cause of a limited time frame, this study did not examine or attempt to estimate the life expectancy of optimally designed and efficiently installed porous lane-marking systems.

Thus, before final decisions can be made regarding overall economy of the porous systems, additional field installations and comparative studies are needed. The economic attractiveness would undoubtedly be improved if appropriate installation equipment and techniques were developed.

ACKNOWLEDGMENT

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The contents of this paper reflect my views, and I alone am responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration.

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# Measurement of Stress in Concrete Pavements

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**Sudden compressive failures of concrete pavements (blowups) are a serious problem for highway maintenance departments. In an effort to predict when blowups will occur, a method of measuring residual stresses within a concrete pavement has been developed. In the procedure, electrical strain gauges are attached to the wall of a corehole by means of a specially designed installation tool. The corehole is overcored and the relief strains are measured. Available theory has been adapted to allow computation of longitudinal stress at the level of the gauges. Laboratory tests have validated the procedure, but results from tests on actual pavements have proved to be somewhat erratic.**

A blowup in a concrete pavement is a buckling failure, usually at a transverse joint and is due to compressive forces in the pavement. Seasonal variations in temperature and moisture content cause the pavement slabs to expand and contract. During contraction of the slabs, the joints open and, if improperly sealed, may receive charges of detritus from the pavement surface, subbase, and shoulder. Subsequent expansion is prevented at these filled joints and compressive stresses result. Blowups rarely occur in new pavements. Such failures usually begin in pavements 5-10 years old and continue for the life of the pavement, whether overlaid or not.

An unusually large blowup, which occurred on June 24, 1975, on OH-21 in Summit County, is shown in Figure 1. This blowup was 0.53 m (21 in) high and involved both northbound lanes for approximately 4.57 m (15 ft).

Blowups usually occur at contraction joints; sometimes they happen at transverse cracks. Joints are particularly susceptible to infiltration by deicers, water, and debris and undergo more severe stress patterns due to traffic than the rest of the pavement. These stress patterns are repetitive and dynamic. Load-transfer devices, such as dowels, that are improperly installed or not functioning properly can cause spalling of the faces of the joint. Many joints inspected after being involved in blowups demonstrate prior deterioration of as much as half of the joint faces. Figure 2 illustrates such disintegration.

The combination of joint deterioration and high compressive stresses in the pavement leads to blowups. Blowups may be gradual or sudden and can involve both large and small areas.

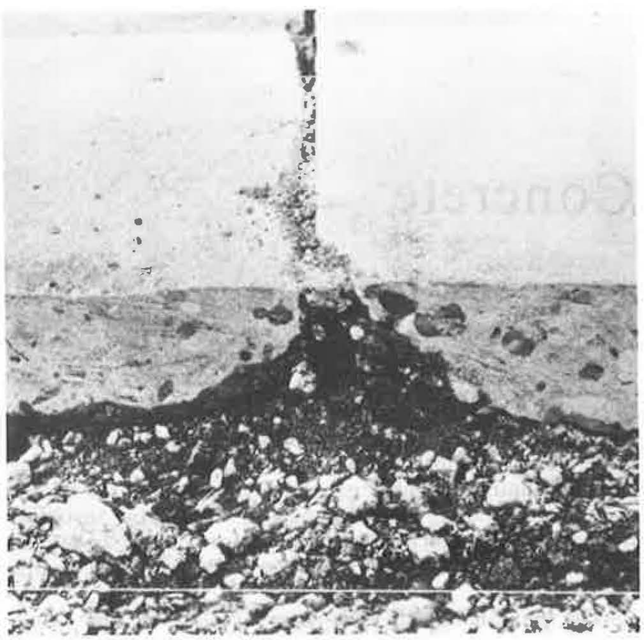
To repair a blowup, full-depth saw cuts are made just outside the zone affected and the concrete is replaced (in Ohio) with bituminous concrete, usually without rebuilding of the subbase. This procedure is expensive and delays traffic, often for several days. The state of Ohio, after waiving its immunity to lawsuit in 1975, has been successfully sued by at least one motorist who was injured by a pavement blowup (Arthur E. Knickel v. State of Ohio Department of Transportation, Court of Claims No. 750329).

