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Bulletin 229

Concrete Pavement: Subbase and Joint Construction

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Contents

Performance of Concrete Pavements as Related to Subbase Construction Methods

PHIL FORDYCE, Senior Paving Engineer, and
J. D. GEESAMAN, Laboratory and Field Studies Engineer,
Highways and Municipal Bureau, Portland Cement Association

Field studies of the performance of concrete pavements built on subbases of various designs and materials show that the method of subbase construction is an important factor. Uniformity of gradation was found to be the one subbase quality having the most influence on pavement performance. Subbase materials having a small maximum size were found generally to perform better than coarser materials because segregation is less likely to occur during placement.

Information was obtained during the construction of 28 projects in many parts of the country representing typical subbase construction methods. The effects on subbase density and gradation of various construction operations are discussed. These include methods of subgrade compaction, subbase mixing, placement and compaction, and fine-grading.

Heavy construction traffic on the completed subbase nullifies the efforts expended to obtain uniformity and results in substandard pavement performance.

● SINCE 1952, the Portland Cement Association has been engaged in an extensive study of subbases for concrete pavements. This study has been carried on in cooperation with the state highway departments of New York, Missouri, Indiana, Michigan and North Carolina. The purpose is to study the relative performance of pavements built on subbases of various designs and thicknesses and representing a variety of materials, in order to establish minimum subbase requirements for a wide range of traffic, soils and climatic conditions.

During the course of these studies it became increasingly apparent that the practices employed in subbase construction have an important influence on pavement performance. One of the main reasons for using subbases is to prevent pumping. Therefore, a careful investigation was made wherever joint pumping was observed on pavements built on subbase. In each case the cause of the pumping was traced to one or more improper construction practices. In some cases gravel had come from a pocket of inferior material in the pit or had become mixed with an excessive amount of overburden material. In others a leveling course of fine-grained soil had been placed on top of the subbase, or the subgrade soil had been mixed into the subbase by construction traffic during wet weather. In one case, for example, the subbase had been exposed to heavy truck traffic throughout an entire winter and at times trucks had become mired down so badly that they had to be towed across the project. By the time of paving, the subbase was badly contaminated and since it was not replaced, pumping developed. The appearance and gradations of the materials under the pave-

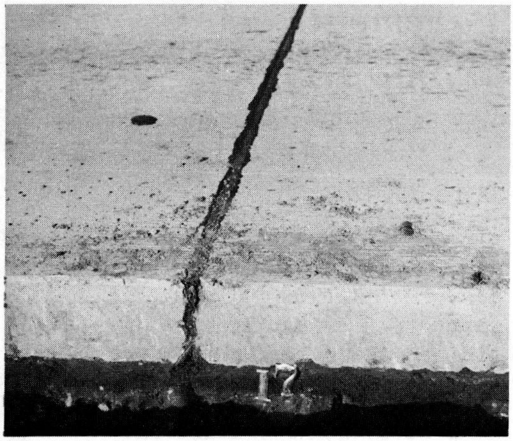

Figure 1.

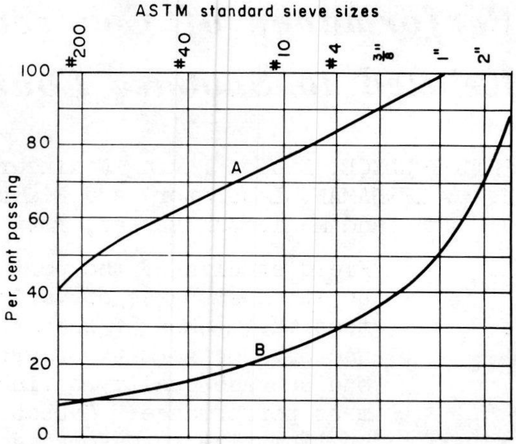

Figure 2.

ment on this project are shown in Figures 1 and 2. Curve "A" shows the gradation of material directly beneath the pavement. This layer varied in depth from 2 to 4 in. Curve "B" shows the gradation of the subbase layer buried below 2 to 4 in. of soil susceptible to pumping.

Another purpose of subbases is to control frost action. In some cases when the subbase was placed in such a manner that it became badly segregated, differential heave has occurred within the granular layer itself. Areas which are deficient in fines act as reservoirs for the collection of water, which can lead to the formation of ice lenses in adjacent areas which have excessive amounts of fine material. When the pavement and subbase are frozen solidly together, the differential heave can cause cracking in the pavement and an uneven surface.

In many cases variable performance occurs within individual projects. On many of these projects the type of soil, sources of materials, and traffic are the same from one end of the project to the other, and the pavement and subbase design and climatic conditions are uniform throughout. On one 10-year old project, for example, there are some sections in excel-

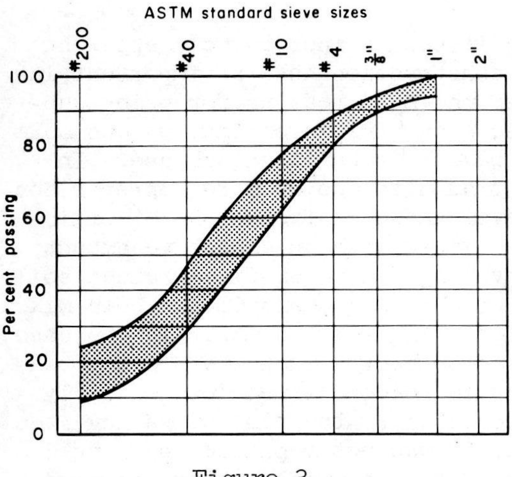

Figure 3.

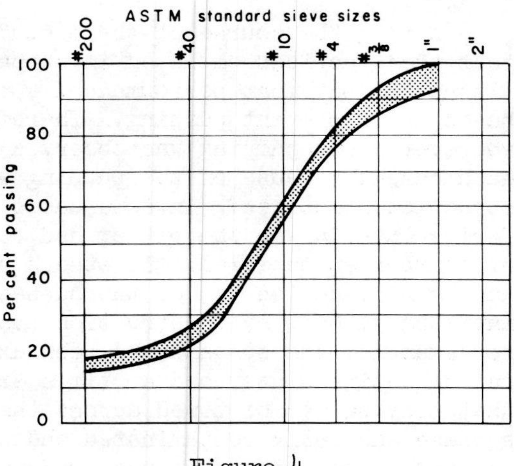

Figure 4.

lent condition, while others have not performed as well as expected. Two sections of this project were selected for detailed subbase sampling, one representing the good performance and one representing the poor performance. In each section six samples of subbase material were removed from under the heavily traveled lane of pavement at 10-ft intervals. Samples from the section with poor performance have wide variations in gradation as shown by the band in Figure 3. The gradation of the sample at one location may be along the top of the band while that of a sample from only 10 ft away may be near the lower limit of the band. On the other hand the gradations of the samples from the section with good performance are nearly identical, all falling within the narrow range shown in Figure 4. This indicated that the poor performance is due to nonuniform support offered by the segregated subbase.

Segregation of subbase material during construction results in variations in density, moisture content and permeability. These may eventually cause variations in the density and moisture content of the subgrade soil, which adds up to possible damage to the pavement due to differential support. Therefore, a subbase with a fairly constant gradation will result in improved pavement performance. Reducing segregation to a minimum by careful handling and mixing during construction is well worth the effort.

Attempting to determine how a subbase was built after it has been in service for some time is a difficult task and often involves making assumptions which may not be true. To obtain more detailed information on the relationship of subbase construction methods and pavement performance, a series of special subbase study sections are being established throughout the country. At present 28 of these sections have been set up in nine states. The locations are shown in Figure 5. On these sections the subbase construction is closely observed, tests are made on the subbase and samples of subbase material are taken for laboratory tests. Thus the future performance of the pavement on these sections can be correlated with subbase construction methods.

These sections are on projects representing a wide range of climate, soil types and subbase design and materials. Routes that carry substantial volumes of heavy truck traffic were chosen in order that the performance trends can be established in a minimum length of time.

A 2,000-ft section of each project included in this study is selected to represent normal construction for that project. Every effort is made to see that these sections are built in the normal manner so that typical construction methods are being studied.

When the section has been fine-graded and all subbase work completed, the tests are made on the subbase just ahead of the paver. Density and moisture tests are made at frequent intervals and samples are taken at each test site for laboratory determination of gradation, plasticity index and to establish standard moisture-density curves. The test sites are staggered transversely so as to include the full width of the lane being studied. Additional tests have been made on several sections. These have included a full series of tests both before and after fine-grading, tests on the subgrade soils, a number of tests in small areas to determine the degree of uniformity, and field CBR tests. Complete written and photographic records of construction methods are made on each section.

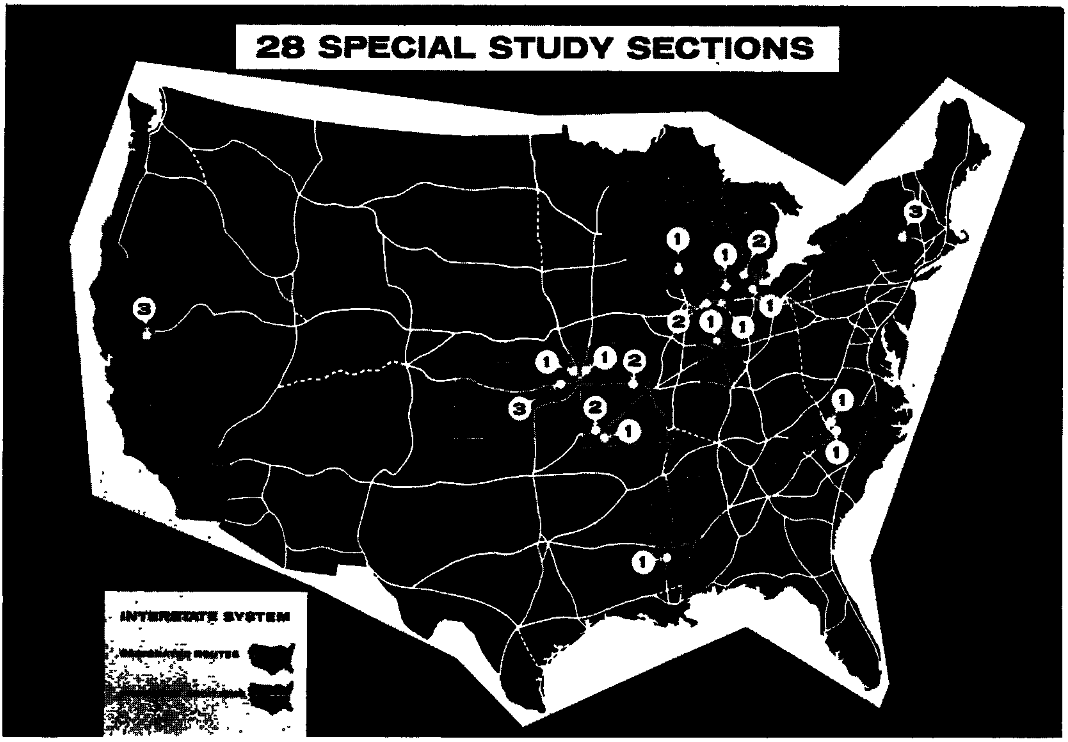

Figure 5.

Detailed pavement condition surveys are made on these sections periodically for a number of years until the performance trends have been established. The initial survey is made after paving and before the road is opened to traffic.

Since the oldest sections are located on projects built in 1954, definite performance trends have not yet been established. However, test results for most of the sections are available. They illustrate subbase conditions resulting from typical construction methods.

The uniformity of subbase density achieved from a given compactive effort depends to a large extent on the uniformity of subbase gradation. The prevention of segregation is extremely difficult for material with a very large maximum size. Figure 6 shows the large range of densities on a section with a bank-run gravel having a 4-in. maximum size. The wide band in Figure 7 represents the wide range in gradations of the subbase material on this section. The curves inside the band illustrate typical gradation variations between individual samples.

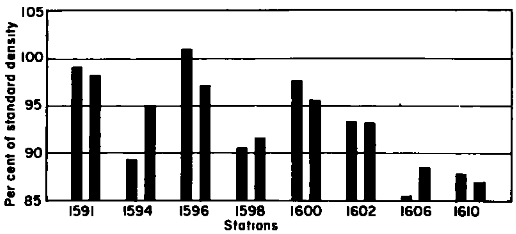

Figure 6.

Figure 8 shows the density test results for a section having a gravel subbase with a smaller maximum size. Densities are uniformly high except at Station 4179. The gradations of the samples from this section are practically identical, all falling within the narrow band in Figure 9, except at Station 4179 which was the location of the lower density. If all the material had the same gradation as that at Station 4179 the compactive effort and moisture content could have been adjusted to give uniformly high densities. However, since the material is different from the surrounding material, but received the same amount of moisture and compaction, the resulting density was different.

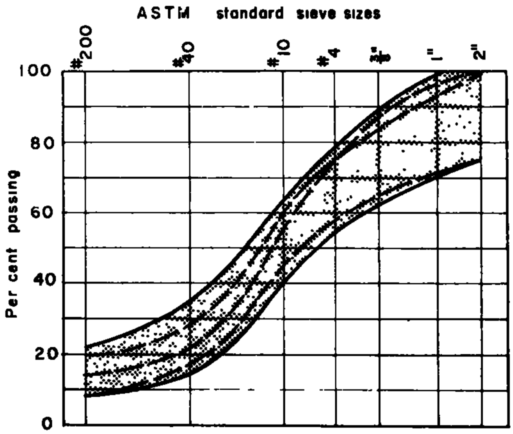

Figure 7.

Figure 10 shows uniformly high densities on a section where a similar material was used and the gradations of samples from this section (Fig. 11) are nearly identical, all falling within a narrow range.

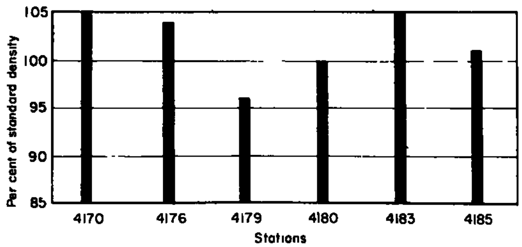

Figure 8.

SUBGRADE

If the subbase is to be compacted to a uniform density and provide uniform support for the pavement, it must lie on a uniform subgrade. Soft spots should be located by proof-rolling or other means, and then excavated and recompacted. If the soft spot consists of a pocket of soil which has sharply different properties from the surrounding soil, it should be replaced with the same type of soil as the adjacent subgrade. Even support cannot be obtained merely by dumping extra granular material on the soft spot as was done at the location shown in Figures 12 and 13.

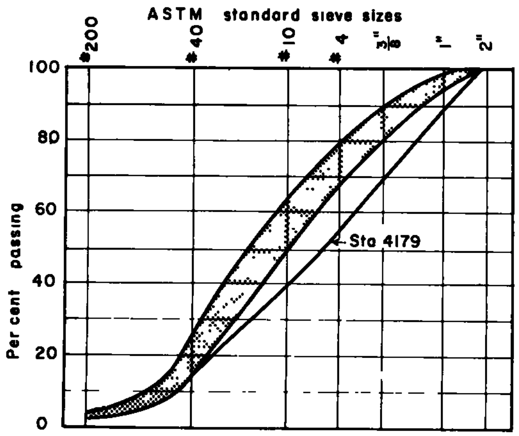

Figure 9.

One of the most important considerations in subgrade construction is moisture control. If the subgrade is too wet the subbase will be ground down into it during construction and construction traffic will cause rutting. If the subgrade is too dry, especially if it contains a significant amount of clay, it will become unevenly wet and soft during the first heavy rains after paving. The moisture content will in-

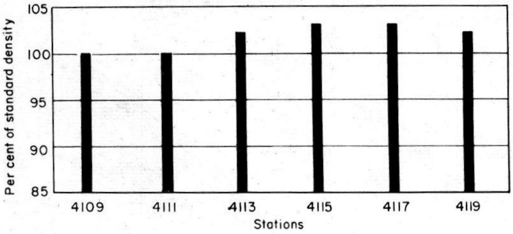

Figure 10.

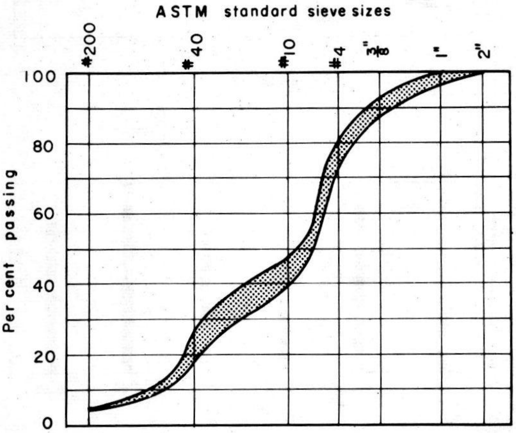

Figure 11.

crease more rapidly at the joints and at the pavement edges causing excessive softening at these locations, uneven support for the pavement and possible damage during its early life.

An example of this can be found in a group of projects with the same pavement and subbase design, built on similar soils and subjected to similar traffic, but in which individual projects are performing quite differently. Some of these projects were built during a period of prolonged drought and the subgrades were placed and compacted without adding water. The pavements have developed structural damage at an early age. The other projects in this group were constructed during periods of normal rainfall. They have no structural damage even though the pavements are older.

Soils containing appreciable amounts of clay will have the lowest permeability and retain the highest degree of stability when they are compacted at or slightly above the standard optimum moisture content (AASHO T99)[1]. Heavy clay soils tend to be somewhat rubbery at this moisture content, which sometimes causes difficulty in compacting an overlying thin layer of granular material to the required high density.

Figure 12.

Figure 13.

[1] In Highway Research Board Bulletin 58 "Compaction of Embankments, Subgrades, and Bases," page 12: "Soils compacted at optimum moisture content have lower permeability and a greater resistance to softening than dry soils at equal densities."

This problem has been successfully solved in many cases by the use of pan-type vibrators in compacting the subbase.

Subbase Moisture-Density

Granular subbases are susceptible to consolidation from vibration which occurs from traffic passing over the pavement. To reduce this consolidation to a minimum, the subbase should be compacted to as high a density as practicable during construction[2]. This is done most effectively when the material is at its optimum moisture content, which usually means that additional water will be required. In order to get uniform density this water must be thoroughly mixed into the material until it has a uniform moisture content. This mixing is also beneficial in reducing segregation.

When the water is added after the material has been spread on the grade, the mixing is effectively accomplished by the use of mortar graders or multiple pass type mixers. If the water is added on the grade and not mixed into the subbase, it will usually penetrate only about ½ in. even in fairly permeable materials.

The mixing of subbase materials and water in pugmills is becoming more widespread. The material is then placed evenly on the grade by aggregate spreaders. The slight additional cost of this method is often offset by the ease in placing and compacting the material.

Ten- to twelve-ton steel-wheel rollers are still extensively used for compacting subbases. They produce satisfactory densities on most granular materials except cohesionless sands. However, if the material has a large maximum size, these rollers tend to ride on the high spots, bridging and leaving loose material in the low spots. Rubber-tire rollers are effective in producing uniformly high densities by adjusting to uneven surfaces and local areas of loose material. They have a kneading action which knits the granular particles together into a dense mass. Heavy rubber-tire units weighing 30 to 50 tons or more have been used on some of the projects included in the special study section group. They are particularly useful in locating soft spots. Vibratory compacters, both the pan type and vibrating rolls, are now widely used to obtain high densities in many types of granular materials. Sometimes a combination of two types of compaction equipment is the most effective means of producing the required density.

FINE-GRADING

The difficulty in fine-grading increases as the maximum size of the subbase material increases. Pit-run gravel containing large stones is

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[2] In Highway Research Board Bulletin 58, page 24, the suggested range of densities for granular subbases for rigid pavements is 100-105 percent of AASHO maximum density when construction traffic does not use the prepared subgrade and 100-110 percent when construction traffic hauls over the prepared subgrade. In the paper "Performance of Subbases for Concrete Pavements under Repetitive Loading," by B. E. Colley and W. J. Nowlen, presented to the Highway Research Board in 1958, one of the conclusions of a laboratory study of the densification of granular subbase materials is that compaction to at least 100 percent of AASHO maximum density is required if densification under traffic is to be small.

Figure 14.

