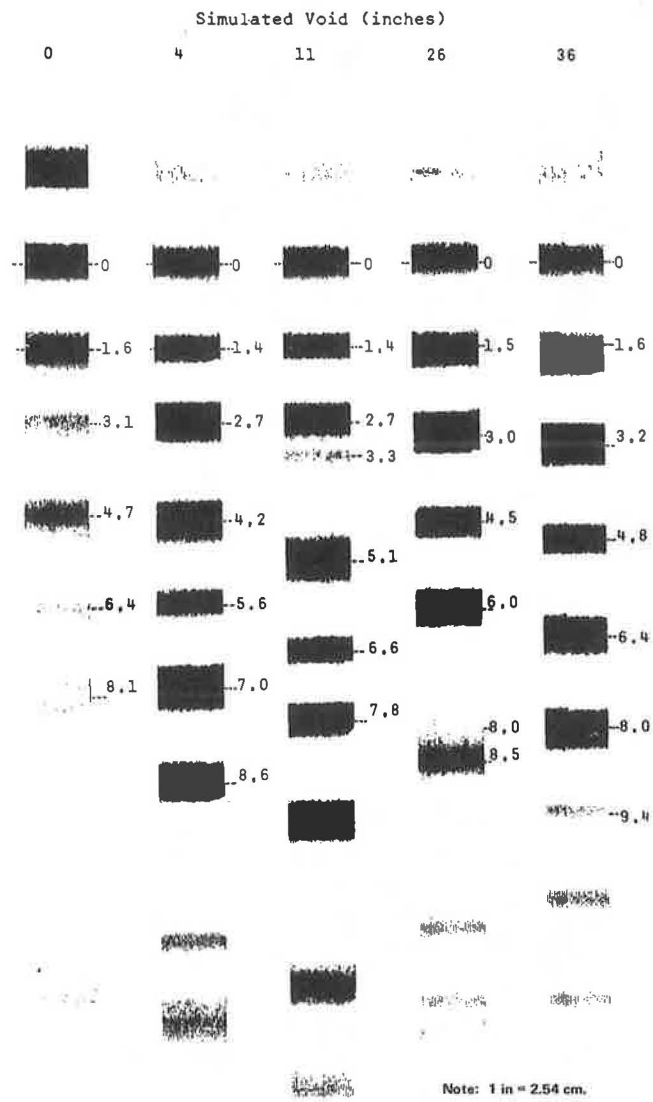


Figure 7. Propagation of radar pulses in air-concrete-soil system.

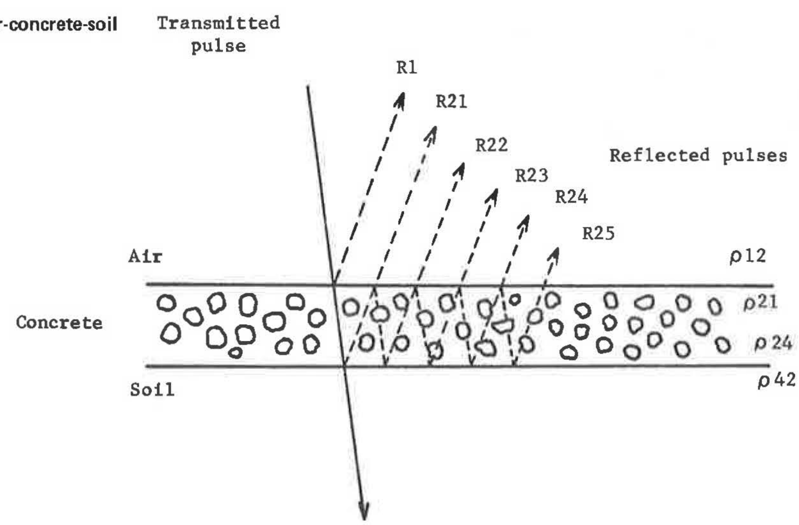
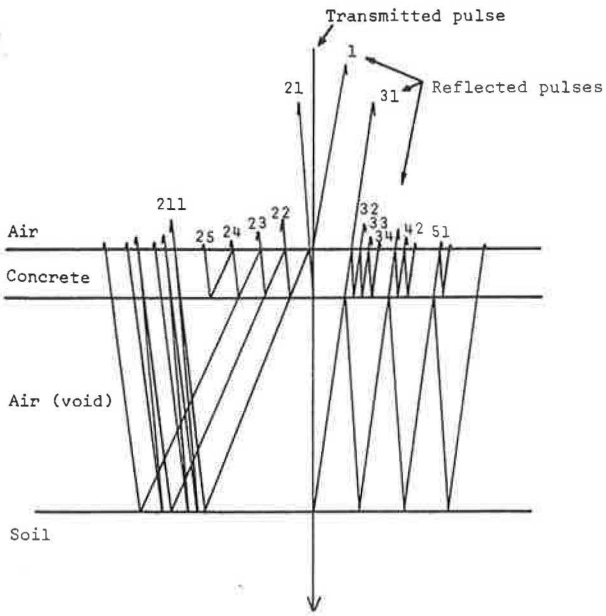


Figure 8. Propagation of radar pulses in air-concrete-air-soil system.