Preventive maintenance involves plowing out the sealant and routing or sawing joints in areas suspected of possible blowups. The joints are then resealed. If the progressive formation of high compressive forces could be anticipated and measured, temporary relief could be furnished by simply sawing a new joint in the pavement and filling it with an appropriate sealant.

Some of the variables that have been studied in conjunction with the blowup problem are ambient tempera-

Figure 1. Blowup on OH-21, Summit County, 1975.

Figure 2. Transverse cross section of pavement slab containing severe D-cracking. White line shows original bottom of slab.



ture, surface temperature, strength of concrete, frequency of joints, types of joints, types of subbase, types of shoulders, drainage, amount of steel reinforcement, construction season, quality of maintenance program, use of deicers and deicing grits, age of pavement, moisture content of the concrete, load-transfer design at joints, type of sealant, sealant reservoir design, polymer sealing device, size of coarse aggregate, source of aggregate, composition of aggregate, overlaying (or resurfacing), and amount and type of traffic.

A National Cooperative Highway Research Program study (1) states that

> Although many studies have been made of blowups, most have been concerned with correlating the number of observed blowups with environmental conditions, the number and type of joints in the pavement, and the lengths of pavement between blowups. Little is known in a quantitative way about stresses at the time of blowup.

## THEORY

The stress at any point P in an elastic, homogeneous, isotropic, and continuous solid can be represented generally as shown in Figure 3 that has the X, Y, and Z axes and any arbitrary set of orthogonal axes.

Assume that a concrete pavement is such a solid. A corehole of radius a is drilled into the pavement in the z direction, as shown in Figure 4. The stresses at a distance r from the center of the hole (at r, $\theta$, z), in terms of polar coordinates, can be related to the basic state of stress by the theory of elasticity. At the periphery of the hole (r = a),

$$\sigma_{rr} = 0 \tag{1}$$

$$\sigma_{\theta\theta} = (\sigma_x + \sigma_y) - [2(\sigma_x - \sigma_y)\cos 2\theta + 4\tau_{xy}\sin 2\theta] \tag{2}$$

$$\sigma_{zz} = \sigma_z - \nu[2(\sigma_x - \sigma_y)\cos 2\theta + 4\tau_{xy}\sin 2\theta] \tag{3}$$

$$\tau_{r\theta} = 0 \tag{4}$$

$$\tau_{\theta z} = -2\tau_{xz}\sin\theta + z\tau_{yz}\cos\theta \tag{5}$$

$$\tau_{rz} = 0 \tag{6}$$

Eight strains are required to evaluate the six perimetric stresses. Three 45° rosettes are arbitrarily placed as shown in Figure 5 and strains are measured in directions 1-9. Here, $\theta$ = 0°, 90°, and 225° and z = depth. Stresses in the nine directions can be evaluated by using Equations 1-6, and these stresses can be manipulated to yield the six stresses in the original state of stress. Using basic strain relationships for a 45° rosette with Hooke's law yields the following:

$$\sigma_x = \tfrac{1}{8}\,[E/(1-\nu^2)]\,[\epsilon_1 + \nu\epsilon_3 + 3(\epsilon_4 + \nu\epsilon_6)] \tag{7}$$

$$\sigma_y = \tfrac{1}{8}\,[E/(1-\nu^2)]\,[3(\epsilon_1 + \nu\epsilon_3) + \epsilon_4 + \nu\epsilon_6] \tag{8}$$

$$\sigma_z = [E/(1-\nu^2)]\,[\nu\epsilon_1 + \epsilon_3 - \nu/2(\epsilon_1 + \nu\epsilon_3 - \epsilon_4 + \nu\epsilon_6)] \tag{9}$$

$$\tau_{xy} = \tfrac{1}{8}\,[E/(1-\nu^2)]\,[\epsilon_1 + \nu\epsilon_3 + \epsilon_4 + \nu\epsilon_6 - 2(\epsilon_7 + \nu\epsilon_9)] \tag{10}$$

$$\tau_{xz} = [E/4(1+\nu)]\,(2\epsilon_5 - \epsilon_4 - \epsilon_6) \tag{11}$$

$$\tau_{yz} = [E/4(1+\nu)]\,(2\epsilon_2 - \epsilon_1 - \epsilon_3) \tag{12}$$

where E is the modulus of elasticity and $\nu$ is Poisson's ratio.