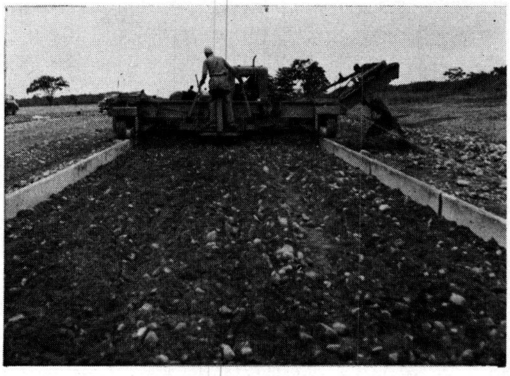


Figure 15.

particularly difficult to fine-grade. After the initial compaction of this type of material, the subbase often has to be loosened by scarification to a depth of about 4 in. and the larger stones near the surface must be removed by hand before the fine-grader can work on it (Figs. 14 and 15). After fine-grading, the large stones which have been brought to the surface have to be removed and the resulting depressions filled with finer material. With small-sized material, such as sand, fine-grading is often effectively accomplished by a motor grader or a tractor-drawn heavy straightedge riding on the forms.

The surface of the subbase is usually loosened and some loss of moisture occurs during fine-grading operations. Because of this, sprinkling and recompacting the top layer after fine-grading is required to restore uniform density to the full depth of the subbase. Generally the heavier compaction equipment is employed before the forms are set. After the surface has been disturbed by fine-grading, restoration of the initial high densities to the top 1 to 2 in. of subbase is often neglected.

When the full width of the subbase is left slightly high after initial compaction, the fine-grading is a cutting operation and the disturbance of the subbase surface and resulting variation in density, gradation and permeability is reduced. However, when the subbase is left high in the middle and low at the sides, variation across the width of the roadway is likely to occur since the edges are brought up to grade by filling in with loose material while the initial density is retained or increased in the center. This is especially true when the subbase immediately next to the forms is neglected during final compaction.

Final compaction is sometimes attempted by means of a series of small steel rollers attached to the rear of the fine-grading machine. This method does not always prove satisfactory in obtaining the required uniformly high densities in the subbase. The rollers are too light for effective compaction, and since they are attached to the fine-grader only one pass is made regardless of the density achieved.

One future maintenance problem associated with fine-grading is the handling of the excess subbase material. When a fine-grading machine is used the excess material is often deposited in a windrow just outside the side forms. If this material is not removed, or spread out and compacted during shoulder construction, it can lead to trouble at the pavement edge.

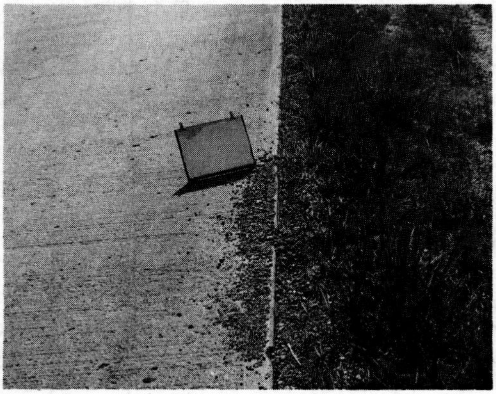


Figure 16.

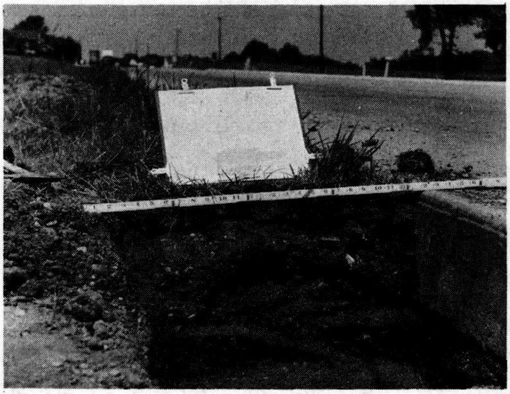


Figure 17.

Figure 16 illustrates one effect of water action which may occur when the windrow of excess subbase material is left in a loose condition during shoulder construction. Figure 17 shows the windrow exposed in a shoulder excavation at the same location. This excess material causes no problem if it is spread out and compacted. Some fine-grading machines deposit this excess material directly into trucks for re-use in another location.

One construction practice which makes it practically impossible to secure a uniform subbase is that of operating the paver between the forms. Not only does the paver tear up a well-built subbase, but the batch trucks necessarily drive over the completed grade. The batch trucks cause considerable rutting and segregation, particularly at turn-around locations and where the forms are removed for their entrance and exit as shown in Figure 18. In wet weather the trucks track mud onto the completed subbase. An example of this is illustrated in Figure 19. The batch trucks generally operate down the middle of the subbase, especially when they have to cross over the fine-grading machine. As a result this heavy traffic produces higher densities in the center of the roadway then the compaction equipment can achieve along the edges. This uneven density across the width of the roadway sometimes results in serious defects in the pavement, such as longitudinal cracking, due to differential support.

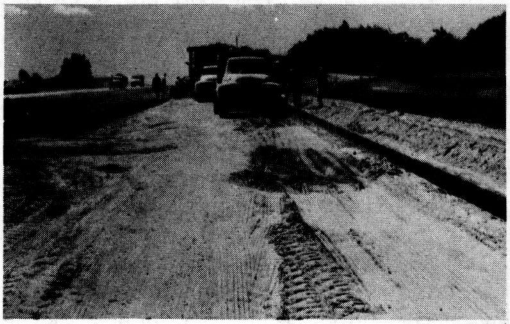


Figure 18.

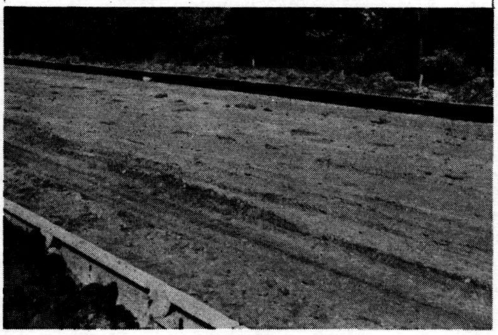


Figure 19.

SUMMARY AND CONCLUSIONS

Uniformity is the key to building a subbase which will enable the pavement to retain its original smoothness for many years. To provide a uniform support for the pavement the subbase must lie on a uniform subgrade and have uniform density, gradation and permeability when the pavement is placed, and must be designed and built so that it will retain these uniform qualities throughout the life of the pavement.

The following construction procedures have been found to contribute towards providing uniform pavement support:

1. Selective grading and cross hauling results in more uniform conditions in the upper part of the subgrade.
2. Close moisture control during grading operations, keeping the compaction moisture content at or slightly above optimum (AASHO T99), provides a more uniformly stable foundation. When the subgrade has dried out before the subbase is placed, loosening of the top 6 in. followed by recompaction at the proper moisture content reduces damage due to differential support caused by uneven softening during the first wet weather after paving.
3. Proof-rolling by heavy rollers locates soft spots in the subgrade. Excavation and recompaction at these locations and replacing or mixing pockets of soil having dissimilar characteristics controls damage from differential soil volume changes.
4. The use of subbase material with a maximum size of about $1\frac{1}{2}$ in. or less reduces segregation and facilitates placement, compaction and fine-grading.
5. Thorough mixing of the subbase material in pugmills or on the grade reduces differential support due to uneven density, moisture content and permeability caused by segregation. By adding the required amount of water for effective compaction during the mixing operation, the moisture content will be uniform for the full depth and width of the subbase during compaction.
6. Compacting the subbase to the highest practicable uniform density reduces consolidation due to traffic vibration. Close moisture-density control is needed after fine-grading as well as before.
7. Realignment of forms which have settled during wet weather prevents variations in subbase thickness due to the removal of excessive amounts of subbase material during fine-grading.
8. Removing or spreading and compacting excess subbase material deposited along the edge of the pavement by the fine-grader reduces excessive water action at that location.
9. Mixing and recompacting a subbase which has been used as a detour or haul road restores the uniformity lost due to heavy traffic passing over the unprotected subbase. Keeping all traffic off the grade following final compaction insures that the benefits to be derived from good subbase construction will not be lost. It is practically impossible to provide uniform support for the pavement when paver and batch trucks are operated between the forms.

In the final analysis it is the combination of pride in workmanship, good inspection and a thorough knowledge of local pavement performance which has the greatest influence on improving the quality of construction and future pavement performance. A regular check of pavements both under construction and in service is the only way to determine what construction methods are best suited for overcoming problems which are associated with local conditions.

Joint Construction in Concrete Pavement

GORDON K. RAY, Manager, Highways and Municipal Bureau,
Portland Cement Association

Research and design engineers have developed a vast storehouse of information on performance of various types of joints in portland cement concrete pavements. The performance of such joints depends to a large degree on the type and quality of their construction. Careless construction practices or incorrect methods of construction resulting from a lack of knowledge of the joint function results in pavements with below standard riding characteristics and maintenance problems. Correct construction procedures for various joint types are outlined and illustrated. Installation of dowels, tiebars and expansion joint fillers is explained.

● DURING THE last two decades, design engineers, research engineers and materials engineers in our highway departments have learned much about the performance of joints in concrete pavements. Laboratory studies, experimental pavements and condition surveys of existing pavements have enabled them to design and specify simpler, better joints for concrete. Construction engineers have not always attached sufficient importance to the precision of joint construction methods to assure that they will function as designed.

There are four general types of joints in use today, each with a different purpose. All will perform properly in pavement if they are properly constructed. To insure proper construction of joints, the construction engineer, the resident and the inspector should understand their function. If they understand how a joint should work and why the design engineer selected a particular type of joint for a particular location, they will do a better job of construction supervision. If they in turn explain to the contractor's personnel how a particular joint will function, it is more apt to be built as planned.

The four joint types, their purpose and design will be discussed. Then each joint type is taken up and satisfactory construction procedures outlined. In some cases there may be several different methods of construction which will produce a suitable joint. Wherever possible more than one acceptable method will be discussed.

JOINT TYPES

All concrete pavement joints can be grouped into one of the following general types: longitudinal, contraction, expansion, and construction. Each has a different but specific function if the pavement, highway or airfield is to give the most satisfactory performance under traffic. All joints connect individual slabs to form a pavement and all permit some type of movement. Slab edges represent structural weaknesses in the slab and joints must be designed to strengthen these edges and prevent differential vertical movement. For this reason nearly all joints are provided with some type of load transfer between slabs.

Longitudinal center joints are joints in the direction of paving and are provided in all street and highway pavement built in lanes over about 15 ft wide. They are also used in some airfield pavement but may be omitted in thicker pavements by some engineers. These center joints are intended to relieve the transverse stresses which develop from wheel loads and slab curling or warping due to moisture and temperature differentials in the pavement and variable subgrade support resulting from soil swelling or shrinking.

These center joints are not intended to open and close. To prevent such horizontal movement they are usually provided with deformed tiebars embedded in both slabs across the joint (Fig. 1). Load transfer between lanes is usually provided by the aggregate interlock which develops below the surface groove. Some states have used a deformed metal plate for this joint to provide a keyway or tongue and groove for load transfer. Some engineers refer to these center joints as hinged joints. In some states using lane-at-a-time paving, construction joints take the place of these center joints.

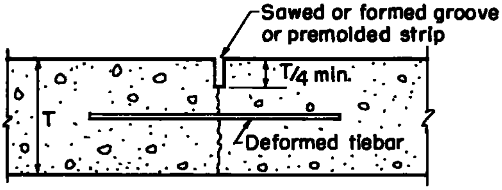


Figure 1. Longitudinal center joint.

Contraction joints are transverse joints used to relieve longitudinal stresses due to contraction as the concrete cools and loses moisture. Contraction joints also relieve longitudinal stresses due to loads and curling or warping and control the location of transverse cracking if properly spaced. Some engineers refer to these as cracker joints, plane of weakness joints, or dummy grooves. They all relieve contraction stresses in the concrete. Since contraction joints generally open somewhat as the slab cools and hardens, some space develops at each joint for later pavement expansion. If all foreign material is kept out of these openings, the contraction joints also serve as expansion joints, making regularly spaced expansion joints unnecessary.

Load transfer to strengthen transverse slab edges and prevent differential movement under traffic is provided by means of aggregate interlock or by means of dowels or other mechanical load-transfer devices (Fig. 2). The need for dowels is determined by the design engineer and is based upon joint spacing, traffic, subgrade support and other factors.

No tiebars are used, since they would prevent pavement contraction and free longitudinal movement at these joints. If dowels are provided for load transfer, they must be coated with a bond breaker to permit free horizontal movement in the pavement. Some engineers carry distributed steel or deformed tiebars through certain transverse joints and discontinue it at others. This restricts longitudinal contraction at these so-called warping joints and causes larger movement at the other joints where dowels are normally provided.

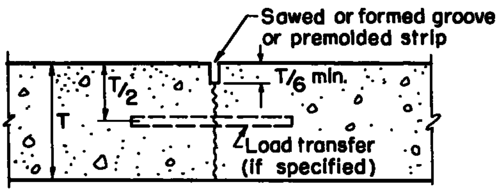


Figure 2. Contraction joint.

Expansion joints are usually transverse joints used to relieve expansion stresses in the concrete by providing room for expansion. An expansion joint is filled with a nonextruding, compressible material. The filler must have sufficient strength partially to resist horizontal slab movement but to permit such movement before crushing or buckling stresses developed in the concrete.

Engineers have found that closely spaced contraction joints provide adequate space for expansion under normal conditions and, therefore, regularly spaced expansion joints are no longer used in most pavement. They are still used, however, adjacent to most bridges and other fixed structures and at certain intersections. They may also be used in the longitudinal direction in certain wide pavement areas such as airfields to protect hangars and drainage structures from expansion stresses in the transverse direction.

In these joints, the expansion filler prevents aggregate interlock from serving as load transfer so dowels or other load transfer devices must be provided between slabs (Fig. 3). These dowels must be free to slip in the concrete to permit horizontal movement in the joint. A recess or socket must be provided at one end of the dowel equal to the thickness of the filler if the joint is to be able to function properly. In some cases where dowels are not used as load transfer devices at expansion joints, the slab edges adjacent to the filler are thickened at this location. While this design may provide the extra strength usually furnished by dowels, it does not prevent differential vertical movement at the joint.

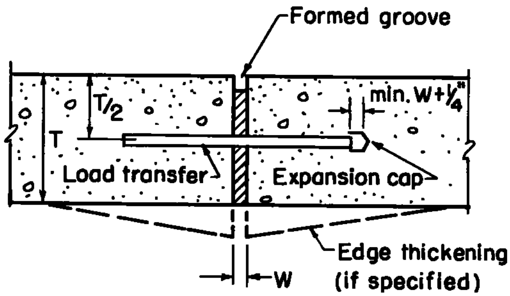


Figure 3. Expansion joint.

Construction joints are transverse header joints put in at the end of each day's run or longitudinal joints between lanes of multiple lane pavement. The purpose of such joints is to divide large pavement areas into convenient sizes for paving. Longitudinal construction joints are usually provided with deformed tiebars or tiebolts to prevent horizontal movement and keyways or tongue and grooves built into slab edges to provide load transfer between lanes.

A transverse construction joint may serve as a contraction or expansion joint if its location coincides with that of a planned transverse joint. If it is to be a contraction joint, a butt-type joint is formed by the header or transverse form and dowels are used for load transfer across the joint. If the joint is to be an expansion joint, a filler is placed against the temporary header. Transverse construction joints which do not occur at regular joint locations

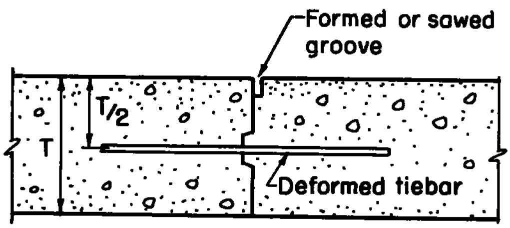


Figure 4. Construction joint.

are generally tied with tiebars to prevent movement (Fig. 4). This is imperative in multiple lane pavements. Keyways may be provided in such cases to insure load transfer since tiebars alone are not adequate for this purpose across butt joints.

CONSTRUCTION PROCEDURES

There are a number of methods of constructing each joint type which are satisfactory if proper attention is paid to each detail of construction. Some of the more common acceptable methods of joint construction are discussed. The method selected must result in a durable, smooth-riding joint which will function as intended without spalling, cracking or differential vertical movement.

Longitudinal center joints are nearly always some type of surface groove. These grooves reduce the pavement cross-section and the resulting plane of weakness in the slab will result in a crack below the groove as transverse stresses develop. The depth of groove must be equal to at least one-fourth of the slab thickness to control longitudinal cracking.

The best method of forming these grooves is by sawing the hardened concrete (Fig. 5). This may be done any time before the pavement is opened to traffic. Sawing may be done wet or dry with diamond or abrasive blades, depending on which method is more economical for the particular aggregate. Usually the center joint can be flushed out, dried and sealed immediately after sawing, eliminating a second cleaning. Self-guided saws are excellent for sawing center joints.

Figure 5.

If a premolded bituminous strip is used to form the surface groove, it must be of the proper dimension and must be vertical and flush with the surface to prevent spalling of the concrete. If it is installed ahead of the finishing machine and longitudinal float, proper alignment is difficult to maintain. If it is installed behind the mechanical finishing equipment, the surface over the joint must be straightedged to remove the bump created during the formation of the groove by a vibrating bar or cutting wheel.

Full-depth deformed metal plates have limited use today. They provide excellent load transfer through the keyway provided by the plate but they must be buried slightly to permit proper finishing. If the plate is too deep unsightly spalling and raveling of the joint may result. A depth of 1/8 in. below the finished surface should be the maximum permitted.

Tiebars across the longitudinal center joint may be supported on chairs driven into the subgrade or may be inserted in the concrete just behind the spreader. This may be done by hand using a simple improvised device or automatically by a wheel which inserts the bars at the proper spacing as the spreader moves forward. The tiebars must be ungreased, of the proper dimensions and placed at the proper intervals shown on the plans. Absolute accuracy as to level and alignment is not critical with tiebars.

Contraction joints are also of the surface groove type. Since these transverse grooves relieve longitudinal contraction stresses which develop during early slab hardening and cooling, while the concrete is relatively weak, the reduction in cross-section does not have to be so great as in longitudinal center joints. Experience has shown that a depth of groove equal to one-sixth of the slab thickness will generally control all transverse cracking.

The best method of forming these contraction joint grooves is by sawing the hardened concrete (Fig. 6). Since contraction stresses develop as soon as the concrete hardens, these joints must be sawed very early. The exact time of sawing depends upon the type of aggregate, curing, meth-

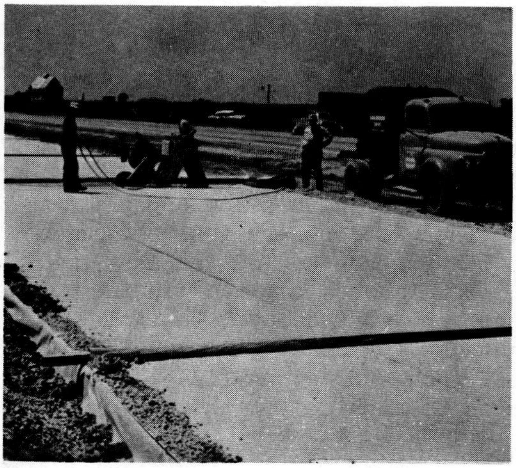


Figure 6.

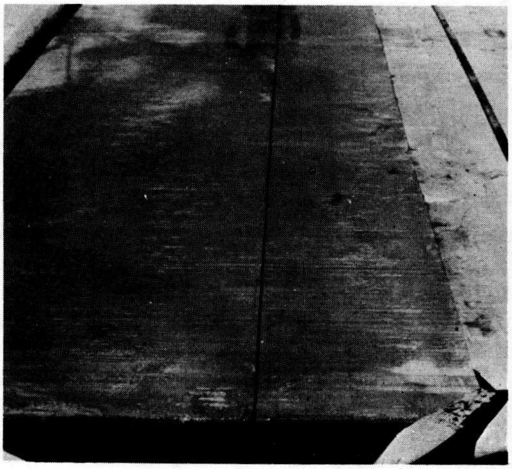


Figure 7.

Figure 8.

Figure 9.

od, cement factor and weather (Fig. 7). Generally all joints should be sawed as soon as possible without damage to the surface. A slight amount of raveling is permissible and desirable since it gives the operator a good gauge of his timing (Fig. 8). If there is no raveling at all, the concrete is too hard and cracks may develop ahead of the saw. All joints should be sawed in succession to provide the plane of weakness at the time of maximum contraction and before the slab gains too much strength. This will insure cracking and uniform opening at all joints. Sawed joints should be thoroughly flushed out immediately after sawing to remove all residue (Fig. 9).

The location for sawing transverse joints may be marked by snapping a string line on the concrete during final finishing operations. The choice of saw blade type and the decision to saw wet or dry should be based on economy and will depend on many local factors. The width of cut should be a decision for the design engineer based on joint spacing and anticipated joint opening. If a crack should develop at the approximate joint location, or if the slab cracks ahead of the saw, sawing should be omitted at that joint location. The crack will function as a contraction joint.