Figures 5 and 6 show, only a finite number of reflections need to be considered for analysis.

Therefore, the radar-echo pattern at the left of Figure 6 for the simulated sidewalk with no undermining is made up of contributions from reflected pulses R1, R21, R22, ..., R25 at least, which are shown in Figure 7. To confirm this, the relative energy and elapsed time of each echo were derived by applying Equations 1-3 and obtaining the approximate mathematical expressions given below. In these expressions, t_c = propagation time for a pulse to travel through the concrete and back, and D_c = slab thickness (in) (1 in = 25.4 mm):

Reflected Pulse	Relative Energy	Elapsed Time (ns)
R1	ρ_{12}	0
R21	$(1 - \rho_{12}^2)\,\rho_{24}$	$t_c = D_c/2.45$
R22	$(1 - \rho_{12}^2)\,\rho_{21}\,\rho_{24}^2$	$2t_c$
R23	$(1 - \rho_{12}^2)\,\rho_{21}^2\,\rho_{24}^3$	$3t_c$
R24	$(1 - \rho_{12}^2)\,\rho_{21}^3\,\rho_{24}^4$	$4t_c$
R25	$(1 - \rho_{12}^2)\,\rho_{21}^4\,\rho_{24}^5$	$5t_c$

Then the energy and time parameters were estimated by using the dielectric constants found in the literature, which are given below:

Material	Approximate Dielectric Constant	Source
Air	1	Morey (3)
Sand saturated with moisture	30	Morey (3)
Silt saturated with moisture	10	Morey (3)
Concrete	6	Moore and Echard (2)

The following table gives the energy and time parameters along with the observed elapsed times for comparison [the estimated elapsed time for pulse R21 was based on a 4-in (10-cm) standard thickness of concrete sidewalk]:

Reflected Pulse	Estimated Relative Energy	Estimated Elapsed Time (ns)	Observed Elapsed Time (ns)
R1	-4.2×10^{-1}	0	0
R21	-3.1×10^{-2}	1.6	1.6
R22	$+5.1 \times 10^{-2}$	3.2	3.1
R23	-8.1×10^{-3}	4.8	4.7
R24	$+1.3 \times 10^{-3}$	6.4	6.4
R25	-2.0×10^{-4}	8.0	8.1

The excellent agreement between the estimated and observed elapsed times is obvious. The agreement between the estimated relative energy and the intensity of the corresponding band on the left echo pattern is less obvious because of the misleading periodic fluctuation in the intensities of echoes R21, R22, etc. However, this fluctuation is explained by the alternate changes in polarity and the resultant canceling effect of the estimated energy of the echoes, as the data given in the final table above indicate.

The echo patterns observed for the different simulated sidewalk underminings can, similarly, be understood by comparing the air-concrete-air-soil system shown in Figure 8 with the air-concrete-soil system shown in Figure 7. Because of the presence of not only more but also different interfaces from which a transmitted pulse can be reflected, the air-concrete-air-soil system would yield more reflected pulses. This is evident in the echo patterns observed for the simulated sidewalks that have various degrees of undermining: There are more bands in the patterns for these sidewalks than there are in the pattern for a simulated sidewalk that has no undermining (Figure 6). The former patterns are more intense, too, because the concrete-air and air-soil interfaces generally have higher reflection coefficients than the concrete-soil interface in an air-concrete-soil system, which is equivalent to a sidewalk that is not undermined. This explains why the radar-echo pattern for a sidewalk that is not undermined is distinctly different from echo patterns for undermined sidewalks and, therefore, why (qualitatively) one can identify or locate undermined areas under sidewalks nondestructively.