The forces causing pavement blowups are essentially longitudinal, i.e., along the direction of the pavement. Consider Figure 5. When the x-axis is oriented in this longitudinal direction and the rosettes are oriented as shown, then the primary stress $\sigma_x$ is given by Equation 7. The four strains required for this determination are given by rosettes 1 and 2 only. Rosette 3 is not required if only $\sigma_x$ is desired. Also, rosettes 1 and 2 may be biaxial only. In the experimental work, therefore, biaxial rosettes are used at locations 1 and 2, as shown in Figure 6, and Equation 7 becomes

$$\sigma_x = \tfrac{1}{8}\,[E/(1-\nu^2)][\epsilon_1 + \nu\epsilon_2 + 3(\epsilon_3 + \nu\epsilon_4)] \tag{13}$$

## MEASUREMENT EQUIPMENT

The obvious approach to an evaluation of these residual stresses is to use electrical strain gauges on the surface of the pavement. However, several studies of blowups (see Figure 2) have indicated that the lower portion of a blown-up joint experienced prior deterioration. The residual stresses in the slab away from the joint are probably reasonably uniform throughout the depth. At the joint, however, the reduced cross sec-

Figure 3. Stresses at a point in pavement.

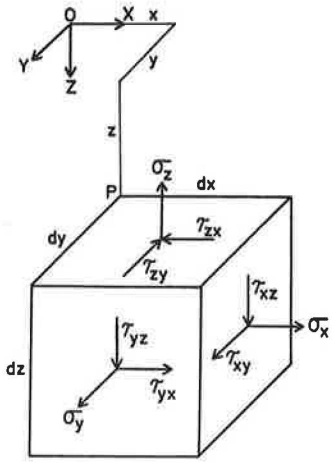
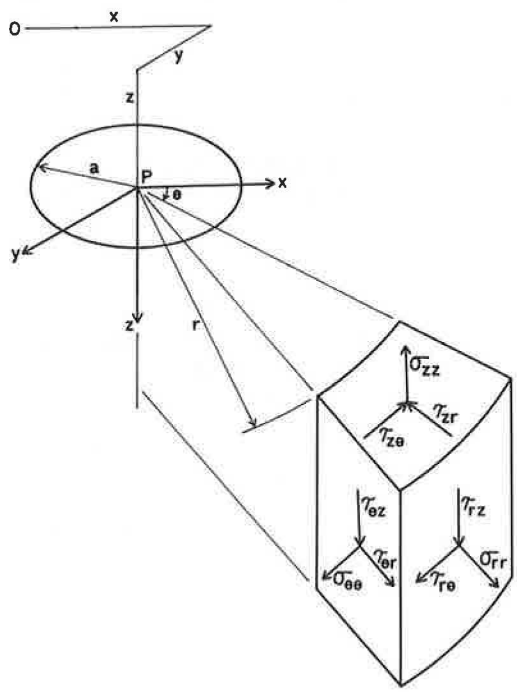



Figure 4. Stresses at a point in cored pavement.




tion causes a nonuniform distribution of stress—a mixture of axial and flexural normal stresses, where the maximum stress is at the bottom of the cross section. Therefore, in order to study stress variations in the slab, it was considered necessary to develop a method to measure strains within the pavement.

During the last 30 years, many devices for evaluating stresses in rock masses have been developed. Several of the most successful of these were considered for this project. The triaxial strain cell developed by Leeman (2) measures nine strains on the surface of a borehole as it is overcored. These strains yield the entire three-dimensional state of stress of the rock at one location for a single borehole, provided that (a) there are no cracks or other discontinuities near the instrument to deform the stress concentration pattern and (b) the rock moduli are known. Leeman's concept was used to develop an installation tool for placement of the two biaxial rosettes required by Equation 13.

Common diameters of core drills used to obtain pavement samples for testing are 38.1 mm (1.5 in) and 102 mm (4 in). It was decided to drill a 38.1-mm diameter borehole and to overcore with a 102-mm diameter bit.