In some areas where very hard coarse aggregates are used, sawing costs may be prohibitive. In these areas, preformed inserts are frequently specified to reduce sawing costs. These inserts may be made of corrugated, paraffin-treated paper or premolded bituminous strips similar to expansion joint fillers. Boards of cane fiber with a low asphalt content have been most successful to date. Those strips are inserted in a groove formed by vibrating a T-bar into the surface behind the last mechanical finishing equipment (Fig. 10). The insert is placed in the groove slightly below the surface and then the surface over the joint must be finished with a scraping straightedge. A crack will develop below the insert since it functions as a surface groove. A fine crack will also develop above the insert which serves as a guide for the saw oper-

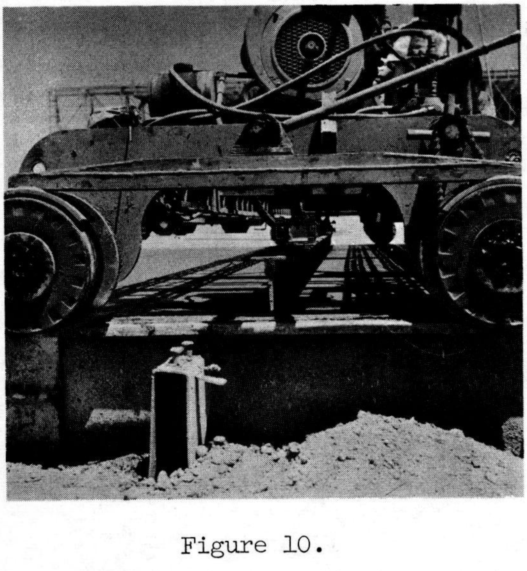


Figure 10.

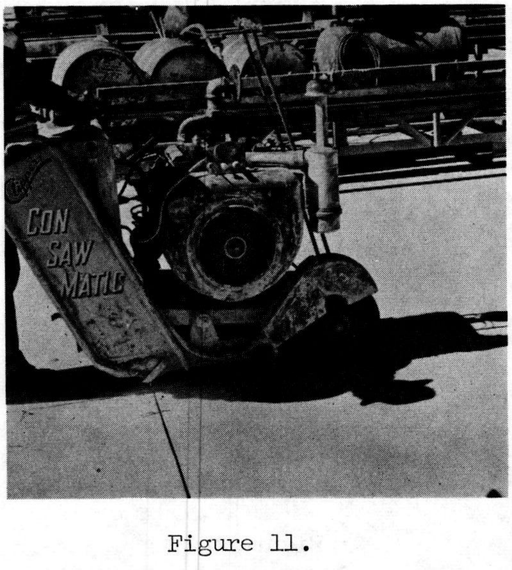


Figure 11.

ator. The sawing out of the insert can be done dry using an abrasive-type blade slightly wider than the insert thickness to remove all paper from the sides of the joint. If bituminous impregnated strips are used, only the top 1 to $1\frac{1}{2}$ in. need be removed for sealing. This type of joint acts as positive crack control and sawing can be delayed until after all paving is completed and the contractor is ready to seal joints (Fig. 11).

One or two highway departments use a premolded bituminous strip which is left in place without any sawing or sealing. This strip also must be placed in a groove made by vibrating a T-bar in the concrete behind all mechanical finishing. The surface over the joint must be carefully straightedged to remove any bump adjacent to the strips. These strips must be flush with the surface to be successful. They must also be vertical and continuous from form to form. If they are buried too deep, if they are tipped, or if gaps or offsets exist between strips, spalling results later.

Dummy grooves hand formed in the plastic concrete are no longer used extensively. They are difficult to build properly. Bumps, spalling and lack of durability are all too common on hand-formed joints. If they are specified, the template used to maintain the groove must be clean and well oiled after each use. It must be removed early enough to prevent damage to the slab. Hand formed joints should be edged with a wide-flange double edger and the surface must be checked with a long straightedge to insure a smooth riding joint (Fig. 12). Edging and hand finishing must be held to a minimum to prevent overworking.

Both the dummy groove and the unsawed, premolded strip types of contraction joints are more difficult to build properly than sawed joints and should only be used when sawing is not permitted. They require much more attention and inspection to insure a surface which is smooth riding and free from spalling or other defects. Overworking of hand formed joints frequently results in mortar concentrations which lack durability and strength.

When dowels are specified in contraction joints, they are generally placed in baskets or assemblies on the subgrade at the joint location prior to placement of the concrete. The baskets must be securely staked to the subgrade to prevent displacement during paving. The basket must be rigid enough to maintain proper dowel alignment and level during paving. Baskets or assemblies which are found to be inadequate must be rejected by the engineer.

Dowels must be of the proper dimensions and spacing and they must be uniformly greased or painted as specified to prevent bond and insure a free-moving joint. They must also be parallel to the centerline and surface of the pave-

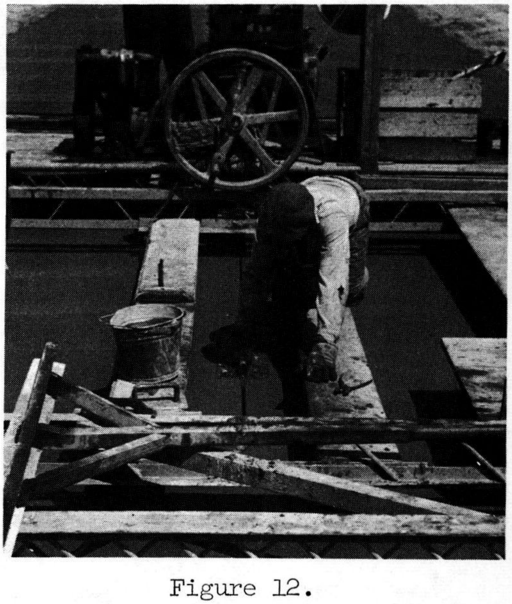


Figure 12.

ment or restraint to slab movement will develop. Proper alignment can be checked at each end with a specially prepared template. Proper level should be checked frequently, using a special level with adjustable legs which can be used on the individual dowels. It is first set on the form at the joint location to adjust the legs so that the level bubble is centered. It can then be used on the dowels to determine whether they are parallel to the surface. This device should be used before concrete is placed and occasionally after machine finishing operations to determine whether or not the dowel assembly does hold the dowels in proper alignment.

To prevent unnecessary dowel displacement, some care must be taken in placing and spreading concrete. The operator should not be permitted to dump concrete directly on the dowels and workmen should not step on them during paving operations.

Expansion joints require that the expansion filler be installed vertically. It must be continuous from slab edge to slab edge with no gaps or offsets between adjacent pieces. To insure proper placement, the filler must be staked securely to the subgrade. It should be shaped to the subgrade or placed in a shallow trench to prevent any concrete from flowing under the filler. Any concrete which bridges the expansion gap will prevent free movement of the joint. If a metal keyway is attached to the side forms, the filler must also be shaped to fit this form.

Since most plans call for the expansion filler to be recessed 3/4 to 1 in. below the pavement surface, some type of installation guide or cap is normally used in construction. This may be a metal channel which fits over the filler or a wooden strip of the same width nailed temporarily to the filler. In any case, the cap is usually close to the surface so that it can be found after the final machine finishing operation. These installation caps must be removed as soon as possible, without damaging the concrete, after it has set sufficiently to prevent slumping into the groove above the joint filler.

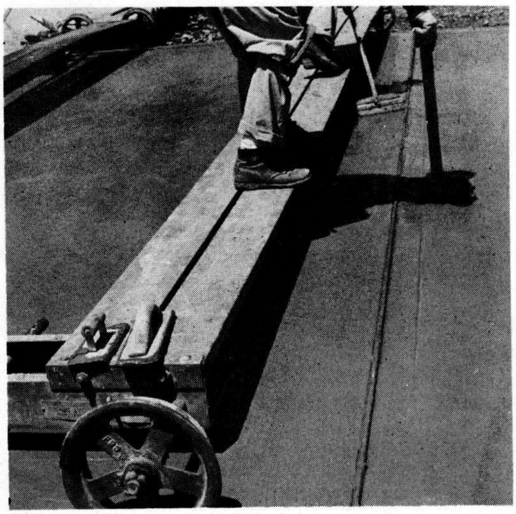


Figure 13.

The cap should be strong enough to maintain the filler in a straight line and it must be cleaned and oiled after each use to facilitate removal. It is normally raised partially to permit edging of the concrete on both sides with a wide flanged double edger and then completely removed. The groove above the filler must be carefully inspected to see that it is as wide as the filler and to insure that there is no concrete bridging above the filler. The groove should be inspected again after side forms are removed to see that there are no plugs of concrete in the expansion space. Such plugs cause spalls in the slab when the joint attempts to close.

The final operation in expansion joint construction is a careful surface check with a straightedge to see that no hump or depression has been created during the edging operation. As in contraction joint construction, any smooth surface left by the edging tool should be roughened with a broom or burlap drag to match the rest of the surface texture (Fig. 13).

Construction joints between lanes or at the end of a day's run also require some attention from the inspector if they are to function properly. Keyways are formed in the edge of the first lane by attaching a metal keyway of proper dimensions to the form at the midpoint of the slab depth. The keyway form should be oiled prior to paving to facilitate removal. If tiebars are used across construction joints, they are usually bent so that one-half projects into the first lane to be paved and the other half lies between the keyway form and edge form. After form removal the keyway is removed and the bent portion of the tiebar is straightened for embedment in the second slab. Tiebolts are made so that the first half is attached to the form by bolts. Frequently these bolts are also used to attach the keyway to the form. After forms have been removed, the second half is attached to the portion embedded in the first lane.

Specifications usually call for slab edge vibration to insure adequate consolidation along the keyway, thus creating a uniformly strong tongue and groove to provide proper load transfer. Both edges of a construction joint should be edged during finishing operations. Special care must be taken during finishing of the second lane to prevent any overhang on the adjacent lane which would spall off under traffic. To prevent this and provide a recess for sealing material along the longitudinal joint, plans should call for a definite groove at this location. Such a groove may be formed during the edging of the second lane or by sawing the groove along the joint after the second lane has hardened.

If dowels are specified across construction joints, they are normally installed by drilling holes of proper dimension in the forms at the specified spacing. The dowels then project through the form, half into the concrete and the other half outside the form. They should be supported to insure proper alignment by welding supporting brackets on the outside of the form. In doweled longitudinal construction joints, form removal may be facilitated by giving each dowel a twist before final hardening of the concrete. The half of the dowel embedded in the first lane must, of course, be properly coated with grease or paint. After hardening, the dowel can then be removed. It is replaced after completion of form removal and fine grading in the adjacent lane. Sectional dowels or plastic dummies which can be attached to the forms are now in use on some airfield projects where doweled longitudinal construction joints are specified.

SUMMARY

Construction of all joints in concrete requires attention to details. Joints are an important part of the pavement and are designed to control cracking and prevent excessive stresses from developing. They must be built properly if they are to do the job and not be a source of trouble to the motorist and the maintenance engineer. Proper joint construction should be demanded, giving the public what it pays for—smooth-riding durable joints free from bumps, spalls, and maintenance problems.

A Theoretical Approach to Design of a Road Joint Seal

EGONS TONS, Research Engineer,
Joint Highway Research Project,
Massachusetts Institute of Technology

● THE SEALING of pavement joints is a common practice. It is done both in new concrete road construction and during subsequent maintenance in the form of joint resealing and crack filling. Even after an old portland cement concrete roadway is covered by a bituminous resurfacing, sealing usually has to continue because of reflection cracks.

A pavement joint undergoes continued changes in width due to such influences as temperature and moisture fluctuations within the concrete slabs. When a joint (or crack) is filled with a sealing compound, strains and stresses caused by every opening or closing movement must develop both within the sealer and along the bond interfaces between sealer and pavement. The mass of sealer in the joint can be pictured as a sort of bridge spanning the gap between two slabs. Just as we try to design a bridge before it is built by using known quantities and methods, we should attempt to design a seal before it is placed in the joint.

The need for a more exact approach to joint sealing has been expressed by various individuals and organizations associated with road construction and research. The 1953 Committee on Joint Materials in Concrete Pavements, in HRB Bulletin 78, states the case as follows:

> "There should, in fact, be a proper relationship between: (1) the amount of change in joint width, (2) the capabilities of the sealing material, and (3) the width of the joint space."

Work on this problem started with a simple joint model test. It was soon discovered that the depth of seal in the joint is another equally important variable.

PURPOSE OF THE STUDY

The purpose of this study was to correlate mathematically and by experiment (a) the joint width, (b) sealed depth, (c) joint expansion, and (d) the extensibility of the sealer, and then hopefully to outline a joint seal design procedure for future practical use in the field.

This paper describes the theoretical analysis and computations of strains in a sealed joint and presents experimental laboratory data to verify the basic theory and to point out its limitations. An approach to a practical joint seal design procedure is given in Appendix B.

THEORY OF JOINT SEAL BEHAVIOR

Basic Assumptions

The theory presented in this paper is based on maximum strain calculations in the sealer due to joint width variations. The following assumptions were made to facilitate the analysis:

The joint cross-section is rectangular (see Fig. 1).

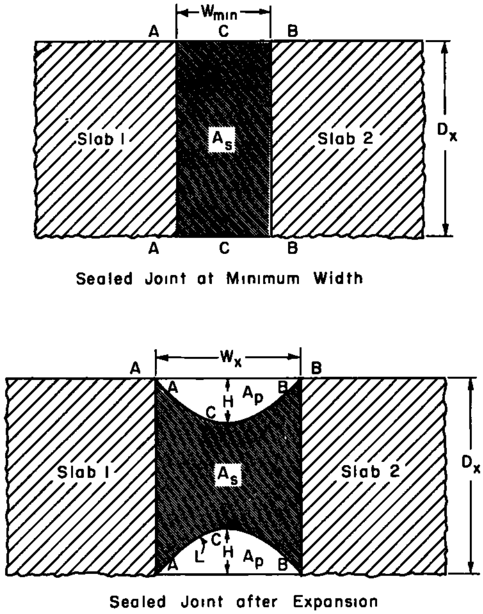
The sealer is a liquid-type homogeneous compound which cannot change in volume but instead changes its shape when the joint varies in width (see Fig. 1).

The majority of sealing compounds currently used fall closely within this group. They show little if any change in volume when extended or compressed.

The curve-in top and bottom surfaces of the sealer resulting from joint expansion are parabolic in shape (see Fig. 1).

This assumption is based both on observations of pavement joints and measurements in laboratory bond-ductility tests. These show that the curve-in line seems to correspond quite closely to a parabola for a wide range of sealed widths and expansions (this is discussed in "Laboratory Tests to Verify Theory and Calculations").

The sealer curves in equally from the top and the bottom of the joint (see Fig. 1).



Key
- W_{min} = minimum joint width
- W_x = joint width of any extension
- D_x = depth of sealer in the joint
- H = maximum depth of the parabolic curve-in line
- L = length of the parabolic arc (line ACB)
- A_s = cross-sectional area of the sealer
- A_p = area of the parabola ABC
- ΔW = amount of joint expansion, in percent
- S_{max} = amount of maximum strain in the sealer, in percent

Figure 1. A sealed joint before and after expansion.

In many cases the bottom of the joint contains foreign matter which prevents adhesion. As will be seen in later calculations, there is a definite advantage in having the sealer curve in from top and bottom (see "Procedure of Strain Computations"). In order to prevent adhesion and to control the depth of seal, appropriate filler materials will have to be used in the bottom of the joint.

The minimum and maximum joint widths are the indicators of the total strains in the sealer, no matter what the width of the joint when it is first sealed.

The minimum joint width has a significant effect on future strain in the sealer. If joints are sealed in fall during moderate temperatures, the compound will not be stretched much the first winter. The following summer the joint will narrow down to its minimum width expelling some of the compound. From then on this smaller volume of sealing material will have to keep the joint sealed at all its various widths.

The strain in the sealer along the parabolic curve-in line is uniformly distributed.

According to observations in laboratory tests, this assumption holds reasonably true for a wide range of joints at various stages of expansion (see "Laboratory Tests to Verify Theory and Calculations").

Notation

Most of the symbols used in this paper are shown in Figure 1. In order to provide a complete list, all of them are summarized below;

W_{min} - minimum joint width
W_x - joint width at any extension
W_{max} - maximum joint width
W_f - joint width at the time of failure in a bond-ductility test
ΔW - linear expansion or change in joint width, in percent

$$\frac{W_x - W_{min}}{W_{min}} \times 100$$

D_{min} - minimum depth of seal
D_x - any sealed depth
D_{max} - maximum depth of seal
H - maximum depth of the parabolic curve-in
L - length of the parabolic arc
S_{max} - maximum total strain in the sealer along the parabolic curve-in line, in percent

$$\frac{L - W_{min}}{W_{min}} \times 100$$

A_s - cross-sectional area of the sealer
A_p - cross-sectional area of the parabola ABC (see Fig. 1)

Minimum Joint Width and Strains in Sealer

For like conditions, the wider the joint at its minimum width, the less the sealer filling will be strained.

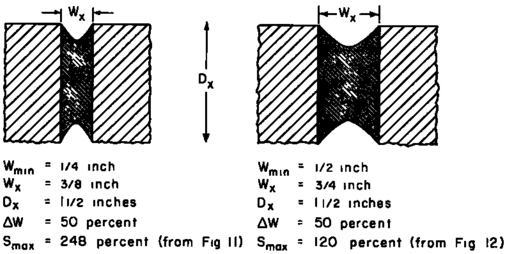


Figure 2. Comparison of maximum strains (S_{max}) in the sealer for two joint widths (W_{min}) with 50 percent expansion (ΔW = 50).

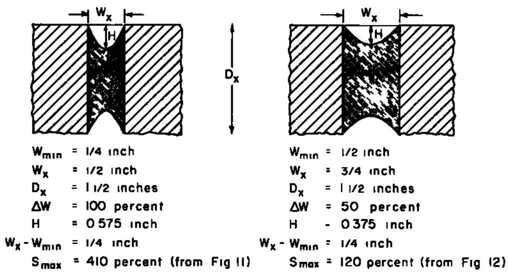
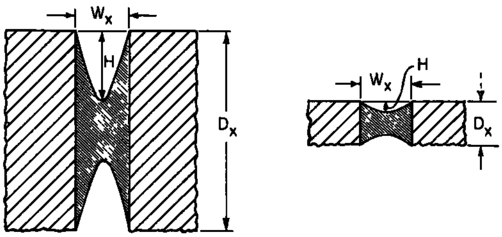


Figure 3. Comparison of maximum strains (S_{max}) in the sealer for two joint width (W_{min}) with $\frac{1}{4}$ in. expansion.

The amount of strain in a common liquid-type sealer is not directly proportional to joint expansion because the surface of the sealer curves down in the joint and does not stretch in a straight line (see Fig. 1). Furthermore, the narrower the joint, the more severe the strain forces for any given percent of joint expansion (see Fig. 2; calculation procedure will be given later). If a narrow joint expands as much as a wider joint, remarkable strain differences will result (see Fig. 3).

Influence of Sealed Depth

While the effect of joint width upon the performance of a seal has been recognized for a long time, the equally important depth of seal has on the whole been left unnoticed. In fact, it is commonly assumed that the deeper a joint is sealed the better. Theoretical calculations indicate the opposite is true; i.e., the shallower the seal, the less "curve-in" and the smaller the strains in the sealer (see Fig. 4). This appears to be confirmed by laboratory tests (see "Laboratory Tests to Verify Theory and Calculations").



W_{min} = 1/4 inch	W_{min} = 1/4 inch
W_x = 1/2 inch	W_x = 1/2 inch
ΔW = 100 percent	ΔW = 100 percent
H = 0.75 inch	H = 0.19 inch
D_x = 2 inches	D_x = 1/2 inch
S_{max} = 550 percent (calculated)	S_{max} = 160 percent (from Fig 11)

Figure 4. Comparison of maximum strains (S_{max}) for 2-in. deep and $\frac{1}{2}$-in. deep seals (D_x = 2 in. and D_x = $\frac{1}{2}$ in.).

