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Bridge Joint Systems—A Performance Evaluation

JAMES J. HILL and ARUNPRAKASH M. SHIROLÉ

ABSTRACT

In this paper bridge joints in Minnesota, from open joints to relatively waterproof strip seals and modular sealed expansion joints, are evaluated. Performance evaluations along with special designs that prevent damage from high-speed snowplow operations are included. Maintenance procedures for rehabilitating joints that leak are also discussed.

Many different types of joint systems have been used in bridge construction in Minnesota during the past three decades. The evolution process in the use of these different systems has primarily been guided by the need to devise and use leak-proof, trouble-free, and zero or low maintenance expansion-contraction joints. The objective of this paper is to review this process and to present an evaluation of different types of joint systems based on performance of more than 2,000 bridge joints in Minnesota.

AVAILABLE DATA

Figure 1 shows commonly used bridge joint systems. The data in Table 1 provide information such as advantages and disadvantages, typical problems, and installed costs of these different joint systems. For the purposes of this evaluation, information such as type and age of installation, current condition, special problems, and type of maintenance required was collected for 2,271 bridge joints. The number of each type of joint evaluated, as well as the number and corresponding percentage of joints that were reported to be leaking, are given in Table 2.

DEVELOPMENT AND USE OF DIFFERENT BRIDGE JOINT SYSTEMS

The original expansion and contraction joints in bridge decks merely consisted of sliding plates as shown in types G, F, and L (see Figure 1). These systems were intended only to carry wheel loads across the joint opening in bridge decks, and movement of water and finer materials were allowed to go unimpeded. As debris and corrosion problems became apparent in joints, bearings, and beam seats, elastomeric and other compression joint seals were used in the attempt to seal deck joints (types J and K). Because of their inability to adhere to the adjacent materials, these seals worked out of the joints. Further, dirt and debris built up over these types of compression seals and caused rapid deterioration of the seals.

The next developmental stage was the use of a variety of concrete joint sealers and waterstops (type Q). These waterstops failed to function when bridge decks expanded and contracted; as a result they tore apart. Further, during construction ends of waterstops make concrete placement difficult. However, because of better performance of some waterstop installations, use of joint systems such as types A, B, C, E, and F became more common. Most of these systems were segmental and experienced leakage through joints between segments. The type C system used a continuous neoprene gland and performed satisfactorily. However, in these systems bolting down of the claw was difficult, and in some instances glands came out of the claw quite readily.

An extrusion type claw (type H) was then used, which held the neoprene glad effectively. Ends of these glands were shaped to conform to the inside of the claw. These glands, when kneaded into the extruded claw, generally became secure. Despite debris collection problems, the glands performed satisfactorily and remained in the claws.

PERFORMANCE EVALUATION OF BRIDGE JOINTS

Data available up to and including 1983 were reviewed to determine the degree to which different joint systems performed satisfactorily. The unsatisfactory performance of various joint systems was found to have caused some damage to adjacent bridge components as well. Common problems experienced with expansion joint devices were investigated.

Leakage

Leaking has been by far the most common problem associated with bridge joints in Minnesota. Fifty-five percent of the 2,271 joints investigated exhibited this problem. Leakage was typically at curb lines, through joints between segments, along the edge of seal bordering the deck, or through the interface between gland and claw. Of 496 segmental joint systems investigated, 366 (74 percent) were leaking. Concentrated leakage through joints between segments of segmental devices of types A, B, E, and F was observed. In some instances leakage was observed between the expansion devices and the adjacent end dam material.

As a result the end dam material was observed to break up, thereby exposing the expansion device directly to traffic. Gland or seal types of joint devices in some cases were found to be ruptured and failed because of traffic debris and tearing under traffic loads.

Remedial actions. The use of a continuous device eliminated the problem of leakage between joint segments. Application of sealers at edges and over end dams after installing the device was found to be effective. Partial repair of ruptured glands by patching a new piece of gland material over the damaged area was quite successful (see Figure 2). The procedure used for patching neoprene glands was as follows:

1. Blow out dirt with compressed air and clean the gland area with a solvent such as methyl ketone or toluene.
2. Place a 0.0625-in. neoprene sheet (0.5 in. wider than the gland and length of damaged area plus 6 in. each side is required) down into the valley of the in-place gland. Form it up each side and draw a line 0.25 in. above where the gland goes into the extrusion. Cut the patch along these lines.

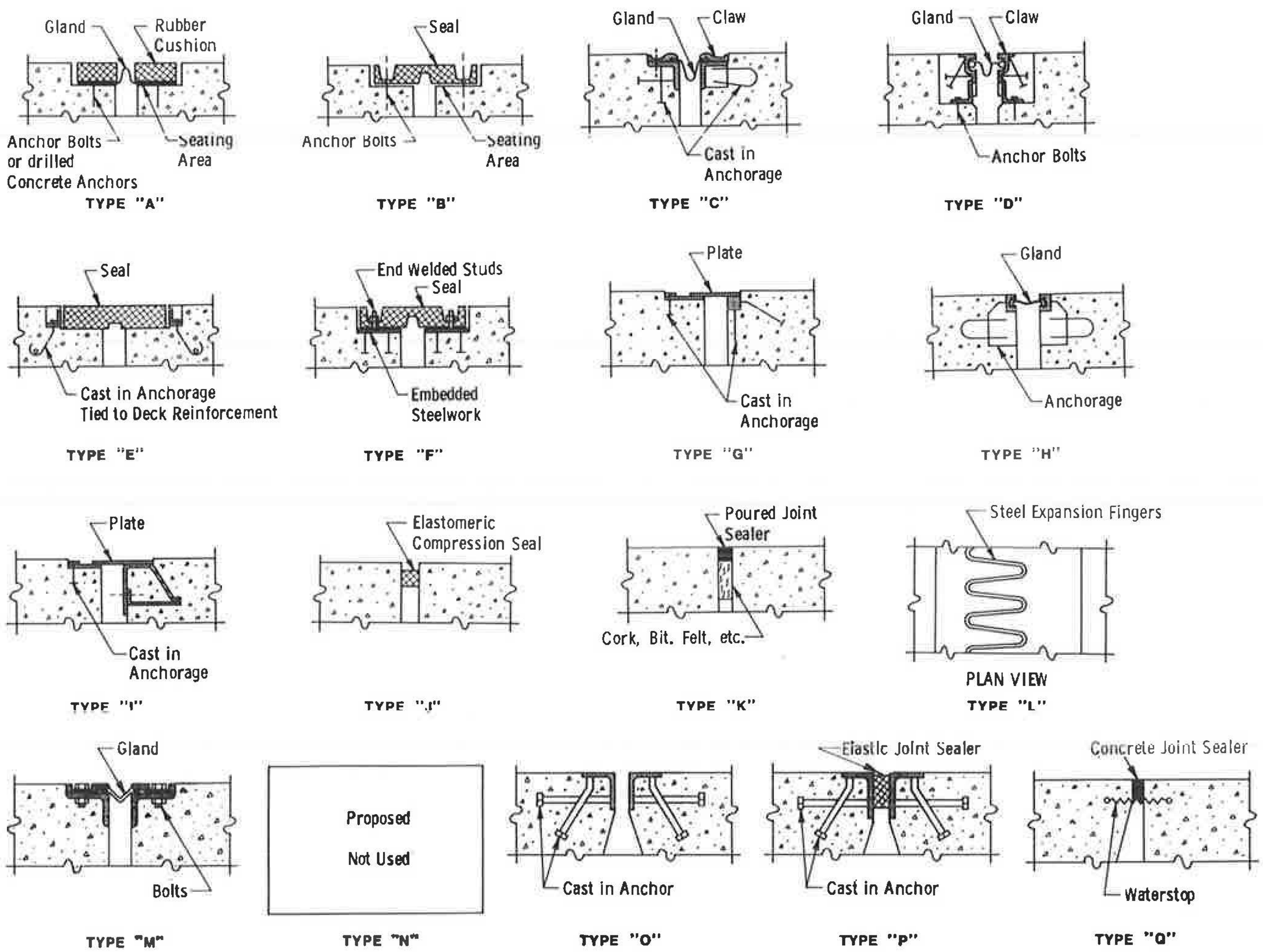

FIGURE 1 Types of expansion devices.

TABLE 1 Data on Types of Expansion Devices Studied

Expansion Device	Period Used From	Advantage	Disadvantage	Type of Problem	Typical Cost[a] ($/linear ft)
A	1975-1977	Easy to install; gland expanded and contracted well	Segmental; hard to recess	Leaked at joints and between rubber and concrete end dam; damaged by snow plows	138.27-151.30
B	1973-1977	Easy to tighten down	Seals above bolts come out	Leaks at joints; dirt fills into bolt holes and causes corrosion	88.12-121.53
C	1976-1980	Gland placement easy	Claw is too short and ineffective	Gland pulls out; hard to tighten down claws	43.50-60.00
D	1978-1981	Armored claw	Complex welding	Anchor bolts pull out	120.00
E	1975-1981	Easy to install	Segmental	Leaked at segment joints	45.50
F	1968-1978	Easy to tighten down	Studs worked loose	Leaked at segment joints	53.57-108.92
G	1965-1975	Simple to install	Leaked below sliding plate; sliding plate forces upward	Sliding plate breaks off	13.24-15.38
H	1977-present	Does not leak	Imported and patented product	Dirt accumulates in gland	85.00-100.00
I	1958-1975	Easy to install	Leaks between sliding plate and adjacent base plate	Sliding plates broken off by traffic and snow plows	25.00-40.00
J	1960-1975	Inexpensive	Seal comes out; leaks	Hard to keep seal in	3.00-5.00
K	1965-1973	Inexpensive	Does not allow compression	Hinders expansion of concrete slab	13.50-16.50
L	1960-present	Good for large expansions and contractions	Bolts in traffic wheel tracks break off and loosen and fall off	Binds up easily from horizontal misalignments	33.26-123.20
M	1958-1967	Easy to install	Joint opening does not stop water	Completely ineffective	60.00-80.00
P	1963-1969	Inexpensive	Compression seal works out of joint leaks	Somewhat ineffective, depends on bond between joint sealer and steel angles	20.00-30.00
Q	1958-1979	Does not leak	Hard to place rubber waterstop in concrete; concrete deteriorates above waterstop	Holds corrosive agents that deteriorate concrete	40.00-70.00

[a]Costs are in place as of time of installation.

TABLE 2 Joints Investigated

	All Joints			Segmental Joints		
Type	No.	No. Leaking	Leaking (%)	No.	No. Leaking	Leaking (%)
A	25	4	16	23	3	13
B	31	14	45	23	14	61
C	73	26	36	15	7	47
D	164	56	34	36	12	33
E	12	9	75	2	0	0
F	12	8	67	12	8	67
G	401	323	81	141	126	89
H	589	46	8	32	3	9
I	121	103	85	33	30	91
J	100	74	74	22	15	68
K	599	467	78	117	112	96
L	43	40	93	9	8	89
M	1	0	0	0	–	–
N	2	0	0	1	0	0
O	32	26	81	5	5	100
P	57	44	77	25	23	92
Q	5	2	40	0	–	–
S	1	0	0	0	–	–
T	3	3	100	0	–	–
Total	2,271	1,245	55	496	366	74

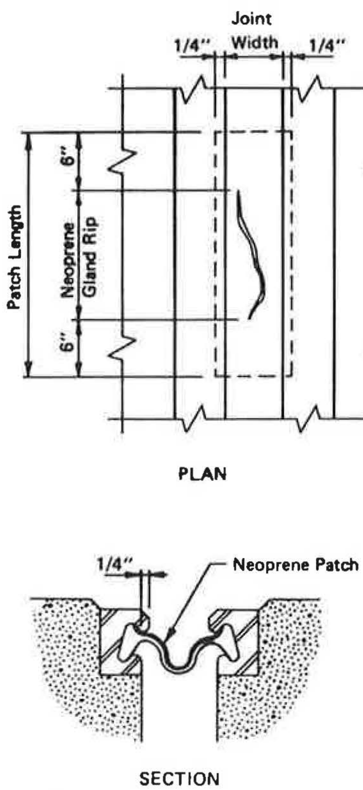

FIGURE 2 Patching neoprene glands.

3. Without puncturing the patch, tuck the extra 0.25 in. of patch on one side into the extrusion groove with a screwdriver or blunt tool.

4. Flip the patch over with a brush, coat the underside of the patch and the in-place gland with "crazy glue" or equal, beginning at the tucked in side. Make sure there are no wrinkles in the patch.

5. Tuck the extra 0.25 in. of patching on the remaining side into the extrusion groove.

6. Coat the exposed ends of the patch liberally with bonlastic adhesive to obtain a water tight patch.

Where ruptured areas were extensive, total gland or seal replacement was found to be desirable.

Corrosion

Uncoated expansion joint devices and those located where chemical debris could accumulate were found to corrode rapidly. Slot covers (mainly in types B, E, and F) were founded sheared off and missing, thereby allowing bolts to corrode and break off. As a result devices lifted up and became subject to severe traffic wear. Corrosion of steel plates used in expansion devices appears to have been accelerated by deleterious entrapments between the plates. Bronze and steel bearing plates corrode quickly and freeze. Such frozen bearings can cause additional stresses in adjacent structural components and shorten their service life.

Remedial actions. To eliminate problems associated with corrosion, it was necessary to remove, clean, straighten, protectively coat, and then replace the expansion device. Where corrosion damage was extensive, replacement of the entire device was considered desirable.

Deterioration

Heavy wheel loads pound improperly placed and exposed plates, angles, seals, and glands to cause rapid disintegration of adjacent materials. Plates and angles bend, warp, and sometimes break off from their anchorages. Types G, I, L, O, and P have been especially prone to this problem.

Remedial actions. Heating of warped plates to restore their original shape and welding back bits and pieces have been of questionable value. Complete replacement of a part or an entire expansion device is preferable.

Restrained Movement

Expansion joints that trap dirt and debris restrain free movement. This can cause disintegration of glands and seals. When seals and glands of types A, B, C, D, K, P, and Q are forced upwards, they are subjected to extreme traffic wear and tear. Restraint on movements at the joint causes spalling and breakup of adjacent materials.

Remedial actions. Movement restraints are located and removed. Partially or completely damaged areas are repaired or replaced. Most of the adverse effects of restrained movements can be prevented with a maintenance program of thorough cleaning, especially each spring.

Settlement and Misalignment

Uneven settlement and vertical misalignment can cause damage to types G, I, L, and Q. The devices warp and break off at their anchorage, thus causing further disintegration of surrounding concrete. Horizontal misalignments cause joint devices such as type L to bind and arrest movement of the bridge deck. As a result, surfaces adjacent to the device are damaged.

Remedial action. When settlement or misalignment results in the joint device failure, it is desirable to replace the device.

Vibrations and Accident Damage

Heavy moving loads cause vibrations that can distress joint devices and fracture joint assemblies. Further, fractured joint assemblies cause damage to adjacent components. Failures of anchorages from inadequate welds, fabrication, or drilled-in anchorages initiate and aggravate vibrational damage. Improperly placed bolts, plates, angles, or seals of expansive devices are easily damaged or sheared off by snowplows, other maintenance equipment, and heavy commercial traffic loads.

Remedial actions. Loose connections and inadequate anchorage generally cause a joint device to vibrate under traffic. In such cases the joint device and anchorage system surrounding the damaged area should be removed and replaced. Joint devices damaged by accidents are either modified in the field to make them secure or are replaced.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are drawn from the evaluation of expansion devices in Minnesota.

1. Joint devices and glands must be continuous and not segmental.
2. Concrete material should be used on either side of the expansion device and the joint should be sealed between the device and the concrete.
3. The expansion joint device should be recessed 0.25 to 0.5 in. below the adjacent concrete.
4. Snowplow guards for glands should be added on expansion devices placed at 20-degree or greater skews. Three-eighths steel bars placed out of wheel tracks will work adequately.
5. Claws of expansion device must hold the device securely. Bolted down claws generally loosen up and allow the gland to easily pull out.
6. Devices must be protected with a coating such as galvanizing.
7. Routine bridge maintenance should include cleaning the gland out and minor repairs to the gland.
8. Cast-in-place plate anchorage systems hold the device securely during construction and in service. Drilled-in anchorages work loose and expose the device and gland to potential damage.

Specification Writing for Bridge Deck Joint Sealing Systems

GUY S. PUCCIO

ABSTRACT

In this paper a simple way to write specifications for expansion joint systems, so as to obtain an economical system with good performance characteristics, is demonstrated. The purpose of the paper is to bring to the design engineer's attention aspects of contract documents that, if not properly handled, can result in controversy or cost overruns. It is demonstrated that if the specifications clearly describe the desired expansion joint, and if the contract drawings show its characteristics and physical requirements and show how it is to be installed, then the right expansion joint can be obtained at the right price through competitive bidding.

The first breakthrough in the development of a satisfactory sealed expansion joint occurred in the early 1960s through the introduction of the elastomeric compression seal. Since then, many alternative expansion joint systems have been available for sealing expansion joints in bridges.

The proprietary nature of these systems made it difficult, if not impossible, to write a universal, meaningful specification. Also, some of the expansion joint systems were failing within a short period of time after installation.

In the quest for improving the performance of expansion joint systems, the Transportation Research Board funded a project to study criteria for developing specifications. Subsequently, a report was written suggesting various criteria for a performance specification. Initially, "segmented seals, bolted to the bridge deck, subjected to varying degrees of tension and compression" and having a moment range of 2 to 4 in. (1) were addressed in the report.

The basic premise was to test these expansion joints as a system in the laboratory and evaluate the results. The systems would be put through several thousand cycles of various testing procedures, which included flexing, impact loading, skew racking, leakage evaluation, and so forth. If the system did not exhibit signs of deterioration or fatigue due to stress and maintained its watertightness, it would be accepted for use in the project.

However, the described tests are valid only when applied within the context for which the report has been written. The report promoted a performance specification that would test tension-compression type solid elastomeric expansion joint systems, the deficiencies of which are well understood.

On the other hand, when applying these criteria to other types of systems, the specification requirements relegate the tests to a material evaluation. For example, applying the cycling concept (e.g., flexing a strip seal 10,000 cycles) will not determine whether a particular system will give satisfactory performance; if a particular rubber material specified were capable of being flexed tens of thousands of times, there would be no need for this test.

Another point is that there must be a discernible engineering difference in or a modification of the use of the joint material specified in order for it to react differently from what could reasonably be expected.

What is being suggested here is that joint systems of like engineering design parameters that are manufactured from identical materials will behave in a similar fashion. Thus engineering education and experience can be used in evaluating and comparing various manufactured expansion joint systems for their conformity to a given specification or their subsequent likelihood of satisfactory performance.

Because there are ample data on physical and mechanical properties of engineering materials, a performance specification of the nature discussed becomes redundant after the initial test, and it becomes cumbersome to administer as well as needlessly restrictive and expensive to the ultimate user.

A side effect of this form of testing is the erroneous premise "more or greater is better." Engineers have adopted criteria for specifying expansion joint systems without field testing or other bona fide documentation as to either their validity or need. Certain requirements have recently evolved that have no real engineering significance other than relating to a given physical parameter chosen by a specifier.

If an expansion joint has been subjected to field performance evaluation and it is constructed of known engineering materials, there is no cause to eliminate the expansion joint from consideration for the sole reason that it has not met an arbitrary physical parameter.

The purpose of this paper is to demonstrate a simple way to specify expansion joint systems in order to obtain an economical system with good performance characteristics.

To begin with, specifications are only a part of the contract documents, and the engineering plans and the proposal are equally important in conveying information to the bidder and to the inspection forces, including the owner's engineers. It is proper use of all these instruments that provides the intent of quality expected. How the design engineer uses his or her knowledge of the subject and combines known parameters within contract plans and specifications is what in essence will determine the type and quality of expansion joint system ultimately incorporated into the project.

GENERAL

The design engineer is charged with the responsibility of selecting and specifying the type of expansion joint he feels will perform best at the most economical cost to the owner. The first and perhaps the most difficult task is the selection of the proper expansion joint, after which the engineer must provide specifications and contract drawings delineating requirements for their manufacture or fabrication and for their installation during construction.

These contract documents must be explicit enough to fully describe what the engineer expects both in terms of materials and performance, without unduly limiting the bidders to either a single source (in the case of a proprietary-type expansion joint) or restricting them to details that may prohibit new improvements in design or materials that may have been made but not incorporated in the contract before the bidding and awarding phases.

There exist many documented cases where improvements that were made in the design concepts or materials of a specified expansion joint were prohibited from being incorporated into an ongoing project because they were made too late for inclusion in the bidding documents, and inadequate contract provisions did not permit their later acceptance or approval for use.

To be sure, changes should always be submitted for approval to the proper authority, so that they can be scrutinized and evaluated with respect to contract plans and specifications. However, even this apparently simple procedure often becomes an impossible task because allowances in the bidding documents prohibit changes, alterations, or substitutes, even when constituting a benefit to the owner.

