THE PUMPING OF CONCRETE PAVEMENTS IN NEW JERSEY, CORRECTIVE MEASURES EMPLOYED, AND FUTURE DESIGNS

William Van Breemen, Engineer of Special Assignments,
New Jersey State Highway Department

SYNOPSIS

Pumping at joints in concrete pavement slabs was first observed in New Jersey in 1930. It occurred on all pavement of standard design in which dowels 3/4-in. in diameter were used for load transfer. A 100 per cent increase in the number of dowels did not eliminate the trouble. The use of crushed stone drains along the edge of the pavement was only partially effective in stopping pumping.

In 1932 a test road was built over a silty-clay soil. One joint with no load transfer device, two with six 3/4-in. round dowels in the 10-foot width of pavement, two with twelve 3/4-in. dowels, and several with various combinations of heavy rectangular and channel type dowels were placed in the slab. Continuous applications of heavy loads under adverse moisture conditions indicated that the use of a load transfer device composed of 2-in. channel-dowels was necessary to prevent faulting and subsequent pumping.

A recent survey of 60,000 channel-dowel joints on heavy duty highways disclosed only three failures that were caused by pumping. No faulting was found at these joints and the failures had occurred by sagging of the pavement. The stone drains along the edge of the pavement were partially clogged with subgrade soil.

A study of pavements laid on sub-bases composed of granular materials lead to the conclusions that their use minimized pumping, reduced damage due to frost action and increased load bearing capacity. All pavements built since 1939 are supported on a layer of bank-run sand, gravel or cinders 8 inches in thickness. To date, where granular materials have been used in conjunction with channel-dowel joints, pavements have remained true to grade, cracks are few and far between, and there have been no indications of pumping, even under heavy truck traffic.

A study of wood for use in expansion joints shows that for most varieties, loads in excess of 500 and less than 1000 pounds per square inch will be required to cause compression of the fibers. If loading is continued, a point is reached where no further compression is obtained. Some varieties may be compressed to as much as 50 per cent of the original thickness. If dry wood is compressed to 50 per cent of its thickness, it will recover to about 65 per cent and remain at that thickness as long as it remains dry. Soaking in water will cause the wood to swell to 94 per cent, and for some varieties more than 100 per cent, of its original thickness. Repeated compression, drying and soaking will result in a permanent reduction in thickness. These tests indicate that wood as an expansion joint filler will have the following merits:

1. Unlike the conventional bituminous fillers, wood will not extrude, regardless of the extent of joint closure or infiltration. (This applies only to wood with the grain direction installed vertically).

2. Unlike other fillers, the wood is expected to retain sufficient swelling capacity and resiliency to prevent the detrimental accumulation and distribution of infiltrated material in the joint spaces which, in many locations, has caused rupturing of the concrete.

Due to the limited time permitted, the following material is perhaps not presented in strict accordance with the "Outline of Proposed Studies" agreed upon by the Committee. However, it is submitted at this time with the hope that a general description of pumping and faulting as observed in New Jersey, together with comments concerning the corrective measures employed, their effectiveness, and future designs, will further the work of the Committee. Because pumping and joint faulting have been so commonly associated, much of what follows necessarily concerns joints and their behavior. For reasons of continuity and completeness, it has been found necessary in frequent instances to include information already presented in a paper prepared by Mr. Harold W. Giffin, Engineer of Survey and Plans, New Jersey State Highway Department entitled "Transverse Joints in the Design of Heavy Duty Pavements". Mr. Giffin's paper which has been published in the 1943 Proceedings of the Highway Research Board is recommended as essential reading to those concerned with the design of concrete pavements.

Pumping and Faulting in New Jersey

Pumping and joint faulting first occurred in New Jersey sometime preceding the winter of 1930-31 on U.S. Route 1 and U.S. Route 130, the principal routes between New York and Philadelphia. By the spring of 1931 it had become severe on those routes and had extended to certain other routes that were carrying an increasing volume of heavy truck traffic. Its occurrence in New Jersey at that early date resulted from the rapid increase in heavy trucking in the immediately preceding years, especially in those locations such as New Jersey where the short distances between centers of large population favored the use of trucks. In consequence, and as a matter of sheer necessity, important revisions in design were made 12 years ago, and from time to time thereafter, for the specific purpose of preventing pumping and faulting.