The various reflected pulses shown in Figure 8 give rise to the composite echo patterns shown in Figure 6 for the different degrees of simulated undermining. Depending on their polarity and elapsed time, some of these reflected pulses will have either a canceling or an enhancing effect on each other, since the radar pulses used in this study have a pulse length of about 1.1 ns. For quantitative determinations of void depths under concrete sidewalks, the elapsed time that corresponds to reflected pulses R31, R32, or R41 (in order of decreasing importance) could be used in the following generalized relation:

$$D_v = 6(t - 0.41\, N_c D_c)/N_v \qquad (4)$$

where

D_v = void depth (in);
t = elapsed time between reflected pulses 1 and 31, 32, or 41 (ns); and
N_v, N_c = number of round trips the above pulses made through the void or concrete, respectively (derived from Equations 1-3).

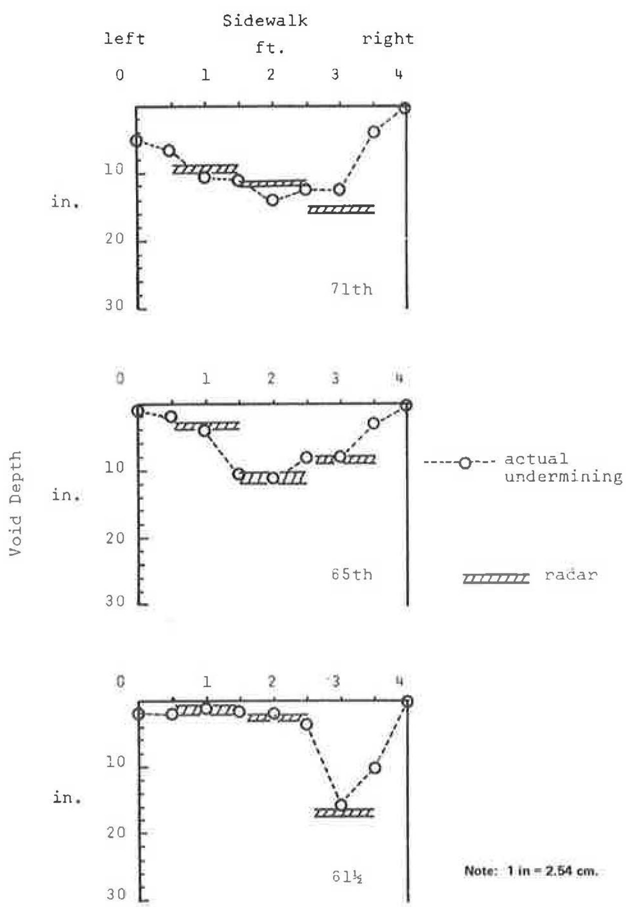
Before this approach can be used, however, one of the reflected pulses in any given composite echo pattern of an unknown void depth has to be identified. It is obvious, from an examination of Figure 6, that developing

Table 1. Comparison of theoretical and observed echo patterns for simulated sidewalk void depths.

Void Depth = 4 in			Void Depth = 11 in			Void Depth = 26 in			Void Depth = 36 in		
	Elapsed Time (ns)			Elapsed Time (ns)			Elapsed Time (ns)			Elapsed Time (ns)	
Pulse	C	O	Pulse	C	O	Pulse	C	O	Pulse	C	O
1	0	0	1	0	0	1	0	0	1	0	0
21	1.4	1.4	21	1.4	1.4	21	1.5	1.5	21	1.6	1.6
31	2.1		22	2.9	2.7	22	3.0	3.0	22	3.2	3.2
41	2.8	2.7	31	3.3	3.3	23	4.5	4.5	23	4.7	4.8
22	2.9		23	4.3		31	5.8		24	6.3	6.4
51	3.4		32	4.7	5.1	24	6.0	6.0	31	7.6	8.0
32	3.5		41	5.1		32	7.3		25	7.9	
61	4.1		24	5.7		25	7.5	8.0	32	9.1	9.4
42	4.2	4.2	33	6.1		33	8.8	8.5	33	10.7	10.9
23	4.3		42	6.5	6.6						
52	4.9		51	6.9							
33	5.0		25	7.2							
43	5.6	5.6	34	7.6							
24	5.7		43	8.0	7.8						
34	6.4										
25	7.1	7.0									
26	8.6	8.6									

Notes: 1 in = 2.54 cm.
C = calculated; O = observed.

Figure 9. Comparison of some radar results with actual undermining profile.



a simple way to identify these pulses is extremely difficult, if not impossible. Therefore, this relatively straightforward approach will probably be used only rarely, and an alternative must be developed.