The reason given for this approach is usually the legality of making changes after the bids are accepted and the contract has progressed. However, it need not be a hindrance if the specification is properly written and the engineering plans are correctly detailed.

One of the most beleaguering tasks the design engineer has to undertake is the selection of equipment or preassembled goods that are of a proprietary nature. This is especially so with expansion joints, because there are clearly many types of joints that are dissimilar in materials and configuration, but will, according to their manufacturers' literature and by appearance, produce the same end result. Expansion joints also invariably cover a wide range of prices, which is often the major criterion used for their ultimate selection; this may result in the cheapest first cost, but not necessarily the most economical life-cycle cost for that particular chosen application.

After selecting the type of expansion joint most appropriate to the physical and environmental conditions for a particular bridge, there is certain information the design engineer must include in the bidding documents, so that he obtains both the highest quality and most economical joint for the project.

BIDDING DOCUMENTS

Once the type and size of expansion joint are selected by the design engineer, the next immediate question is, How does one specify the expansion joint system so that the specifying agency and/or owner obtains the quality desired and remain within budget limitations.

Some engineers simply choose the easy way out by using an entire proprietary specification and adding the words "or approved equal." Proprietary specification refers to describing, in worded detail, patented features, the context of which constantly refers to or mentions a brand name. This procedure, without question, is the least professional and the one that most often will cause problems with contractors and other manufacturers not specifically listed.

The design engineer who adopts a proprietary specification, without deliberately intending to, will most likely

1. Create a specification that is too restrictive by specifying patented components or features of the proprietary product;
2. Eliminate competitive bidding, thereby substantially increasing construction costs;

3. Prohibit the use of improvements made in the product selected after the specification has been made part of the contract because of references to specific features or catalog information; and

4. Require additional cost factors to be used by the contractor when bidding to account for unknown elements and uncertainties with respect to alternates.

The discussion here is centered on the two following points:

1. What the design engineer must include or delete from the contract documents to obtain the highest quality expansion joint at the most economical life-cycle costs level, and

2. What the bidders or producers of expansion joints need to know so that they can furnish the proper joint at the lowest cost to the user or owner.

It is current knowledge among engineers that a poor specification can cause manufacturers or contractors, who have high quality goods or services to offer, to overbid. In other words, the contractor who knows how a product or service is to be tendered will account for those elements that would produce the quality he knows is expected, while the uninformed one will not.

Because every type of expansion joint has both attributes and shortcomings, it is wise to include some form of performance requirement in the bid documents.

To formulate a good specification, the engineer must first select the type of joint he desires and decide which joints he will permit as alternatives. To allow any type of joint during the bidding process would be a mere sentimental gesture and accomplish nothing in the way of quality. There could be cases where cost may be the primary objective in lieu of all other considerations, but it is generally not an acceptable criterion.

In order to base the specification on known acceptable parameters, it would be wise to develop a general classification system for expansion joints. This will enable the specifier to confine or simplify the wording and any other special conditions required that would normally be repeatable for use on future projects.

One type of joint classification for purposes of establishing bidding documents might be as follows:

I. Open joints (with or without drainage appurtenances)
II. Compression seals (unarmored or armored)
III. Strip Seals
   A. Elastomeric retainers or headers
   B. Steel retainers or headers
IV. Elastomeric joints (tension-compression type)
V. Modular and multiseal units
VI. Finger or tooth joints
VII. Aluminum (should be used as special classification)
VIII. Others (can have as many classes as there may be types)

Open joints fall into the category of gaps or openings in the concrete or steel deck with or without armor protection. Today engineers use troughs, gutters, and so forth beneath the gap to collect runoff water, thus protecting the bearings and structural elements below the deck surface. Open joints have all but been abandoned as a viable system. The spaces and troughs fill with debris and silt, eventually causing failure of the system or high life-cycle costs because of periodic maintenance demands.

Compression seals can be used with steel (armor) joint edges or with sawed concrete joint faces. Compression seals are made by several manufacturers, and there is considerable technical data published on the subject. Material characteristics and quality for the seals have been standardized, and the current ASTM specification appears to adequately cover these parameters. The use of general construction procedures and installation techniques along with these physical and material specifications will adequately ensure satisfactory performance.

Strip seals, on the other hand, are generally comprised of two basic components--the strip seal gland and the device that contains or secures the gland to produce a watertight seal. Therefore, although the gland is elastomeric in nature, the retaining device, whether mechanically locked, bolted in place, or molded as an integral part of the gland, can be elastomeric, aluminum, or steel.

Examples of the various types of strip seal systems, as currently produced, are

1. Elastomeric: Fel-Span, Elasto Dam, Trojan;
2. Aluminum: Alu-Strip, On-Flex, Delastiflex, and Acme Titan; and
3. Steel: Pro-Span, Maurer, Acme Strip Seal, and Gen-Strip CD.

The use of proprietary names, in giving these illustrations, is for clarity only, and is not intended to either promote or slight any manufacturer.

Category IV (in the outline given previously) would include the proprietary elastomeric molded type of joints similar in construction and configuration to Transflex and Waboflex.

Modular and multiseal expansion joint systems are more complex because they normally entail some form of expertise and use various engineered mechanical features to provide the movement range desired. The only caution to be given is that similar systems be specified for a particular project, without opening the specification to such broad implications as to allow steel versus aluminum or box-shaped lock-in seals versus strip seals to compete with one another.

Because of the nature of aluminum compared with steel and the differences in the ambient environment throughout the North American continent, this category, whether used with strip seal or any other type of joint system, should be considered as a special item unto itself when specifying. It is strongly suggested that when specifying aluminum, the designer give due consideration to all ramifications. For example, some concerns that must be evaluated and considered in design include fatigue life, brittleness in cold climates, bimetal or galvanic corrosion possibilities, coefficient of expansion in relation to the substrate or embedment materials, impact attenuation with reference to its weight versus load distribution, special handling and welding equipment required for original manufacturing and particularly for future maintenance work, salt corrosion of the aluminum, as well as oxides that may react unfavorably with embedment concrete.

After the basic type of expansion joint has been determined and evaluated, the specification can be written. For example, if an elastomeric joint (type IV) is selected, the bidding documents would describe and limit the contract to molded elastomeric joints with similar properties; if an aluminum (type VII) constructed joint is desired, those manufacturers that have acceptable aluminum joints would be competing; similarly, if it is desired to incorporate into the contract a high-quality expansion joint such as the Maurer or Acme MSB series, with heavy-duty, high-strength components, the design engineer should specify that type of joint and include within the specifications those features or approved alternatives that will be permitted.

This is an important point because when specifying two or more completely different types of expansion joint systems for use on the same project, the least expensive type will undoubtedly be bid lowest. If this is the intent of the design engineer, then why specify more than one type?

Also, another problem is created because two specifications must be written for the same item of work because a single specification describing two different systems would be confusing and inadequate.

Finally, a well-written specification allows for one or more approved equivalents (no two similar joints are absolutely equal), and will thereby create competitive bidding, thus obtaining the best economical joint.

The specification should include, but not be limited to, the following items:

1. Description: General description of the type of expansion joint desired, allowing for approved equivalents.
2. Materials and performance: This section should contain material requirements spelling out testing procedures and sampling techniques or methods.
3. Construction: Construction requirements and installation procedures should be detailed only to the point that the supplier and contractor know what is expected. This section should also include shop drawing requirements, site preparation, and special conditions not otherwise anticipated in the course of installation.
4. Method of measurement: Method of measurement should clearly delineate the limits and how measurements are to be taken so that there is no question as to the quantity to be paid for.
5. Basis of payment: The last section merely contains the elements for consideration in payment.

The foregoing general list is a suggested guide. There are many formats to writing specifications, and there is absolutely nothing wrong in adopting any style as long as it is clear as to exactly what is intended.

More specifically, some of the items to be considered in writing a specification are as follows. The general description should denote the type of expansion joint desired and indicate to some degree the basic quality required. It is also important, when bidding, to know sizes and quantities of material samples required for testing purposes, including the time that the samples should or will be taken. If samples are not required, the certifications desired by the owner should be spelled out.

Additional uncertainty is created when the material specification describes one material, when actually another is to be furnished. For example, if a preformed elastomeric compression seal is fully described, and a lock-in type seal is mandated, the material specification will most likely not be applicable to a large degree. Both seals are made from similar materials, but they operate under dissimilar engineering concepts. Therefore, if the specification is applied with indifference, the lock-type seal will normally fail the recovery tests (physical characteristic), ensuring difficulties between the supplier and materials testing agency. The specifying authority must be flexible enough to recognize real differences in products, especially those material attributes that relate to performance criteria, and not create additional "red tape" and undue delay when evaluating an equivalent item.

On the other hand, material specifications must be explicit to the point of including alternatives and equivalents either by describing them or by reference, so that material testing agencies will be aware of differences, and approved procedures will remain simple and unencumbered by bureaucratic nitpicking.

Painting and coating requirements should be outlined in this section of the specifications. Whether using a primer, rich zinc paint, special epoxy paint, metalizing, or hot dip galvanizing, parameters such as thickness, areas to be coated, and restrictions in their use should all be described. Often the painting specification for structural steel is used. This should be avoided, if possible, because expansion joints are fabricated (manufactured) products; therefore procedures, as outlined for large monolithic units, cannot always be followed. This is especially true when manufacturing the more intricate and complicated multisealed or modular-type joints. For instance, because of the time it takes to fabricate and assemble component parts to make a composite unit, it is not always possible to paint the joint within a few hours of the grit-blasting operation.

Another point of concern is to make allowances for repairing galvanizing, paint, or other coatings when the assembly must be made in short lengths that can be safely and adequately handled and then spliced together for final assembly.

After specifying general material requirements, a short section on service expectations and performance would be applicable. Physical parameters could also be interjected at this point.

In the case of waterproof or sealed expansion joints, a field test consisting of flooding the joint and observing it over a brief period of time (1 hr) would be beneficial to determine if the initial installation is satisfactory.

Only prolonged field use should be considered a barometer by which performance and life expectancy may be judged. No amount of laboratory testing can guarantee that an expansion joint system will perform to expectations. There are too many interacting variables, which are independent of the quality of the system, that could affect the field performance of the system.

The known physical characteristics of materials, including their life expectancy under given conditions of stress, should enable the specifying engineer to evaluate any system proposed without requiring exotic, redundant, or unwarranted long-period testing procedures.

Known physical characteristics are understood to be those properties that have been adopted and accepted by current industrial standards, such as:

1. Specifying A36, A588, or grade 50 steel immediately connotes its yield point, tensile strength, chemical analysis, unit weight, and other well-defined parameters; and
2. Specifying neoprene rubber by suitable ASTM designation would immediately account for material indices such as tensile strength, elongation, durometer, and compression set.

Societies such as ASTM, American Concrete Institute, American Steel Construction Institute, and the like spend many years developing material specifications that can be easily (if need be) modified or tailored to one's needs.

Major points to cover under the construction section are items directed to either or both the contractor and manufacturer, such as special handling, unique field operations or techniques required for proper installation, and subsequent satisfactory performance.

This would involve other items of work such as tolerance for setting grades and acceptable deviations in placing sealers or other appurtenant components needed for installation.

Certainly structural steel tolerances should be accepted as standard, because in the manufacture of armored expansion joints, machining is not one of the operations, and the state in which steel is received by the manufacturer will affect the final overall dimensions and straightness of the joint. Small deviations in measurements and physical characteristics that do not affect the performance of either the sealer or joint system should also be permitted.

Regarding method of measurement, in most cases there is a definite advantage to using payment by lineal feet supplied versus a lump-sum arrangement. Most bidders can and often do give their lowest unit price if they can be certain that any changes in quantities furnished, as ultimately needed on the job site, will be paid for.

Although a specification may be well written, certain other data and information are needed to achieve the desired end results. These data are normally described in the engineering contract plans.

The well-known cliché, "a picture is worth a thousand words," is especially relevant to the subject under discussion. Intentions, ideas, and desires of the design engineer can be graphically incorporated in the contract plans, with sufficient notes to ensure the meaning of the specifications.

The contract plans should clearly indicate physical characteristics such as (a) anchorage system desired or minimums required; (b) typical sections of the joint, including slider plate assemblies and general treatment at pedestrian walk areas, without detailing every dimension to the nth degree; (c) blockout geometry, when used; (d) details and geometry of supporting structural members, where applicable; and (e) specific notes dealing with the joint system's manufacture or fabrication and installation or erection procedures, especially when relating to other required standards or codes.

The kind of information to be contained in the general notes on drawings, other than special installation instructions or restrictions, would be related to painting or galvanizing, field splicing of seals or metal members, class of steel or other metals, welding code requirements, material requirements for specific components not indicated elsewhere, and other sundry items to either emphasize or clarify the intent of the drawings and specifications.

There are two items most often omitted from the contract plans, but nonetheless important, that would be significantly beneficial to manufacturers and suppliers: the ambient temperature range and the anticipated or design joint movement. These data are necessary when bidding projects because products of manufacturers differ slightly in movement ratings, and a determination of which alternates would be acceptable may have to be made.

Certain details must be clearly dimensioned and delineated, such as the size and spacing of rebars, studs, gusset plates, or other relevant anchorage systems, as well as plate sizes for slider assemblies. These are items normally designed by the engineer and are generally independent of the type or make of expansion joint used.

Proprietary cross sections may be used to depict type and materials desired by the engineer for obtaining an end result; however, notes allowing for minor deviations in dimensions and in design configuration to other equivalents can and should be used so as to allow both the specifying agency, the contractor, and the manufacturer latitude so they can adapt the expansion joint that is ultimately selected and approved to the specific structure.

The use of overall dimensions, as applied to the expansion joint system, is considered necessary in order to furnish proper details for fabrication drawings. However, once again, the engineer is cautioned not to overdo the dimensioning when using a proprietary design. The product specified will, undoubtedly, conform to all the general dimensions, but any equivalent product will have minor variations in dimensions, which in most cases will not affect the true intent of the design.

Therefore, it is maintained that, for practical purposes, it is not necessary to show every detail of an expansion joint if it is proprietary, because extraneous information may tend to confuse the actual purpose of why it was shown in the first place.

It is necessary, however, to detail and annotate any item added to a proprietary joint system that would be expected to be furnished, regardless of who the ultimate supplier may be. This would include attachment brackets, structural shapes made a part of the system, and the like.

On the other hand, designs of nonproprietary expansion joints for manufacture or fabrication and installation by the general contractor must contain all necessary detail dimensions. In this case, the engineer is conveying information to the uninformed or nonspecialist. This same reasoning should equally be applied to those portions of proprietary expansion joints that are actually nonproprietary, such as rolled steel sections added for anchorages or slider plate assemblies and so forth. The bidder needs to know the sections required, the type and grade of steel, the thicknesses where appropriate, and basic design details of the unit or assembly.

The only method the engineer has of assuring a clear understanding of his contract documents is either the use of an example (naming a proprietary product) to denote quality or, in the instance of general drawings, showing enough detail for its manufacture or fabrication. For example, all weld sizes and lengths, specific material requirements, as well as exact sizes of all parts must be shown on nonproprietary components or expansion joints; and typical sections of proprietary expansion joints should be used where they will convey the engineering parameters desired. This will enable all bidders to evaluate the contract documents in a similar manner and conform to the same standard.

SUMMARY AND CONCLUSIONS

There is, perhaps, much more work needed in writing adequate and competitive specifications for expansion joint systems. The purpose of this paper is to bring to the design engineer's attention some of those aspects of contract documents that, if not properly handled, would either result in controversy or additional costs to the user.

When specification or drawings are not clearly understood, problems will arise that both engineers and suppliers do not want. The supplier wants to be able to bid his product and should be given the opportunity to do so, whenever possible, within the limitations of that product. The design engineer wants and should receive the quality he desires for the most economical costs obtainable.

In conclusion, specifying agencies should write specifications that clearly describe the desired expansion joint and should draft contract drawings that indicate graphically the inherent characteristics and physical requirements of the joint. The drawing should also show how the joint is to be placed, connected, and installed. In this way the specifying agency will relieve itself of much controversy, and at the same time they will obtain the right expansion joint at the right price through competitive bidding.

REFERENCE

1. Howard Needles Tammen and Bergendoff. Bridge Deck Joint-Sealing Systems: Evaluation and Performance Specification. NCHRP Report 204. TRB, National Research Council, Washington, D.C., June 1979, 46 pp.

Vertical Movement of Jointed Concrete Pavements

I. MINKARAH, J. P. COOK, and S. JAGHOORY

ABSTRACT

The vertical deflection of a concrete pavement under truck loading may be the determining factor in predicting the service life of the pavement. Consequently, it is of prime importance to know the effect of different variables on this vertical movement. To study experimentally the effects of the variables, a test pavement was constructed as part of US-23 in Chillicothe, Ohio. Data have been collected on this pavement continuously since 1972. To isolate the variables, the pavement was divided into 10 sections of approximately 10 joints each. Vertical measurements were taken by using a truck with a measured axle load. The measurements provided continuous plots of the vertical movements as the test truck traveled over the joint. Measurements were repeated at different speeds to determine the effect of truck speed on pavement deflection. Measurements were also repeated both morning and afternoon to study the effect of pavement curl. The measurements were analyzed statistically to determine the relative effects of the different variables on the behavior of the slab. The analysis indicated that there is a significant effect on slab behavior caused by difference in the subbase, location of the truck on the pavement, speed of the truck, and time of measurement (morning versus afternoon). Only a minor effect was noted due to spacing of joints, types of dowels, and a configuration of the saw cut.

The vertical movement of pavements is affected by wheel loadings and expansion and contraction caused by temperature and moisture changes. Portland cement concrete pavements are usually jointed to accommodate this movement. The results of uncontrolled pavement movement may be cracked slabs, pavement blow-ups, and bridges tilted or pushed out of skew.

Horizontal movements are usually assumed to be a sinusoidal variation of expansion and contraction, thus causing the joint to open and close. Of course, many other factors affect this movement. The vertical movement depends on both traffic loads and the curl of the pavement caused by temperature change.

STUDY OBJECTIVE

The objective of this research was to determine the actual magnitude of the vertical movements of the pavement. Because there are several factors that may affect movement, each factor was considered as a variable. The variables were then isolated to determine the effects of each. At the risk of "reinventing the wheel," even those assumptions that are commonly accepted as fact were challenged. The factors considered to be of prime importance were type of subbase, coating of dowel bars, joint spacing, configuration of the saw cut, and use of skewed joints. Combinations of these variables were incorporated into a test pavement, and were studied for a period of 8 years by actually measuring pavement movement (1-3).

TESTING PROGRAM

The test pavement is a section of the southbound lane of US-23 approximately 0.6 mile (1 km) long. The pavement is a tangent section on an easy grade. Truck loads are heavy, but the average daily truck traffic is not high.

The test section is reinforced concrete, 24 ft (7.3 m) wide and 9 in. (229 mm) thick. Most of the pavement is laid over a granular subbase, except for a 776-ft (237-m) section, which is laid over an asphalt-treated base. Spacing of the joints was set at 17, 21, and 40 ft (5.18, 6.4, and 12.2 m). The dowels used were standard steel dowels and plastic-coated dowels. The configuration of the joints also varied. There were 0.5-in. (12.7-mm) joints, 0.25-in. (6.4-mm) joints, and one set of joints with a beveled saw cut. Data about each of the variables are given in Table 1.

Instrumentation

Vertical movements were measured with a linear motion transducer and a strip-chart recorder. The

TABLE 1 Joint Groups

Group No.	Joint No.	Total No. of Joints	Type of Joint	Spacing (ft)	Type of Subbase	Type of Dowels	Remarks
1	1-7	7	0.125-in. bevel saw cut	40	Granular	Standard	
2	8-16	9	Standard 0.25-in. saw cut	40	Granular	Standard	Chlorinated rubber base cure
3	17-24	8	Standard 0.25-in. saw cut	21	Stabilized	Standard	
4	25-34	10	Standard 0.25-in. saw cut	40	Stabilized	Standard	
5	35-44	10	Standard 0.5-in. saw cut	17	Stabilized	No dowels	Right forward skew and plain pavement
6	45-53	9	Standard 0.25-in. saw cut	21	Granular	Plastic coated	
7	54-63	10	Standard 0.25-in. saw cut	40	Granular	Plastic coated	
8	64-73	10	0.5-in. sawed	40	Granular	Standard	
9	74-84	11	Standard 0.25-in. saw cut	40	Granular	Standard	
10	85-94	10	Standard 0.25 in. sawed	21	Granular	Standard	
	95-96	2	Standard 0.25 in. sawed	40	Granular	3M Coated	
	97-100	4	Standard 0.25 in. sawed	40	Granular	Standard	
	101	1	Expansion	40	Granular		

Note: 1 in. = 2.5 cm, 1 ft = 0.3 m.

transducer was an Edcliff 1500 ohm with a 1.75-in. (44.5-mm) stroke. The strip-chart recorder was a Sanborn Model B-1000 with variable amplification so that the size of the curve could be varied to suit the gradations on the chart paper for improved accuracy.