Figure 5. Location of strain gauges.

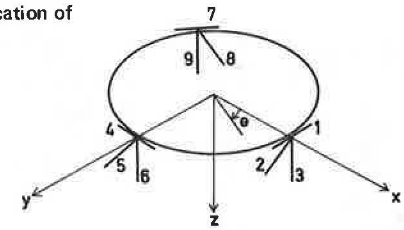
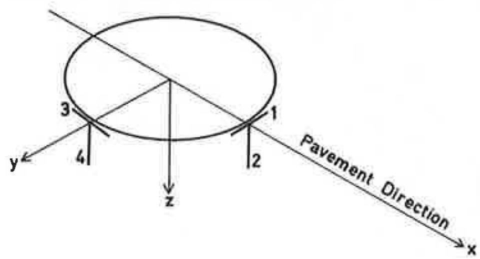



Figure 6. Location of biaxial rosettes.




The tool was formed from a 295-mm (11.62-in) length of stainless steel pipe with a 25.4-mm (1-in) outside diameter (OD) and a 3.18-mm (0.12-in) wall. One end was capped and the other was threaded to engage a coupling to a compressed air hose. Two 3.18-mm diameter holes were drilled through the pipe 38.1 mm from the capped end. These holes, normal to each other, furnished four air outlets equally spaced around the capsule wall. A 63.5-mm (2.5-in) length of rubber with a 25.4-mm inside diameter (ID) and a 3.18-mm wall was slipped over the four holes. The clearance from the rubber to the end of the capsule was 6.35 mm (0.25 in), so that the air holes were at the mid-length of the rubber tube. Epoxy was applied to the inside of each end of the rubber tube to attach it to the metal pipe. Each end of the rubber tube was also wrapped tightly with 8-10 turns of nylon cord, also glued in place.

This installation tool with gauges attached to its outer surface would fit easily into the 38.1-mm borehole. However, to prevent the possibility of gauges striking the edge of the hole, two metal guides were devised, one at each end of the rubber tubing. These guides, cut from 4.76-mm (0.19-in) stainless steel plate, had a 25.4-mm ID and a 31.8-mm OD and three projections 120° apart. The projections have an OD slightly less than the diameter of the borehole.

The guide at the top of the tubing is permanently attached to the pipe. The bottom guide, if permanently attached, will be very likely to strike the gauges unless the installation tool is withdrawn very carefully. In the field, therefore, the lower guide was attached lightly with Vaseline and remained in the bottom of the borehole until all measurements were taken. The final installation tool shown in Figure 7 has two biaxial rosettes attached and is ready for placement in a borehole. The upper guide and two of its projections can be seen just at the top of the rubber tubing.

## LABORATORY TESTING

Two specimens were manufactured for laboratory testing of the installation tool and verification of the stress theory. These specimens measured approximately

Figure 7. Installation tool.

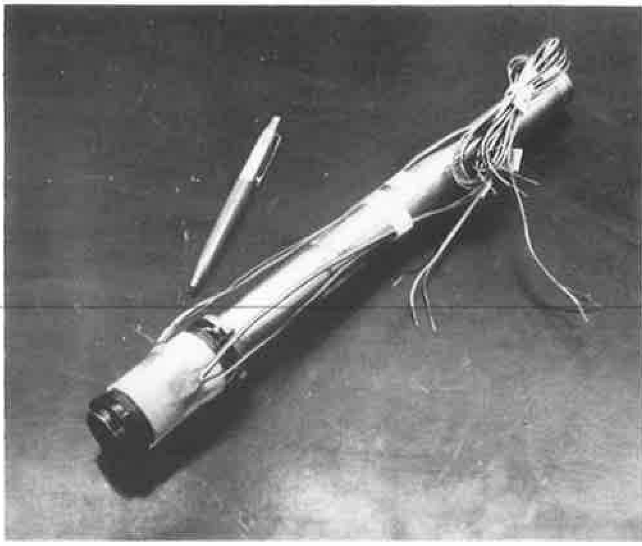
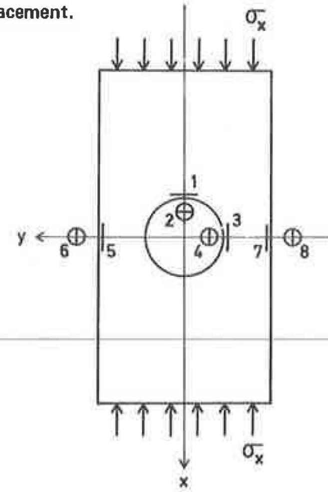


Figure 8. Specimen in testing machine.