Type of Pavement and Subgrade Involved

In 1931, practically all of the pavements that were pumping, and had faulted joints, had been constructed as follows:

Thickness: 9 in. uniform.
Slab length: Mostly 35 ft. Some variables to maximum of 68-2/3 ft.
Width: 10 ft. minimum - 14 1/2 ft. maximum.
Reinforcement: Single mat 2 in. below top surface. 3/8 in. longitudinal bars 7 1/2 in. c. to c.
Corner Reinforcement: 1/2 in. hairpin bars, all corners, 2 in. below top surface.
Load Transfer: Six 3/4 in. round dowels, 20 in. long, 3 at each end of joint 12 in. c. to c.
Joint Filler: 1/2 in. Thickness Premoulded Bituminous. Poured bitumen at top.

Practically all of these pavements had been laid directly upon whatever native subgrade soil was encountered. Except in very infrequent locations where the soil was found to be exceedingly unstable the subgrades had received no treatment other than shaping to grade and rolling. Inasmuch as subgrades varied from practically clean sand and gravel to silts and clays considerable variation in behavior resulted under identical conditions of pavement design and traffic. Many long stretches of pavement laid on inferior subgrades were carrying heavy truck traffic shortly after completion and in some locations serious pumping and faulting had developed after but three years of service.

Conditions Noted in 1931

Detailed investigations were made early in 1931 to determine the causes and extent of pumping and faulting. Conditions noted at that time appear to have been more or less typical of the pumping and faulting occurring in other parts of the country from time to time, especially recently as a result of heavy, wartime hauling. These conditions have been so adequately described by numerous observers in recent years that no general description appears necessary. However, some comments concerning certain conditions noted in 1931 may be appropriate.

1. The direction of the fault, step, or offset in the pavement surface was invariably such that as the wheels left one slab end ("leaving end") and crossed the joint space they dropped down onto the depressed slab end ("receiving end"). Faulting of as much as 3/4 in. was frequent.

2. The "leaving ends" were not depressed. (With but very few exceptions the "leaving ends" are still not depressed - have never been mud jacked even where immediately adjacent to "receiving ends" that have required mudjacking several times.)

3. Only minor pumping was occurring at the cracks. No cracks had faulted even in areas where joint pumping and faulting were severe. (In general, this condition still obtains today. Compared to the number of joints that have pumped, faulted, and required mudjacking, and repetitions of mudjacking, the number of pavement failures at cracks has been insignificant. Within recent years, however, some cracks have faulted. These instances will be discussed later.)

4. The water contributing to pumping was found to be almost entirely surface water that had infiltrated to the subgrade through open joints, and at points along the shoulder line. (This was substantiated by the fact that pavements on high fills and in cuts behaved more or less alike with respect to how soon they started to pump after rain began to fall, and with respect to the severity of pumping and faulting.)

5. The joint fillers or sealers did not exclude surface water. The independent movement of the slab ends had done much to destroy their sealing value, especially during cold weather when the bituminous sealing materials were brittle. It was noted that considerable quantities of water were swept into open transverse joint spaces by traffic.

In March, 1931, a section of the depressed end of a slab at a joint which had faulted 1 in. was removed for examination. The pavement was on a 2 foot fill. Heavy rains had fallen two days previously. The following note was made:

"The subgrade under the portion of the slab removed is clay - covered with a layer of water 1/8 in. to 1/4 in. deep over an area 5 ft. wide parallel to the joint. Apparently this subgrade is impervious to water. Unless pumped out, the water seems to have no means of escape other than by evaporation or slight seepage. A hole drilled through the pavement 14 ft. from the joint showed comparatively dry, firmly compacted subgrade." Soil sample No. 354253 shown in Table 2 represents the subgrade soil in this area.