One joint in each group of 10 was selected for vertical measurements. Adjacent to the joint a small hole was dug down to the level of the base of the pavement. An 8-ft-long (2.4-m) section of No. 14 reinforcing bar was then driven vertically down into the subgrade until the top of the bar was flush with the level of the subgrade. This furnished a solid base for the movable plunger of the transducer. The body of the transducer was mounted on the side of the pavement slab. Figure 1 shows the mounting setup.

Measurements

Loads for the measurement program were supplied by the Ohio Department of Transportation (DOT). A loaded truck was weighed and then sent to the site. Because it was impossible to furnish exactly the same load for each set of measurements, all of the loads were reduced to a common base of 10,000 lb (4550 kg) before reducing any data.

For each set of measurements, the truck made three runs over the test joint at 55 mph (88 km/h) in the center of the lane, at 55 mph at the edge of the pavement (where the transducer was located), and at 10 mph (16 km/h) at the edge of the pavement.

Two sets of measurements were made on each test day. The first set was taken in the morning when the pavement was relatively cool and then repeated in the heat of the afternoon. Sets of measurements were taken during each of the four seasons of the year. Figures 2-7 are typical curves showing the vertical movement.

ANALYSIS OF DATA

The data obtained from the recorder show deflection of the joint as a function of time (i.e., the time it takes for the truck to pass over the joint). Figures 2-7 are typical time-deflection curves. The curves show two peaks, a small one corresponding to

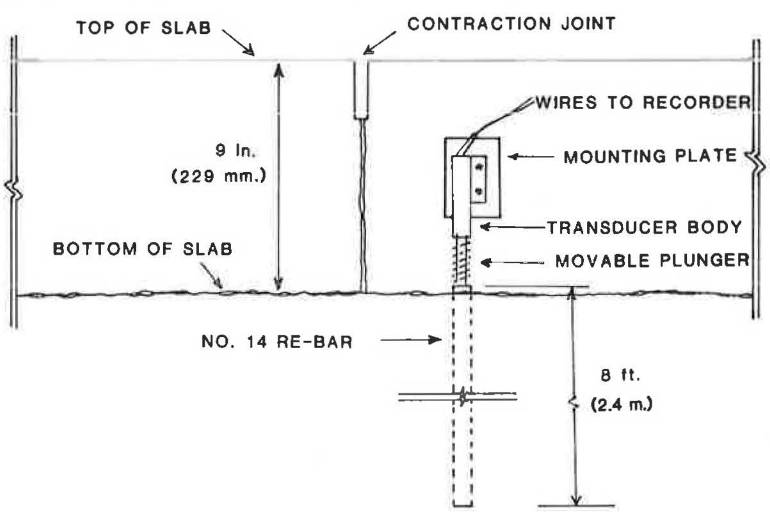

FIGURE 1 Mounting of transducer.

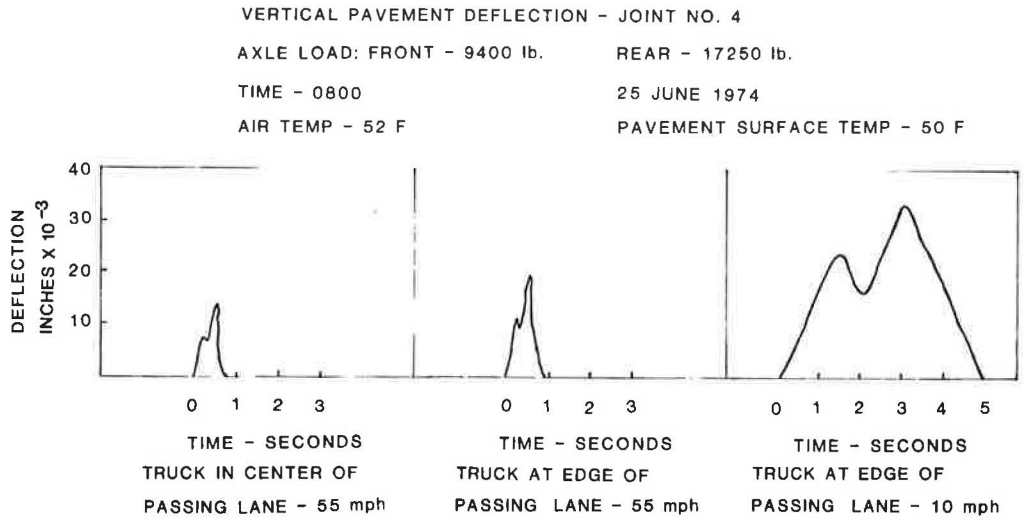

FIGURE 2 Vertical pavement deflection, joint 4, morning.

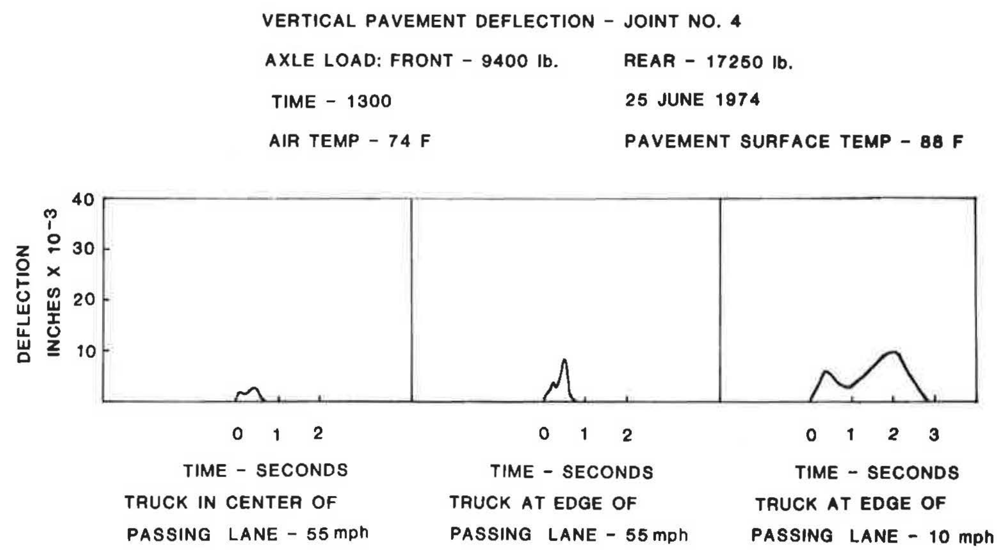

FIGURE 3 Vertical pavement deflection, joint 4, afternoon.

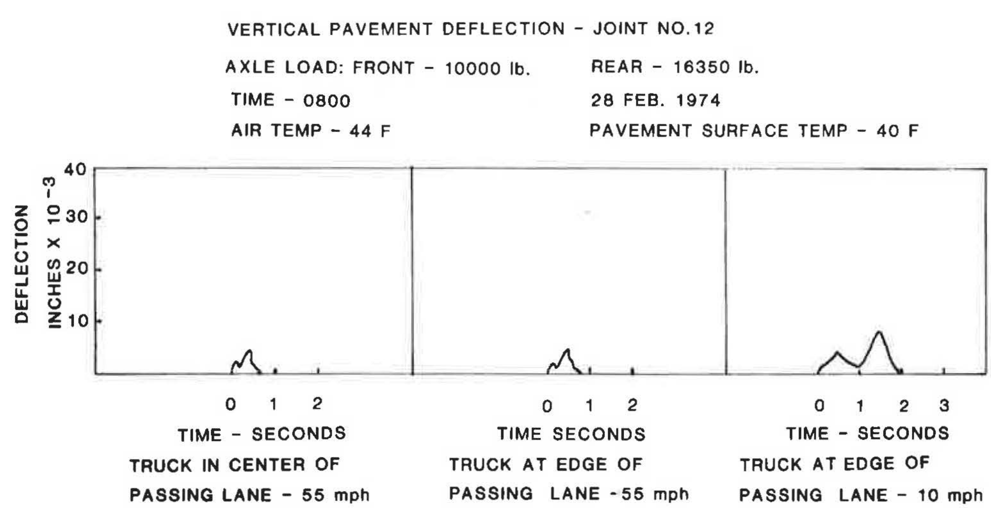

FIGURE 4 Vertical pavement deflection, joint 12, morning.

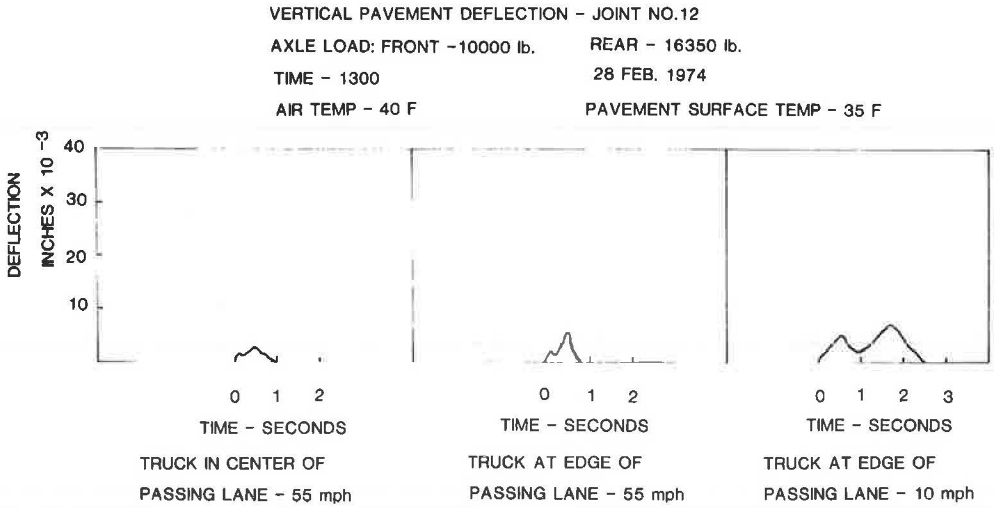

FIGURE 5 Vertical pavement deflection, joint 12, afternoon.

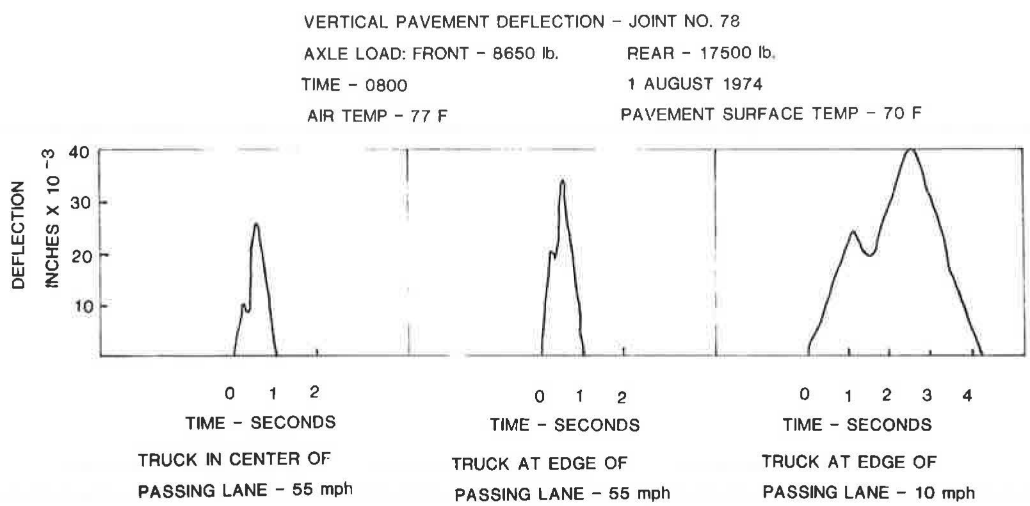

FIGURE 6 Vertical pavement deflection, joint 78, morning.

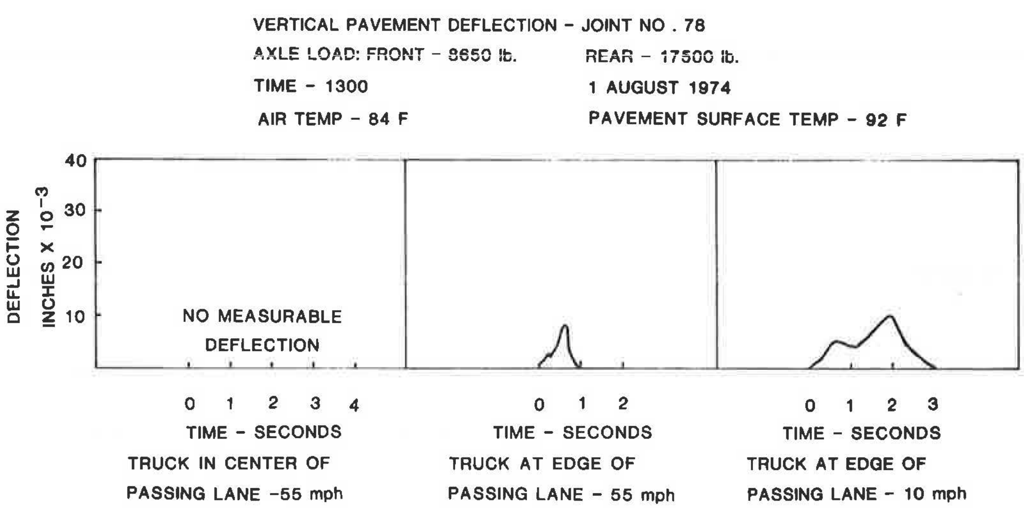

FIGURE 7 Vertical pavement deflection, joint 78, afternoon.

the front wheel passing over the joint and a larger one corresponding to the rear wheel passing over the joint. The axle weights for the truck differed slightly each time a set of readings was taken. Consequently, the measured deflection under both the front and rear axles was converted to the equivalent deflection that would be caused by a 10,000-lb (4550-kg) load. The conversion, of course, was linear. Thus the curves provided two points where both the load and the deflection are known. These points were used in the analysis.

Several variables affect the magnitude of the vertical deflections. Some variables were incorporated into the pavement in such a way that each one could be isolated by comparing two groups that are identical except for the variable under study. This comparison was then made by using a standard two sample t test. A normal distribution could not be used in this case because the sample size corresponding to one variable at a time was relatively small. The null hypothesis ($\mu 1 - \mu 2$) = 0 was tested at the level of significance $\alpha = 0.05$. The hypothesis is rejected if for (n1 + n2 - 2) degrees of freedom, t (calculated) < $-t\ \alpha/2$ or > $t\ \alpha/2$. Rejection of the hypothesis means that there is a significant difference between the pavement sections being compared. That is, the variable under study does affect the behavior of the pavement.

The effect of the location of the truck in the traffic lane and its speed became apparent from the beginning of the study. The deflection measured at the edge of the pavement with the truck in the center of the lane traveling at 55 mph (88 km/h) was always small, regardless of the season or the time of day (see Figures 2-7). Sometimes the deflection of the pavement was so small that it was not measurable. This usually occurred when the mid-slab temperature was greater than 80°F (26.5°C). However, there were cases when the movement measured 0.03 in. (0.76 mm). These movements are for a 10-kip load in the center of the lane, with the truck traveling at 55 mph.

The effect of truck speed can be observed from a comparison of the movement corresponding to a truck at the edge of the lane traveling at 55 mph to the same truck at the edge of the lane traveling at 10 mph. Figures 2-7 show that the movement is always larger for the slower speed. The means and standard deviations, converted to the equivalent 10,000-lb load, are as follows:

Truck at Edge of Lane at	X Bar	Sigma
10 mph	8.46×10^{-3} in.	8.33×10^{-3} in.
55 mph	5.98×10^{-3} in.	6.91×10^{-3} in.

The remaining variables studied were spacing of joints, type of dowels, type of subbase, time of measurement (morning versus afternoon), and temperature and seasonal effects.

Spacing of Joints

Joints in the test section were spaced at 21 ft (6.4 m), 40 ft (12.2 m), and one section at 17 ft (5.2 m) with a right forward skew. The statistical analysis indicates that joint spacing does not significantly affect the vertical deflection of the pavement. This can be seen from the data in Table 2 by comparing Group 3 to Group 4, Group 6 to Group 7, or Group 9 to Group 10. In each of these instances the isolated variable is slab length. In all cases, t (measured) is less than t ($\alpha/2$), in which α = 5 percent. This might not appear too unusual, because the weight of the slab tends to neutralize part of the lift-off due to curl of the pavement caused by temperature.

Type of Dowels

Two types of dowels were used in the project: standard steel and plastic coated. The main function of the plastic coating is, of course, corrosion control. Again, the analysis indicated no significant difference between the two types. Both types were functioning well up to the end of the project in 1981. A comparison of Group 6 to Group 10 and Group 4 to Group 7 showed no significant effect due to the dowels.

It did come as a surprise that there was no significant difference between the means of Group 3 and Group 5. The variable in this instance is dowel versus no dowel. Of course, aggregate interlock is compensating for the dowels. Unfortunately, only one section of the pavement was left without dowels, so there is not enough information to draw a relevant conclusion.

Type of Subbase

Two types of subbases were incorporated into the pavement: granular and stabilized. The analysis indicates that there is a significant difference in the vertical movements due to the subbase. This can be seen from a comparison of Group 4 to Group 9, Group 3 to Group 10, Group 3 to Group 6, and Group 4 to Group 7. In all these cases, t (measured) is greater than t ($\alpha/2$). In comparing Group 3 to Group 6 and Group 4 to Group 7, the dowels are also different, but if we accept the conclusion that the effect of dowels is not significant, then the difference is due to the subbase.

The vertical deflections of sections on stabilized bases were consistently smaller than those on granular bases. This apparently is one of the reasons why stabilized bases are superior to granular bases in controlling pumping and faulting of joints. The means and standard deviations of vertical movement of joints on granular and stabilized bases are

TABLE 2 Comparison of Means of Maximum Deflections Based on a 10-Kip Load

Comparison Group	Comparison Variable	Degrees of Freedom	$t_{Calc.}$	$t_\alpha = 0.025$	$t_\alpha = 0.05$	Remarks
3 to 4	Slab length	11	-0.110	2.201	1.796	Accept hypothesis
3 to 6	Type of dowel, type of base	11	4.146	2.201	1.796	Reject hypothesis
3 to 10	Type of base	11	2.079	2.201	1.796	Accept hypothesis
3 to 5	Dowels versus no dowels	11	1.220	2.201	1.796	Accept hypothesis
4 to 5	Type of dowels, slab length	11	-1.228	2.201	1.796	Accept hypothesis
4 to 7	Type of dowel, type of base	11	-3.00	2.201	1.796	Reject hypothesis
4 to 9	Type of base	11	-3.06	2.201	1.796	Reject hypothesis
6 to 7	Slab length	11	1.373	2.201	1.796	Accept hypothesis
6 to 10	Type of dowels	11	2.183	2.201	1.796	Accept hypothesis
7 to 9	Type of dowels	11	-0.180	2.201	1.796	Accept hypothesis
9 to 10	Slab length	11	1.279	2.201	1.796	Accept hypothesis

as follows. The measurement again corresponds to a 10-kip axle load with the truck traveling at 10 mph at edge of the pavement:

Type of Base	Mean of Maximum Deflections	Standard Deviation
Granular	10.28×10^{-3} in.	10.45×10^{-3} in.
Stabilized	5.71×10^{-3} in.	5.60×10^{-3} in.

Temperature and Seasonal Effects

Vertical measurements were taken in the morning and repeated in the afternoon. The means of the maximum movements of the joints were compared. The results revealed a significant difference between the two means for most groups, with the morning deflections larger than the afternoon deflections (see Table 3). This is to be expected because the surface temperatures of the pavement are cooler in the morning than in the afternoon, thus affecting the shape of the slab and the position of the edge of the pavement with respect to the base. The sections that did not fit this pattern were those that had the short spans, that is, Group 3 and Group 10 with 21-ft (6.4-m) spans and Group 5 with a 17-ft (5.2-m) span and no dowels. This may be because the daily change in the shape of the slab for the shorter spans is not as pronounced.

TABLE 3 Comparison of Means of Maximum Deflection at 8:00 a.m. and 1:00 p.m.

Group	Degrees of Freedom[a]	$t_{Calc.}$	$t_{\alpha} = 0.025$
1 and 2	9	2.95	2.262
3	6	1.52	2.447
4	5	2.79	2.571
5	6	2.28	2.447
6	6	4.52	2.447
7	7	2.45	2.365
8 and 9	9	3.19	2.262
10	6	0.568	2.447

Note: Truck at the edge of the pavement at 10 mph (16 km/h), corrected for an equivalent load of 10 kips.