Procedure of Strain Computations

Figure 1 shows the shape of the sealer cross-section before and after extension. If the joint width before expansion is W_{min} (minimum width) and after expansion becomes W_x and the joint has been sealed to a depth D_x, the increase in the joint cross-sectional area is $(W_x\text{-}W_{min}) \times D_x$. Since the sealing material acts like a liquid and is not able to change its cross-sectional area A_s, the two parabolic areas A_p will be equal to this increase (see Fig. 1) and the maximum curve-in value H can be calculated as follows:

$$2A_p = (W_x - W_{min})D_x$$

$$A_p = \tfrac{1}{2}(W_x - W_{min})D_x$$

but A_p-2/3 HW_x (equation for a parabolic area)

from which $H = 3/2 \dfrac{A_p}{W_x} = \dfrac{3/4\ D_x\ (W_x - W_{min})}{W_x}$

where H = the maximum curve-in distance
A_p = area of one of the parabolas (area ACB)
W_x = the width of the sealed joint after expansion
W_{min} = minimum joint width

Any cross-section of a sealed joint can be divided into numerous layers. If the width of the joint is increased, the outer layers which follow the parabolic curve will be stretched most (see Fig. 5). The length of this outer skin can be computed by using the formula for the arc length of a parabola:

$$I = \tfrac{1}{2}\sqrt{W_x^2 + 16H^2} + \frac{W_x^2}{8H}$$

$$\log_e \frac{4H + \sqrt{W_x^2 + 16H^2}}{W_x}$$

where L = length of arc ACB (see Fig. 1)
H = the maximum curve-in distance
W_x = the width of the sealed joint after extension

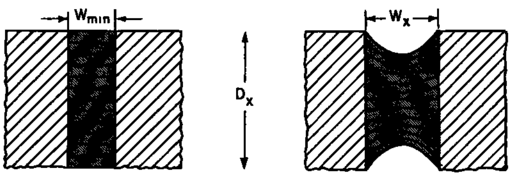


Figure 5. Visual strain comparison in the sealer at different levels for W_{min} = $\frac{1}{2}$ in. and W_x = 3/4 in.

Once the length of the curve-in line is known, the actual maximum strains in the surface of the sealer under various conditions can be calculated:

$$S_{max} = \frac{L - W_{min}}{W_{min}} \times 100$$

where S_{max} = maximum strain in the sealer, in percent
L = length of arc ACB (see Fig. 1)
W_{min} = minimum joint width, also equal to minimum L

The calculated maximum strain S_{max} in the sealer under ideal conditions depends, (a) upon the minimum joint width, (b) the amount of Joint expansion, and (c) the depth of seal. The numerous and repetitious calculations to correlate these factors were done by an electronic computer and the results were compiled in curve form1/.

Discussion of Theoretical Curves

By this procedure, the maximum strains in the sealer can be computed for any combination of depth of seal, joint width and joint expansion up to 200 percent2/; nine sets of curves are included for illustration (Figs. 6-14). They show that strains in the sealer can be decreased by either increasing the minimum joint width or by decreasing the depth of seal.

Figures 6 to 9 show the variation of maximum strain (S_{max}) occurring along the parabolic curve-in line at various joint openings for eight minimum joint widths (W_{min}) and four different seal depths (D_x). Thus Figure 8 gives comparisons for joints which have all been sealed to a depth of 2 in. (quite a common practice). The curves indicate considerable differences in strain developments. For example, a joint with a minimum width of 1 in. (W_{min} = 1) expanding 50 percent, or an additional $\frac{1}{2}$ in., will induce an 87 percent strain in the outside layer of the sealer (S_{max} = 87). If, similarly, a $\frac{1}{4}$-in. joint ($W_{min} = \frac{1}{4}$) opens 50 percent, or 1/8 in., the maximum strain in the sealing compound would be about 342 percent (S_{max} = 342) which is nearly four times as much. The differences in strain could be more surprising if the $\frac{1}{4}$-in. joint would have to take the same expansion ($\frac{1}{2}$ in.) as the 1 in. wide. In this case the strain induced in the outside skin of the sealer would be 780 percent or about nine times as much as in the wider joint.

Figures 10 to 13 show the variation of maximum strains (S_{max}) in the sealer with change in joint width for eight sealed depths (D_x) and four minimum joint widths (W_{min}). For example, if in Figure 11 the maximum strains (S_{max}) in the sealer are compared for sealed depths of $\frac{1}{2}$ and 2 in. at 50 percent expansion the same 87 percent versus 342 percent strains are found, just as in the previous example.

Figure 14 presents the calculated curves in a compounded form correlating ΔW, W_{min}, D_x, and S_{max}. To get it, Figure 13 (W_{min} = 1) was expanded and a ratio D_x/W_{min} was introduced which makes this figure valid for various joint widths and depths of seal. Thus, if the stretchability

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1/ A fully derived equation for the maximum strain in the sealer (S_{max}) along the surface is given in Appendix E.

2/ Theoretically, the two parabolas (see Fig. 1) intersect at 200 percent of joint expansion no matter what joint width or depth is taken. A mathematical proof is given in Appendix E.

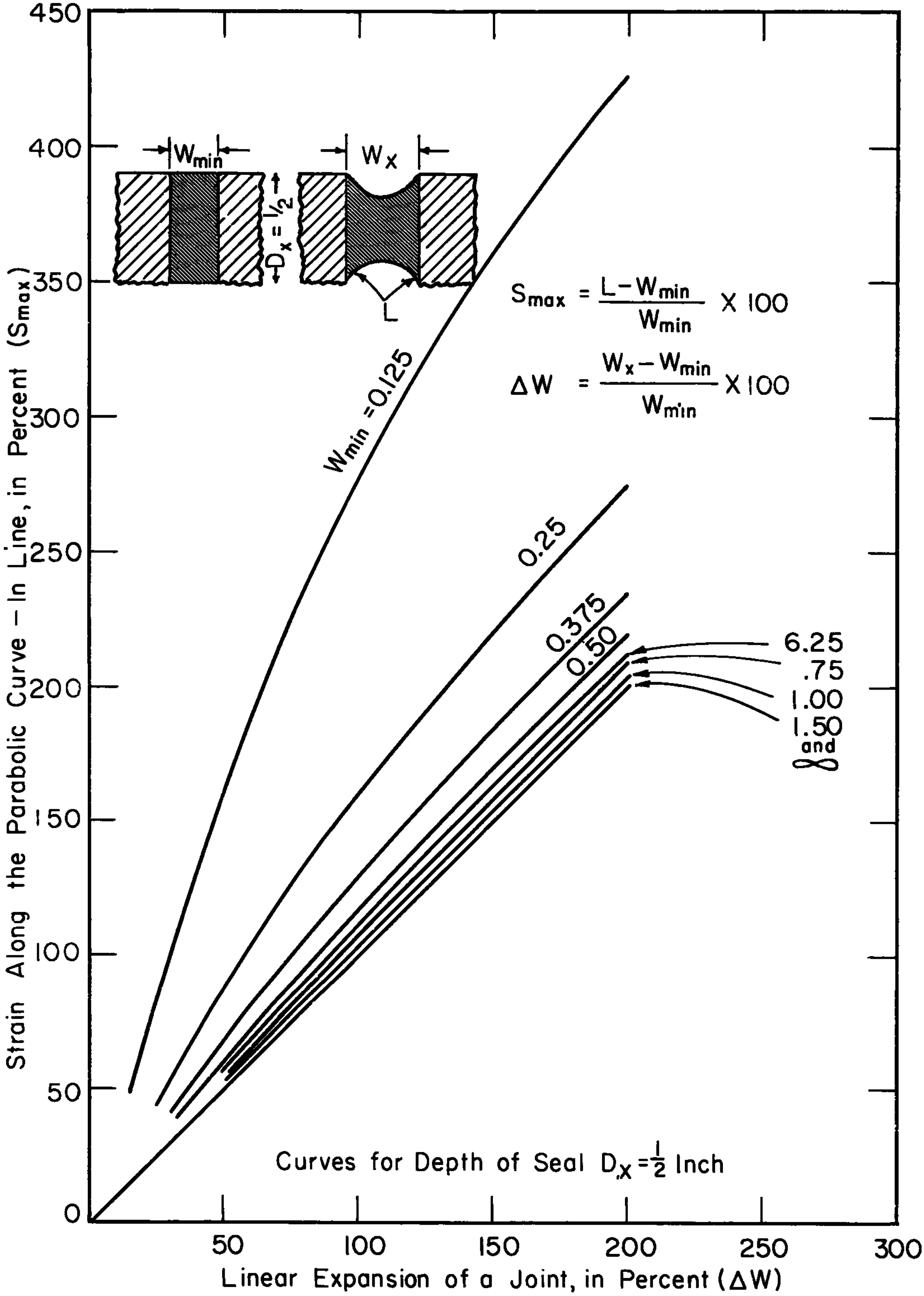

Figure 6. Comparison of strains in the sealer for various joint widths and $\frac{1}{2}$-in. depth.

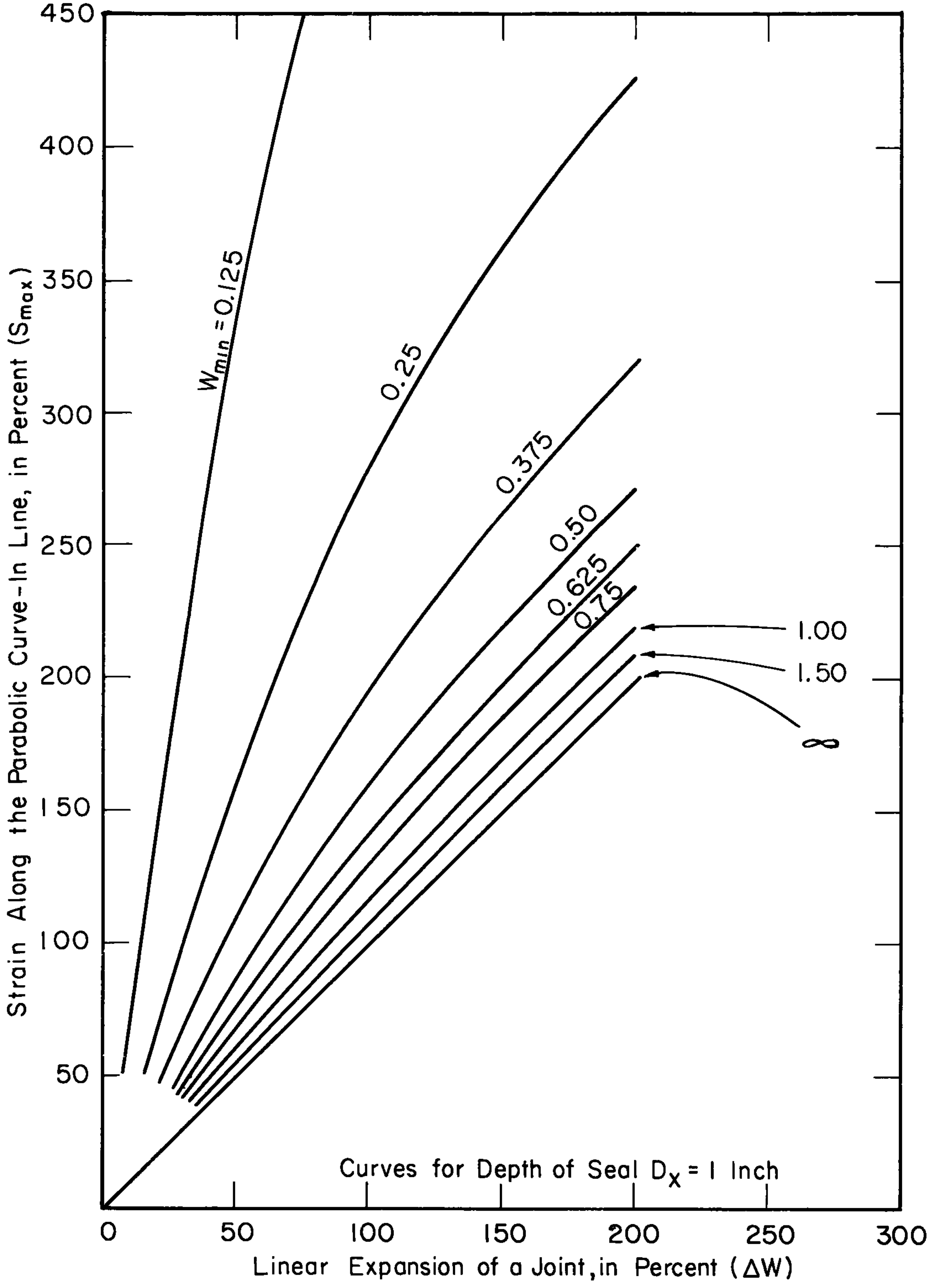

Figure 7. Comparison of strains in the sealer for various joint widths and 1-in. depths.

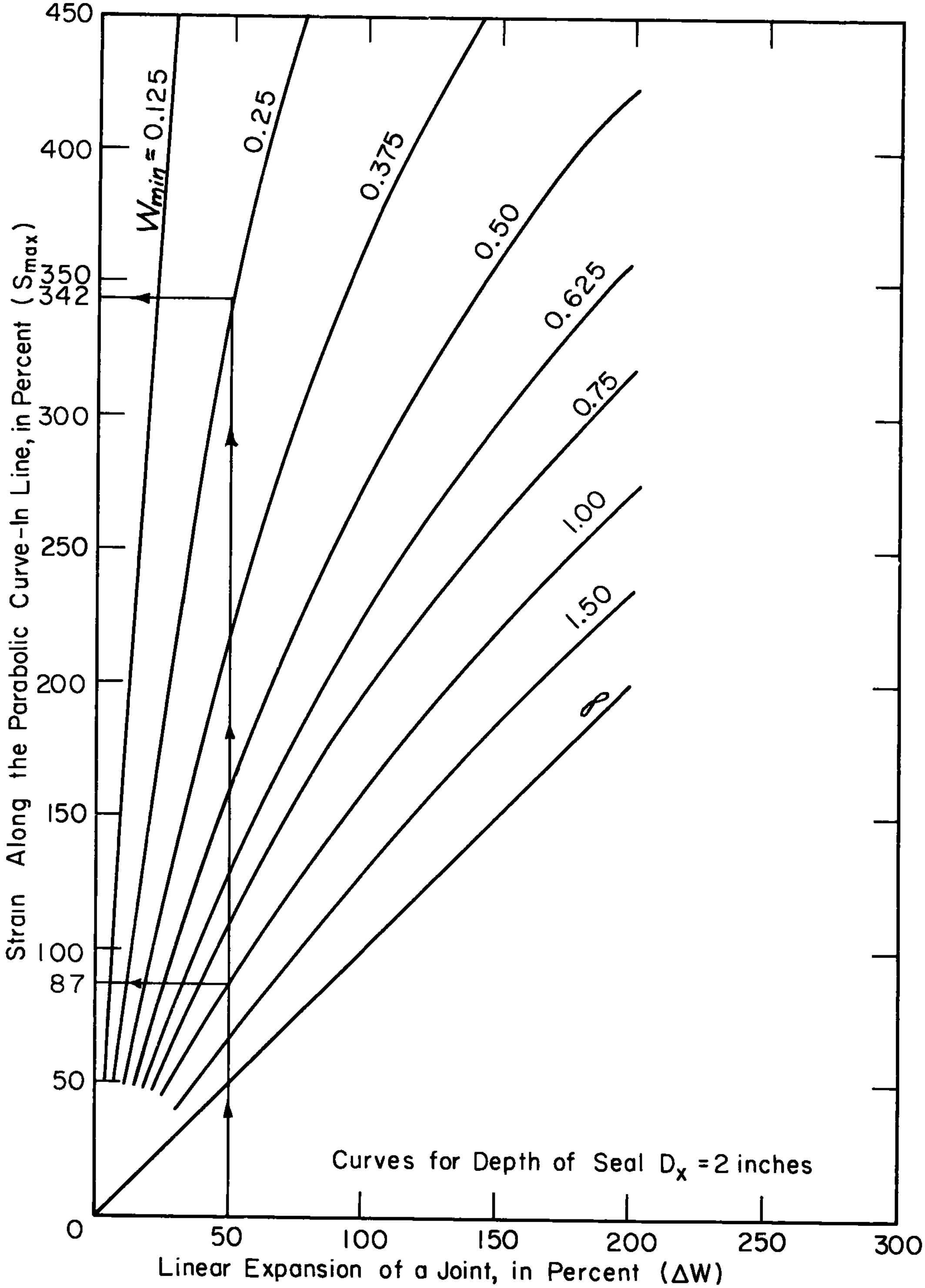

Figure 8. Comparison of strains in the sealer for various joint widths and 2-in. depth.

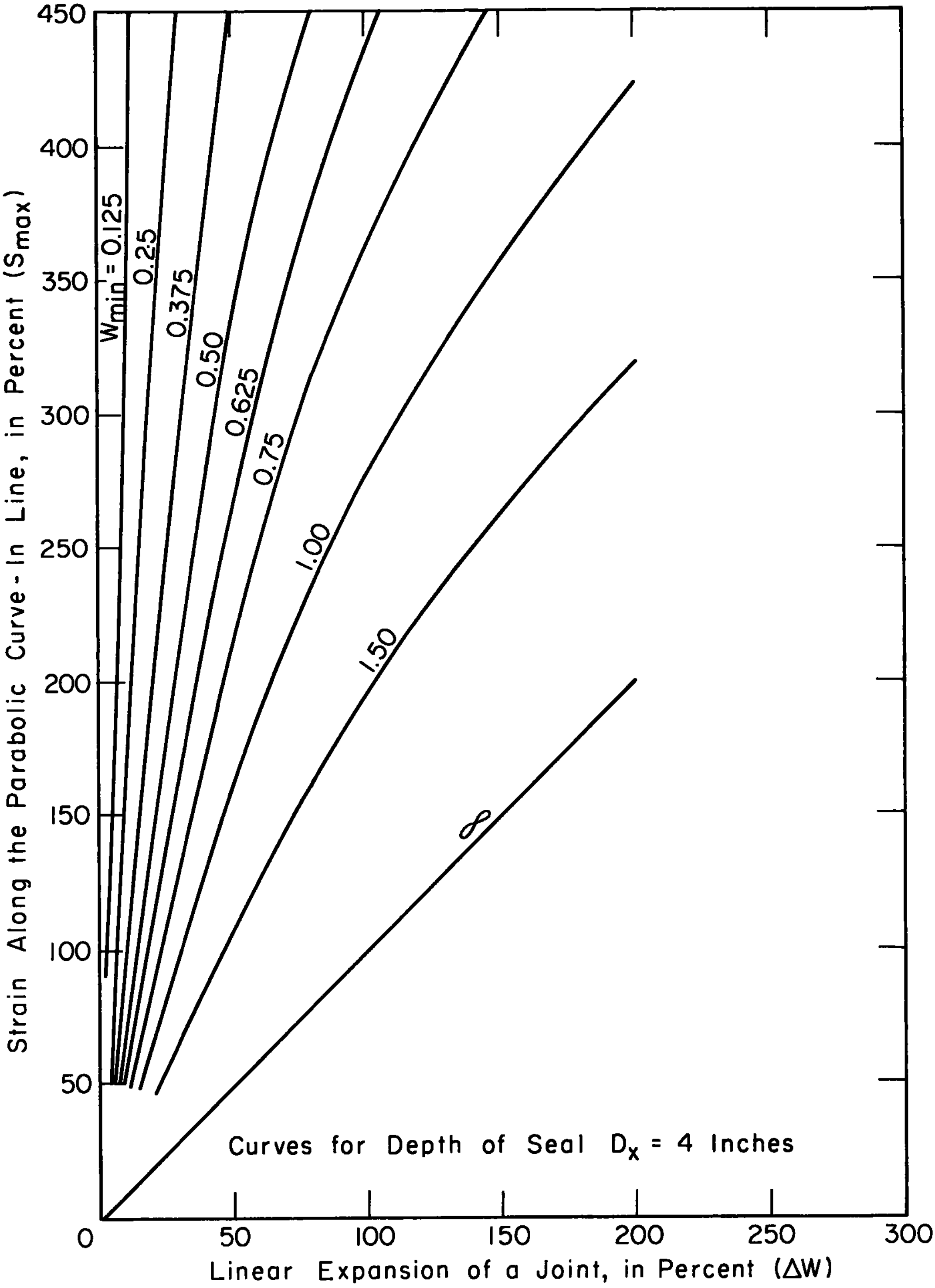

Figure 9. Comparison of strains in the sealer for various joint widths and 4-in. depth.