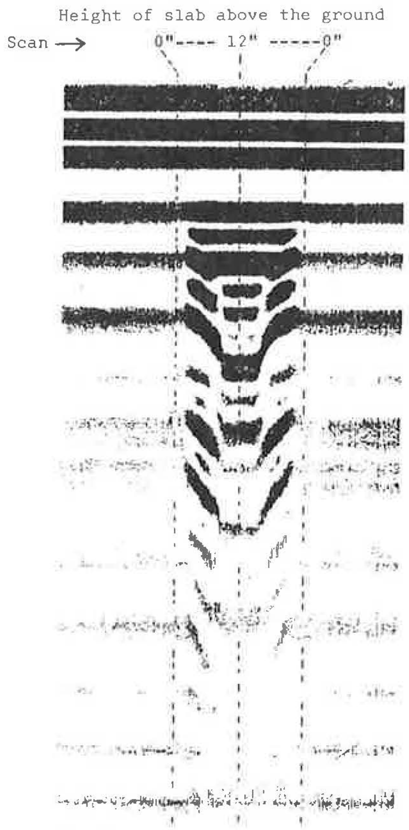
To verify some of the stationary radar-echo patterns observed in the two undermined sidewalks, in addition to those in Figure 6 for the simulated undermined sidewalks, we developed an alternative approach that can be described as "echo pattern matching". In this approach, elapsed times that correspond to an assumed void depth are estimated for the major reflected pulses shown in Figure 8 by using Equation 4. This estimation generates a theoretical or expected echo pattern that consists of expected elapsed times for the assumed void depth. When this is repeated over a range of assumed void depths, at practical intervals, the estimates produce a set of theoretical echo patterns with which any given unknown echo pattern could be compared or matched. The corresponding void depth of the theoretical echo pattern that best matches the unknown echo pattern would, therefore, be the most probable unknown or sought-for void depth.

By using this approach, the radar-echo patterns for the four simulated void depths in Figure 6 were compared with theoretical echo patterns (see Table 1). Except for the fact that some of the calculated reflected pulses for the two shallow voids were not observed, the agreement is extremely good. A more exact calculation of the theoretical patterns, including the polarities and relative magnitudes of the reflected pulses, would show how pulses separated by less than one full pulse width of 1.1 ns overlapped and canceled each other, which accounts for the absence of some close reflected pulses of opposite polarities in the observed echo patterns.

The good agreement between the observed and theoretical echo patterns for the simulated sidewalk underminings indicates that radar subsurface probing can be used to reliably determine void depths in these ideal cases. To test the reliability of the technique under the less ideal situation of a real undermined sidewalk—the void geometry of which can often be more complicated than the wide trenches used to simulate undermined sidewalks—stationary echo patterns were recorded at several selected spots along the two study sidewalks. Then each echo pattern was matched against a set of theoretical or expected patterns corresponding to a range of void depths at 1-in (25.4-mm) intervals.

Figure 9 shows some examples of results obtained when radar-determined void depths were compared with the measured actual voids or underminings. As the figure shows, the radar-determined depths have an uncertainty of at least 1 in because it was often difficult to exactly match an unknown pattern with a single theoretical pattern. The overall agreement in these examples could be described as fairly good. It must be emphasized, however, that there were some unknown radar-echo patterns that, although they belong to different sidewalk spots of almost identical actual void depths and would therefore be expected to match each other, did not match well. Such discrepancies could have been caused partly by the complicated geometry that characterized some of the undermining. It is suspected that, since the radar pulses are transmitted in a 90° conical shape, a gully with an extremely small width-to-depth ratio would probably have an echo pattern significantly different from that of a gully of identical depth but with a larger width-to-depth ratio. Variations in moisture content and the occasional presence of rocks in the soil base would also affect the radar-echo pattern.

Figure 10. Simulated linear radar scan of a segment of sidewalk.