[a]Number of pairs minus one.

The surface temperature of the slab was measured at the same time as the vertical deflections. An attempt to correlate surface temperature and deflection was unsuccessful. Figure 8 is a typical plot of vertical deflection versus surface temperature. It is obvious from the plot that there is no direct relationship. Vertical deflection is a function of the shape of the slab, which depends on both top and bottom temperature of the pavement. The vertical movement is a function of the temperature gradient across the depth of the slab.

To further study the effect of temperature on the shape of the pavement, continuous measurements were taken simultaneously at three locations across the slab depth (i.e., top, middle, and bottom). These measurements were taken in both the spring and fall. Figures 9-12 show hourly temperature variation across the depth of the slab for the period of measurement. Although the data do not represent a large enough sample to have strict statistical validity, enough data are given to indicate definite patterns of variation across the slab.

1. The bottom slab temperature is seasonal and changes gradually.
2. The surface temperature fluctuates during the day, as expected.
3. The mid-slab temperature also varies a great deal and does not necessarily fall in between the top and bottom temperatures. Sometimes it is higher than both.
4. The temperature at the surface and the middle of the slab peaks early in the afternoon when the sun's rays strike the slab at more or less a normal angle. The temperature at the bottom of the slab reaches its peak value sometime in the early evening. All three temperatures hit their low points in the early morning.
5. In the spring the top and middle temperatures were within the bounds of the maximum and minimum air temperatures, whereas the bottom temperature

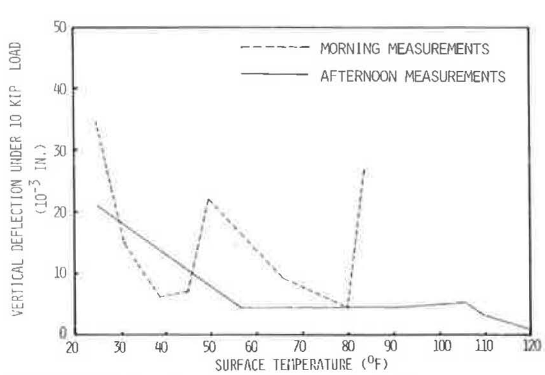

FIGURE 8 Vertical deflection versus surface temperature, group 6.

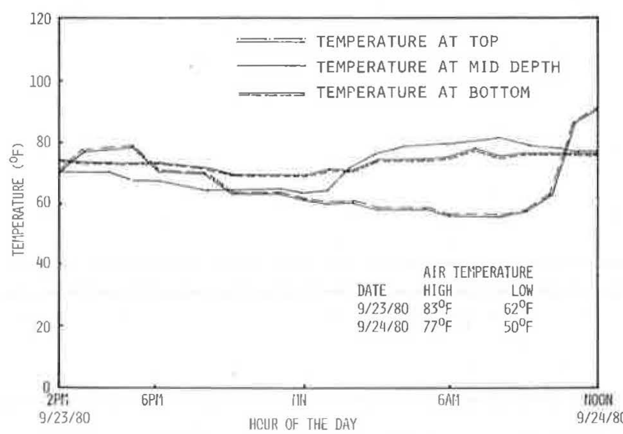

FIGURE 9 Hourly temperature variation across depth of pavement on September 23 and 24, 1980.

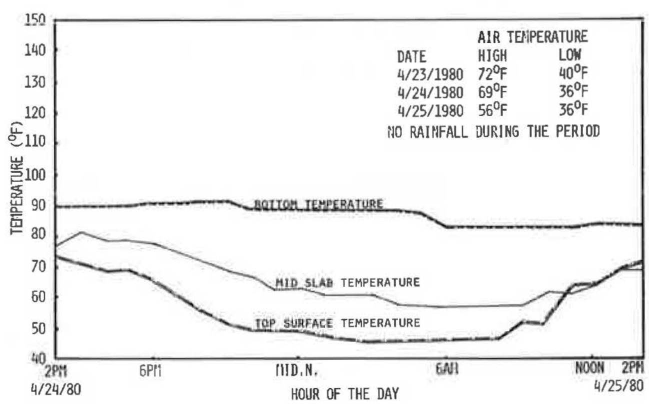

FIGURE 10 Hourly temperature variation across depth of pavement on April 24 and 25, 1980.

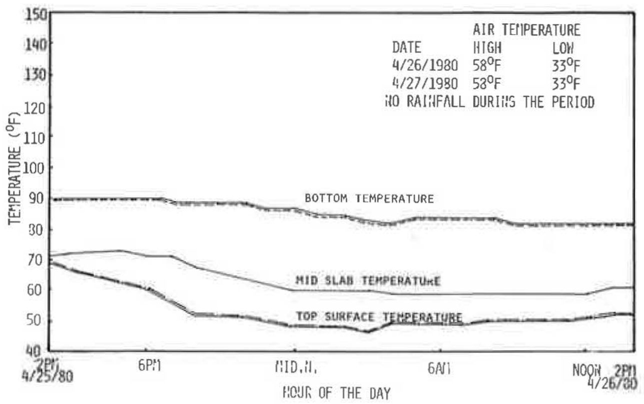

FIGURE 11 Hourly temperature variation across depth of pavement on April 25 and 26, 1980.

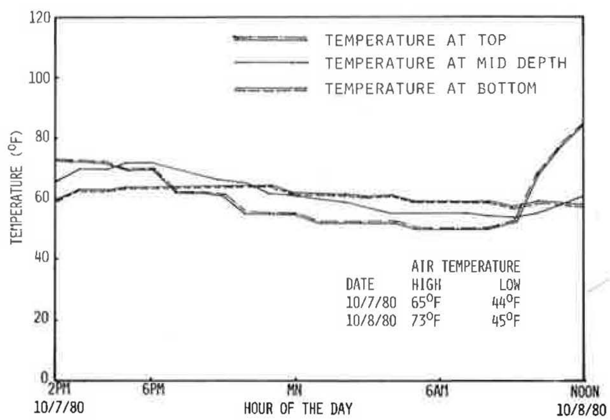

FIGURE 12 Hourly temperature variation across depth of pavement on October 7 and 8, 1980.

remained steady and unexpectedly high. In the fall all the measured slab temperatures remained within the bounds of the maximum and minimum air temperature range.

6. Surprisingly, the temperature of the top of the slab seldom exceeds the bottom temperature. Consequently, it would appear that during the summer the slab tends to remain concave, rather than changing shape from concave to convex during the day.

7. In the fall the pavement shape changed during the day. It was generally concave during the morning rush hour and convex during the afternoon rush hour. In one instance (October 8-9, 1980), the top of the slab was warmer than the bottom or almost equal to the bottom during almost the entire recording period. This indicates that the slab also may remain convex for extended periods of time.

It is apparent that it is not possible to know the shape of the slab from only one factor (i.e., air temperature, surface temperature, or time of day). Slab shape is affected by long-term as well as short-term changes in the temperature pattern. Seasonal changes affect bottom temperature more, whereas daily fluctuations affect surface temperature more. The curl, which depends on the differential between top and bottom temperature, has the largest effect on the vertical movement. Therefore, more data are needed, not only to determine the magnitude of the movement, but also to determine when this maximum movement will occur. Smaller movements in heavy traffic are more critical than large movements in lighter traffic.

CONCLUSIONS AND RECOMMENDATIONS

Analysis of the data indicate that the following factors have a significant effect on vertical pavement deflections: difference in the subbase, location of the truck on the pavement, speed of the truck, and time of measurement (i.e., morning versus afternoon).

The effect of the subbase came as no surprise. It makes sense that a stabilized base should provide more support for the pavement and be less susceptible to compaction over the years.

The effect of truck location was also expected. Because the transducer was mounted on the edge of the pavement, it stands to reason that the deflection would be more pronounced when the truck was closer to the measuring device.

It is often assumed that high-speed truck traffic is one of the major factors responsible for pavement deterioration. This study confirms recent results showing exactly the reverse. Low-speed traffic causes the greater deflection. Apparently, the truck moving over the joint at high speed simply does not give the pavement time to deflect.

Time of measurement must be studied further. In this work deflections were found to be greater in the morning than in the afternoon as a general rule. However, measurements of top, middle, and bottom slab temperatures indicate that the pavement remains concave for most of the day during the spring. In the fall, winter, and summer the temperatures would indicate that the shape of the pavement is changing during the day. This needs further study.

Spacing of joints, type of dowels, and configuration of the saw cut had only a minor effect on permanent deflection. It was expected that the configuration of the saw cut would have little effect on deflection. The same may be said for the type of dowels. However, the fact that joint spacing had little effect on deflections was somewhat unexpected. Skew joints also show virtually no effect on deflections when compared with normal joints.

All of the magnitudes of the movements appear small, if the absolute values are considered. However, it should be remembered that fatigue failure can be a major consideration. Fatigue, by definition, is repetitive loading below the yield stress of the material, and it does not take too long for the average daily truck traffic to build up to several million cycles, which would cause failure.

ACKNOWLEDGMENT

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Rigid Pavement Joint Resealing: Field Application, State of the Art

JOHN W. BUGLER

ABSTRACT

In the 1970s the New York State Department of Transportation initiated and executed field performance studies of formed-in-place sealant for future use in a statewide joint resealing program. It was determined after 3 years of service that of the six formed-in-place sealants tested, hot-poured polyvinyl chloride, conforming to ASTM D3406, performed best. A joint resealing program was initiated in Region 10 (Nassau and Suffolk counties) in 1979. There were initial field application problems. The problems are described, and the solutions used are explained.

Until 1958 New York State constructed concrete pavements with transverse expansion joints generally spaced every 100 ft. At that time the state amended the specifications to include the use of contraction joints spaced every 60 ft 10 in. The width of the joint was 0.375 in. and remained so until 1968.

Liquid formed-in-place sealants were in use until 1963, at which time the specifications were amended to require the exclusive use of 0.8125-in. (uncompressed width) preformed compression seals.

BACKGROUND

Performance of 0.8125-in. Preformed Compression Seals

The service life of the 0.8125-in. preformed compression seals was from 2 to 3 years (1). An explanation for the seal having such a short service life is as follows (2):

> Past experience had shown that due to slab contraction, transverse joints might open an additional 3/8-in. In other words, joints might be as wide as 3/4-in. during cold periods in winter. State specifications require preformed sealers to be 13/16-in. wide--1/16-in. wider than the anticipated maximum joint opening. This was in an effort to ensure that pressure against the joint faces would be maintained throughout the winter months. To consistently construct transverse joints exactly 3/8-in. wide was, of course, difficult if not impossible. Many joints were constructed slightly wider or narrower. Joints wider than 3/8-in. sometimes opened beyond 13/16-in. during winter, and thus the sealer was not in compression. When joints were too narrow, it was difficult to install the preformed sealer without stretching it. Also, in narrow joints it sometimes was subjected to more compressive stress than it was designed to withstand.

In March 1968 the specifications were amended to increase the joint width to 0.625 in. The uncompressed width of the preformed sealer was increased to 1.25 in.

This was an improvement, in that the 1.25-in. seal had to recover only 80 percent of its uncompressed width to be able to effectively seal the joint in the dead of winter, whereas the 0.8125-in. seal had to recover 92 percent.

Performance of 1.25-in. Preformed Compression Seals

After 7 years of service, 65 percent of the seals examined in the field were found to have taken a compression set of 0.375 in. (3). Also 51 percent of the joints examined were found to have moderate bottom-of-joint infiltration (3).

FIELD RESEARCH

For the purpose of effectively resealing pavement joints as the need arose, a field study involving the application and performance evaluation of six different formed-in-place sealers was initiated by the New York State Department of Transportation (NYSDOT) (1).

A research report by Bryden et al. (1) stated that "polyvinyl chloride coal tar performed best of any liquid sealer, and the material itself is in excellent condition after three winters."

Maintenance Resealing Program

Regarding the limits on the effective service life of preformed compression seals, NYSDOT advised all regions to initiate condition surveys in the sixth year of service; and as the need arose, they should initiate a maintenance joint resealing program (4).

Results of Regional Survey

The survey of preformed compression seals was conducted in the tenth year of their service. It was found that, although the seals appeared to be doing their job at moderate temperatures, a high percentage of those examined in winter were not sealing the joints.

Initiation

Following department guidelines (4), a small joint resealing contract (5,000 linear feet) was executed by using liquid polyvinyl chloride coal tar, which conformed to ASTM D3406. This first joint resealing contract was actually supplementary to a larger rehabilitation contract, and department personnel believed that it should limit the quantity of materials used, because there was limited experience with formed-in-place sealers.

In writing the original specification, an intensive literature search was performed and correspondence was made with both industry and other jurisdictions, all of which resulted in an inclusive specification. There were, however, some field application problems.

FIELD APPLICATION PROBLEMS AND SOLUTIONS

Problem: Joint Overfilling

The first maintenance joint resealing contract experienced problems with joint overfilling. The specification called for sealing the joint to a level no higher than 0.25 in. from the road profile. More than 50 percent of the joints sealed failed by an unacceptable margin to meet that design criterion.

The solution was as follows. The industry was contacted about the problem (Posh Chemical, Inc., of Port Washington, New York). The manufacturer responded by designing and manufacturing a new applicator wand (Figure 1, left).

The cutoff valve on the original applicator wand was located 4 ft from the discharge tip (Figure 1, right). This made it difficult for the operator to judge when to close it as he approached the end of his pass. He would invariably overfill the last 2 ft of the joint.

The operator had the problem of having to hold the wand up over the joint as he made his pass. It was difficult to keep the elevation of the wand tip

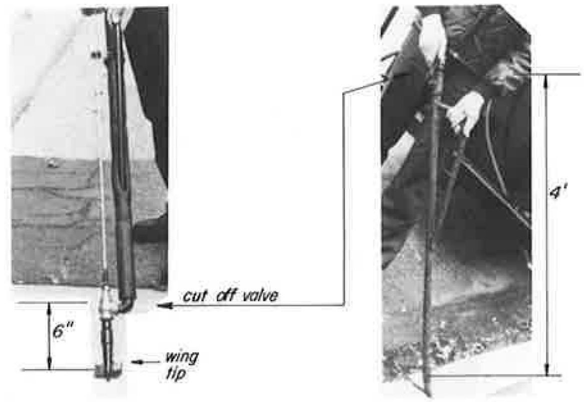

FIGURE 1 Applicator wand: new (left) and original (right).

constant. These problems were eliminated with the introduction of the new applicator wand:

1. The cut-off valve was located 6 in. from the discharge tip, and
2. The applicator wand discharge tip was redesigned to include a set of wings (Figures 2-4), thereby allowing the operator to glide the applicator wand along the joint as he made his pass.

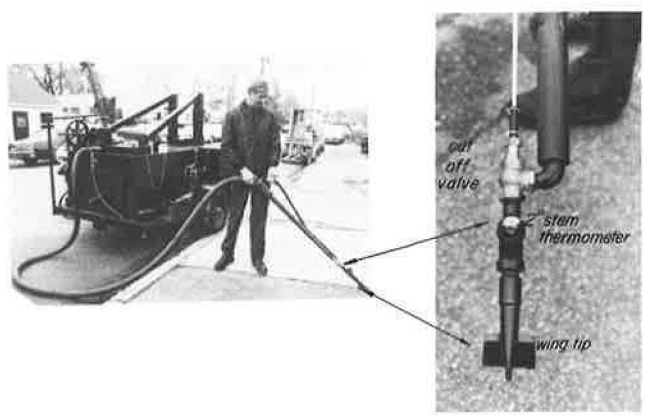

FIGURE 2 Posh field extruder with insulated hose and insulated applicator wand.

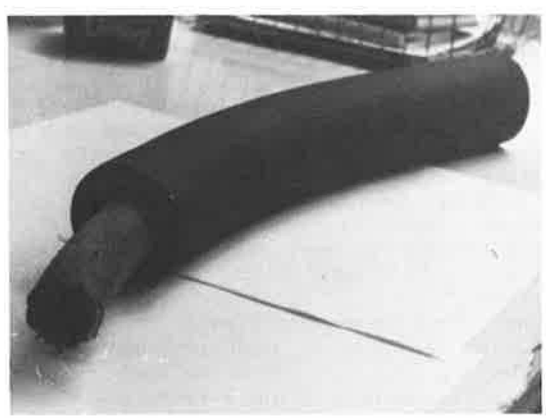

FIGURE 3 Section of insulated hose.

FIGURE 4 Wing tip.

These improvements gave one additional benefit: time. The time needed to seal a 12-ft joint was reduced by one-half.

Since introduction of this new applicator, Region 10 has resealed an additional 300,000 linear feet of contraction joints, and no additional problems with joint reservoir overfilling have been noted.

Problem: Incomplete Sandblasting of Joint Face, Leading to Intermittent Bond Adhesion Failure

The first large maintenance resealing contract (92,000 linear feet) executed in Region 10 experienced problems. These were 2- to 4-in. bond adhesion failures for the full depth of the seal.

Representatives from the industry and the Materials Bureau of NYSDOT were called. It was the conclusion of all concerned that the heart of the problem was improper and incomplete joint sandblasting.

Field investigation of another joint resealing project in progress revealed faulty sandblasting. Some of the joint faces examined after sandblasting were found to have less than 50 percent of the joint face thoroughly clean.

Many contractors have little experience in sandblasting highway pavement joints. Their approach and methods are more applicable to plane surfaces (i.e., bridge decks, structural steel).

Figure 5 shows a method used by an out-of-state contractor with some success. Other jurisdictions (Iowa and Pennsylvania) were contacted and they confirmed that they also had experienced similar problems with sandblasting pavement joints.

The original specification for sandblasting was as follows: "Both faces of the joint shall be throughly cleaned by sandblasting or high pressure water blaster, to a depth of the bottom of the proposed sealer."

The specification was modified by adding the following: "The sandblast or high pressure waterblast joint cleaning operation shall be such that when completed the concrete joint surface which is to receive the new joint sealant shall be free of all constituents of the lubricant adhesive used to place the original preformed compressive seals; all tar and asphalt; all discoloration and stain; as well as any and all other forms of contamination, leaving a clean, newly exposed concrete surface."

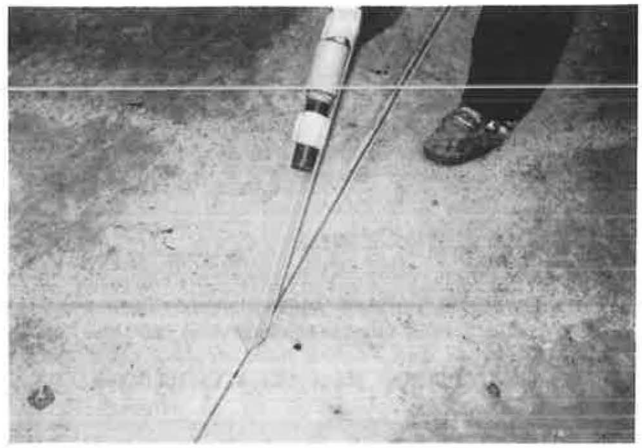

FIGURE 5 Sandblasting equipment.

This upgrading was done to preclude such inquiries as, "What do you mean by clean?"

Field inspection forces were advised to assign one inspector to oversee the sandblast operation at all times. Previously, one inspector was used to cover both the sandblast and the joint sealing operations.

Sandblast operators have been observed holding the nozzle several feet above the joint and walking the length of the joint, moving the nozzle from left to right as they walked. On the return pass they would airblast the joint. To the undiscerning it would appear that they executed two separate sandblast passes; such was not the case.

To properly sandblast the joint face it is necessary that the operator hold the sandblast nozzle very close to the pavement. This is unpleasant but necessary.

There is also the problem of the joint seal apparatus catching up with the sandblast operation in about the fourth hour of operation. This is because the joint seal operation is 4 to 6 times faster than the sandblast operation (using one sandblast operator).

Therefore, it is recommended that contractors consider using high-capacity compressor-sandblast units; thus they would be able to operate two or more sandblast units simultaneously. It is also recommended that contractors execute their sandblast operations far ahead of the joint seal operation, so as to preclude their coming together before the day's end. Finally, it is recommended that there should be correspondence with the sandblast equipment industry, urging them to consider development of a sandblast nozzle more applicable to the needs of pavement joint sandblasting.

Inspectors were advised to use their clipboards or other similar device to cast a shadow on the pavement surface near the joint reservoir when inspecting the quality of the sandblasting. This was necessary because sunlight reflecting off the pavement surface will close the eye's pupil, such that it will be difficult to see into the joint reservoir with any discernment.

Some inspectors have reported that, by using a Sear's "inspection mirror" (similar to a dental mirror), they are able to successfully expedite the inspection of the joint reservoir after sandblasting.

Problem: Some Joints Were Not Sandblasted at All

The problem of some joints not being sandblasted is unique to joint faces that had their pore structure impregnated with constituents of the lubricant adhesive used in the placing of the original preformed compression seals.