The 3/4 in. round dowels were found very much out of parallel and the concrete around them was porous. The midsection of each dowel was bent for a length of about 4 in. in the shape of a flat "S", and the dowel holes had been enlarged by a crushing or wearing action of the dowels on the concrete for about 2 in. inward from the faces of the joint. These conditions are probably typical of most faulted joints installed prior to 1931.

In 1932, as a temporary measure, the number of 3/4 in. round dowels per joint was increased from six to twelve, and various devices were tried to maintain them in prroer alinement. Periodic inspections made of these joints disclosed that the additional number of dowels probably retarded the rate of failure to some extent but that serious faulting developed nevertheless.

In addition to increasing the number of dowels, shallow crushed stone drains of various description were constructed immediately adjacent to the edge of the pavement in some locations, and a special effort was made to construct them in a manner most likely to carry off any free water that might tend to collect under the pavement. Drains of this type were installed in 1933 in some sections of U.S. Route 1, immediately adjacent to a heavy trucking lane laid in 1932 with joints having twelve 3/4 in. round dowels. Their effectiveness is difficult to evaluate, however, inasmuch as the joints adjacent to the drains are now faulted just about as much as those where the drains were omitted. There is at present an average faulting of about 1/4 in. - maximum 1/2 in. ± and there are some indications of pumping. Probably, due to progressive pumping of subgrade material into them, these drains have become less effective from year to year. The average daily truck traffic on this lane (1940 count) is about 1300, and many of the trucks are heavily laden truck-trailer units. The subgrade soil is essentially clayey and corresponds more or less to Sample No. 354,234. (Table 2).

Test Road

A circular concrete test road was constructed in 1932 on silty-clay soil, primarily to make an accelerated test of various joint types, and to further observe the process of pumping. Some of the joint types tested were:

One plain joint - no dowels - no load transfer - consisting simply of a 1/2 in. width space filled with premoulded bituminous filler.

Two joints with six 3/4 in. round dowels in each (same design as used prior to 1932), 1/2 in. premoulded bituminous filler.

Two joints with twelve 3/4 in. round dowels (same as temporarily adopted in 1932), 1/2 in. premoulded bituminous filler.

Several other joints having various combinations of heavy rectangular dowels, plus bearing angles for the dowels, plus sheet metal flashing.

Traffic consisted of one Mack truck pulling a loaded trailer at approximately 9 miles per hour. The maximum axle loads were:

Mack Truck		Trailer	
Front	- 5,500 lbs.	Front	- 13,400 lbs.
Rear	- 17,300 lbs.	Rear	- 32,800 lbs.

As concerns this test, the following remarks seem appropriate:

1. From August 1 to 18, during dry weather, the equipment completed approximately 1300 trips around the track at various speeds while numerous deflection readings were made. No visible or measurable deterioration of any kind was noted during this period except slight pumping at the plain joint during a shower on the afternoon of August 3.

2. During the morning of August 18 water was applied by means of sprinkling wagons, and in the afternoon and evening heavy rains fell. Shortly after the application of water the pavement at the plain joint started to pump noticeably and pumping to a lesser degree started at most of the other joints. By 8:00 P.M., after 840 trips that day, the plain joint had faulted 3/16 in. By midnight, 1120 trips, it had faulted 1/2 in. and a crack appeared 5½ ft. from the depressed end. Running on the track ceased at midnight.

3. The following morning an examination disclosed no faulting at any other joints, nor any cracking, except at the plain joint as noted above.

4. On August 22, after 4270 trips since August 18, more cracks were evident adjacent to the plain joint which had become faulted 1½ in. None of the joints with dowels had as yet faulted, and no cracking had developed in their vicinity. Sometime after the plain joint had faulted 1 in. or more the "leaving" slab end at this joint also became depressed, accompanied by cracking.