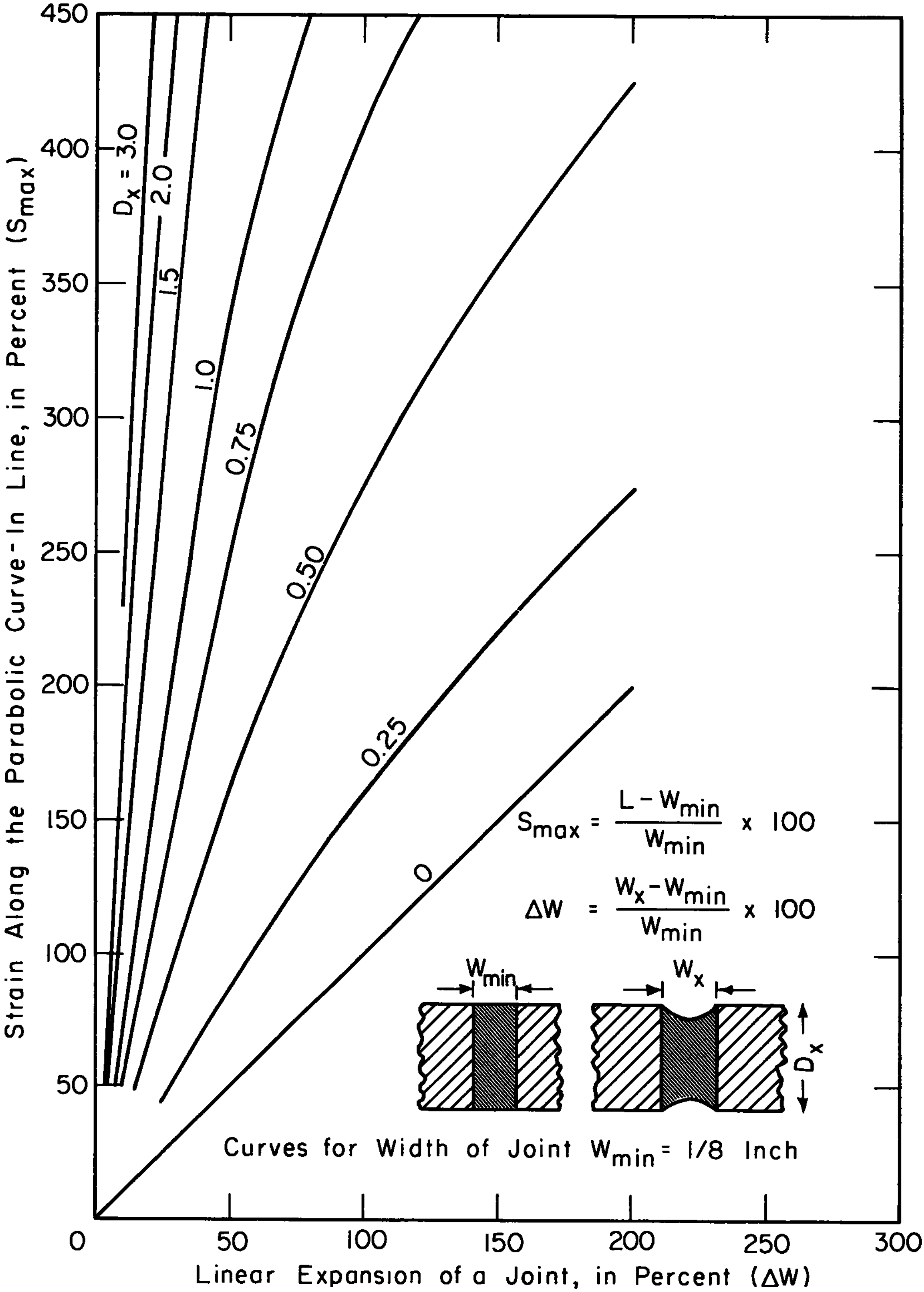

Figure 10. Comparison of strains in the sealer for various depths of seal and 1/8-in. width.

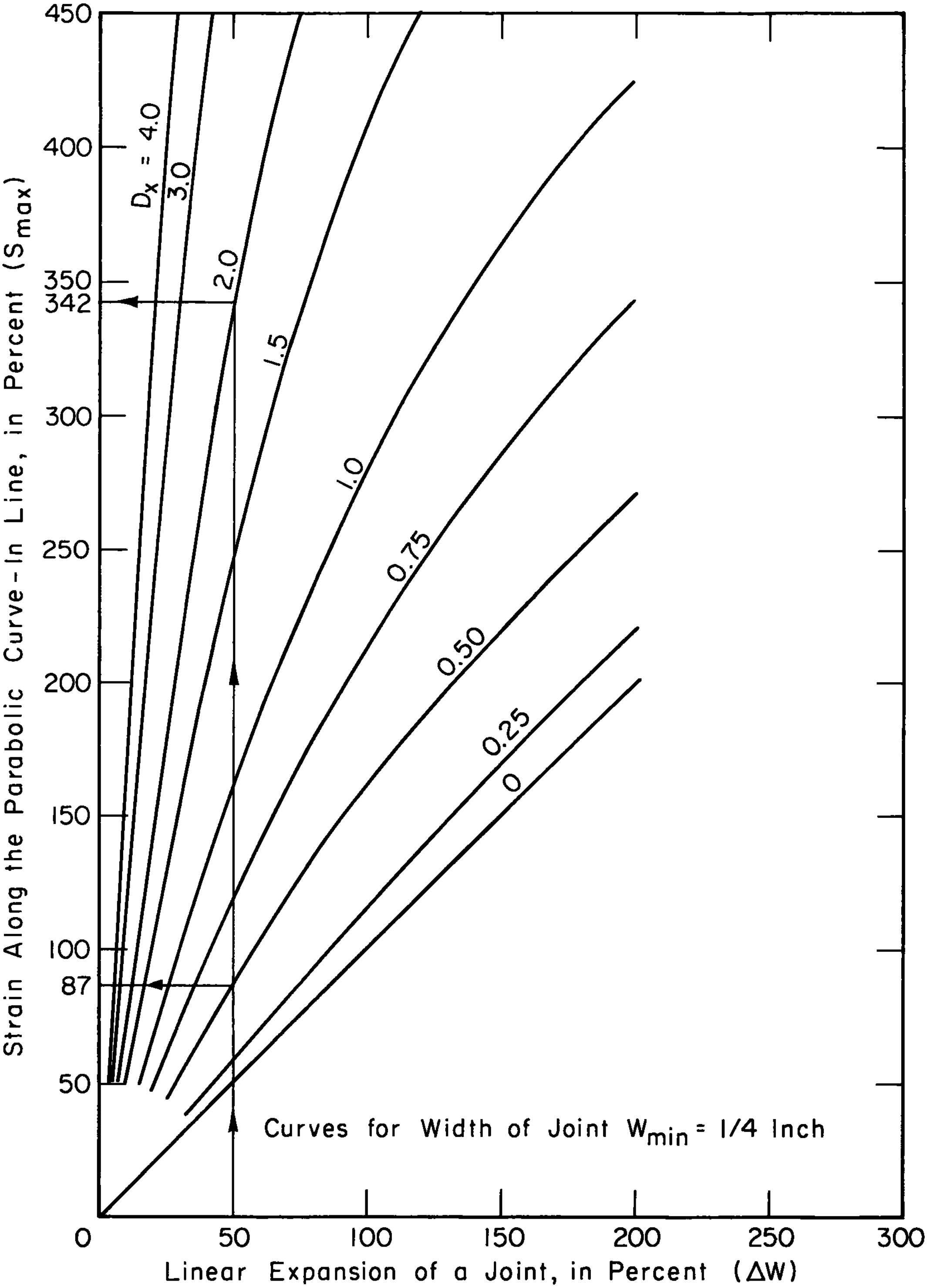

Figure 11. Comparison of strains in the sealer for various depths of seal and $\frac{1}{4}$-in. width.

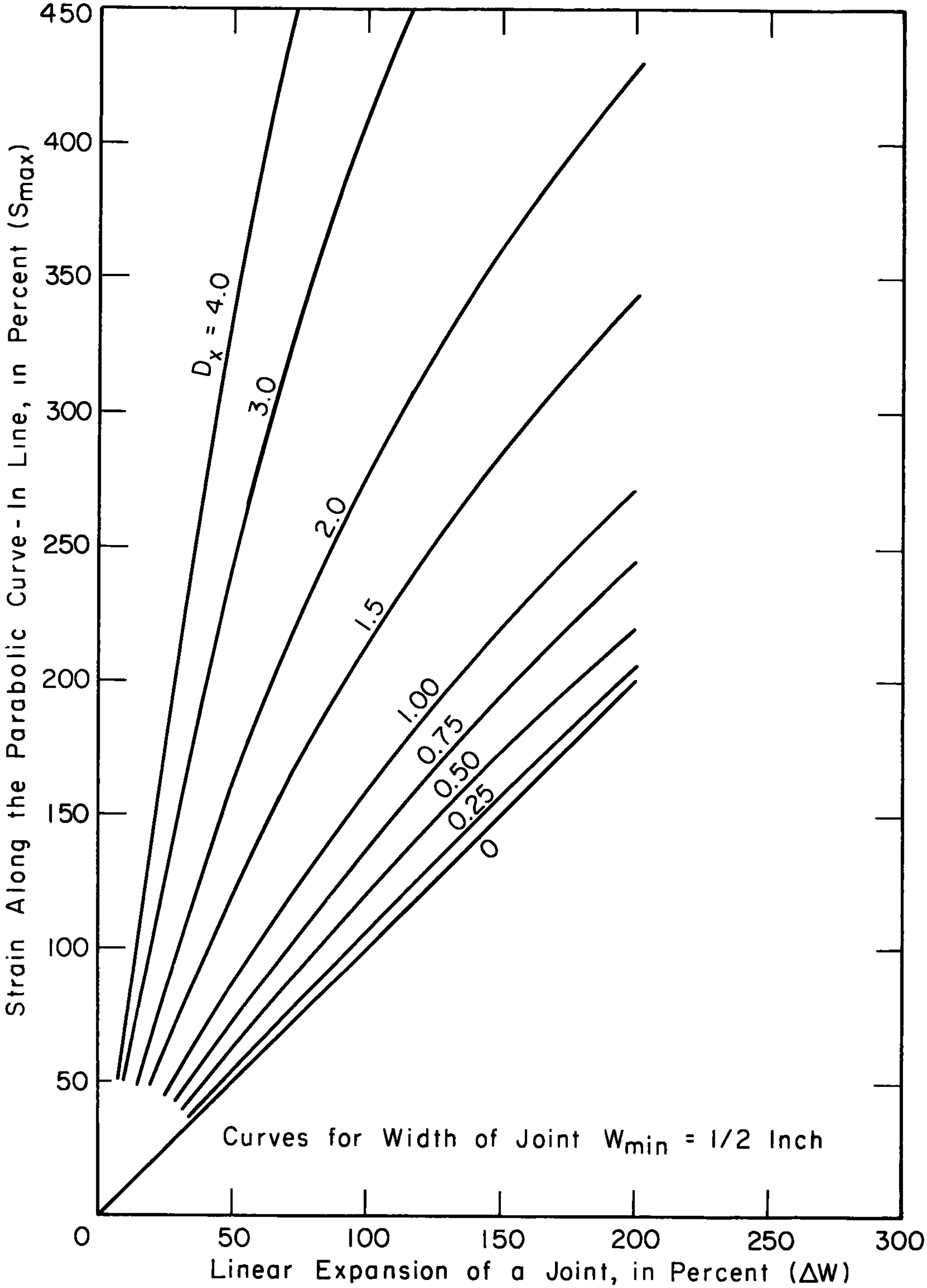

Figure 12. Comparison of strains in the sealer for various depths of seal and $\frac{1}{2}$-in. width.

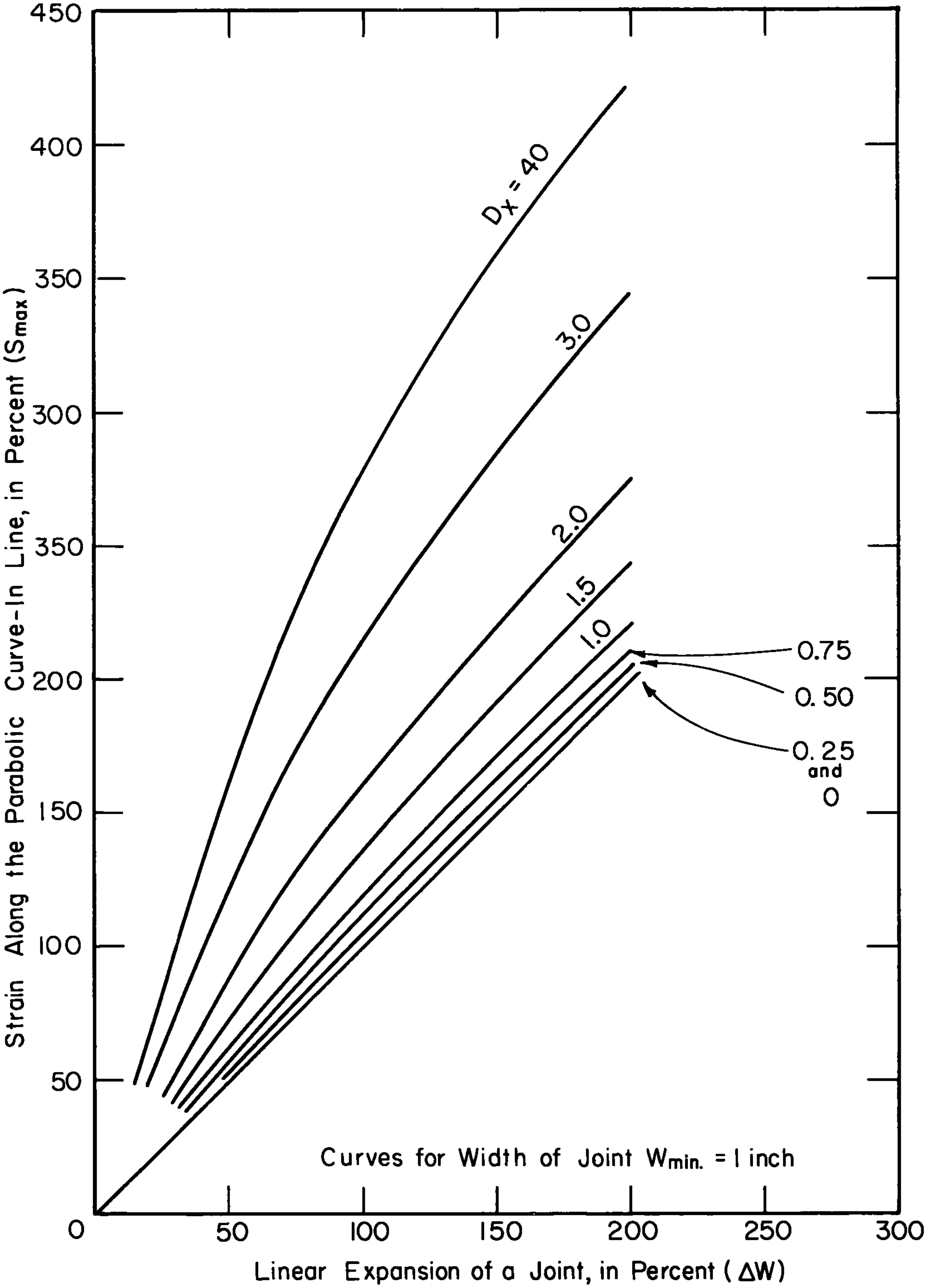

Figure 13. Comparison of strains in the sealer for various depths of seal and 1-in. width.

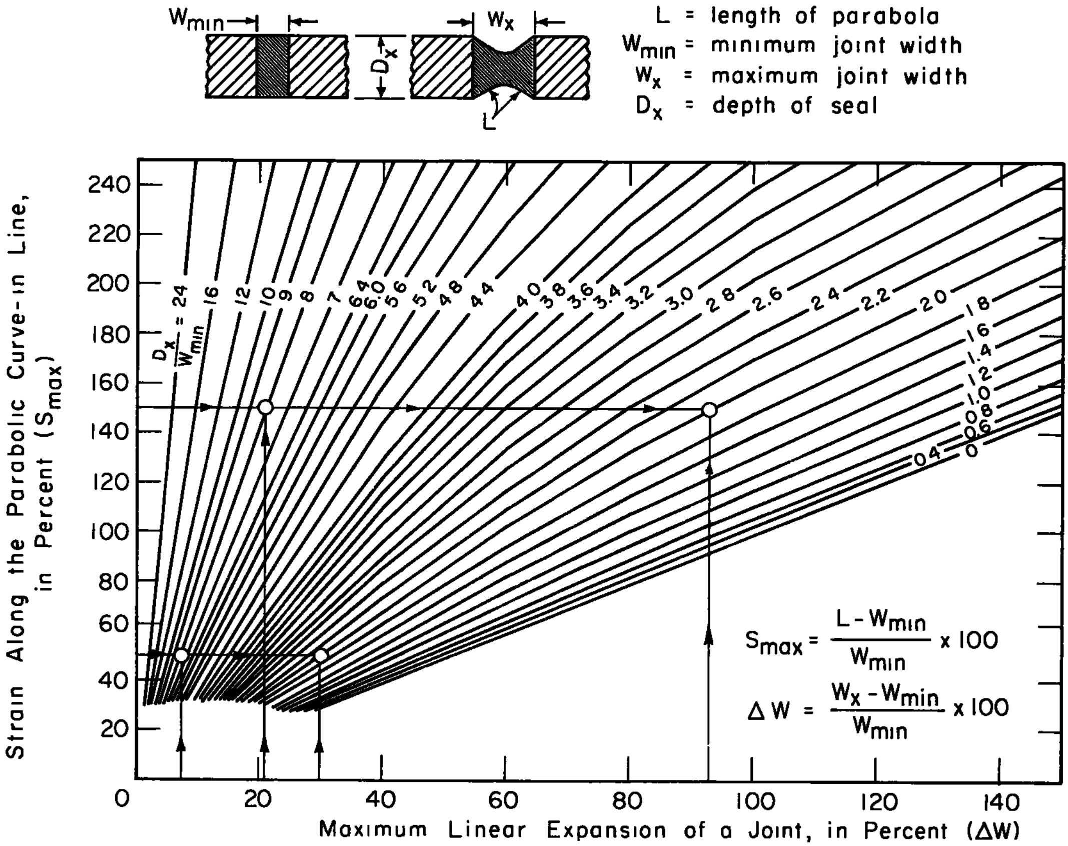

Figure 14. Relationship between ΔW, W_{min}, D_x and S_{max} in a sealed joint.

of the sealer (S_{max}) and the amount of joint expansion (ΔW) are known, D_x can be found for any desired joint width (W_{min}).

Figure 14 clearly indicates the importance of the depth of seal and joint width ratio (D_x/W_{min}). The lower this ratio the more stretchable the seal will be, other factors being the same. For instance, a sealer with maximum allowable strain S_{max} = 150 percent placed in a 3/8-in. joint can take only 21 percent of joint expansion (ΔW) if D_x/W_{min} = 8 and 93 percent if D_x/W_{min} = 2. In other words, the shallower a certain joint is sealed, the more strain a given sealer will take before it fails.

A similar comparison can be made with a low extensibility sealer (S_{max} = 50) as illustrated in Figure 14. The comparative benefits obtained are much smaller in this case. For D_x/W_{min} = 8, ΔW is 13.5 percent and for D_x/W_{min} = 2, ΔW is 30 percent.

Present data is insufficient to recommend a definite D_x/W_{min} ratio for joint seals. This will depend upon type of sealer used, the service it is put to and other requirements yet to be defined. As an estimate, this ratio (D_x/W_{min}) might be around one and may go as high as four for most practical applications. From a sealing standpoint a 3/8-to-$\frac{1}{2}$-in. depth of seal is probably the minimum that should be attempted as it would be difficult to place anything shallower.

LABORATORY TESTS TO VERIFY THEORY AND CALCULATIONS[3/]

Test Outline

The parabolic curve-in surface was observed in numerous bond-ductility tests in this laboratory during the past four years using various sealing compounds; additional tests were recently performed to obtain more accurate measurements to justify the assumptions in "Theory of Joint Seal Behavior."

In the new series a modified bond-ductility test was adopted. The length of the specimens was increased to 6 in. in order to minimize the effect of curve-in from the two ends of the test blocks. The opening between the blocks (W_{min}) as well as the depth of seal (D_x) were varied to check the different ranges of the theoretical curves in Figures 6 to 14. The maximum sealed depth in these tests was limited to 3 in., as this is usually the maximum for pavement joints.

The test procedure, preparation and testing were similar to that outlined in Appendix A, except that the temperature was 80 F during this test. Previous laboratory observations have indicated that the basic curve-in pattern (parabola) is similar at 80 F and 0 F if the same liquid-type, homogeneous sealer is strained between two blocks (joint). However, the total strain that can be applied before the seal fails is usually less in the cold test.

In these test series two sealing compounds were used: (a) a hot-poured rubber-asphalt and (b) a cold-applied, two-component synthetic polymer. They were chosen because they are well-known materials, appeared to represent a fairly large group of sealers currently used and this laboratory has already had extensive experience with them.