With the passing of time the preformed compression seals take a compression set. Often in such cases the seal slips down into the joint reservoir, leaving the top 0.5 in. of the joint face exposed (Figure 6). With the passing of time the exposed surface weathers and, as a result, appears to be clean. It is not sandblast clean. It is, however, clean enough to give the appearance of having been sandblasted. Field investigation has revealed instances of cursory joint sandblasting, such that the area under the exposed 0.5 in. had not been touched by the sandblast. Looking down from a standing position, however, the joint appears to have been sandblasted. This apparent condition is reinforced by the fact that the sandblast operator invariably leaves his signature (sandblast abrasions) on the pavement area around the pavement joint.

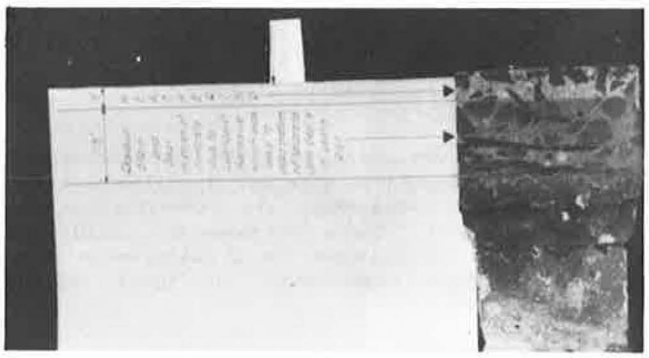

FIGURE 6 Seal slippage.

The constituents of the lubricant adhesive are such that bonding of the formed-in-place sealant to the joint face is impossible. The problem is compounded by the fact that unless the inspector physically inspects the joint (close up), he will fail to discern the problem.

The solution to this problem is closer inspection.

Proper sandblasting is the most critical part of the joint sealing operation. The failures in the field were bond adhesion failures, basically because of improper sandblasting. According to Tons (5), "if there is a true bond between the sealant and the concrete, the sealant should fail in cohesion rather than in adhesion."

Another point of consideration is the joint face surface area. A sandblasted joint face has a much greater surface area for bonding when compared with a sawcut joint face without sandblasting. A sandblasted joint face enables a properly constituted formed-in-place sealant to achieve a significant increase in net bonding force at the joint face. Also, by doing this, the performance of the sealant is optimized during periods of extension.

Problem: Failure to Maintain the Design Shape Factor

The shape factor (depth-to-width ratio) of formed-in-place sealants has a decided effect on the amount of tensile stress induced into the sealant during periods of extension (6). The amount of strain imposed on the extreme fiber of the sealant is largely determined by the shape factor (6).

Adhesion failure occurs when the tensile stress in the sealant exceeds the bonding force exerted at the concrete joint face, thus causing the sealant to pull away from the joint face. Figure 7 shows the strain imposed on the extreme fiber of a sealant (being extended 0.5 in.) at different joint design shape factors.

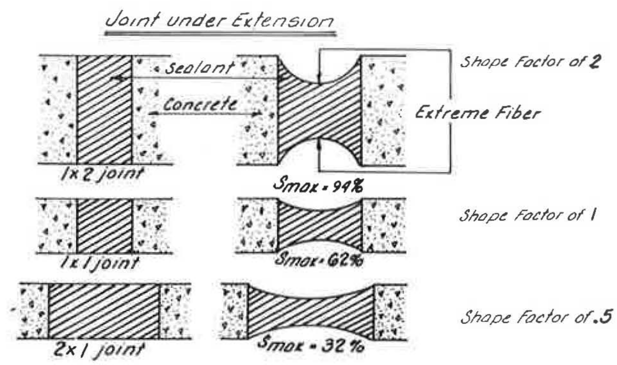

FIGURE 7 Strain on extreme fiber of sealant for a 0.5-in. extension of different joint designs (shape factors).

A joint with a shape factor of 2, when extended 0.5 in., will increase the length of the extreme fiber by 94 percent of its original length (7). A joint with a shape factor of 1, when extended 0.5 in., will increase the length of the extreme fiber by 62 percent of its original length (7). By simply reducing the depth of the sealant to 1 in., the strain is decreased by 50 percent.

There are advantages in keeping the strain concentration to a minimum. Therefore the specifications call for a deformable bondbreaker to be inserted in the joint reservoir, thus creating a formed joint geometry that will keep stress concentrations within the performance limits of the sealant. However, if insufficient compression is exerted on the deformable bondbreaker, the sealant will make its way around it, resulting in a depth-to-width ratio outside the limits of design.

Figure 8 (left) shows a deformable 1-in.-diameter bondbreaker placed inside a 1-in.-wide joint reservoir. There is virtually no compressive force being exerted on the bondbreaker. Field inspection revealed that the sealant had passed around the periphery, resulting in a depth-to-width ratio outside the limits of design.

Figure 8 (right) also shows a 1-in.-diameter bondbreaker. However, this time it is inserted into a 0.75-in.-wide joint reservoir. Therefore, it is in compression. Field inspection revealed the sealant to be contained within the limits of design.

The solution to this problem is closer inspec-

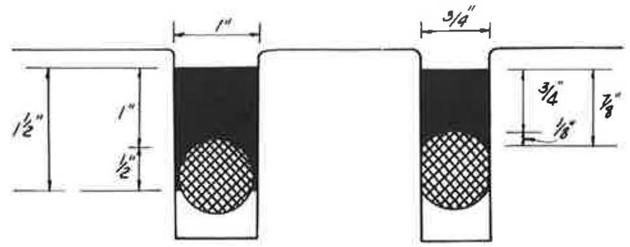

FIGURE 8 Bondbusters.

tion. Inspectors were advised to check the diameter of the stitched cotton piping cord used in the joint and compare it with the width of the joint. They were also advised that the diameter of the cord should be approximately 25 percent greater than the width of the joint.

Because joint widths vary due to moisture, temperature, and degree of infiltrated incompressibilities, it may be necessary to have on hand cord, the diameter of which is not readily available; thus the contractor may have to make a special order. The readily available cord diameters are 0.375, 0.5, 0.625, 1, and 2 in. Special orders take 3 weeks to execute. Figures 9 and 10 show how the cord is inserted and how it should look when it is in place.

FIGURE 9 Placing stitched cotton piping cord.

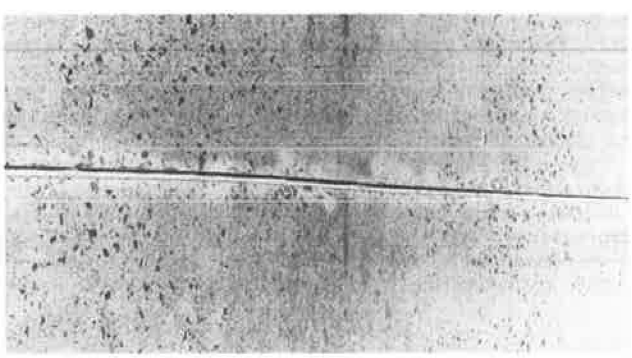

FIGURE 10 Cord in place.

Problem: Heat Losses Endemic to the System

Unless equipment operators are experienced (and very often they are not), they will find it difficult to achieve the recommended pouring temperature at the start of the work day. To preclude this condition, the specification has been amended to include the following: "At the start of the day's operations special procedures may be necessary in order to achieve a sealant temperature consistent with specification. The contractor shall ascertain from the manufacturer of the apparatus he is using the procedures necessary and be able to so execute these procedures prior to his commencement of joint sealing operations."

Problem: Heat Losses Endemic to the Hoses and Applicator Wand

The hoses are usually 15 ft long, and the applicator wand is 4 ft long. The heat losses from the hoses and the wand are significant if they are not insulated. In the field heat losses of 20°F, at ambient temperatures near 70°F, have been experienced.

The problem was solved by amending the specification to read: "The hoses and the applicator wand shall at all times be insulated. The material and method of insulation shall be in compliance with the recommendations of the joint seal apparatus manufacturer and meet with the approval of the Engineer. The material and methods shall be submitted to the Engineer for his approval, two weeks prior to the commencement of joint sealing operations."

Problem: Joint Seal Apparatus Thermometers Out of Calibration

On occasion it was noted that the thermometers of the joint seal apparatus were out of calibration by as much as 25°F. Therefore the specification was amended to read: "These thermometers...shall be turned over to the Engineer for a calibration check two weeks before commencement of joint sealing operations."

Problem: Oil Residue

One final problem noted was failure to purge the flush oil residue (remaining from the previous day's flushing of the system on completion of work) from the hoses and applicator wand at the start of the work day. Therefore the specification was amended to read: "The first gallon of material to flow out of the applicator wand at the start of the day shall be considered spoil and as such be discarded into a container so designated."

IMPORTANCE OF TEMPERATURE OF MATERIAL AT TIME OF PLACEMENT

Barksdale and Hicks (8), working with ASTM D3406 at a time when the industry recommended a pouring temperature of 250°F, reported that "polyvinyl chloride...when poured at a temperature 40°F above the recommended pouring temperature, was found to perform significantly better (in bond adhesion) than specimens poured at the recommended pouring temperature."

The industry has since amended its recommended pouring temperature to the 290°F range. The reason for the significant improvement in bond adhesion is because the surface tension of the material is significantly lower at the temperature range of 290°F, and as such it better penetrates the pore structure of the concrete joint face.

CONCLUSION

Formed-in-place sealants conforming to ASTM D3406, when applied in conformance with the specifications, have performed satisfactorily. With the exception of one section on the first contract in Region 10, there have been virtually no failures in bond adhesion, material cohesion, or extrusion. It would appear that rigorous inspection with regard to field application is the key to successful performance.

(Editor's note: A rigorous specification is currently in force and is working well in Region 10 in New York State. A copy of the specifications is available from the author.)

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Materials and Methods for Sealing Cracks in Asphalt Concrete Pavements

JIM CHEHOVITS and MARK MANNING

ABSTRACT

In recent years significant advances in both materials and methods for sealing cracks in asphalt concrete pavements have been made. Crack sealing has been transformed from a poorly performing and many times ineffective fill-in type of maintenance task to a viable and cost-effective preventive maintenance technique that can extend the life expectancy of roadways. Many aspects of the maintenance technique of sealing cracks in asphalt concrete pavements are examined herein. The subject is covered by qualitatively examining the cracking mechanism and the consequences of not maintaining adequately sealed cracks. The influences of climatic conditions and traffic on crack formation and subsequent movements are discussed. Physical characteristics of sealant materials required for application and acceptable performance are discussed, as well as testing methods for determining these characteristics. Physical properties and specification conformance of materials that are currently used as crack sealants are presented. Advantages and disadvantages of two basic types of sealant application configurations are discussed along with equipment and application methods that are used in crack sealing.

Asphalt concrete highways comprise approximately 1.9 million miles or 93 percent of the surfaced roadways in the United States, with portland cement concrete roadways comprising the remaining 7 percent (1). Asphalt concrete roadways range in type from low traffic volume seal-coated roads and subdivision streets to high traffic volume full-depth asphalt concrete Interstate highways. The majority of asphalt concrete roadways are at least several years old and are exhibiting cracking of varying types and extents.

Crack sealing in asphalt concrete pavements is thought by many to be an ineffective, low-priority pavement maintenance task that is performed only after other pavement maintenance activities such as overlays, seal coats, and fog seals are completed, and only if time, budgets, and manpower are sufficient. Because of this belief, cracks in many miles of highways are not sealed each year, which results in accelerating pavement deterioration because of intrusion of moisture and increased oxidation of the pavement binder.

Significant strides in both sealing materials and techniques for sealing cracks have been made during the past decade. Crack sealing is no longer a slow and ineffective task. With currently available materials and equipment, crack sealing has advanced from being a low-priority maintenance task to being a viable and effective preventive maintenance technique that can greatly increase the life expectancy of asphalt concrete highways.

Many aspects of crack sealing technology for asphalt concrete pavements are presented and discussed in this paper. The subject is covered by examining crack formation, types, movements, and growth. Properties of sealant materials, testing methods, and specifications are summarized. In addition, sealant application techniques and equipment used for sealing cracks are discussed.

CRACKING IN ASPHALT CONCRETE PAVEMENTS

Crack Formation

Asphalt concrete pavement systems are typically composed of a compacted subgrade, a granular base course, and an asphalt concrete surfacing layer. In contrast to rigid portland cement concrete pavements, asphalt pavements are designed as flexible systems that can deform without cracking when subjected to vehicle loadings, contraction and expansion due to thermal effects, or subgrade movements and changes in volume (2). Various factors can influence the degree of flexibility of asphalt pavements, including ambient temperature, aggregate characteristics, asphalt cement stiffness, temperature susceptibility of the asphalt cement, asphalt content of the mixture, and degree of compaction. Each of these factors can have a significant effect on pavement stiffness and flexibility. However, asphalt cement stiffness and temperature susceptibility characteristics are of special interest with respect to pavement cracking.

Recent research has indicated that the viscosity of asphalt cement in service can increase by as much as ten- to fifty-fold in 4 years because of aging effects from oxidation (3). The rate and magnitude of aging is related to many factors, including degree of mixture compaction, source of asphalt cement, aggregate absorptive characteristics and climate (3). With very old pavements, recovered asphalt cement viscosities at 140°F of 500,000 poise and greater are common, which indicate that viscosity can increase because of long-term in-service aging by as much as 125 times (assuming an initial viscosity at 140°F of 4,000 poise).

Asphalt concrete pavements that contain asphalt cement that has aged significantly are not as flexible as when originally constructed because of the increased stiffness of the asphalt cement. The increased pavement stiffness results in a pavement that has a lessened ability to redistribute stresses caused by thermal deformation or other loading effects. Cracking then occurs when the pavement is subjected to heavy traffic loadings, cold temperatures, rapid temperature decreases, or subgrade movements.

Considerable research on the influences of asphalt concrete mixture and component properties on crack formation has been performed (4-10). The majority of these studies has determined that several mixture properties can influence crack formation; however, properties of the asphalt cement have been found to have the most significant effect. In general, stiffer grade asphalt cements result in increased cracking at low temperatures. In addition, asphalt cements with high degrees of temperature susceptibility, as indicated by penetration index (PI) or pen-vis number (PVN), have been found to have a greater cracking potential than asphalts with lower degrees of temperature susceptibility.

Cracking Types and Occurrences

Several different types of cracking may occur in asphalt concrete pavements as the pavement ages. Most cracking can be classified as either temperature or fatigue related (2,11). Reflective cracking of underlying cracks through newly constructed asphalt concrete overlays is another common type of cracking.

Thermal Cracking

Thermal-related cracking appears as both transverse and longitudinal cracks and results from the inability of the asphalt concrete to redistribute horizontal tensile stresses that develop along the length and width of the pavement as ambient temperature decreases. In properly designed and constructed pavements, transverse cracking, which extends the full pavement width and at large spacings (greater than 100 ft), is usually the first type of cracking to occur. As the pavement ages and the asphalt cement stiffens, transverse cracks appear at lesser spacings and may be present in old pavements at spacings of less than 10 ft (12).

Longitudinal thermal-related cracking occurs when the pavement stiffness is such that thermally induced stresses in the transverse direction cannot be adequately distributed by the pavement. Cracking generally appears as a single crack near the center of the pavement width for two-lane pavements, or at a spacing of approximately 10 to 15 ft for wider pavements. Thermal-related transverse cracking tends to appear in most pavements within 1 to 3 years, whereas longitudinal cracking begins at a somewhat later age.

Fatigue Cracking

Fatigue or alligator cracking is generally caused by the inability of the pavement to redistribute stresses resulting from vertical deformations caused by traffic loadings or base or subgrade failure. Fatigue cracks generally appear in a rather close block-type pattern spaced at between 4 and 12 in. Cracking is many times localized and present where poor drainage, inadequate base thickness, or poor compaction of the base or subgrade was attained. Fatigue cracking may also be prevalent in heavily traveled vehicle wheelpaths.

In properly designed and constructed pavements, fatigue cracking appears in wheelpaths as the pavement nears the end of its design life. However, fatigue-type cracking can occur within a short time following pavement construction in areas in which construction deficiencies or overloading occur. Following periods of wet weather, fatigue cracking can also appear adjacent to open thermal cracks because of the weakening effect of surface moisture intrusion into base and subgrade layers through the open cracks.

Reflective Cracking

Reflective cracking in asphalt concrete overlays is caused by transference of horizontal or vertical movements of discontinuities in underlying pavement materials into a localized area of the overlay (13). Cracking then results when the ability of the asphalt concrete overlay to adjust to these movements is exceeded. Typical cases of reflective cracking at overlays include

1. Cracking above joints in portland cement concrete due to localized horizontal movement induced by thermal expansion and contraction of the concrete at the joint (reflective cracking may also result from vertical movements of faulted slabs at joints);
2. Cracking above cracks in portland cement concrete pavements due to horizontal or differential vertical movement of the slab sections;
3. Cracking above transverse and longitudinal thermal cracks in asphalt concrete pavements due to localized horizontal movement resulting from thermal effects or localized differential vertical faulting-type movements resulting from vehicle loadings; and
4. Cracking above fatigued areas in asphalt concrete pavements due to localized differential vertical movements at the fatigued area.

Reflective cracking of asphalt concrete overlays and its prevention are major concerns when resurfacing old pavements because reflective cracking can greatly reduce the useful life of overlays.

Crack Movements

Cracks in asphalt concrete pavements can experience movements in both horizontal and vertical directions (2). As previously discussed, horizontal movements result from thermal expansion and contraction of the pavement. Horizontal movements of transverse cracks in a full-depth asphalt concrete pavement of as much as 0.4 in. have been observed in Kansas from summer to winter (note that data are from correspondence with Glenn Koontz, materials engineer, August 10, 1983). Larger movements are common in colder areas. These thermally induced movements mainly occur on a seasonal rather than a daily basis (14). Even though air temperatures may change significantly during a 24-hr period, pavement temperature variations are small because of the heat-retention effects of the base and subgrade below the pavement surface. On a seasonal basis, however, the subgrade and therefore the total pavement system experience significant temperature changes. In general, cracks are open their widest in the winter and are narrowest in the summer.

The magnitude of horizontal, thermally induced movement at cracks is dependent on the spacing between cracks in much the same way as thermal movements at joints in portland cement concrete pavements depend on joint spacing (14). In general, cracks that are widely spaced may experience greater horizontal movement than closely spaced cracks. Another observation is that transverse thermal cracks may not experience uniform movement or, in other words, some thermal cracks may experience large movements while adjacent cracks move only slightly. Thermal cracks generally experience greater horizontal movement than fatigue cracks because of their greater spacing. Reflective cracks in an asphalt concrete overlay on a jointed portland cement concrete pavement will tend to move horizontally with seasonal temperature variations in a manner similar to the underlying joint.

Differential vertical movement at cracks may result from moving vehicle loadings because the crack creates a discontinuous pavement surface structure that can have reduced load transfer capabilities. The reduction in load transfer capacity depends on the width of the crack. Load transfer will be the least when the crack is widest (in the winter) and greatest when the crack is narrowest (in the summer). Reduction or loss of load transfer capacity in a pavement at a crack may result in differential vertical movements when loaded. If underlying base and subgrade layers are saturated with moisture, differential vertical movements may be greater because of the weakening effect of moisture.

Crack Growth

In some cases cracks have been observed to grow or widen with time (data from Glenn Koontz). This is especially true with thermal transverse and longitudinal cracks. Crack growth is hypothesized to occur as a result of three possible mechanisms.

First, when a hairline crack (in the summer) opens during the winter because of thermal contraction, it can become partially filled with debris. When the pavement warms, expansion is restricted by the debris and expansive stresses are relieved through either flow and yielding of the warmed asphalt concrete, warping of the pavement, or spalling at the crack. The crack that was originally hairline or very narrow may then be as wide as 0.5 in. During the second winter the pavement contracts, which again widens the crack. The open crack then is blocked by additional noncompressibles that once again restrict pavement expansion in warm weather, resulting in an even wider crack the following summer. This cycling continues each year and the crack widens by approximately the amount of pavement contraction that occurs at the crack each year.

The second possible widening mechanism comes from the observation that cracks that are not blocked with noncompressibles may not completely return to their original summer width after a winter contraction cycle (data from Glenn Koontz). A possible explanation is that during the pavement expansion cycles the pavement is warmed, which significantly reduces its stiffness. During the expansive cycle, frictional restraining forces between the asphalt concrete and the base may be of sufficient magnitude to cause yielding and viscous flow in the asphalt concrete so that the crack does not completely return to its original width.

The third possible widening mechanism is that of noncompressibles being plowed into the bottom of the crack from the base or subgrade when the crack closes as the pavement warms and expands. Many times, especially when milling asphalt pavements during recycling operations, it is noted that cracks may widen toward the bottom of the asphalt concrete layer, which may support this hypothesis. Crack widening at the bottom of the pavement may also be due to stripping in the asphalt concrete caused by water entry at the crack.