203x305x610 mm (8x12x24 in).

One specimen contained 6x12-2/4 mesh; the other did not. Both contained a 38.1-mm diameter hole at the center of the 305x610-mm face. The hole extended completely through the 203-mm dimension and was preformed with pipe. One of the specimens is shown in Figure 8. Leads from four biaxial rosettes, two within

Figure 9. Gauge placement.




the holes and two on the 203x610-mm sides, are shown attached to the switch and balance unit. The rosettes on the sides of the specimens furnished data for computations of modulus of elasticity (E) and Poisson's ratio ($\nu$). Placement of gauges in Figure 9 shows the hole enlarged for clarity.

The most acceptable biaxial rosette proved to be Micro-Measurement EA-06-250TA-120. The lead finally chosen was Micro-Measurement 326-DFV three-conductor stranded tinned-copper 26-gauge vinyl-covered wire. Devcon 5-min epoxy and a pressure of 414 kPa (60 lbf/in$^2$) gave acceptable results in the laboratory.

A maximum stress level of 6210 kPa (900 lbf/in$^2$) was chosen to ensure elastic action of the specimens and to permit retesting. The load was applied in three increments of 2070 kPa (300 lbf/in$^2$) by a Baldwin universal testing machine with a capacity of 136 000 kg (300 000 lb).

Equation 13 was used to find $\sigma_x$. Results from the last two laboratory tests are shown in the table below (1 kPa = 0.145 lbf/in$^2$). Sample 1 contained mesh; sample 2 did not.

| Actual Stress (kPa) | Sample 1 (kPa) | Sample 2 (kPa) |
|---|---|---|
| 2070 | 2430 | 2400 |
| 4140 | 4780 | 4890 |
| 6210 | 7160 | 7360 |

These values, although approximate, are considered to be adequate estimates of stress level. One small discrepancy in procedure involves placement of gauges 2 and 4. It was decided to align gauges 1 and 3 on the x and y axes because of their large and sensitive readings. Gauges 2 and 4 are therefore approximately 6.35 mm (0.25 in) off the point under consideration. The chief reason for the discrepancy is probably the nonhomogeneity of the material. The mesh, which had a 152x152-mm (6x6-in) grid centered on the hole, did not appear to affect the results.

## FIELD TESTING

Six field tests were conducted, all in regions undergoing high blowup activity. Because repaired blown-up joints can absorb considerable pavement growth before stress builds up, the joint to be tested was always chosen in a stretch of pavement relatively free of blowup patches, spall patches, and so on. Usually a reasonably straight

Figure 10. Core and overcore.


section of roadway was chosen, primarily as a safety precaution. In three of the six sites, the joint chosen was adjacent to a bridge.

The two corehole locations were marked on the pavement with a felt-tipped pen. For several reasons, the coreholes were located on a transverse line 1.83 m (6 ft) from the joint. The selected joint was in good condition, so it was felt that stresses at the joint would not differ appreciably from those 1.83 m away. The distance was also chosen to avoid the effects of contact irregularities at the joint and to avoid dowel bars across the joint. Two holes, placed symmetrically either 0.914 m (3 ft) or 1.22 m (4 ft) from the edge of the lane, were used so that an average stress level could be computed across the lane (assuming a linear variation transversely).