The following measurements and observations were made:

1. The maximum curve-in distance H was measured at different percentages of extension (joint expansion) on each specimen and compared with the calculated values (curve). This was taken as an indication of how closely the actual curve-in line approaches a parabola (see "Laboratory Test Data").
2. The uniformity of strains along the curve-in surfaces was checked by marking them and observing the strains between various points visually.
3. The amount of curve-in from the top and the bottom of the specimen was compared.
4. Cohesion and other failures were closely watched and recorded.

The test data on the H measurements are compiled in a graphical form in Figures 15 to 22. A discussion of the test results follows:

Laboratory Test Data

Figures 15 to 18 summarize the H readings obtained on specimens of hot-poured rubber-asphalt sealer of varying depth (D_x). The horizontal axis gives the simulated joint expansion while the vertical one denotes the calculated maximum curve-in values (H). The solid line represents the computed H curve for a certain depth of seal (D_x) on each figure,

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3/ A test road using various joint widths and depths of seal was installed near Syracuse, New York, in September 1958. It is briefly described in Appendix D.

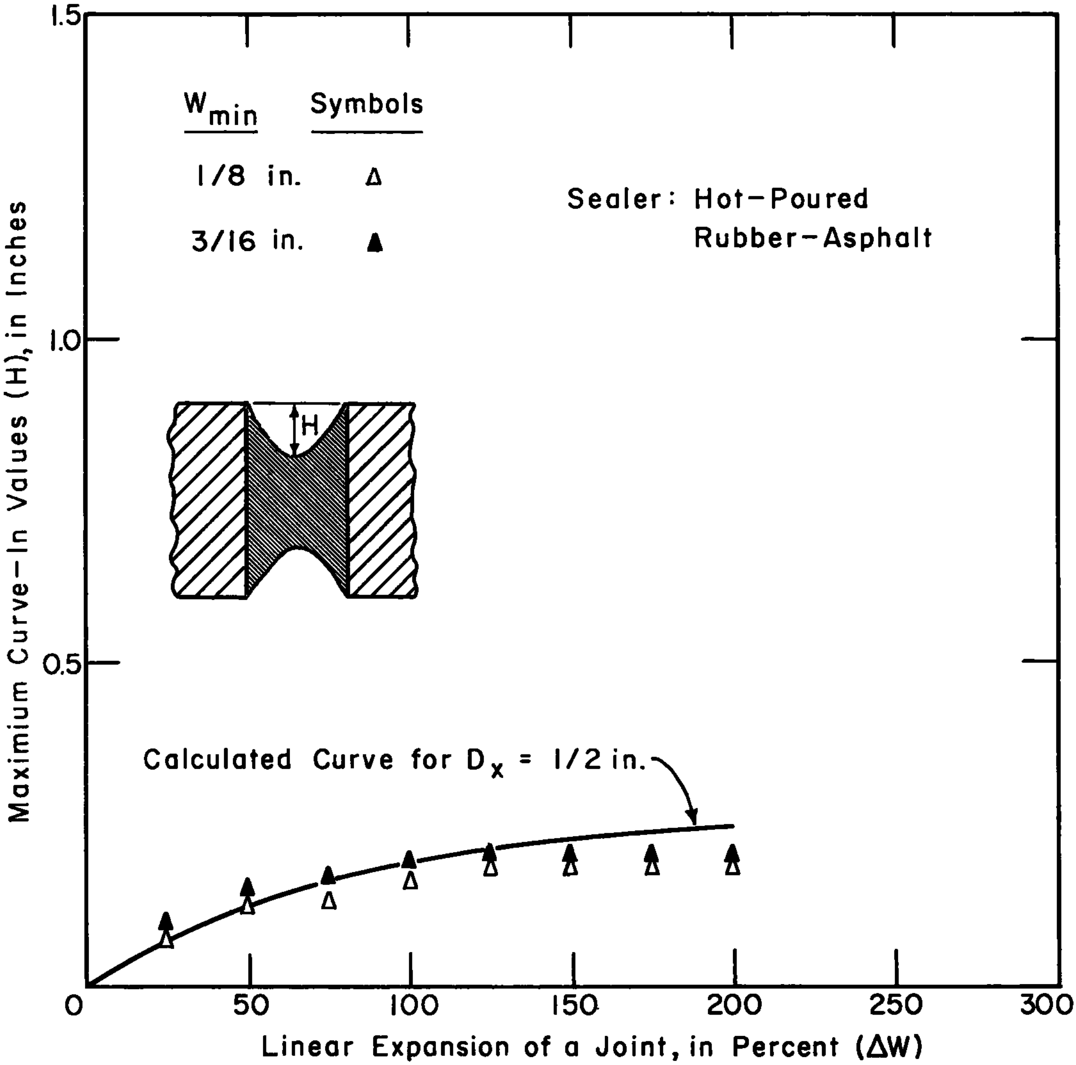

Figure 15. Comparison of calculated curve-in values (H) with those measured in the laboratory for $D_x = \frac{1}{2}$ in. and varied joint width (W_{min}).

with which the measurements obtained in the laboratory strain test are compared[4/] The mathematical equation for the curves is:

$$H = 3/4\ D_x \left(1 - \frac{100}{100+\Delta W}\right)$$

It must be pointed out that H is independent of W_{min} and will be identical for any minimum joint width (W_{min}) and one depth (D_x) at points of equal ΔW.

4/ The mathematical equation for the parabolic H curves in Figures 15 to 22 is: $H = 3/4\ D_x \left(1 - \frac{100}{W+100}\right)$ (derived in Appendix E). If the measured H values at different depths of seal (D_x) and various expansions (ΔW) satisfy this equation, the curve-in line has to approximate a parabola.

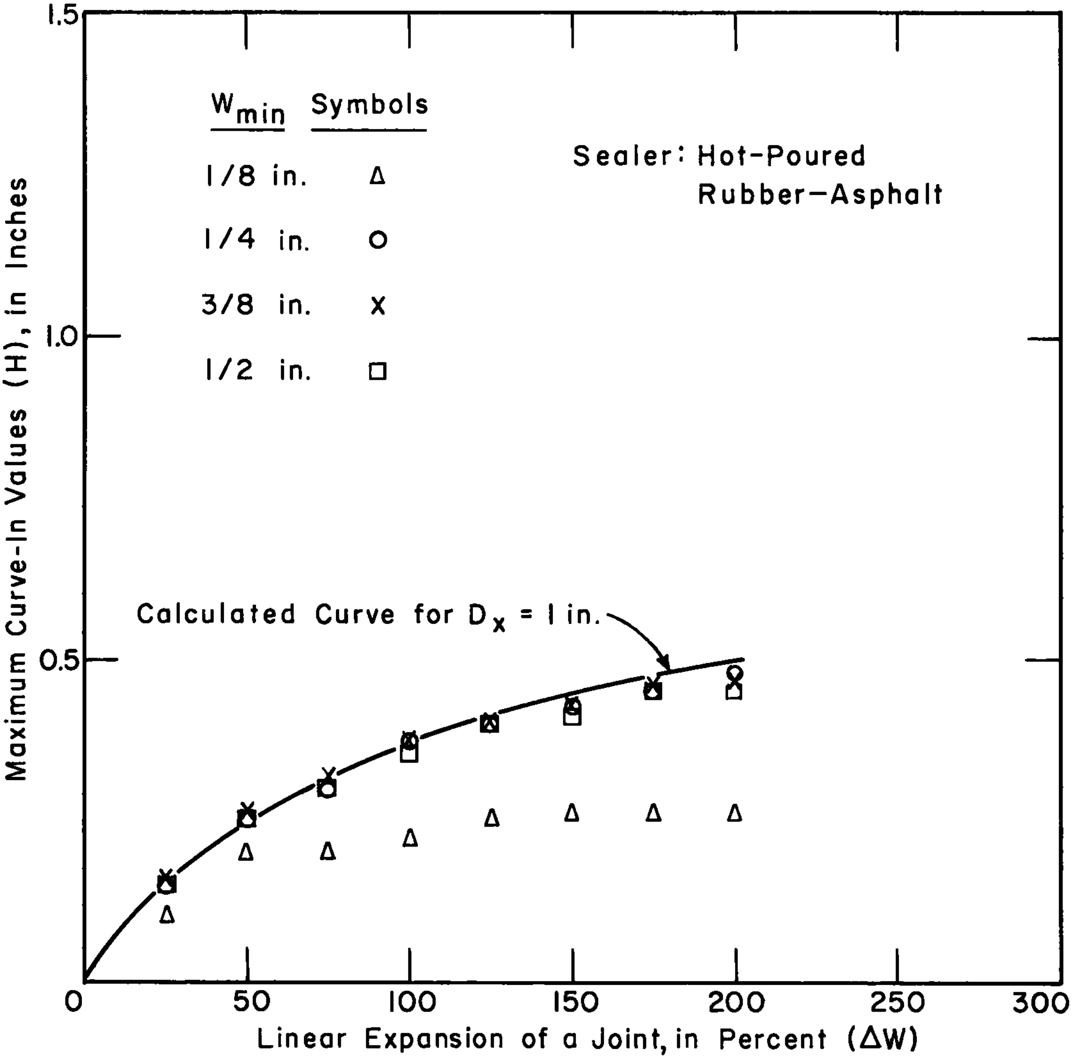

Figure 16. Comparison of calculated curve-in values (H) with those measured in the laboratory for D_x = 1 in. and varied joint width (W_{min}).

The hot-poured rubber-asphalt specimens were strained to the maximum theoretical limit which is 200 percent. The sealer in the ½-in. deep specimens followed the calculated H value curve very well (see Fig. 15). It did not break at 200 percent extension but continued to stretch in a thin band without showing any signs of failure. When the depth of seal was increased to 1 in. (see Fig. 16) the narrow 1/8-in. specimens showed a marked deviation at about 75 percent of expansion with some visible indications of separations within the sealer. When the sealed depth of 2 in. was tested a similar departure from the basic curve was noted for the ¼-in. specimens while 3/8-in, and ½-in. wide seals fell below the curve after about 100 percent expansion (see Fig. 17). Finally, when the compound was tested in 3-in. deep specimens, only the 1-in. wide seal (W_{min} = 1) followed the curve closely (see Fig. 18). Again inside separations

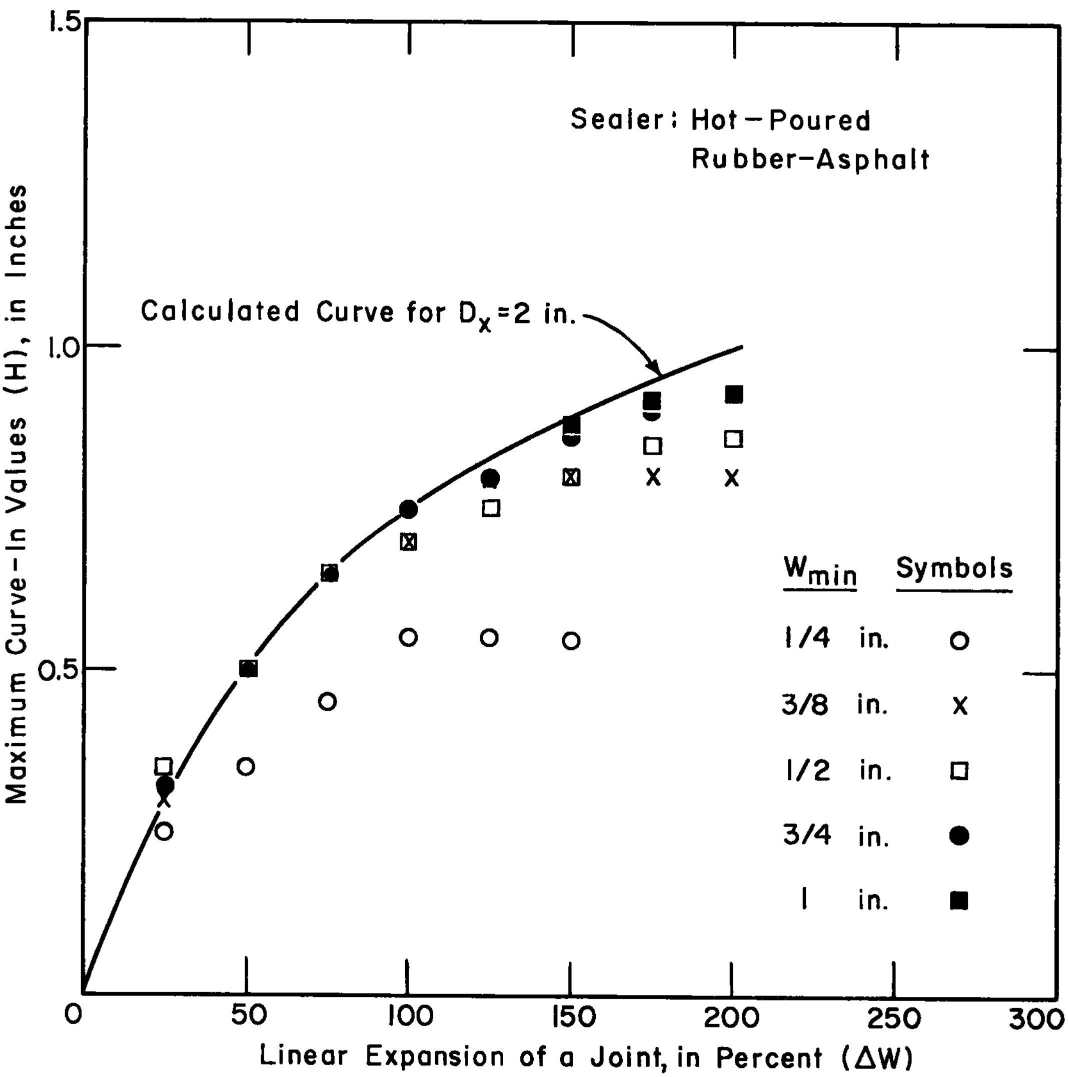

Figure 17. Comparison of calculated curve-in values (H) with those measured in the laboratory for D_x = 2 in. and varied joint width (W_{min}).

and even openings in the outside surfaces of the sealer were registered as soon as the H measurements dropped about 10 percent below those calculated.

The maximum curve-in value measurements for the cold-applied rubber polymer are summarized in Figures 19 to 22. The force required to pull this type of seal apart was considerably higher than that for the rubber-asphalt compound. Due to some limitations in the strain apparatus the specimens in these series were extended only by 100 percent. The trend of the actual measurements was very similar to the previously discussed rubber-asphalt results. Even the separations and openings in the sealer when the H values started to drop below the theoretical curve were of the same nature.

The results obtained on the two sealers indicate that the H measure-

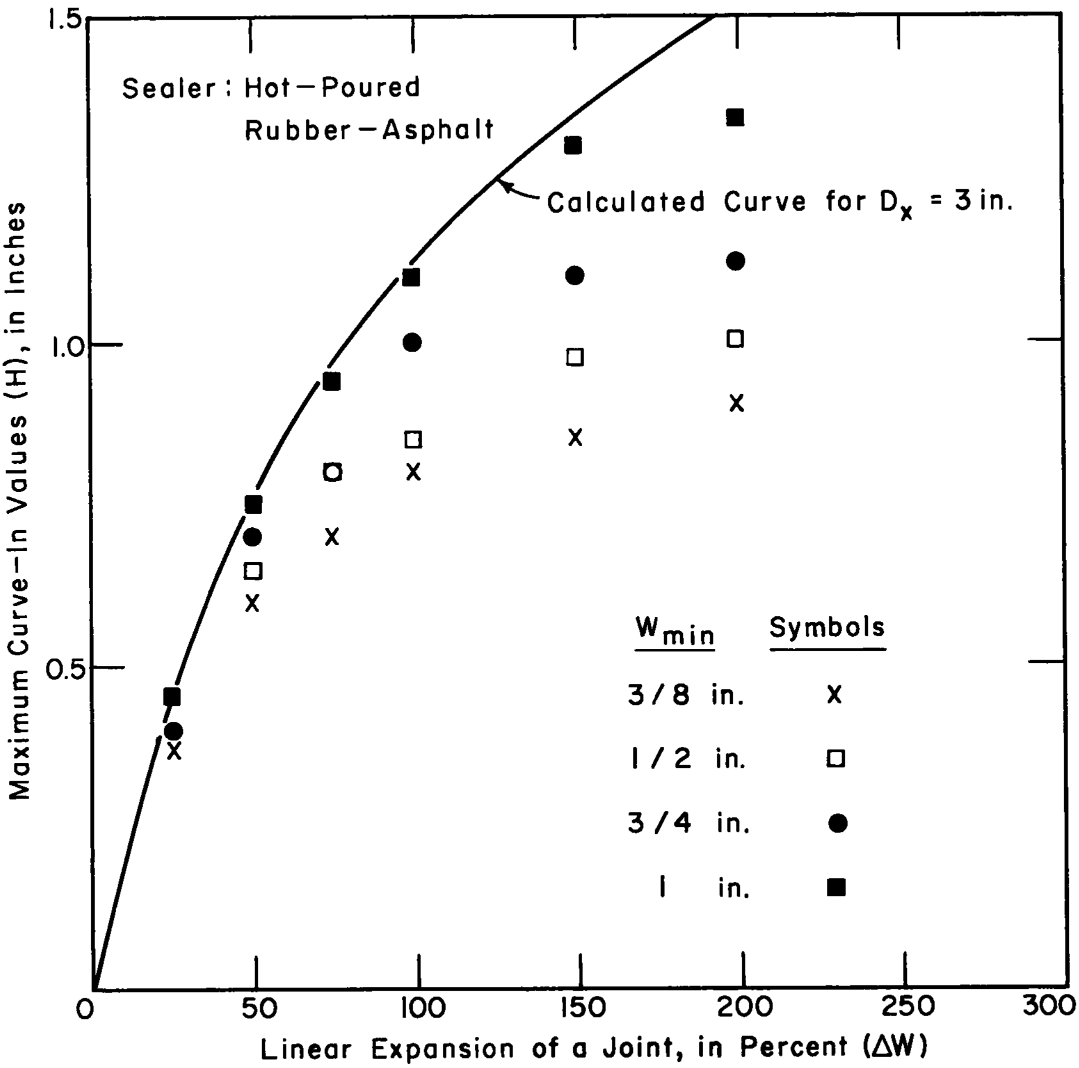

Figure 18. Comparison of calculated curve-in values (H) with those measured in the laboratory for D_x = 3 in. and varied joint width (W_{min}).

ments taken in the laboratory were comparable to the calculated ones, which in turn means that the curve-in line of the surface of the sealer closely approached a parabola[4]. The apparent deviations from the parabolic curvature occurred:

1. When the sealer was placed to a shallow depth and unreasonably large strains were applied (around 200 percent extension) (see Fig. 23).
2. When the seal lost homogeneity and started to form air spaces inside (see Fig. 24). This is much more likely to happen when a narrow and deeply sealed joint expands. Observations so far indicate that in case of homogeneous materials, as those used in the tests and under the conditions described, such air spaces form when the tangent to the parabola has an angle of 15 to 25 degrees to the vertical (see Figs. 24 and 25). Ap-

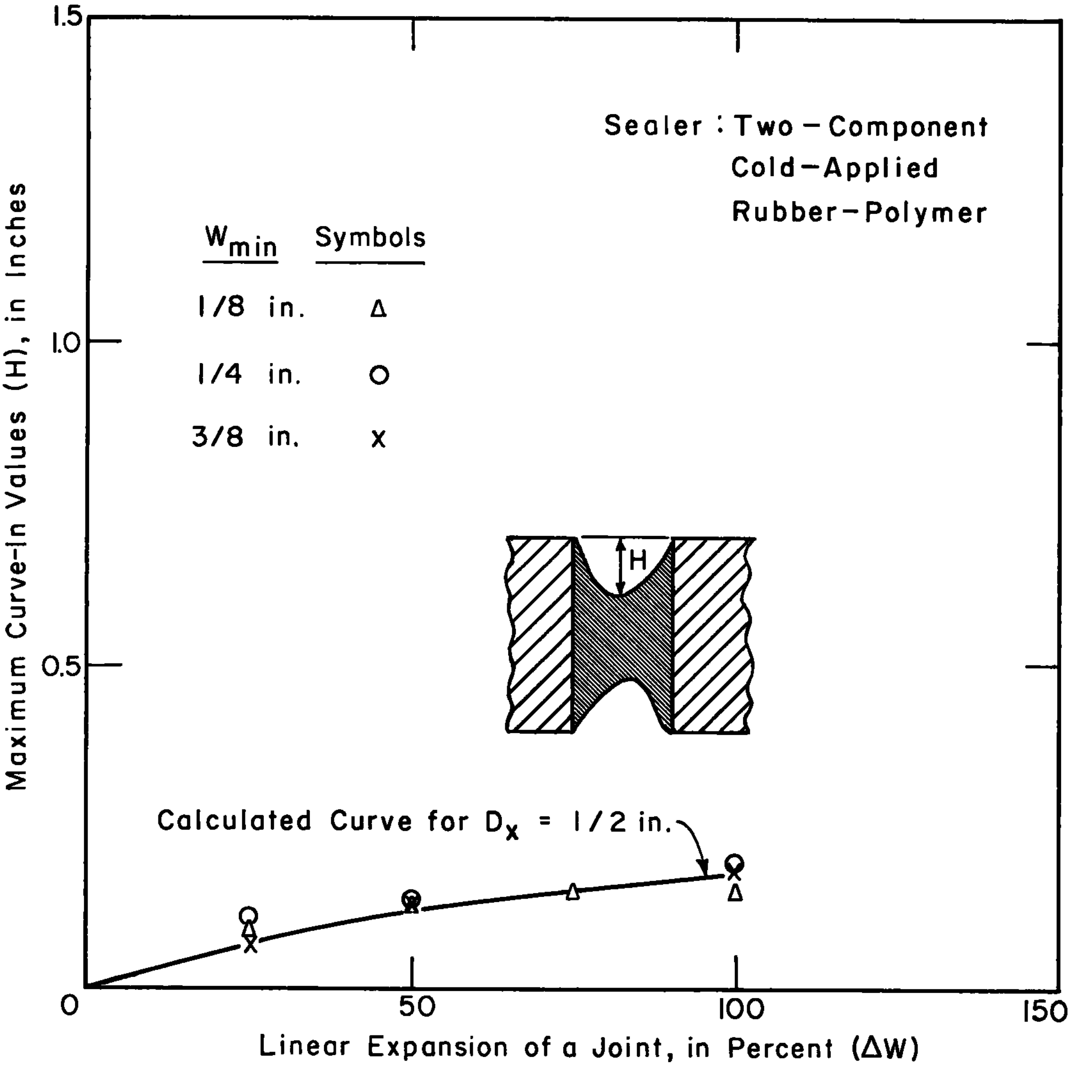

Figure 19. Comparison of calculated curve-in values (H) with those measured in the laboratory for $D_x = \frac{1}{2}$ in. and varied joint width (W_{min}).

parently this phenomenon is related to the magnitude of the shear and tension forces in the sealer along the joint walls.