Consequences of Inadequately Sealed Cracks

When cracks in asphalt concrete pavements are not maintained and are left unsealed, deterioration of the pavement immediately adjacent to the cracks is hastened. There are three ways that deterioration can occur. First, the faces of the cracks are exposed to the environment, and the binder at the exposed crack faces begins to oxidize and harden more quickly than if the pavement was not cracked. The increased stiffening of the binder then can result in raveling and further deterioration of the asphalt concrete at the crack face, thus widening the crack.

The second major type of deterioration caused by inadequately sealed cracks consists of water entry through the crack into the base and subgrade of the pavement. The presence of excess water in the pavement base or subgrade tends to reduce both the compressive and shear strengths of these structural layers in areas immediately below and adjacent to the crack. Because the base and subgrade are weakened in the presence of excess moisture, deflections of the pavement surface may increase when loaded, thus promoting further cracking and deterioration (11). If water entry continues, eventually fatigue-type cracking and potholes may occur.

The third type of deterioration, which can occur when cracks are not adequately sealed, is entrance of noncompressibles into the crack, which restricts crack closure during warm weather, as previously discussed. The noncompressibles may also cause compressive stresses at the crack faces, resulting in spalling and loosening of the asphalt concrete. In some cases, with highly oxidized pavements, the stresses resulting from noncompressibles are relieved by heaving of the pavement near the crack, resulting in bumps.

An additional effect of improperly sealed cracks is that crack growth is not controlled or restricted due to the entrance of noncompressibles, as previously discussed.

When cracks in asphalt concrete pavements are adequately sealed and maintained, crack growth is lessened because of the rejection of surface noncompressibles, and pavement life is increased because of minimizing deterioration caused by entrance of surface water into the underlying base and subgrade.

SEALANT MATERIALS

Required Properties for Acceptable Performance

For a material to perform adequately as an asphalt concrete crack sealant, it must have sufficient flexibility throughout the range of temperature encountered in service to remain bonded to the crack faces. The general requirements of ASTM D3405, "Joint Sealants, Hot-Poured, for Concrete and Asphalt Pavements" (15), are typical of those required for adequate performance of a crack sealant as follows:

> The joint sealant shall be composed of a mixture of materials that will form a resilient and adhesive compound capable of effectively sealing joints and cracks in concrete and asphaltic pavements against the infiltration of moisture and foreign material throughout repeated cycles of expansion and contraction with temperature changes, and that will not, at ambient temperatures, flow from the joint or be picked up by vehicle tires. The material shall be capable of being brought to a uniform pouring consistency suitable for completely filling the joints without inclusion of large air holes or discontinuities and without damage to the material. It shall remain relatively unchanged in application characteristics for at least six hours at the recommended pouring temperature in the field.

These general requirements may be separated into nine specific characteristics that are important in roadway sealants:

1. Ability to be easily and properly placed in a crack through application equipment,
2. Adequate adhesion to remain bonded to the asphalt concrete crack faces,
3. Adequate resistance to softening and flow at high in-service pavement temperatures so that the sealant will not flow from the crack and therefore prevent tracking,
4. Adequate flexibility and extensibility to remain bonded to crack faces when extended at low in-service temperatures,
5. Sufficient elasticity to restrict the entrance of noncompressible materials into the crack,
6. Sufficient pot life at application temperatures for application of the total amount of prepared material,
7. Resistance to degradation from weather to ensure long in-service life of the sealant,
8. Compatibility with asphalt concrete, and
9. Low cure time to permit opening to traffic as soon as possible after application.

Testing Methods

Many testing methods that are applicable to sealants for cracks in asphalt concrete pavements are contained in testing specifications for portland cement concrete joint sealant materials. Testing specifications for concrete joint sealant materials contained in the ASTM standards (15) that are applicable are

ASTM Specification No.	Title
D1191	Standard Methods of Testing Concrete Joint Sealers
D3407	Standard Methods of Testing Joint Sealants, Hot-Poured, for Concrete and Asphalt Pavements
D3408	Standard Methods of Testing Joint Sealants, Hot-Poured, Elastomeric-Type for Portland Cement Concrete Pavements

In addition, several other standard and nonstandard tests can be used to determine crack sealant properties. A list of properties that should be determined for asphalt concrete crack sealants and testing methods that can be used to determine these properties is given in Table 1.

The following is a brief discussion of testing applicability and the mechanics of performing testing for determining various sealant properties.

Application Characteristics

The application temperature viscosity of a sealant can be determined effectively by using a Brookfield viscometer and testing at the application temperature. Sealants with an application temperature viscosity less than approximately 70 poise are self-leveling when applied, generally pump easily, and can penetrate cracks less than 0.375 in. wide. Sealants with viscosities at application temperatures greater than approximately 70 poise are generally not self-leveling and may not penetrate narrow cracks.

TABLE 1 Testing Methods to Determine Sealant Properties

Property	Testing Methods	ASTM Specification No.
Application characteristics	Brookfield viscosity at application temperature	D3236
Adhesion	Low temperature bond	D1191, D3407, D3408
High temperature softening resistance	Flow Softening point	D1191, D3407 D36, D2398
Flexibility and extensibility at low temperatures	Low temperature bond Mandrel bend Ductility at 39.2°F	D1191, D3407, D3408 —[a] D113
Elasticity	Resilience	D3407, D3408
Pot life	Extended heating	D3407
Weathering resistance	Weatherometer	G23, G53
Cure time	—[b]	—[b]
Compatibility	Compatibility test	D3407

[a]A nonstandardized test used by several state agencies.
[b]A general characteristic of the type of material being used. Hot-pour materials generally cure when cooled to ambient pavement temperature (1 hr), whereas cold-pour solvent-based and emulsified materials take longer and may require several days to several weeks to cure to a nontracking condition.

Adhesion

Adhesion, as well as low temperature flexibility characteristics of sealants, may be determined by using the bond test as specified in ASTM D1191, D3407, or D3408. The test consists of pouring sealant between two concrete blocks, trimming the excess, extending the blocks at 0.125 in. per hour a specified distance at low temperature, and then allowing the blocks to return to their original spacing at ambient temperature. This extension and return sequence is then repeated a specified number of times. To pass the test, there must not be any adhesive or cohesive failure greater than 0.25 in. deep at the sealant block interface or within the sealant. The bond test in ASTM D1191 uses 1-in.-thick specimens that are extended 50 percent at 0°F for five cycles, whereas ASTM D3407 specifies 0.5-in.-thick specimens extended 50 percent at -20°F for three cycles, which is a more difficult test than D1191.

High Temperature Softening Resistance

The resistance of sealant to softening at high in-service pavement temperatures needs to be determined to guard against possible tracking by vehicles and flow of the sealant from the crack. Surface temperatures of asphalt concrete pavement can commonly be as high as 150° to 190°F on bright sunny summer days. The flow test, in accordance with ASTM D1191 or D3407, can aid in determining high temperature softening resistance. The test consists of casting a 3.2-mm-thick x 40-mm x 60-mm sample of the sealant on a tin plate, which is then placed in an oven at 140°F on a 75-degree angle for 5 hr. The amount of sag of the specimen during this time is measured in millimeters. Paving grade asphalt cements will generally flow in excess of 50 mm within 1 hr, whereas acceptable crack sealants flow much less. The maximum flow for ASTM D1190 sealant is 5 mm in 5 hr, and for D3405 sealant it is 3 mm in 5 hr.

The ring and ball softening point test (ASTM D36 or D2398) can also be used as an indication of high temperature softening resistance. To resist in-service tracking, the sealant softening point should be at least 20°F higher than the maximum expected pavement surface temperature.

Low Temperature Flexibility and Extensibility

The flexibility and extensibility of sealant materials at low service temperatures may be determined in conjunction with adhesive characteristics by using the low temperature bond test in accordance with ASTM D1191, D3407, or D3408, as previously discussed. A simple nonstandard test that can be used to determine low temperature flexibility characteristics is the mandrel bend test. The test consists of casting a 0.125-in. thick x 1-in. wide x 4-in. long sample of the sealant, conditioning at a specified low temperature, and then bending over a 1-in.-diameter mandrel 90 degrees at a uniform rate in 10 sec. A passing test is one in which the sample does not crack. Common testing temperatures used by several specifying agencies are 0° and 10°F. Ductility at 1 cm/min at 39.2°F also provides an indication of the degree of low temperature extensibility of a sealant material.

Elasticity

The elastic characteristics of a sealant may be determined by using the resilience test as specified in ASTM D3407 or D3408. The test is conducted by using a 6-oz. tin sample of sealant and a standard penetrometer with a ball penetration device instead of the penetration needle. Basically the test measures the amount that the ball penetration device rebounds after being forced into the sealant sample; it is expressed as a percentage. Pavement grade asphalt cement generally is nonelastic in this test and has a resilience between 0 and -50 percent, whereas concrete joint sealants may have resiliences as high as 90 percent. Sealants with resiliences in excess of approximately 40 to 50 percent generally have adequate ability to reject noncompressible materials from the sealed crack. Sealants with high degrees of elasticity, as indicated by high resilience values, tend to be relatively strong materials that may be prone to pull-off types of failures when poor bonding conditions exist.

Pot Life

Sealant materials must have sufficient pot life during application to permit acceptable preparation and application of the entire batch of prepared material. For hot-pour-type sealants, pot life may be determined by using extended heating (for example, 6 to 8 hr) at application temperatures in an indirect heated melting unit and testing the material before and after the extended heating period to determine changes in physical characteristics caused by the heating period.

Resistance to Weathering

Weathering resistance of sealant may be qualitatively evaluated by testing in a carbon arc or ultraviolet-type weatherometer unit (ASTM G23 or G53). The sealant material is evaluated visually and qualitatively after exposure for hardening, embrittlement, cracking, shrinkage, blistering, and so forth.

Compatibility with Asphalt Concrete

Sealant materials for use in asphalt concrete must be compatible with the asphalt concrete. Compatibility can be determined by using the asphalt compatibility test specified in ASTM D3407. This test consists of sawing a 0.5-in.-wide by 0.75-in.-deep groove along the diameter of a compacted asphalt concrete specimen (Marshall or Hveem), filling the groove with sealant, and then placing it in an oven at 140°F for 72 hr. After removing from the oven, the specimen is examined for incompatibilities, which may consist of bubbling, blistering, or formation of an oil-like exudate. Asphalt-based sealants are generally compatible with asphalt concrete, whereas tar-based sealants may not be.

Available Sealant Materials

A wide range of materials with varying properties are currently used to seal cracks in asphalt concrete pavements. The majority of these materials can be grouped into three basic classifications based on their physical characteristics and degree of temperature susceptibility modification: unmodified asphalts, asphalt-rubber, and polymer-modified asphalt.

A listing of several typical physical properties of an unmodified asphalt that meets requirements of ASTM D3405 is given in Table 2. Cone penetration data from Table 2 as a function of temperature are plotted in Figure 1. From Figure 1, differences in the slopes of the plots, which indicate different temperature susceptibilities, can be noted. In addition, from Table 2 differences in resilience and thus elasticity of the materials are noted.

Unmodified Asphalt

This classification includes various grades of asphalt cement, emulsified asphalts, cutback asphalts, and asphalts that contain various types of mineral or fibrous fillers. Common specifications for these materials are as follows:

Material	Specifications
Asphalt cement	ASTM D3381, D946, D312 AASHTO M226, M20
Emulsified asphalts	ASTM D977, D2397 AASHTO M140, M208
Cutback asphalts	ASTM D2027, D2028 AASHTO M82, M81
Filled asphalts	Various state highway department specifications

As a class, unmodified asphalts have a high degree of temperature susceptibility. At low pavement service temperatures (approximately 0°F), unmodified asphalts are very stiff and brittle, whereas at high pavement service temperatures (approximately 140° to 160°F), they are very soft and

TABLE 2 Typical Physical Properties of AC-10 Asphalt, Asphalt Rubber, and Polymer-Modified Asphalt

Property	Test Specification	AC-10 Asphalt Cement	Asphalt-Rubber	ASTM D3405 Polymer-Modified Asphalt
Cone penetration, 150 g, 5 sec (dmm)				
0°F		9	14	20
77°F	ASTM D1191	130	60	75
140°F		>300	220	130
Resilience, 77°F (%)	ASTM D3407	−30	40	70
Flow, 140°F, 5 hr (mm)	ASTM D3407	>50[a]	10	1
Softening point (°F)	ASTM D36	115	170	190
Bond, 0°F, 1 in., 50% extension	ASTM D1191	Fail	Pass	Pass
Bond, −20°F, 0.5 in., 50% extension	ASTM D3407	Fail	Fail	Pass

[a]In 1 hr.

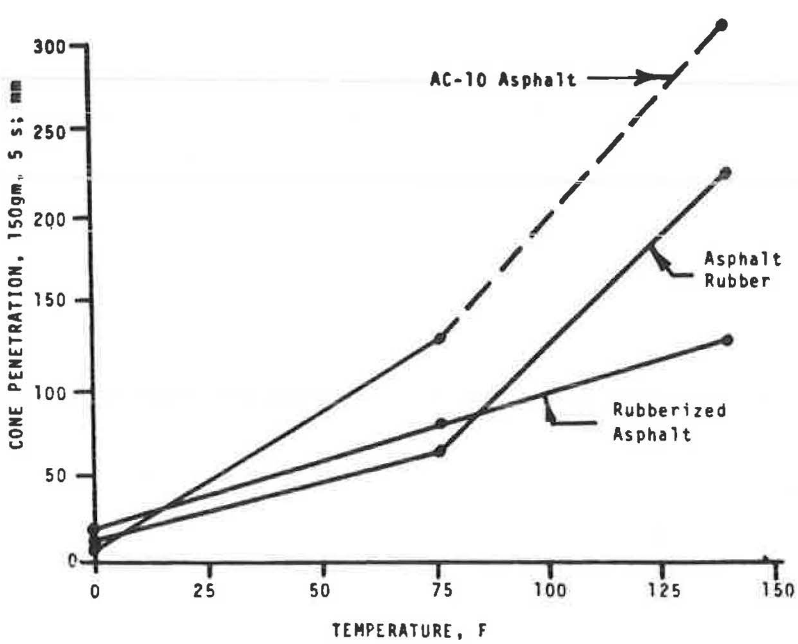

FIGURE 1 Cone penetration for sealant materials at 0°, 77°, and 140°F.

semifluid. In addition, unmodified asphalts have little or even negative resilience values, as indicated in Table 2. Generally, the useful life as a sealant of an unmodified asphalt is less than 1 year.

Unmodified asphalts are the least expensive type of material currently in use, costing approximately \$0.08 to \$0.15 per pound. Because of their high degree of temperature susceptibility when compared with other sealant materials, unmodified asphalt sealants tend to experience tracking in warm weather and may crack easily in cold weather. Because of their low degree of elasticity, unmodified asphalts will permit penetration of noncompressible materials into sealed cracks.

Asphalt-Rubber

Asphalt-rubber is a mixture of paving grade asphalt cement and between 15 and 30 percent granulated reclaimed crumb rubber particles. When the asphalt and rubber are heated to approximately 350°F, a reaction between the two occurs. The rubber particles absorb fractions of the asphalt, which results in swelling, and the rubber may partially dissolve in the asphalt (16,17). The degree of reaction is dependent on the physical and chemical characteristics of the asphalt and rubber as well as reaction temperature and time period (18). Reacted asphalt-rubber has radically different properties than the base asphalt cement or unreacted blends of asphalt and rubber. The reacted asphalt-rubber has a much higher viscosity and greater elasticity than the unreacted material and also has a lower degree of temperature susceptibility, as evidenced by greater high temperature stiffness and lesser low temperature brittleness (this can be noted in Figure 1). The asphalt-rubber reaction has been studied extensively and reported in the literature (16-18). In addition, much effort has been placed in studying properties in the laboratory of reacted asphalt-rubber materials (16-19). In most of this work, however, asphalt-rubber was studied for use in stress-absorbing membranes, interlayers, and waterproofing membranes, and not as a crack sealant material.

Specifications currently in use for asphalt-rubber sealants generally specify the grade(s) of asphalt cement that may be used, the percent and type of rubber, and the gradation of rubber. Several agencies specify additional requirements at low and high in-service temperatures, which provide an indication of the degree of temperature susceptibility modification achieved. These additional requirements may be a mandrel bend test at low temperature, ring and ball softening point, and 39.2°F ductility. Also, the Asphalt Rubber Producers Group has developed a guide specification for asphalt-rubber sealant materials (20).

As a class, asphalt-rubber sealants have improved temperature susceptibility characteristics and higher elasticity than the unmodified asphalt sealants. Properly formulated asphalt-rubber sealants can provide an effective and lasting seal for many types of cracks in asphalt concrete pavements in all but the coldest of climates. Working transverse thermal cracks in cold climates, which are sealed with asphalt-rubber, may separate when the pavement contracts in the winter. Asphalt-rubber sealants currently in use are more expensive than unmodified asphalt sealants and cost approximately \$0.20 to \$0.30 per pound.

Polymer-Modified Asphalts

Polymer-modified asphalt hot-poured sealant materials are compounded with asphalt cements, plasticizers, and various types of polymers and other ingredients to provide sealant materials with a high degree of temperature susceptibility modification, and thus greatly improved performance when compared with unmodified asphalt sealants. Polymer-modified asphalt sealant materials can be formulated to be capable of high degrees of extension at low service temperatures, while having softening points in excess of 200°F, which will minimize tracking in even extremely hot climates. In addition, polymer modification can impart high degrees of elasticity if desired. These materials are commonly used as joint sealants in portland cement concrete pavements; however, they can perform extremely well as crack sealants in asphalt concrete when appropriately installed.

Various standard concrete joint sealant specifications are currently used to specify these types of materials and include ASTM D1190, D3405 and AASHTO M173. In addition, several modifications to these specifications are used by various state agencies to provide improved performance. It is important to note when specifying asphalt crack sealing materials using concrete joint sealant specifications, that the sealant material must be compatible with asphalt concrete, as indicated by the ASTM D3407 compatibility test procedure. The physical requirements for various polymer-modified asphalt sealant specifications are given in Table 3. From the limits in Table 3 and the test data in Table 2, improvements in properties can be noted for polymer-modified asphalt as compared with unmodified asphalts and asphalt-rubber sealants.

Polymer-modified asphalt sealant materials are excellent long-lasting crack sealing materials in nearly all climates and conditions. The cost of these materials varies widely (from approximately \$0.30 to \$0.70 per pound), depending on the specific type of material.

TABLE 3 Polymer-Modified Asphalt Specification Requirements

Property	ASTM D1190, AASHTO M173	State-Modified M173	ASTM D3405	State Low-Modulus D3405
Cone penetration (dmm)				
77°F	90[a]	50-90	90[a]	110-150
0°F	—	—	—	40[b]
Flow, 140°F (mm)	5[a]	5,10[a]	3[a]	3[a]
Resilience, 77°F (%)	—	25[b]	60[b]	60[b]
Bond	0°F, 50%, 5 cycles	0°F, 100%, 5 cycles	−20°F, 50%, 3 cycles	−20°F, 100%, 3 cycles
Ductility, 77°F (cm)	—	35, 40, 50[b]	—	35[b]
Prolonged heating (hr)	—	6	6	6
Tensile adhesion (%)				
77°F	—	—	—	600[b]
−20°F	—	—	—	300[b]

[a]Maximum.
[b]Minimum.

SEALANT AND USE APPLICATION

Proper use and application of sealing materials to cracks in asphalt concrete pavements is essential for optimum performance and maximum life of the seal. Factors that need to be considered when sealing cracks include seal geometry, expected crack movement, crack cleaning techniques and equipment, and sealant application techniques and equipment.

Seal Geometry

Two basic seal geometries can be used when sealing cracks. The first is commonly called an overband or band-aid configuration, and the second is called a sealant reservoir. Each of these geometries has inherent advantages and disadvantages in different sealing situations.

Band-Aid Configuration

The band-aid type of sealant configuration consists of applying a 3- to 4-in. width of sealant approximately 0.125 to 0.25 in. in thickness on top of the crack on the cleaned pavement surface, as shown in Figure 2. The advantage of sealing cracks with this geometry is mainly ease and quickness of application. The band-aid type of configuration is attained by pumping sealant over the crack and then leveling with a wiping or "squeegee" operation.

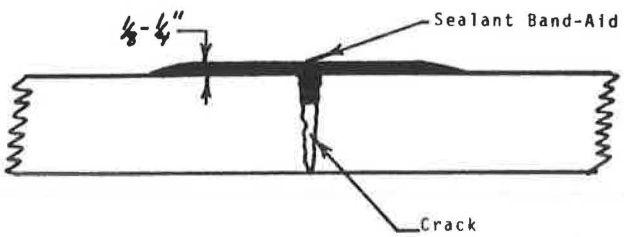

FIGURE 2 Band-aid sealant application configuration.