A 38.1-mm corehole was drilled at the mark with the lower vertical elevation in order to avoid fouling the other location with drilling paste. When the truck-mounted drill was not available, a 110-V Milwaukee portable core drill was used to drill both the corehole and the overcore. Thin-walled diamond-tipped drilling bits were used. Considerable water was used in these operations and was carried to the site in five 0.0189-m$^3$ (5-gal) plastic bags with pouring spouts. Pressure for the water cooling the portable drill was furnished by a 0.0114-m$^3$ (3-gal) metal can with a hand-operated piston. A Sears 2500-W alternator with a gasoline motor furnished the power for both the drill and the Sears compressor, a standard 110-V model with a capacity of 827 kPa (120 lbf/in$^2$).

The compressed-air hose was inserted at very low pressure into the hole to dry the corehole wall. After a short drying period, the corehole surface was prepared for gauge installation. Three different materials were used, in the following order: chlorothene aerosol (degreaser), M-Prep conditioner (mild acid), and M-Prep neutralizer. These liquids were applied to gauze pads that were held by a pair of forceps and rotated thoroughly against the corehole wall.

The installation tools were prepared in the laboratory and came to the site with gauges already mounted. In the laboratory, a piece of Teflon tape was placed around the rubber tube to prevent epoxy, applied at the site, from reaching the tube itself. The two biaxial rosettes were attached face down to a piece of double-backed Scotch tape; gauges 1 and 3 were centered a distance apart that equalled a quarter-circumference of the corehole (see Figure 6). This tape was then placed around the Teflon-covered portion of the tubing. By means of guidelines on the tubing, the rosettes were placed so that, when the tubing expanded, they would be as close as possible to the desired locations in the corehole. Leads long enough to reach the readout equipment were attached to the rosettes in the laboratory, were labeled, and were held to the tool with masking tape.

Epoxy was mixed for 90 s and applied to the two rosettes with a Q-tip within the next 30 s. The tool, attached to the compressor, was carefully inserted into the corehole to the desired depth. The compressor regulator was usually set at 414 kPa (60 lbf/in$^2$). Pressure was allowed into the tool; the tubing was expanded; and the rosettes were forced against the wall of the hole. A 20-min set time was used and was measured from the beginning of the epoxy mixing.

While the epoxy was drying, the four leads were attached to a Vishay model SB-1 switch and balance unit. This unit had been connected in the laboratory to a Vishay V/E20A multipurpose direct-reading digital indicator.

After the 20-min period, the pressure was released and the tool was extracted from the hole. Ambient temperature was measured and the four gauges were zeroed. The gauges were waterproofed with a thick coating of Vaseline. The leads were disconnected, banded, and pushed into the hole just above the rosettes. A tapered rubber stopper, also coated with Vaseline, was pressed into the top of the hole. These precautions were taken to prevent drilling paste from the overcoring from reaching the gauges.

The corehole forms a discontinuity in the pavement. The original state of stress has been distorted by this discontinuity, but the relationships between the original and the distorted stresses were developed earlier. The distorted stresses are now represented by the four zero-strain readings. In order to measure the residual stresses, it is necessary to remove the original state of stress. Overcoring releases the residual stresses and is equivalent to superimposing stresses of opposite sign on the original stress state.

A 102-mm (4-in) diameter diamond drill was centered on the corehole and was forced through the pavement. A typical core and overcore are shown in Figure 10. The small hole near the bottom of the overcore was drilled later so the rosettes could be used for measuring material properties.

The overcore was removed from the pavement and the rubber stopper was extracted. The leads were attached to the readout equipment and strain readings were taken. While the second hole was being prepared, it was possible to take additional readings on the first sample at approximately 10- to 15-min intervals. These readings showed a disturbing tendency to change as time elapsed. Two possible reasons for these changes are temperature fluctuation and creep.

The corehole acts as a temperature sink and, as with stresses, causes considerable distortion of the temperature pattern near the hole. Both drilling operations also, because of drill friction and cooling water, change the temperature distribution in the overcore. When the cool wet overcore is removed from the slab, it begins to move toward the ambient temperature. A computer model for the overcore was developed (3) and possibly explains most of the change in strain in most of the overcores.

In the laboratory, creep was not found to be a serious problem. In several tests, the stress level of 6210 kPa (900 lbf/in$^2$) was held for 10 min and no appreciable changes in strain readings were recorded. This stress is greater than that measured at five out of six sites in the field. Therefore it can be concluded that creep is probably not as important as temperature in causing these time-dependent changes in strain.