Visual observations of the distribution of the strain along the curve-in surface of the sealer indicated that it is uniform at low percentages of extension but tends to vary slightly when the strains get high and the sealer has sagged deep into the opening (joint). For all practical purposes the assumption of a uniform strain along the surfaces of the sealer seems reasonable.

The amount of curve-in at the bottom of the specimen was also measured and was found to average out identical to the one on the top.

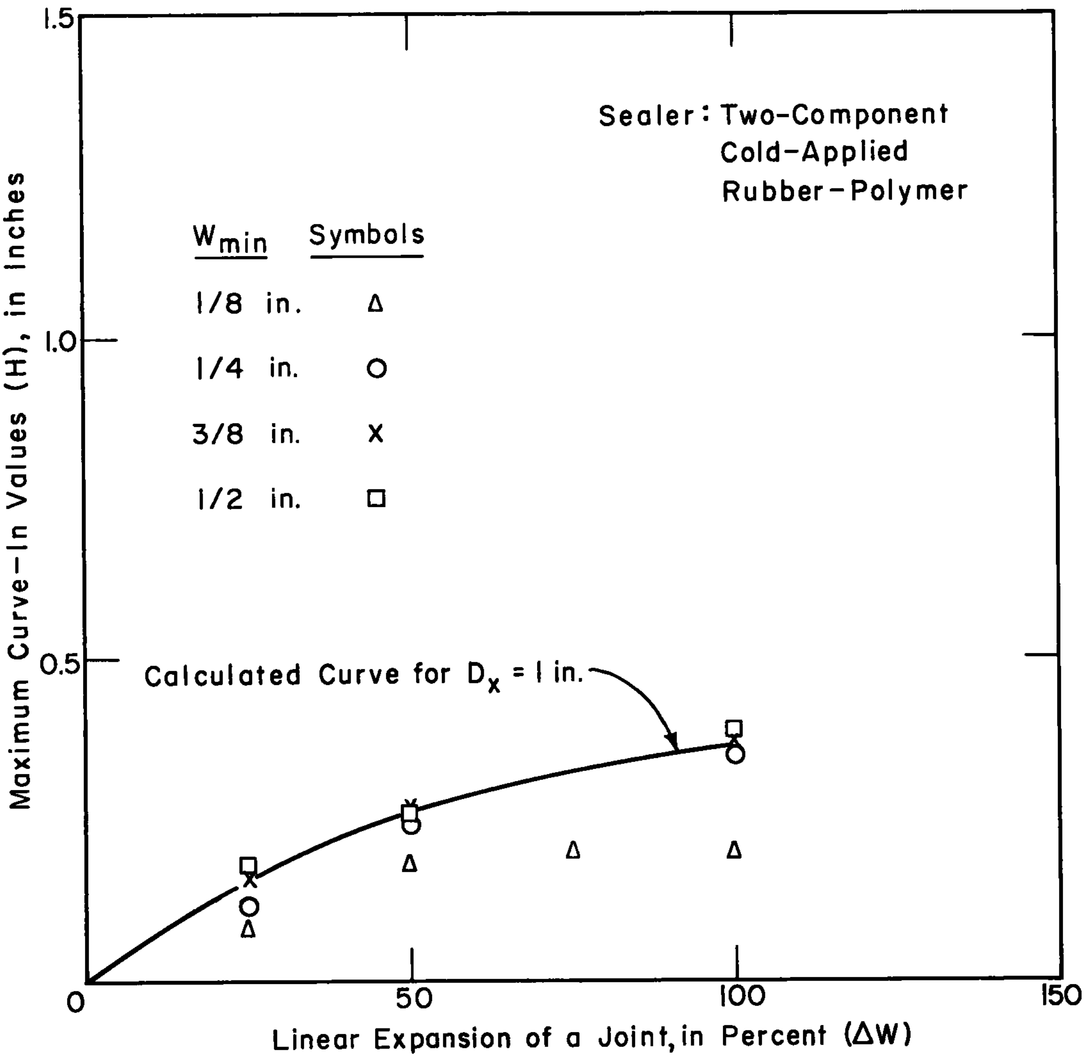

Figure 20. Comparison of calculated curve-in values (H) with those measured in the laboratory for D_x = 1 in. and varied joint width (W_{min}).

CONTEMPLATED PRACTICAL APPLICATION

Evaluation of Bond and Ductility

In order to design a seal, the properties of the sealing material have to be known, particularly the maximum strain (S_{max}) the sealer can endure. The present bond-ductility test does not test the sealer to failure[5/]. The strain pattern in the sealer during the four-hour extension period appears to be complex and different from what would be encountered in a road joint; due mainly to the small size of the specimens and the chance for the material to curve in from all four sides. Measurements and computations of the maximum strain in the sealer after the specified

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5/ See Federal Specification SS-R-406C, Method 223.11.

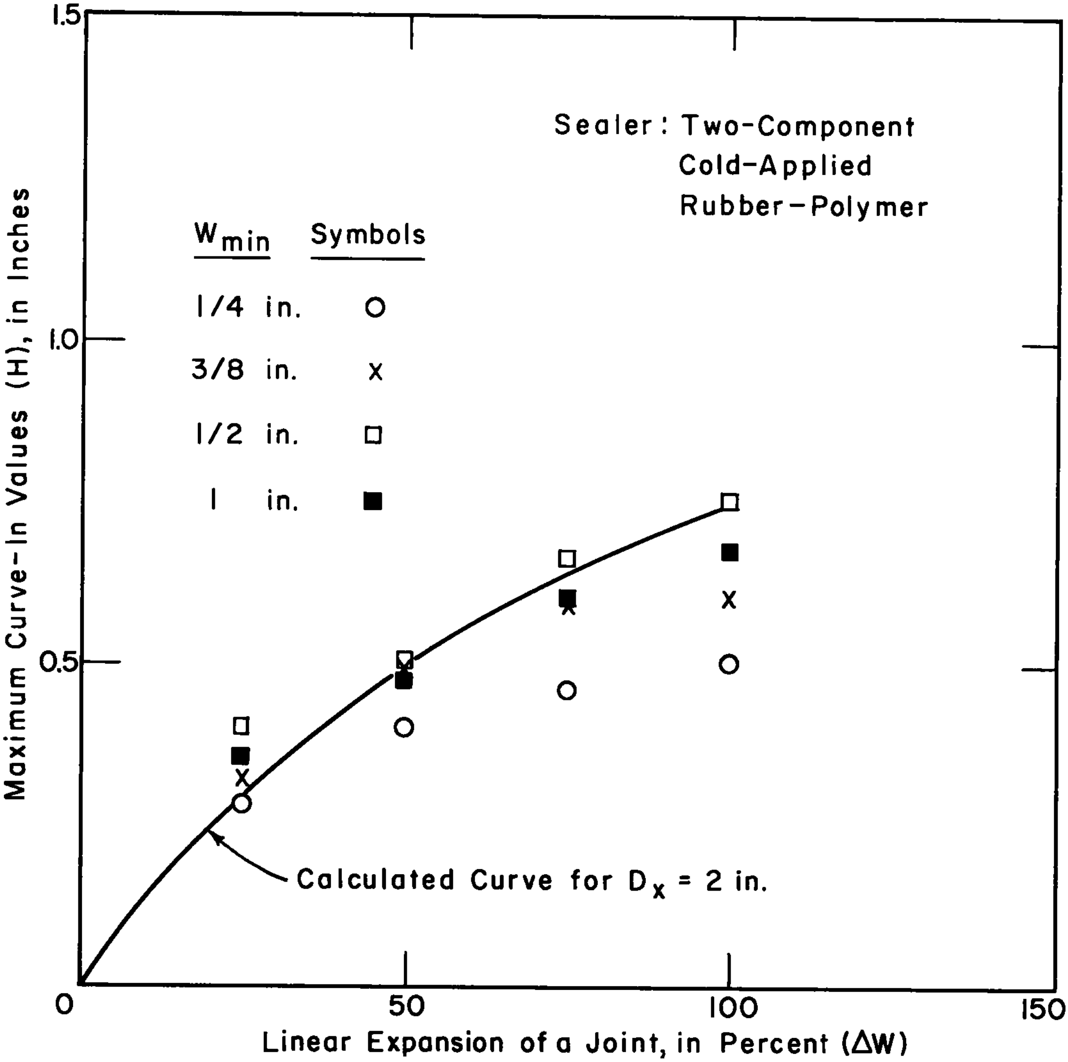

Figure 21. Comparison of calculated curve-in values (H) with those measured in the laboratory for D_x = 2 in. and varied joint width (W_{min}).

50 percent extension indicate that the strain is about 62 percent. Under present practice if the sealer meets this specification it is then used to seal any size joint which might induce much greater strains. If such a compound is placed in a $\frac{1}{4}$-in. joint, sealed to 2-in. depth and expanded 50 percent (1/8 in.), the maximum strain in the sealer will be 342 percent, or more than five times as much as it was tested for.

The best way to predict the performance of a sealer would be to test it in a joint similar to that in which it is going to serve. This is often impractical and would involve difficulties in the standardization of the test. If the maximum strains in the sealer can be correlated mathematically for different joint widths and depths, the test can still be standardized. In Appendix A of this paper it was attempted to outline

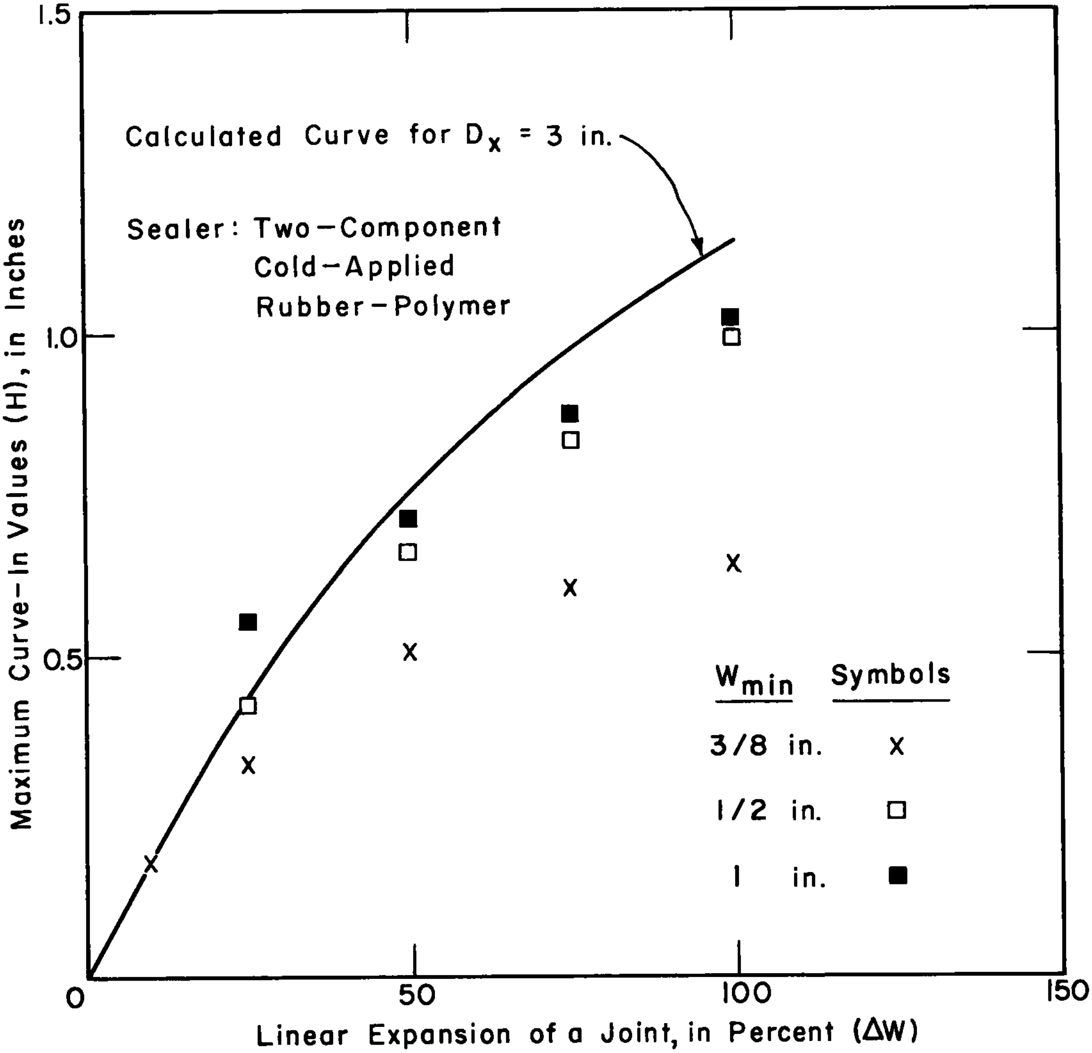

Figure 22. Comparison of calculated curve-in values (H) with those measured in the laboratory for D_x = 3 in. and varied joint width (W_{min}).

what might be a more realistic approach to the evaluation of the capabilities of different sealing materials. The new test would use longer (6 in.) test blocks, 1 in. deep and spaced $\frac{1}{2}$ in. apart. The sealing material would be placed in the opening, cooled to 0 F and extended to failure in cohesion or bond. For the design of joint seals a safety factor of 2 would be applied; i.e. only one-half of the obtained failure strain value would be used in the actual design.

Actual Design

Once the strain capabilities are known, the necessary joint width and depth of seal can be determined for any known joint expansion. An outline of how this can be done is given in Appendix B.

Other Important Variables

It should be pointed out once more that this method of designing a seal for road joints is concerned primarily with the proper geometric relationships. Careless sealing techniques, insufficient adhesion, excessive shear at the joints and other influences might render a seal ineffective no matter how well it is proportioned.

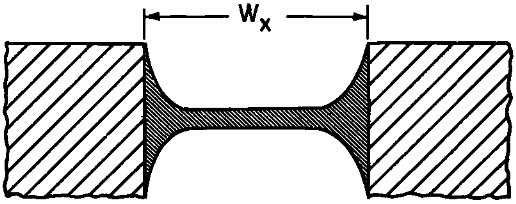

Figure 23. At joint expansion approaching 200 percent a sealer in a shallow joint might not follow a parabolic curve-in line.

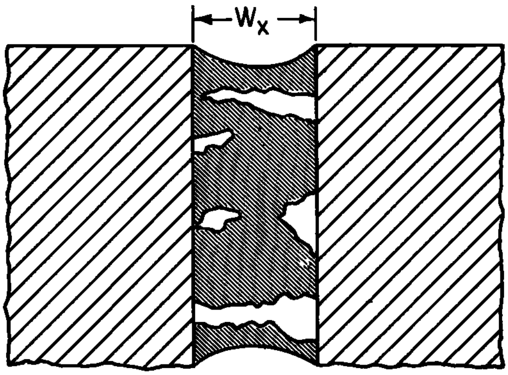

Figure 24. In most cases if a sealer does not follow the parabolic curve-in line an internal (cohesion) rupture is imminent.

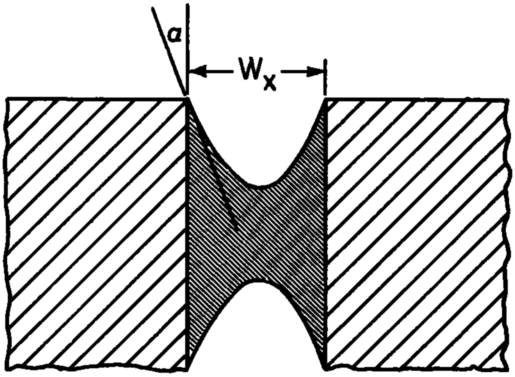

Figure 25. The tangent angle **a** was found to be 15 to 25 degrees at the time of most failures of deep seals.

CONCLUSIONS

This paper outlines a procedure for esti ating tension strains in a homogeneous liquid-type sealer used for sealing joints and cracks in pavements. The assumptions and theoretical calculations have been verified by laboratory test. The major conclusions from this study are:

1. Laboratory tests indicate that if a homogeneous liquid type sealer is placed in a rectangular joint and subjected to strain (expansion) the curve-in surface closely follows a parabolic curve (except under certain conditions discussed in "Laboratory Test Data").
2. Maximum strains in the sealer can be calculated by using parabolic equations and relationships.
3. The calculations show that for like conditions, the greater the minimum width of the joint, the less the sealer will be strained for the same percentage of joint opening.
4. The shallower the joint is sealed, the less the sealer will be strained when the joint opens, other conditions being the same.
5. Observations in the laboratory show that if a sealer does not follow the parabolic curve-in line closely and appears sound from the outside, inner cohesion separations and formation of air spaces are taking place.

6. The present bond-ductility test does not indicate the actual strain capabilities of a sealer.

7. A bond-ductility test in which the sealer is strained to failure should be a better way to evaluate the material.

RECOMMENDATIONS

Additional research is needed to broaden the scope of this basic theory and to further check its limitations.

1. A bond-ductility test for testing the sealer to failure needs to be standardized and perfected.
2. Performance of various types of joint sealer compounds should be studied in laboratory and field tests to determine agreement with the theory and to observe what is happening when the sealer does not follow the predicted strain pattern.
3. The influence of temperature on the maximum allowable strains for various sealers should be studied.
4. The strain and stress distribution at various stages of extension as well as the tangent angle (see Fig. 25) at the time of failure or departure from the parabolic curve-in line should be further investigated in a laboratory.
5. The optimum ratio of D_x/W_{min} should be studied for various compounds.
6. In addition, it is felt that research and data gathering are needed on the following subjects:

    a. Adhesion of sealer to joint walls.
    b. Influence of shear movements at the joints on the performance of the seal.
    c. Accumulation of joint movement data in various parts of the country.
    d. Development of a durability test for joint seals.

7. The final goal should be to define types and shapes of sealers the pavement engineer needs, so that the manufacturers can make them.

ACKNOWLEDGMENTS

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APPENDIX A

Outline of a Bond-Ductility Test for Evaluation of Sealers

Background

In January 1958, Kuenning[6] presented some experimental test data from a bond-ductility test on one hot-poured rubber-asphalt sealer. The basic outline for the current paper was ready at that time and it was encouraging to note that Kuenning's test values agreed with the author's theoretical calculations. Kuenning used what may be a more meaningful bond-ductility test for sealers. He tested his specimens to failure rather than using the usual 5-cycle, 50 percent extension method. He also used specially prepared longer test specimens to come closer to actual road joint conditions.