Several disadvantages of this type of geometry exist. First, a pavement sealed in this manner is unsightly because of the wide dark bands of sealant. Second, the sealant material is on top of the pavement and is exposed to abrasion from vehicle tires that can wear it away soon after application. In cold climates, snowplow operations can dislodge the sealant. The third disadvantage is that when working thermal cracks are sealed in this manner, the sealant is subjected to relatively large and localized tensile strains immediately above the crack, which can promote early failure. For a sealant to perform well in this situation, it must be capable of large extensions at low temperatures. In addition, the sealant must have sufficient high in-service temperature stiffness to resist pickup and tracking by vehicle tires.

Sealant Reservoir

The sealant reservoir type of configuration consists of a widened crack in a rectangular shape cut approximately 0.5 in. wide and 1 in. deep in the pavement surface. The crack is then filled to surface level with sealant, as shown in Figure 3. The reservoir can be efficiently cut with commercially available sawing and routing equipment designed specifically for this purpose.

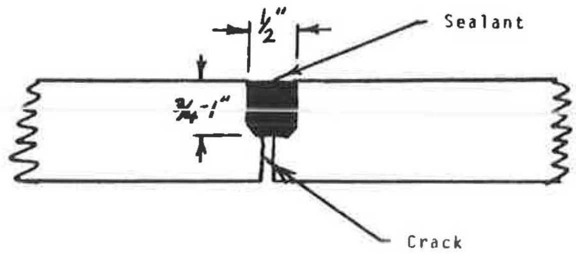

FIGURE 3 Sealant reservoir application configuration.

Crack sealing using a sealing reservoir type configuration has several advantages. First, the sealant is applied only to surface level, resulting in a neat appearance when compared with the band-aid configuration. Second, the sealant is not on top of the pavement surface, and therefore it is not directly exposed to abrasion by vehicle tires. The crack widening operation also cleans the crack faces, which provides intact surfaces for the sealant to adhere to. Another advantage when compared with the band-aid configuration is that the sealant is subjected to a lesser amount of strain when the pavement contracts in cold weather because of the increased width of the sealant. In very cold areas where large crack movements are expected, a v-shaped reservoir, which is between 0.25 and 0.375 in. deep at the center and 1.5 to 2 in. wide, can provide improved performances when compared with a standard widened reservoir. This type of reservoir may also be cut with commercially available equipment.

The main disadvantage of using the sealant reservoir geometry is that the widening operation is an extra step and an added cost. With commercially available equipment, an operator can widen between approximately 4,000 and 8,000 linear feet of cracks (depending on asphalt concrete characteristics) in an 8-hr shift at a total cost of between $0.05 and $0.08 per foot.

Crack Preparation Methods

In order for sealant material to adhere appropriately to the pavement and to ensure maximum sealant life, the crack must be prepared in a manner that provides intact bonding surfaces that are free of moisture, dust, loose aggregate, or other contaminants. Various methods and equipment types can be used to clean cracks. Many times several of the following cleaning methods need to be used to adequately prepare the cracks for sealing.

Compressed Air

Compressed air at a minimum of approximately 80 psi can be used to remove relatively loose debris, dust, and slight amounts of moisture from cracks. For dry cracks that are relatively clean and at least 0.5 in. wide, use of compressed air may be the only cleaning operation required before sealing.

Low Pressure-High Volume Air

Low pressure-high volume air flows can be used to clean cracks and can be provided by several pieces of commercially available equipment. In contrast to use of an air compressor, the low pressure-high volume air blowing devices are smaller and more portable. These devices can adequately clean many cracks of loose debris, dust, and slight amounts of moisture.

Wire Brushing

A power wire brushing operation can aid in cleaning and removing relatively loose deteriorated asphalt concrete from cracks and can greatly improve the adhesion of the sealant in the pavement. Several different devices are available commercially.

Crack Widening

Crack widening is performed when sealing relatively narrow cracks (less than 0.375 in. wide) using the sealant reservoir geometry or when the faces of the crack are deteriorated to the point that they must be cut back to provide intact asphalt concrete. Following crack widening, the crack should be cleaned with an air-blowing operation or wire brushing before sealing.

Hot Compressed Air

Devices are commercially available that receive compressed air from an air compressor, heat the air, and then direct the air to the crack. These devices can remove loose debris and dust from cracks, as well as dry out and remove excess moisture before sealing, which can aid in extending the sealing season in cold or damp weather. An added benefit of the hot compressed air cleaning operation is warming the pavement, thus promoting an improved seal with hot-pour sealants.

Application of Sealants

Two basic sealant classifications with respect to mode of application exist: cold pour and hot pour. Cold-pour sealants, as the name implies, are applied by pouring at ambient temperatures. Cold-pour sealants cure or set up as the fluidizing medium, generally either hydrocarbon solvent or water, evaporates. Many times cold-pour-type sealants require sanding immediately following application to prevent tracking.

Hot-pour-type crack sealants must be melted and then heated to the manufacturers' recommended application temperature before being applied to ensure development of maximum adhesion and to provide appropriate sealant consistency for penetration into cracks. Many sealant materials may degrade if overheated; therefore sealants should not be heated in excess of the manufacturers' recommended safe heating temperature. Several different types of equipment can be used to melt and apply hot-pour crack sealing materials.

Melter Applicator Units

A sealant melter applicator unit is a device specifically designed to efficiently melt and then apply hot-pour-type pavement sealant materials. Most commercially available units also have an agitation system that assures uniform temperature and consistency of the sealant at application. Melter applicator units generally are constructed in a tank-within-a-tank type of configuration, in which sealant is melted in the inner tank and the space between the tank shells is filled with a heated heat-transfer medium (generally heat-transfer oil) that provides indirect heating. Indirect heating is necessary for many types of sealant to guard against localized overheating and possible sealant degradation. Sealant at the proper application temperature is generally applied to the crack through a pump-fed applicator wand and nozzle. It is important that the melter applicator unit being used is capable of safely heating the sealant to the proper application temperature. Several currently available melter applicator units can be used to melt and apply as much as 5,000 to 8,000 lb of sealant (approximately 15,000 to 24,000 ft of cracks) in an 8-hr day.

Pour Pots

Sealant may also be applied through hand-operated gravity feed pour pots. For hot-pour sealants, the sealant first is melted in a kettle, and then the pour pot is filled. The pour pot is then used to apply the sealant. Pour pots cannot efficiently apply sealants that are of high viscosity at application temperatures. Pour pots may also be used to apply some types of cold-pour unmodified asphalt sealants such as emulsified sealants.

Crack Sealing Cases

Many types of cracks in asphalt concrete pavements in several different situations should be sealed to ensure maximum pavement life. Crack sealing, if performed adequately and soon after crack development, can be an economical and effective preventive maintenance technique. In addition, crack sealing can be performed along with other types of pavement maintenance and rehabilitation functions.

Transverse and Longitudinal Thermal Cracks

Thermal cracks should receive the highest priority when sealing cracks because they occur before the pavement has significantly deteriorated. It is important that these cracks be sealed with a sealant and in a manner and configuration that will assure that the seal can adjust to various crack widths as the pavement contracts and expands. Sealing thermal cracks soon after development will aid in limiting crack growth and minimize moisture-related deterioration while extending the life of the pavement.

Fatigue Cracks

Sections of pavements that experience fatigue cracking have failed structurally. Therefore, sealing fatigue cracks will not increase pavement life to the extent that sealing transverse cracks will. Sealing fatigue-type cracks, however, aids in retarding further deterioration by minimizing moisture intrusion; therefore the useful life of deteriorated pavement areas can be increased by extending the time to reconstruction.

Reflective Cracking

Reflective cracking in asphalt concrete overlays may appear within a year after construction of the overlay. Sealing of reflective cracks, especially reflected thermal-related cracks, will aid in ensuring that the overlay does not prematurely deteriorate and provides useful service throughout its design life. Sealing cracks in the pavement surface that is being overlaid will also aid in minimizing deterioration by preventing moisture from reaching the base and subgrade. When milling old pavements, crack sealing should be considered after completion of milling operations before construction of an overlay.

Shoulder Joints

Although joints between portland cement concrete pavements and asphalt concrete shoulders are not cracks in asphalt concrete pavements, the need exists for sealing this joint. Maintaining an adequate seal in shoulder joints aids in minimizing deteriorations of the asphalt concrete shoulders as well as along the edges of the concrete pavement. Sealant material used to seal shoulder joints should be capable of conforming to varying joint widths that can occur as shoulders settle and move.

SUMMARY

The many aspects of the maintenance technique of sealing cracks in asphalt concrete pavements are examined. The mechanism of cracking, crack movement, and consequences of not sealing cracks are discussed. In addition, properties and specifications for sealant materials and application techniques are covered. In summary, several specific statements are presented.

1. Cracking is a normal occurrence in asphalt concrete pavements and occurs mainly because of aging of the binder and loading of the pavement.
2. If cracks are not effectively sealed, pavement deterioration is hastened because of the detrimental effects of moisture intrusion into the pavement structural system.
3. Cracks in asphalt concrete pavements may experience significant movement from summer to winter. Therefore it is essential that the crack sealant material be capable of extending and flexing at low ambient temperatures so that it can maintain the seal as the pavement moves.
4. Several different types of sealant materials are currently used for crack sealing. The properties, effectiveness, and life expectancy of these materials vary widely.
5. Two basic types of sealant geometries (band-aid and widened reservoir) are currently in use; each has advantages and disadvantages in specific situations.
6. Equipment specifically designed for high production crack sealing is currently available.
7. With available materials, equipment, and techniques, crack sealing in asphalt concrete pavements today is a lasting and cost-effective preventive maintenance function that can extend the useful life of asphalt concrete pavements.

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Field Performance of a Low-Modulus Silicone Highway Joint Sealant

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ABSTRACT

A field study undertaken to evaluate the performance of Dow Corning 888 silicone highway joint sealant in various climates and pavement conditions indicates that sealant performance remains high for 6 years and beyond. Nine-year-old joints in Georgia and Michigan are performing well. Pavement seals in Connecticut, Georgia, Illinois, Indiana, Iowa, New Mexico, Michigan, Minnesota, and South Dakota, covering four major climatic zones (wet, freeze; no freeze; dry, freeze; and dry, no freeze) were inspected and evaluated. The study also identified factors that affect performance. Of these, installation procedures and shape of the actual seal are the most influential and also the most controllable. The inspections revealed that Dow Corning 888 silicone highway joint sealant can overcome inadequacies in field installation procedures and provide a reasonable seal life.

There are more than 200 low-modulus silicone highway joint sealant installations across the country. Project sizes vary from 3 joints to 30 miles of jointed pavement. Many projects have been installed by state agencies to evaluate these new sealants. Others are part of demonstration projects. There are several installations where the sealant was installed on regular construction projects.

The sites for this nine-state field study were selected to evaluate Dow Corning 888 low-modulus silicone highway joint sealant with various seal ages, climatic zones, traffic levels, and joint conditions. The study revealed that Dow Corning 888 silicone sealant offers excellent seal integrity and longevity. Performance variations between installations primarily reflect differences in joint design and care taken during installation.

UNIQUE SEALANT PROPERTIES

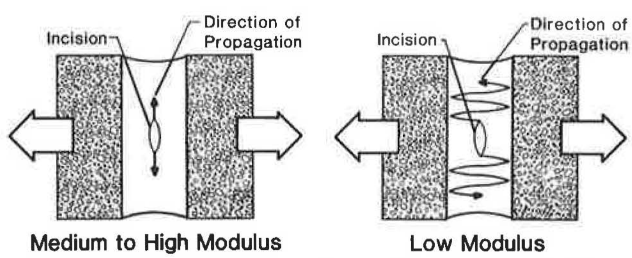
Silicone sealants are widely used in concrete construction. They are one-part materials consisting of long chain silicone polymers, curing agents, and fillers. The applied sealant cures to an elastomer on exposure to water vapor in air, and forms a continuous silicone-oxygen-silicone network. This silicone-oxygen linkage is transparent to ultraviolet radiation and is responsible for the superior weatherability of silicone sealants.

Silicone sealants can be differentiated from one another by their modulus (i.e., their ability to stretch and recover their original shape). The lower the modulus value, the greater is their ability to elongate and recover and thus withstand the cyclic movement of concrete pavement joints. The modulus, ultimate elongation, and joint movements for typical high-, medium-, and low-modulus silicone sealants are given in the following table (1):

Type	Modulus (psi)	Ultimate Elongation (%)	Cyclic Joint Movement (%)
High	>100	<500	±25
Medium	40-100	500-1,200	±40
Low	<40	>1,200	±50

Silicone sealants, in general, are set apart from other sealants by their ability to resist compression set. This allows them to withstand repeated movement caused by climatic changes. Typical recovery values after compression for low-modulus silicone sealants are 90 to 100 percent compared with recovery values of 80 to 90 percent for urethane and 70 to 80 percent for polysulfide sealants. This combination of resistance to compression set and low-modulus characteristics enables the sealant to expand when the joint opens. Recovery from compression is a key feature that distinguishes silicones from other sealants (1).

Another property of Dow Corning 888 silicone sealant is its resistance to tear propagation. Usually sealant tears propagate perpendicular to the direction of stress (1). In contrast, the low-modulus silicone has a very ragged tear that propagates slowly back and forth almost parallel to the direction of stress (Figure 1).



Note: Blocks of concrete are shown pulling on the sealant with an incision to initiate a tear.

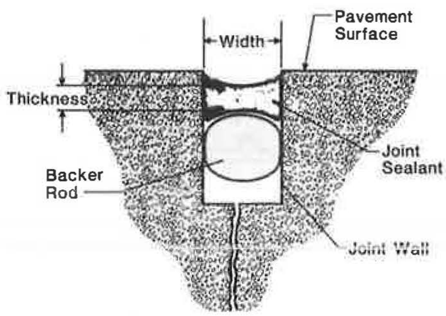
FIGURE 1 Difference in tear propagation between low-modulus and higher-modulus silicone sealants.

Joints filled with the low-modulus silicone sealant can be repaired by patching with new sealant. New silicone will form a strong bond with the cured sealant. Thus small failures can be repaired without replacing the entire joint seal.

FACTORS AFFECTING SEAL PERFORMANCE

Although many factors influence sealant performance, the most important factors are shape of the applied sealant, joint spacing, sealant physical properties, joint condition, and proper seal installation. Of these, proper installation and the shape of the applied sealant are the most influential, and also the most controllable.

The smaller the ratio of applied sealant thickness to width, the lower is the stress applied to the silicone rubber [Figure 2 (2)]. A thickness-to-width ratio, or shape factor, of 0.5 to 1.0 with a thickness range of 0.25 to 0.5 in. is recommended. This produces a thinner seal than recommended for other sealants, but it is acceptable because of the ability of the silicone sealant to bond to the joint walls and because of its excellent cohesion.



Shape Factor: Sealant Thickness/Sealant Width

FIGURE 2 Joint terminology and shape factor *(2)*.

Proper installation procedures are necessary to ensure that the physical properties can be maximized. The joint must be clean and dry, free of sawing debris, and free of any particles or film of old sealant. The backer rod, which controls sealant depth, must be correctly placed. The sealant must be tooled immediately after application to recess it beneath the pavement surface and to apply sufficient pressure to force the sealant against joint walls to ensure a good bond.

STRESSES AFFECTING SEALANTS

Adhesive stress is the tensile stress between the sealant and the joint wall. Factors that can cause the sealant to separate from the joint wall include weak sealants, wet or dirty joint walls, inadequate tooling, high stress brought on because of an improper shape factor, and sealant hardening.

Cohesive stress is developed within the sealant when the joint opens. If the sealant is insufficiently elastic or has weak interparticle bonds, it will split. Also, if the thickness-to-width ratio is too great, high cohesive stress will cause an otherwise acceptable sealant to fail.

Peeling stress develops at corners of the sealant where it bonds to joint walls. It is caused by joint movement and can be accentuated by improper installation or tooling.

Compressive stress is caused by joint closing. If the sealant is too fluid, or if the joint closes too far, the sealant will extrude from the joint.

Figure 3 (2) shows the effect of these stresses on the sealant and the sealant-joint wall interface. Anything that reduces stress or strain on the sealant or increases the bond strength between the sealant and the joint wall without reducing sealant elasticity will improve sealant performance.

ASSESSING SEALANT PERFORMANCE

The function of a highway joint sealant is to prevent water and foreign matter from entering the joint. Consequences of sealant failure include sub-

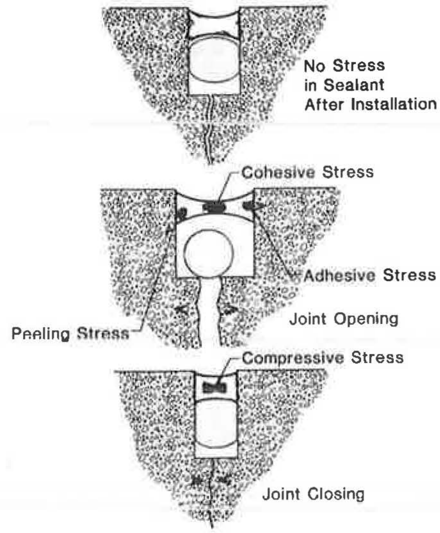


FIGURE 3 Stresses in field-poured sealants *(2)*.

grade weakening, pumping, faulting, accelerated D-cracking, blow-ups, and joint spalling.

In evaluating the low-modulus silicone highway joint sealant, four performance properties were examined: (a) adhesion, (b) cohesion, (c) surface defects, and (d) spalling or the presence of foreign material in the joint.

Adhesive failure is a common failure with any sealant. With silicone sealant, such a failure may be caused by lack of an initial bond or by the loss of bond. Also, a large shape factor, especially in the narrow joint (where the sealant is extended more than 100 percent), is a common cause of a loss of bond. When failure occurs in such cases, the joint wall usually has residue on it.

An adhesive failure with no sealant residue on the joint wall indicates a firm bond was never established because of improper cleaning before sealant installation. Insufficient tooling or contamination of joint walls with dirt, sawing residue, old sealant, or moisture can prevent a good bond. Figure 4 shows the results of common installation problems.

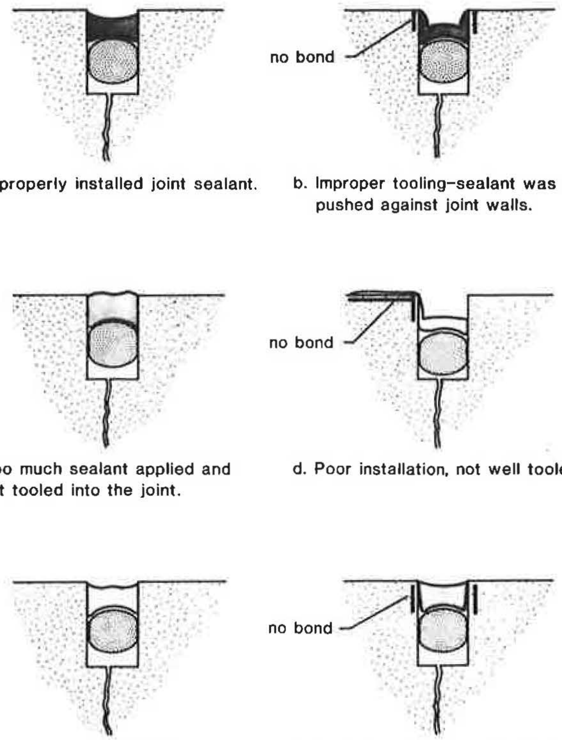
Cohesive failure is purely material failure. The sealant is unable to stand the internal tensile stress caused by the joint opening. Significant amounts of sealant usually remain on the joint wall.

If failure is near the joint wall it may be difficult to distinguish between adhesive and cohesive failure. Examination of the joint wall is the key. Adhesive failure leaves little sealant on the joint wall. Cohesive failure leaves more sealant on the wall, and it will still be firmly bonded. Cohesive failure of low-modulus silicone sealant is uncommon, except when the seal is too thin (usually less than 0.125 in.).

Joints were also checked for damage due to spalling caused by incompressibles. Spalling caused by incompressibles is distinguished from chipping of the leave slab by the size and shape of the particles. Close inspection usually reveals that chipping caused by snowplows is distinguishable from spalling by many small, thin pieces of concrete broken away from the slab at a 45-degree angle.

MEASURING SEALANT PERFORMANCE

A severe test was developed to identify and measure adhesive and cohesive failures. The end of a thin,



a. A properly installed joint sealant.
b. Improper tooling–sealant was not pushed against joint walls.
c. Too much sealant applied and not tooled into the joint.
d. Poor installation, not well tooled.
e. Sealant installed well but was in compression before completely cured.
f. Sealant applied in wet joint, bond is at top of joint wall.

FIGURE 4 Joint cross section illustrating installation problems.