The second test was identical to the first. After the overcore was removed from the second hole, both holes were patched with cold-mix asphalt concrete or ready-mix concrete.

In order to compute modulus of elasticity and Poisson's ratio, laboratory testing was required. It was decided to use the same biaxial rosettes used in the field tests and still attached to the overcores. The samples had to be capped with sulfur compound, so it was necessary to drill a small hole near one end in order to use the leads, as shown in Figure 10.

Because overcoring is equivalent to removing the residual compressive stresses in the slab, the effect is the same as an application of tensile force. The signs of all terms in Equation 13 must be reversed. Results of all tests are shown in the table below (1 kPa = 0.145 lbf/in$^2$).

| County | Sample No. | Residual Stress (kPa) | County | Sample No. | Residual Stress (kPa) |
|---|---|---|---|---|---|
| Ashtabula | 1 | - | Stark | 1 | +50 |
| | 2 | -1160 | | 2 | +1840 |
| Clermont | 1 | -480 | Summit | 1 | -920 |
| | 2 | -2740 | | 2 | -740 |
| Hancock | 1 | -3910 | Warren | 1 | -6840 |
| | 2 | -7030 | | 2 | -6220 |

## CONCLUSIONS

Of the six sites inspected, it was felt that the final two sites in Hancock and Warren counties offered the best evidence of the efficacy of the field procedure. The tests were performed without problems and the results were reasonably consistent. Stresses (see the table above) at the Hancock site were -3910 and -7030 kPa (-567 and -1020 lbf/in$^2$) and at the Warren site they were -6840 and -6220 kPa (-992 and -902 lbf/in$^2$).

This long stretch of I-75 was in such deplorable condition and was under such extensive repair that selection of a sound joint was difficult. In Warren County it was necessary to use a joint in the passing lane. It would seem that stress levels of 6890 kPa (1000 lbf/in$^2$) were accompanied by considerable joint deterioration. OH-21 in Stark County was also in poor condition. The fact that the computed stress at one corehole remained tensile throughout the test indicated that the gauges were not properly bonded to the corehole wall.

This project uncovered areas that deserve additional study. Two of the most important questions raised are (a) what the effect of temperature change on the strain gauge readings is and (b) at which stress level remedial maintenance should be initiated.

Variations in temperature in the corehole and overcore seriously affect the strains being measured. If the temperature on the wall of the corehole differs from the general temperature at that level of the slab when the strain gauges are zeroed, then the measured strains will not be correct. If the overcore temperature differs from that of the test region when gauges were zeroed, then the measured strains will not be correct.

One approach to solving this problem would be an experimental field study of the temperature variations involved. Another approach would be to base the upper-limit stress and temperature on a standard field test procedure. If the gauge installation and overcoring could be performed on a strict time basis, then the strains measured would at least be comparable from test to test.

As indicated earlier, a stress level of 6890 kPa (1000 lbf/in$^2$) is accompanied by extensive deterioration at the joints. A residual stress of 3450 kPa (500 lbf/in$^2$) measured at an ambient temperature of 26.7°C (80°F) is a possible upper limit that would trigger initiation of stress relief maintenance.

If a stress value at a particular temperature were chosen as an upper limit dictating stress relief, stress measured at other than the standard temperature would have to be corrected. Correction could easily be accomplished by using the results of the computer model (3). For example, if the ambient temperature were 21.1° C (70° F) when strains were measured, the model, by using the assumed material properties, will indicate an increase of about 2070 kPa (300 lbf/in$^2$) in longitudinal stress at 26.7°C (80°F).

Several facets of the test procedure require more study. It is difficult to install the two biaxial rosettes in a 38.1-mm (1.5-in) corehole. Some consideration should be given to a 50.8-mm (2-in) or possibly even larger diameter. However, a larger corehole with the same 102-mm (4-in) overcore increases the possibility of fracturing the wall of the overcore. All 12 of the overcores for this project remained intact throughout all testing.