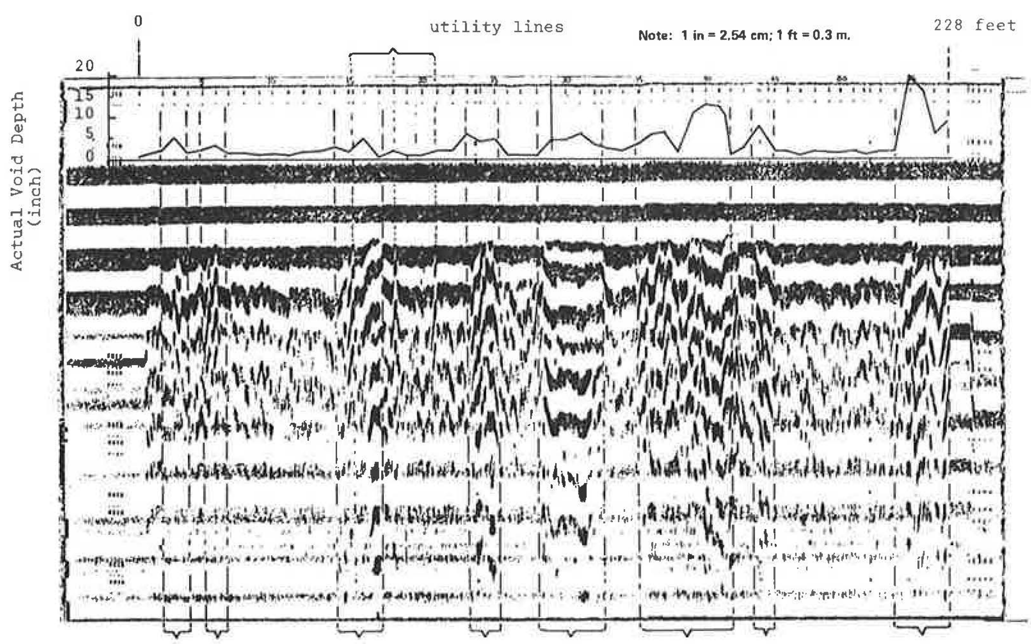
Linear Sidewalk Scan

It has been shown that the void depth at any spot in an undermined sidewalk could be determined from the stationary radar-echo pattern observed for that spot. In an actual application, however, it may not be necessary to determine the exact void depth but only the length of sidewalk that is undermined. In such a case, the sidewalk can be scanned by pulling the radar transducer over its length while the facsimile chart recorder provides a continuous record of the variations in void depth as reflected in the echo pattern. This procedure is referred to as a linear sidewalk scan.

In this research, the radar transducer was placed over a piece of old concrete sidewalk seated on solid ground, and it was possible, simply by gradually raising the concrete slab from the ground, to simulate the scanning of a sidewalk that has a gradual increase in void depth along its length. The recorded pattern simulated a linear scan of such a simulated sidewalk. A scan obtained from such a simulation, to a maximum void depth of approximately 12 in (30 cm), is shown in Figure 10. This figure vividly shows that, as expected from conclusions drawn in the earlier discussion, the echo pattern that corresponds to the presence of any void beneath a sidewalk is distinctly characterized by more reflected pulses of very high amplitudes than that for a solid or nonundermined sidewalk.

In Figure 11, the linear scan of one of the two selected sidewalks is shown superimposed on a graph that represents the actual undermining that had occurred beneath the middle portion of the sidewalk. As previously discussed, the radar-echo pattern for solid sidewalk is characterized by few and weak reflected pulses. Based on this, one could say, from the radar scan, that there were two stretches of relatively less undermined sidewalk (between benchmarks 7 and 14 and 45 and 54) and four shorter stretches between them. It is obvious that the agreement with the physically measured undermining is extremely good.

Figure 11. Linear radar scan of 228-ft segment of sidewalk along Ponderosa Drive in Fairfax County and graph (at top) showing undermining physically measured after concrete slabs were removed.



CONCLUSIONS

From the foregoing discussion it is fairly evident that the radar subsurface profiling technique could meet the need for improved accuracy in estimating the extent of undermining that needs repair. In areas where repairs are anticipated, it would be desirable to evaluate sidewalks over their full length. Such an evaluation should include two scans of each sidewalk at approximately the one-third-width points for good coverage. Although no such studies have been incorporated in this research, it has been established that the radar technique evaluated here is capable of evaluating voids under pavements and could be useful in the development of undersealing and drainage-repair contracts (2).

The conclusions reached in this paper can be summarized as follows:

1. The characteristic radar-echo pattern for an air-concrete-soil system, or a nonundermined concrete sidewalk, is distinctly different from that of an air-concrete-void-soil system, or an undermined sidewalk. This is because, in general, more reflected radar pulses with high energies are observed with increasing void depth.
2. It is feasible to determine from a radar scan the extent of undermining that has occurred beneath a length of sidewalk.
3. If necessary, it is possible to determine the depth of voids beneath an undermined sidewalk through echo pattern matching. However, because of complications in the echo pattern caused by some factors, this may sometimes be difficult to achieve.

ACKNOWLEDGMENT

We thank assistant resident engineer P. D. Gribok and inspector J. Phillips of the Fairfax Residency, Virginia Department of Highways and Transportation, for their help in the many arrangements required for the successful completion of the field work for this study. In addition, we are grateful for the assistance rendered by R. W. Gunn and J. W. French of the staff of the Virginia Highway and Transportation Research Council. We also express gratitude to J. F. Koca of the Soil and Exploratory Techniques Group, Federal Highway Administration, for helpful suggestions.

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Publication of this paper sponsored by Committee on Pavement Maintenance.