0.75-in.-wide metal ruler is pushed into the sealant at intervals of 3 to 6 in. along the joint. Cohesive failure is apparent when the ruler is pushed into the sealant. Twisting the ruler pulls the sealant away from the joint wall (Figure 5) and severely tests the bond between them. Any adhesive failure is noted and measured in inches. This test permits year-round inspection, not just in winter when joints are open for visual inspection.

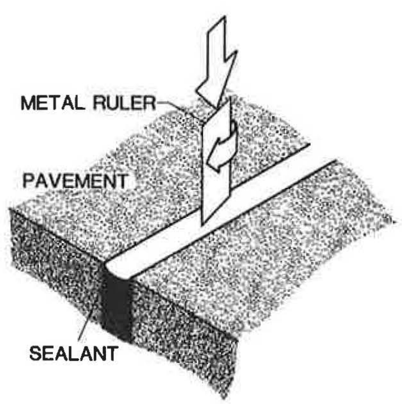


FIGURE 5 Graphic representation of test procedure for adhesive/cohesive failures.

An adhesion/elongation test evaluates sealant strength and the sealant-joint wall bond. Three cuts are made in the sealant: 2-in. cuts along each wall and a cut across the sealant at one end of the 2-in. cuts. The 2-in. tab thus formed is lifted out of the joint at a right angle to the surface. A mark is drawn across the tab at a height of 1 in. Then, with the ruler held along it, the tab is pulled up at a steady rate. The location of the mark along the ruler when the sealant begins to fail is noted, as is the type of failure: adhesive or cohesive. This test can also be conducted at any time of year because silicone sealant properties are not especially temperature sensitive.

In this test, an inch change in length equals 100 percent elongation. Typical values recorded in the field ranged from 200 to 500 percent. However, the amount of elongation is insufficient to describe the results because elongation is a function of sealant cross-sectional area.

Adhesive failure is the sign of a weak bond. Cohesive failure indicates that the sealant has sufficient bond strength to withstand joint movement.

PERFORMANCE SUMMARY

Locations for sealant performance evaluation were selected to represent different climatic zones: wet, no freeze (Georgia); wet, freeze (Connecticut, Illinois, Indiana, Iowa, Michigan, and Minnesota); dry, freeze (South Dakota); and dry, no freeze (New Mexico). Sealant age was also a consideration. A complete list of test sites is given in Table 1.

TABLE 1 Sites Inspected

Location	Climatic Zone	No. of Sites	Sealant Age (years)
Georgia			
I-75	Wet, no freeze	1	6
I-16	Wet, no freeze	1	5
I-85	Wet, no freeze	2	6
I-20	Wet, no freeze	2	4
Connecticut, I-84	Wet, freeze	1	2
Indiana, US-31	Wet, freeze	1	4
Illinois, IL-5	Wet, freeze	1	1
New Mexico, I-94	Dry, no freeze	1	2
Minnesota, I-94	Wet, freeze	1	5
Iowa, R-30	Wet, freeze	1	5
South Dakota, I-29	Dry, freeze	2	4
Michigan, I-69	Wet, freeze	1	1

Georgia

Georgia was selected for the first inspections because, since 1974, the state has sealed many miles of pavement with Dow Corning 888 low-modulus silicone sealant. Six sites were inspected in detail in May 1983, and others were examined visually (see Table 2).

Georgia's use of low-modulus silicone pavement joint sealant has received considerable attention. Published reports indicate the sealant is performing well, and detailed inspections verify this (3). Numerous pavement and bridge deck sealing projects were observed while traveling with Georgia Department of Transportation (DOT) engineers.

The oldest silicone sealant installation in Georgia, located on the northbound lane of I-75 at milepost 189 near Forsyth, was installed in 1974. Heavy traffic prevented detailed inspection. Cursory examination showed that the sealant was still performing well.

All of the joints inspected were resealing projects covered by Georgia DOT specifications. Joints were sawed and cleaned by sandblasting, although occasionally a wire brush was used.

Ten joints (240 linear feet of sealant) were inspected at each site. The results are summarized in Table 2. At several sites it was noted that the

TABLE 2 Georgia Inspection Summary

Location	Pavement[a]	Installation Date	Age (years)	Pavement Distress	Sealant Failure	Comments
I-85						
Northbound lane, milepost 17	9-in. PCC over 1 in. of AC sand; JPCP at 20-ft spacing with dowels	Summer 1977	6	Old spalls filled with silicone	3 percent, adhesive	Insufficient tooling
Northbound lane, milepost 22	9-in. PCC over 1 in. of AC sand over 12-in CTB; JPCP at 20-ft spacing	Summer 1977	6	None	0.5 percent, adhesive	Insufficient tooling
I-75, northbound lane, milepost 204	9-in. PCC over 3 in. of AC sand over 8-in. CTB; JPCP at 20-ft spacing	May 1977	6	Localized chipping	None	
I-20						
Eastbound lane, milepost 116	9-in. PCC over 12-in. CS; JPCP at 30-ft spacing with skewed joints	June 1979	4	Minor faulting	None	
Westbound lane, milepost 129	9-in. PCC over 12-in. CS; JPCP at 30-ft spacing with skewed joints	June 1979	4	Minor faulting	<1 percent, cohesive	Sealant too thin, <0.0625 in.
I-16, southbound lane, milepost 4	9-in. PCC over 4 in. of AC sand over 8-in. CTB; JPCP at 30-ft spacing	Fall 1978	5	None	0.5 percent, adhesive	

[a]Note that PCC = portland cement concrete, AC = asphalt concrete, JPCP = jointed plain concrete pavement, CTB = cement-treated base, and CS = crushed stone.

asphalt shoulder sealant and paint stripe at the pavement edge acts as a dam, trapping water, sand, and small stones in the joint recess. In time, this could accelerate joint and sealant damage.

The most sealant failure was found on the northbound lane of I-85, at milepost 17. Here sealant on the pavement surface and along the joint wall above the recess indicates incomplete tooling. The same condition was also noted in the southbound lanes of I-85, but resulted in only 0.5 percent adhesive failure.

Connecticut

The Connecticut test site is on the eastbound lanes of I-84 south of Manchester at the end of the Wyllyss exit turn off. The four-lane pavement is on a long uphill grade. Three lanes are long-jointed portland cement concrete (PCC) pavement. An asphalt shoulder serves as a truck lane. This pavement is subjected to as much as 0.5-in. of vertical movement caused by differential frost heave. Seven transverse joints (originally 0.875 to 1.25 in. wide) and the corresponding longitudinal joint (0.5 to 0.75 in. wide) were sealed with silicone sealant in September 1981.

This installation was satisfactorily done. The sealant is well tooled against the joint walls, and the average recess is 0.375 in. Except for two large adhesive failures 2 and 4 ft long, only small failures were found in the remaining joints. The 2-ft failure appeared to be caused by too thin an initial bond area. The joint with the 4-ft failure did not appear to have been throughly cleaned. Old asphalt sealant was found under the backer rod, and the joint wall of the leave slab contained some residue.

Adhesive failures totaled 87 in., or 3 percent of joint length, and the two large failures accounted for 72 in. of this. Overall, the sealant is still performing well.

Indiana

The Indiana test site is on US-31 northwest of South Bend. The silicone sealant was installed as a demonstration in May 1979. Twenty-five joints from station 209+30 north to station 218+90 were inspected.

The sealant was installed in new pavement with 40-ft joint spacing. Joints were sawed 0.25 to 0.375 in. wide and cleaned with an airblast. Using a roller, 0.375-in.-diameter closed backer rods were installed 0.5 in. deep in the joint. Silicone was pumped into the joint and tooled to a 0.25-in. depth with a tooling foot, a device attached to the applicator that produces the intended sealant recess.

This site is typical of most test installations. Because of inexperience or experimentation, sealant application is uneven in the first few joints. Joints sealed later look neater and correctly installed.

Of 240 linear feet inspected, there was no bond for 48 ft, or 20 percent of the total length. This adhesive failure is classified as lack of bond development caused by contamination of joint walls with sawing residue. Airblasting alone cleans unevenly, and adhesive failures can be expected. This pavement is also heavily tined, making joint walls prone to damage from snow removal equipment. Most joints have a foot or more of chipping.

Illinois

In the summer of 1982, low-modulus silicone sealant was used to seal joints in 5 miles of PCC overlay in the eastbound lanes of the East-West Tollway between Naperville Road and IL-59. The 8-in. overlay was laid over 10 in. of original concrete pavement on a crushed stone base. Random joint spacing ranges from 12 to 18 ft.

According to engineers interviewed at the site, the contractor used the following installation procedure. Joints were sawed within 24 hr of construction and again 2 weeks later. A sealing crew followed immediately with a high pressure waterblast. Sealant was pumped into wet joints for the first 2 miles of the project. Informed of the proper cleaning technique, the contractor switched to wire brush cleaning for the last 3 miles.

The sealant was inspected approximately 1 week after installation by pulling up on the ends. Few joints failed; those that did were resealed.

Inspection of 10 seals placed in wet joints revealed lack of bond in 9. About one-third of total sealant length failed adhesively. Most had bonded at one time, but only to the top 0.125 in. of the joint wall that had time to dry before the sealant was applied. The appearance of the sealant also indicates that tooling was insufficient to create intimate contact between the sealant and the joint wall.

Despite these problems, the silicone sealant is still in place and functioning on this heavily traveled road where bond is only 0.125 in. An average recess of nearly 0.5 in. contributes to this performance by preventing tires from pulling the sealant out of the joint.

New Mexico

In September 1981 about 7,200 linear feet of silicone sealant was installed on I-25 south of Albuquerque. The site begins 300 joints south of milepost 219 in the northbound lane. Joints are skewed on 18-ft spacing in plain jointed concrete.

No failures, either adhesive or cohesive, were found in 10 joints inspected in December 1983. This site illustrates the importance of the shape factor and the sealant thickness-to-width ratio. Joints range from 0.5 to 0.625 in. wide and sealant thickness ranges from 0.375 to 0.5 in. Thus the shape factor varies from 0.6 to 1.0, the correct range. Sealant recess averages 0.1875 in.

Cursory inspection of the other 290 joints revealed only 3 with any visible distress. All three are in the outer wheelpath where the sealant was used to fill spalls. A total length of 56 in. (0.6 percent of the total) has been replaced with asphalt.

The silicone sealed joints have a neat appearance and performance has been satisfactory. There are essentially no failures in 7,200 linear feet of silicone sealant.

Minnesota

Twenty-five joints on the eastbound lane of I-94, previously sealed with hot-poured asphalt, were resealed with low-modulus silicone on October 24, 1978. The joints, spaced 20 ft apart, are located just east of the first service crossover west of the Sauk Center interchange.

The joints were sawed with a diamond blade to a width of from 0.625 to 0.75 in., then sandblasted and airblasted. A 0.75-in. Ethafoam backer rod was rolled into the joint to a 0.75-in. depth. A tooling foot on the sealant applicator tooled the sealant to a depth of 0.25 in. The right lane was sealed first and opened to traffic within 30 min of sealant application.

This site was inspected by Dow Corning representatives in April 1979. Joint appearance was reported good, with the left lane looking better than the right. This is understandable because of traffic volume. As the work progressed, the applicators became more adept. Some adhesive failure occurred at the centerline, where the sealant had been used to fill large corner spalls. Overall, the silicone sealant looked good after its first winter.

In 1983 the overall seal condition was very good to excellent. The corner spalls had been replaced by asphalt concrete as a part of a maintenance program for the entire pavement. The silicone was removed from the spall area before patching, and the spall repair crew somewhat damaged adjacent sealant. Approximately 60 in. of chipping by snow removal equipment was observed, but the silicone sealant held the chips firmly in place.

There is less than 1 ft of adhesive failure in the total joint length of 240 ft. After 5 years the sealant is still performing well.

Iowa

The Iowa test site is an excellent example of the importance of proper joint cleaning and sealant application. The site is located on country road R-30 between F-31 and IA-44 northwest of Des Moines. Forty joints are north and five joints are south of the first gravel crossroad south of F-31. The 6-in. concrete pavement was constructed in the summer of 1978, and the joints, spaced every 40 ft, were sealed in September.

The joints were divided into nine sections of five each. Each section was sawed to different widths, and three different cleaning methods were used (Table 3). Both sandblasting and waterblasting were followed by an airblast. Waterblasted joints were allowed to dry for 4 hr before applying the sealant.

TABLE 3 Combinations Used for Joint Sealing, Iowa Site

Section	Joint Width (in.)	Cleaning Method	Backing Material
1	0.25	Waterblasting	Ethafoam
2	0.25	Airblasting	Ethafoam
3	0.5	Airblasting	Ethafoam
4	0.5	Airblasting	Tape
5	0.375	Airblasting	Ethafoam
6	0.375	Airblasting	Tape
7	0.25	Sandblasting	Ethafoam
8	0.5	Sandblasting	Ethafoam
9	0.5	Sandblasting	Tape

Various tooling methods produced variable sealant recesses. An immediate inspection stated that the installation was only fair because of overall sloppiness. Uniformity of joint width was poor, and a rough surface hindered installation.

An inspection in April 1979 revealed no evidence of adhesive failure but did discover areas where not enough sealant had been applied to "wet" the joint. As a result, no bond had developed.

Twenty joints, 10 cleaned by sandblasting, and 5 cleaned by each of the other methods, were inspected in August 1983. The airblasted-only joints have an average of 50 percent adhesive failure. Four of the waterblasted joints averaged 16.5 percent adhesive failure. In the fifth, the sealant was less than 0.09375 in. thick, and the joint was full of small gravel from the road intersecting at this point; it failed totally. Only 1 percent of the joints that had been sandblasted and airblasted failed adhesively, and that failure is attributed to not enough sealant being applied. Nine feet of cohesive failure was noted in one sandblasted joint where the sealant was only 0.03125 in. thick.

It should be noted that proper sealant shape, proper thickness-to-width ratio, and proper sealant recess below the pavement surface would have improved performance at this site, regardless of the cleaning method.

South Dakota

Low-modulus silicone sealant was installed in a 30-mile pavement rehabilitation project on I-29 in South Dakota. In the fall of 1979 the sealant was installed in 13 miles of the northbound lanes extending north from the Iowa line. The next year the sealant was installed in the northern 17 miles of the project in the southbound lanes. The rehabilitation consisted of partial depth patches to repair spalls caused by deteriorating Unitube joint formers. About 75 percent of each joint was patched, so much of the sealant was applied in new joints. Joints were sawed and cleaned with a waterblast followed by an airblast.

At this site the sealant is subject to three different joint conditions: (a) joints previously sealed with hot asphalt, (b) new joints sawed from patches, and (c) a patch on one side and old concrete on the other. Performance of the silicone sealant in these joints is influenced by how well the joints were formed after patching and how well asphalt sealant residue was removed from old surfaces. Most of the adhesive failures noted in 1983 were in resealed joints in which an asphalt film remained on joint walls. The patched joints have much better adhesion because the waterblast process removes saw fines more effectively than old sealant.

Poorly formed joints appear to have caused problems during application and tooling. In many joints the surface of the sealant is wavy, as shown in Figure 4c, rather than concave, as in a properly tooled joint.

Joints installed in the northbound lanes in 1979 exhibited more uneven application and adhesion problems than those installed the next year. Also, adhesive failures were inversely proportional to the length of the patch. The majority of one particular joint, less than one-half of which had been patched, failed adhesively. All other joints averaged 5 percent adhesive failure, and the failures occurred almost exclusively in the unpatched portion of the joint where residual sealant remained.

The seals in the southbound lanes looked much better. Only 21 in. (1 percent) of the total joint length inspected showed any failures. Some chipping of high spots in patches was also noted.

Michigan

Low-modulus silicone sealant was installed in the eastbound lanes of I-69 between the Clark Road overpass and the Airport Road exit in 1982. Joint spacing is 40 ft and the pavement has concrete shoulders.

Both transverse and longitudinal joints in the highway and the shoulder are sealed with silicone. Joints were sawed 1 in. wide and sandblasted before installation.

Detailed inspection in 1983 found no adhesive or cohesive failure in 1,950 linear feet of transverse and longitudinal joints inspected in 1983. Joint width ranged from 0.875 to 1 in. and sealant thickness ranged from 0.375 to 0.9375 in., giving the proper shape factor. Typical sealant recess was 0.25 in. Tooling appeared adequate, although considerable excess sealant was noted on the pavement surface.

CONCLUSIONS

Among the variables influencing the performance of Dow Corning 888 sealant examined in this study were climate, age, joint cleaning methods, installation procedure, joint design, sealant shape factor, sealant recess, traffic, pavement condition, and joint spacing. Inspection of 14 highway sealant installation projects indicates that two factors are paramount: joint wall cleaning and installation techniques. The data in Table 4, which summarize the results from all 14 sites, clearly demonstrate this.

The various sites inspected included four cleaning techniques: airblasting only; wire brushing followed by airblasting; waterblasting followed by airblast; and sandblast followed by airblast.

The Iowa site vividly demonstrates the superiority of sandblasting. The South Dakota site shows the importance of removing old sealant residue from joint walls in resealing projects and indicates that high-pressure waterblasting is unable to do this effectively.

Wire brushing, as on I-16 in Georgia, is effective for removing saw residue in new or resealed joints. However, this technique is not recommended unless the joint is sawed. There is no data to indicate that it removes old sealant effectively.

Installation is very important. Sealant should never be applied to a wet or damp joint. After the sealant is pumped into the joint it must be tooled to push it against the joint walls. This can be done with a special foot on the applicator nozzle or by hand using a variety of trowel-like devices. The fewest failures were found at joints where the width of the tooling foot matched the joint width or where the sealant was carefully tooled by hand.

Joint design and sealant shape factor are also important, especially when joint cleaning and installation techniques are marginal. The correct shape factor reduces stresses in the sealant and increases its life. In the sites inspected, sealant thickness varied considerably. At a few sites very thin seals failed cohesively. However, no problems could be attributed to very thick application. Seals with shape factors of less than 0.5 and greater than 2.0 were performing well after 5 years, which indicates that Dow Corning 888 sealant is forgiving of poor joint design and some application techniques.

Other variables appear to have only a minor effect on sealant performance. Climate and age were expected to be major factors, and may prove to be so with time. However, samples taken at several sites and analyzed to determine the effect of aging indicate that the modulus (elasticity) of the sealant

TABLE 4 Inspection Summary

Location	Date	Age (years)	Cleaning Method	Tooling	Failure (%) Adhesive	Cohesive
Georgia						
I-75	1977	6	Sandblasting	Good	0	0
I-16	1978	5	Wire brush	Good	0.5	0
I-85	1977	6	Sandblasting	Fair	2.0	0
I-20	1979	4	Sandblasting	Good	0	0.4[a]
Connecticut, I-84	1981	2	Sandblasting	Good	3.0	0
Indiana, US-31	1979	4	Airblasting	Fair	20.0	0
Illinois, IL-5	1982	1	Waterblasting	Fair	31.0[b]	0
New Mexico, I-25	1981	2	Sandblasting	Good	0	0
Minnesota, I-94	1978	5	Sandblasting	Good	0.3	0
Iowa, R-30	1978	5	Airblasting	Poor	50.0	0
			Waterblasting	Poor	16.5	5.0[a]
			Sandblasting	Poor	3.0	4.5[a]
South Dakota, I-29	1979	4	Waterblasting	Good	3.0	0
Michigan, I-69	1982	1	Sandblasting	Good	0	0

[a]Sealant installed thinner than recommended.
[b]Installed in wet joint.

changes very little with age, as indicated by the data in the following table:

Number	Age	Modulus (psi)
1	4 years	24
2	4 years	26
3	2 years	28
4	5 years	29
5	7 days	20-25
6	27 days	25-30

Because of their excellent aging characteristics, silicone sealants appear to be capable of preventing pavement distress for much longer periods than conventional asphalt sealants.

The data developed in this study indicate that Dow Corning 888 low-modulus silicone sealant can overcome minor installation inadequacies and provide extended seal life. The data demonstrate that performance remains high for 6 years and more.

Longer-term performance has not been established because of the length of service of present installations. More study will be required over longer time periods to collect and analyze standardized performance data and illustrate long-term performance. This study is one point in time of the performance history of the installations surveyed.

RECOMMENDATIONS

Pavement joint sealant systems must be based on the calculated joint movement. After the working range of the joint is determined, the sealant shape can be selected to ensure that sealant strains will be within the manufacturer's recommendations.

Detailed specifications should include joint design, material acceptance, preparation, sealant installation including equipment, and inspection (4). Regular monitoring of the job site is necessary to assure that the specifications are followed precisely.

A long-term study should be undertaken to evaluate the performance of all types of sealants in a standardized manner. Such a study could establish life-cycle cost data for use in planning cost-effective pavement rehabilitation strategies. Joint sealing is critical to pavement life and should be addressed in a professional manner.

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