If a larger overcore diameter is attempted, then the probability of striking the steel mesh is very high. Encountering mesh causes two problems: The drilling time is greatly increased and mesh near a gauge installation has an effect on the strains being measured. It is recommended that a 50.8-mm core be tried with a 102-mm overcore and that a metal detection device be used to locate the mesh pattern. The installation tool would not need any modification.

A stacked strain-gauge rosette would permit both gauges to be placed accurately at the desired location. A larger corehole would permit larger rosettes to be used. Two rosettes at 120°, instead of 90°, apart would allow larger rosettes to be used. A gauge with option W (Micro-Measurement) allows easier lead attachment and a more stable mechanical connection, and complete encapsulation offers more protection to the grid.

The adhesive used for attaching the gauge to the corehole wall during this project seemed somewhat sensitive and performed well most but not all of the time. The setting time of 20 min severely delayed the field tests. Adhesives manufacturers should be consulted to find a 5-min adhesive that can be used under field conditions.

Overlays should not affect the field test in any way.

If a stress-measurement program is undertaken, the following recommendations are in order. The testing equipment and crew should be completely self-contained. A two-person crew, one skilled in strain-gauge techniques and the other reasonably good mechanically, is sufficient. A van, small truck, or Jeep-type vehicle can carry all the equipment required and should have a rear bumper suitable for holding a portable drilling rig. This unit should carry signs, cones, and other equipment needed for traffic safety. With sharp drilling bits, a metal detector, and a 5-min adhesive, it should be possible for such a crew to complete a test in about an hour. It may be possible, as experience accumulates, to dispense with one of the two holes at a test site.

If stresses exceeding the upper limit are encountered consistently in a region, then stress-relief measures could be initiated. Most states cut a full-depth 102-mm gap completely across the pavement and fill the gap with asphalt material. Michigan has designed a hydraulic device, mounted on a maintenance truck, that forces a 102-mm strip of preformed foamed plastic filler into a 76.2-mm (3-in) wide gap cut by a Vermeer saw. Although some of the strips have been pulled out by traffic, the joint is considered successful at this time.

For new construction, several recommendations can be made. To avoid infiltration from below and from the sides, a thin asphalt concrete base should be placed before the slab is poured, and shoulders should be paved. The joint should be sealed by a preformed polychloroprene sealer that should also seal the joint vertically at each side of the slab. Special care should be used in placing dowels at the joint, so that no misalignment occurs.

Some thought might be given to different thicknesses for the traffic and passing lanes. Heavy commercial vehicles use the traffic lane most of the time and the severe repetitive compressive stresses they cause would be reduced by using a thicker pavement.

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*Abridgment*

# Evaluation Methods for Bridge Deck Membranes

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An evaluation of 44 applications of waterproofing membranes placed on rehabilitated concrete bridge decks in New York state is reported (1). The applications were not designed as field experiments but were made by a variety of state maintenance forces under normal working conditions.

Unsound concrete had generally been removed, but the level of residual chloride in unexcavated portions of the deck was not determined. Evaluations were based on copper-copper sulfate half-cell potential and electrical resistivity measured between one and five years after membrane placement. In a few instances, measurements were also made at the time of membrane placement, but more commonly placement was not observed. The paper deals primarily with the evaluations of data collected under these less-than-ideal circumstances.

Thirty of the applications were of one of two liquid-applied membranes that had been New York State Department of Transportation (NYSDOT) standards between 1958 and 1974. Their performances were compared and found to be superior to that of a representative national sample of about the same age distribution studied under a National Cooperative Highway Research Project study.

The national experience with membranes at that time was generally considered unsatisfactory. These standard membranes were then used as the norm against which the performances of 14 experimental membranes (both liquid and sheeting type) were evaluated. The experimental membranes, as a group, were found to have performed better at comparable ages than the standard membranes, but not outstandingly so.

In addition to the evaluation of specific membrane applications, the study resulted in the following more generally applicable conclusions:

1. Three of the only four membrane installations that consistently performed better than 75 percent of the norm group were observed to have been preceded by careful leveling and smoothing of the deck surface.
2. The physical condition of membranes examined in core samples, including the integrity of their bonds, was generally consistent with the level of electrical resistance measured at the site (Table 1).
3. The physical condition of concrete examined in core samples, including rebar corrosion, was generally