Outline of the Test Procedure

More direct research work is needed for giving a definite test procedure. This is only a projected example of thinking what such a test might look like as compared to the present test[7].

1. The test blocks could be longer, shallower, and the spacing between them should be decreased. For instance, some observations indicate that 6-in. long, 1-in. deep specimens spaced $\frac{1}{2}$ in. apart to receive the sealer are a promising combination.
2. The molded specimen could be extended in a suitable machine at a low temperature until the seal fails. Failures at the very ends of the specimens should be neglected. This could be used as an indication of seal performance.
3. The preparation of the bond surfaces of the blocks should be brought closer to actual pavement conditions than it is in the present test. It might well be wise to introduce moisture into the blocks before the sealer is poured.
4. Otherwise, the specimen preparation and testing features could be similar to the present test[7].

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[6] Kuenning, W. H., "Laboratory Tests of Sealers for Sawed Joints." HRB Bull. 211 (1959).

[7] Federal Specification SS-R-406C, Method 223.11.

APPENDIX B

Probable Joint Seal Design Procedure

Step 1

Extend the sealer in a bond-ductility test until it fails[8]. The test blocks should be made and arranged so that a $\frac{1}{2}$-in. wide ($W_{min} = \frac{1}{2}$), 1-in. deep and 6-in. long gap can be filled with the sealer. Note the block distance (W_f) at the time of failure.

Step 2

Compute the percent of expansion at the time of failure

$$\Delta W = \frac{W_f - \frac{1}{2}}{\frac{1}{2}} \times 100$$

and enter the value at the bottom of Figure 26. From this point intersect the curve and note on the left side the maximum allowable strain (S_{max}) for the sealer. Factor of safety (SF = 2) is already included in the left side figures.

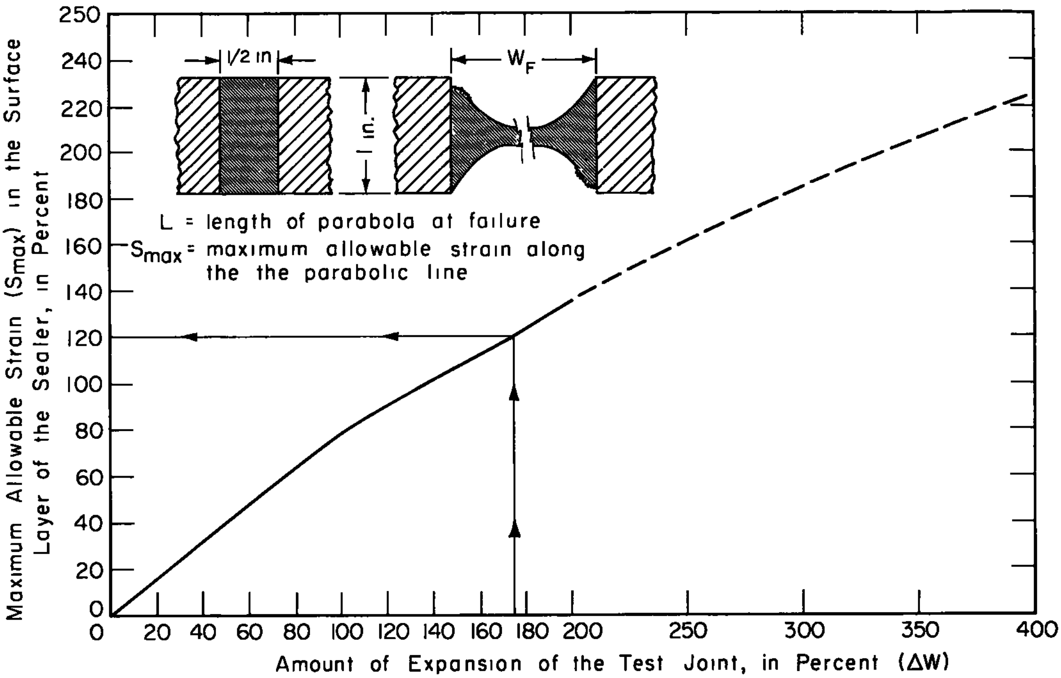

Figure 26. Allowable strains in the sealer, as determined by a bond-ductility test in a laboratory.

Step 3

Assume desirable slab length and estimate from field measurements on other pavements the maximum joint width variation ($W_{max} - W_{min}$). If such

[8] An outline of the procedure is given in Appendix A.

data are not available, assume a reasonable coefficient of expansion for the concrete[9] and, taking the maximum temperature differential for a year, calculate the maximum change in joint opening ($W_{max} - W_{min}$).

Step 4

Assume desirable joint width (W_{min}) and calculate the maximum joint expansion (ΔW) in percent, using the value obtained in Step 3.

$$\Delta W = \frac{W_{max} - W_{min}}{W_{min}} \times 100$$

Step 5

Take the maximum joint expansion value (ΔW) from Step 4 and enter from the bottom of Figure 27. Then using the maximum allowable strain (S_{max}) found in Step 2 enter Figure 27 from the left side. Where the two lines intersect the D_{max}/W_{min} ratio will be indicated. Using the in Step 4 assumed W_{min} value, D_{max} can be calculated.

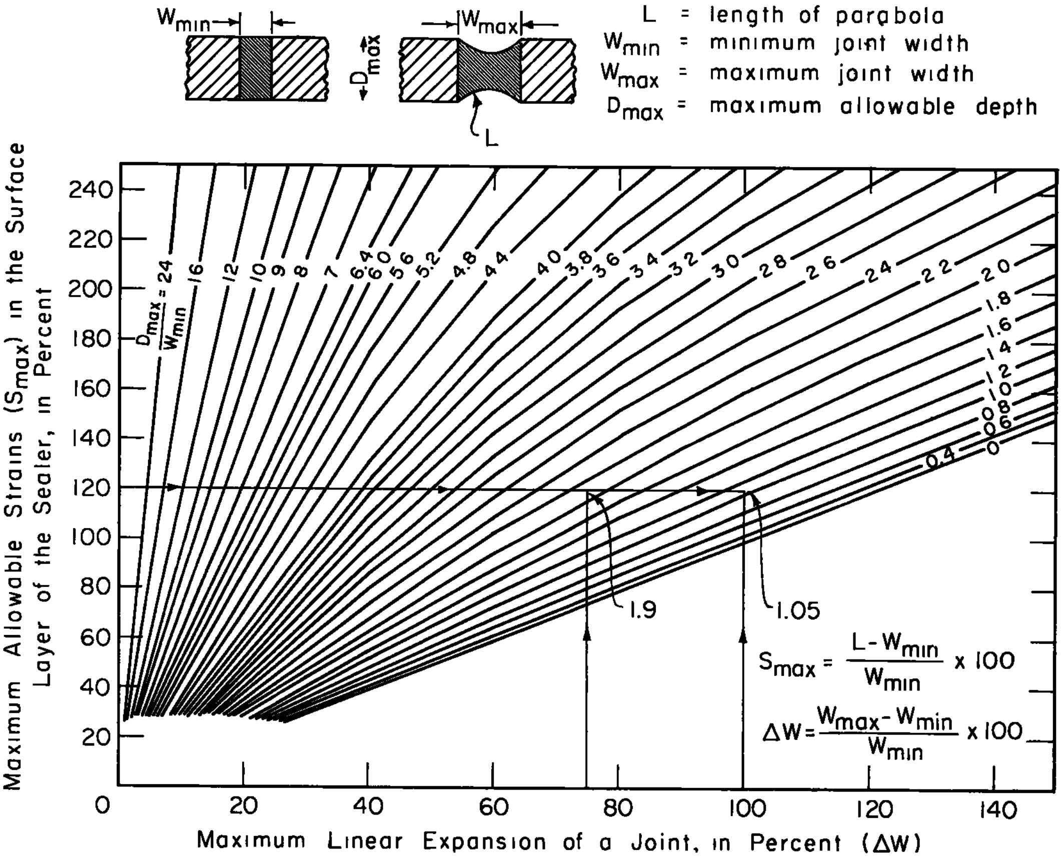

Figure 27. Maximum allowable filling depth for a known sealer for a given joint width and expansion.

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[9] Values of 4.0, 5.0 and 6.0 x 10^{-6} have been suggested in the literature.

Step 6

The found depth value (D_{max}) should be equal to greater than the minimum joint width (W_{min}) but never less than $\frac{1}{2}$ in.[10]. If this is not so, take smaller slab length or wider minimum joint width and repeat Steps 3, 4 and 5.

Step 7

Finally, check by Figure 28 whether the sealer does not curve in too deep for the seal as determined in Step 5. If H exceeds $\frac{1}{2}$ in.[10], foreign matter might accumulate on the top of the sealer.

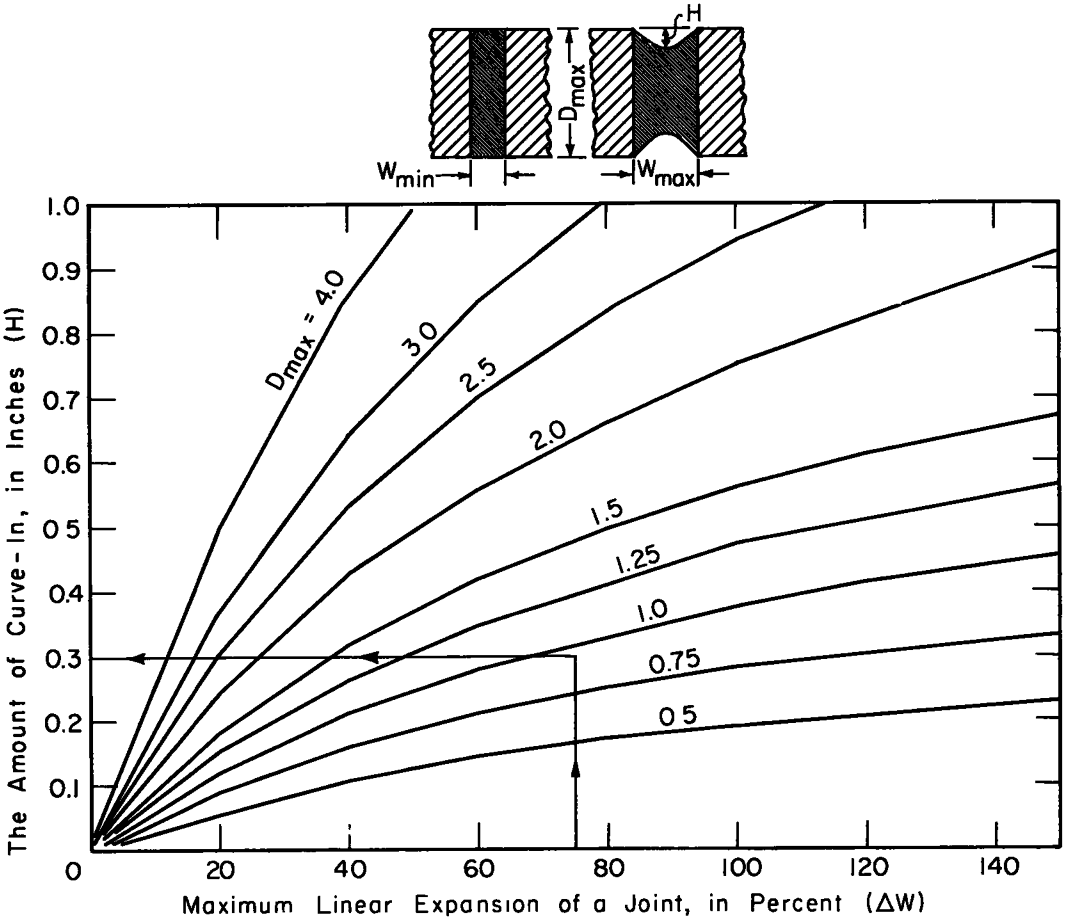

Figure 28. Maximum curve-in (H) values.

EXAMPLE

Step 1

A hot-poured rubber-asphalt sealer was tested in the laboratory in a $\frac{1}{2}$-in. opening. It failed at 1.37 in. (W_f = 1.37) or after 0.87-in. extension.

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[10] See discussion in Appendix C.

Step 2

The percent of test joint expansion at failure was 0.87:0.5 - 174 percent. Entering this value at the bottom of Figure 26 and intersecting the line, 120 percent for maximum allowable strain in the sealer is obtained.

Step 3

State "X" specifies contraction joints only, spaced 75 ft apart. Measured joint movements at numerous places of similar pavements indicate that the maximum joint expansion is about 0.375 in.

The same State "X" does not have field measurements to rely upon. They know that the difference between maximum and minimum yearly temperatures is about 105 F. They estimate the coefficient of expansion for the concrete to be 4.0×10^{-6}. Thus the maximum joint expansion can be calculated: $75 \times 12 \times 105 \times 4.0 \times 10^{-6} = 0.378$ in.

Step 4

They would like to have as narrow joints as possible, but they realize that for 75-ft spacing 1/8- and $\frac{1}{4}$-in. joints might not be sufficient. They assume 3/8 in. ($W_{min} = 3/8$) for the first trial. This means that the maximum joint expansion is 3/8:3/8 or 100 percent ($\Delta W = 100$).

Step 5

The sealed depth design curves are found in Figure 27. Entering 100 percent at the bottom and 120 percent from the left side, a point of intersection is obtained. It happens to be at a point where D_{max}/W_{min} = 1.05 or $D_{max} = 1.05/0.375 = 0.4$ in. This is slightly below the minimum specified 0.5 in. and maybe a $\frac{1}{2}$-in. joint would be more desirable in this case[11].

Step 6

A $\frac{1}{2}$-in. joint (after going through Steps 4 and 5 again) can have maximum depth of 0.95 in. which is all right ($\Delta W = 75$; $D_{max} = 0.95$).

Step 7

The $\frac{1}{2}$-in. wide and 0.95-in. deep seal will curve in about 0.3 in. at $\Delta W = 75$ according to Figure 28, and is acceptable.

11/ Reduction of slab length would permit keeping the 3/8-in. joint.

APPENDIX C

Discussion of Proposed Design Procedure

Step 1 describes the physical dimensions of the bond-ductility specimen. The $\frac{1}{2}$-in. width and 1-in. depth were suggested because this combination so far gave the most uniform and reliable strain values in laboratory investigations. The 6-in. length appeared to be the minimum needed to eliminate the influence of the specimen ends on the strain pattern in the center of the seal.

Step 2 tells how to use Figure 26. The curve for this figure is taken from Figure 12 ($W_{min} = \frac{1}{2}$, and $D_x = 1$ in.) using only one-half of the calculated strain values (safety factor of two). It was extended for possible failures beyond the 200 percent limit (ΔW) where the validity of the parabolic curve ends. This part of the curve, therefore, is only an approximation and should have significance only in the laboratory test, to the results of which a safety factor of two is applied. Kuenning has shown that the maximum allowable strain (S_{max}) for a certain hot-poured rubber-asphalt sealer was around 110 percent (SF = 2 is included).

Step 3 and Step 4 are self-explanatory.

Step 5 describes how to use Figure 27 which is basically identical to Figure 14. It shows the correlation between joint width, depth of seal, joint expansion, and the maximum strain in the sealer. Even though the chart permits strain estimates for extensions up to 150 percent, it is questionable whether in actual practice this can ever be reached. Extreme expansions might affect the ability of the sealer to recover when the joint closes.

Step 6 calls for $\frac{1}{2}$-in. minimum sealed depth. This value was assumed to be the shallowest seal that can be placed under field conditions.

Step 7 calls for a maximum allowable curve-in of $\frac{1}{2}$ in. This was considered the maximum depth at which the surface of the sealer could be kept clean through the suction action of passing traffic.

Finally, it should be emphasized that more thought and data are needed to check this seal design procedure and the assumptions.

APPENDIX D

Test Road to Check Joint Seal Design Theory

The basic outline of the joint seal design theory was presented at the last meeting of Committee D-3 of the Highway Research Board. It was considered useful to test on the road various combinations of joint width and depth of seal. Through cooperation of the State of New York and Committee D-3 some 140 transverse joints were sealed in September 1958. The joint widths were $\frac{1}{4}$, 3/8 and $\frac{1}{2}$ in. The depth of the sealer in the joints was $\frac{1}{2}$, 1 and 2 in. A hot-poured rubber-asphalt sealer was used to fill the joints. The joint expansion and the curve-in depths will be measured periodically and compared with the predicted theoretical values.

APPENDIX E

Miscellaneous Equations

A. <u>Equation for Curves in Figure 14</u> (also Figures 6 to 13 and Figure 27 in Appendix B)

$A_p = \frac{2}{3} H W_x$ (parabolic area) and

$H = \frac{3}{2} \frac{A_p}{W_x}$ but $A_p = \frac{1}{2} (W_x - W_{min}) D_x$, therefore

$$H = \frac{3}{2} \frac{\frac{1}{2} (W_x - W_{min}) D_x}{W_x} = \frac{3}{4} \frac{D_x}{W_x} (W_x - W_{min})$$

$$L = \frac{1}{2} \sqrt{W_x^2 + 16 H^2} + \frac{W_x^2}{8H} \ln \frac{4H + \sqrt{W_x^2 + 16 H^2}}{W_x}$$

Substituting H

$$L = \frac{1}{2} \sqrt{W_x^2 + 9 \frac{D_x^2}{W_x^2} (W_x - W_{min})^2} + \frac{W_x^2}{6 \frac{D_x}{W_x} (W_x - W_{min})} \ln \frac{3 \frac{D_x}{W_x} (W_x - W_{min}) + \sqrt{W_x^2 + 9 \frac{D_x^2}{W_x^2} (W_x - W_{min})^2}}{W_x}$$

$$= \frac{1}{2W_x} \sqrt{W_x^4 + 9 D_x^2 (W_x - W_{min})^2} + \frac{1}{6} \frac{W_x^3}{D_x (W_x - W_{min})} \ln \frac{1}{W_x} \left(3 \frac{D_x}{W_x} (W_x - W_{min}) + \frac{1}{W_x} \sqrt{W_x^4 + 9 D_x^2 (W_x - W_{min})^2} \right)$$

$$S_{max} = \frac{L - W_{min}}{W_{min}} \times 100;$$

$$S_{max} = \left[\left[\frac{\frac{1}{2W_x} \sqrt{W_x^4 + 9 D_x^2 (W_x - W_{min})^2} + \frac{1}{6} \frac{W_x^3}{D_x} \frac{1}{W_x - W_{min}} \ln \frac{1}{W_x^2} \; 3 D_x (W_x - W_{min}) + \sqrt{W_x^4 + 9 D_x^2 (W_x - W_{min})^2}}{W_{min}} \right] - W_{min} \right] \times 100$$

B. Theoretical Joint Expansion Limit for a Sealer in any joint is 200 Percent.

If $A_s = (W_{min} \times D_x)$ at 0 percent expansion (ΔW) the cross-sectional area of the joint at 200 percent expansion will be:

$$A_s + 2A_p = 3\,(W_{min} \times D_x) \text{ or}$$

$$2A_p = 2\,(W_{min} \times D_x) \text{ and } A_p = (W_{min} \times D_x)$$

$$\text{but } A_p = \frac{2}{3} H W_x = (W_{min} \times D_x) \text{ from which}$$

$$H = \frac{3}{2}\,\frac{W_{min} D_x}{W_x} \quad \text{at 200 percent expansion } W_x = 3W_{min}$$

$$\text{or } H = \frac{3}{2}\,\frac{W_{min} D_x}{3W_{min}} = \frac{1}{2} D_x$$ or the sealer has to break (theoretically) in the middle at $\Delta W = 200$

C. Equation for Curves in Figures 15 to 22.

$$A_p = \frac{1}{2}\,(W_x - W_{min})\,D_x$$

$$H = \frac{3}{2}\,\frac{A_p}{W_x} = \frac{3}{2}\cdot\frac{1}{2}\,\frac{(W_x - W_{min})\,D_x}{W_x} = \frac{3}{4}\,D_x\,(1 - \frac{W_{min}}{W_x}) \qquad (1)$$

$$\text{as } \Delta W = (\frac{W_x - W_{min}}{W_{min}}) \times 100 = 100\,\frac{W_x}{W_{min}} - 100$$

$$\text{or } \Delta W + 100 = 100\,\frac{W_x}{W_{min}}$$

$$\text{or } \frac{1}{\Delta W + 100} = \frac{1}{100}\,\frac{W_{min}}{W_x}$$

$$\text{or } \frac{W_{min}}{W_x} = \frac{100}{\Delta W + 100} \text{ which substituted in (1) gives } H = \frac{3}{4} D_x\,(1 - \frac{100}{\Delta W + 100})$$

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