5. On August 23, after 5310 trips since August 18, cracks developed in the pavement adjacent to both of the joints with six 3/4 in. round dowels. No measurable faulting was evident. But due to the erosion of the subgrade and the formation of cracks near these joints, the pavement at the joints had sagged somewhat. Eventually these joints did fault slightly. On this date, no faulting or cracking was visible at any other joints that had more or heavier dowels.

It is necessary to mention here that the conditions of test were much more severe than normally occur in service. After August 18, the equipment was kept running continuously each day for 12 hrs. until the pavement at the plain joint became practically impassable. Prior to August 18, dikes had been constructed along the edges of the pavement to prevent the escape of surface water; in fact for a time the entire roadway was flooded and the slabs were more or less under water. The supposition was that those joints that best survived the ordeal would probably be most satisfactory in actual service. In consequence, there was a distinct difference in the behavior of the joimts in the test road as compared to joints in service. No joints, other than those with six 3/4 in. round dowels faulted measurably. The typical failure of the joints with various types of load transfer consisted of a pumping out of subgrade soil from under the pavement at the joints followed by a sagging of the pavement at the joints - both adjacent slab ends depressing the same amount. As the test progressed, the cracks also pumped violently - but none faulted. The absence of appreciable faulting, even at the joints with the weakest load transferring devices, appears to be explained by the flooded conditions, magnitude of load, and the lack of sufficient repetitions of load to permit the dowels to progressively crush, chew out, or wear away enough concrete to appreciably decrease their effectiveness. In short, the attempt to accelerate the faulting of the joints by the application of a limited number of heavy loads under extremely adverse subgrade conditions was, for the most part unsuccessful.

Although all of the joints with sturdy load transferring devices eventually pumped and sagged, to date, with but very few exceptions, no comparable behavior has been observed at similar types of joints that have been in service on heavy trucking routes for from 6 to 10 years. Considering the very rapid rate of faulting and pavement failure at the plain joint as compared to the joints with load transfer, this test at least indicated that load transfer at all points of interruption in the continuity of the pavement, such as at joints and cracks, is definitely beneficial.

Revisions in Design

The use of 3/4 in. round dowels was abandoned in 1933 and a much stronger joint structure was designed and adopted the same year. Except for occasional minor revisions, the joint so designed was used in conjunction with all concrete pavements constructed during the period 1933-1942. Load transfer was furnished by 2 in. channels, 20 in. long, an average of 12 in. apart. Bearing angles were provided above and below the dowels adjacent to the joint faces to prevent the dowels from crushing or wearing out the concrete at points of high bearing pressure. Sheet metal flashings were provided to exclude as much surface water as possible.

Effectiveness of Heavy Joint Design

More than 60,000 channel-dowel joints were installed during the period 1933-1942. Many are in service in pavement laid directly on soil susceptible to pumping which also carries extremely heavy truck traffic. To date, failure at these joints due to pumping has been negligible. A recent survey of the heavy trucking routes involving an examination of several thousand channel-dowel joints disclosed pavement failure due to pumping at only three joints. However, in these instances, no faulting had occurred. Instead, the pavement immediately at the joints had sagged to a maximum of 5/8 in. These joints are all in a heavy trucking lane (laid 1936) within a few hundred feet of each other in an area where subgrade and drainage conditions are particularly adverse. Also, in this area, appreciable sagging of the pavement was found at several cracks, and one crack had faulted ½ in. Pumping in varying degrees is general in this vicinity and, unless prevented, the pavement is expected to become progressively depressed at many other joints and cracks. The stone drains constructed immediately adjacent to this lane were found partially clogged by pumped-out subgrade material. The average daily truck traffic over these joints (1940 count) is 1050. The subgrade is represented by Samples No. 354,253 and 354,254. In considering the performance of these joints it is desirable to add that joints in an adjacent roadway constructed in 1928 with six 3/4 in. round dowels pumped badly and faulted as much as 1 in. within three years after construction, under less severe traffic conditions.

Some 20 miles south of this pumping area faulting of 1/4 in. and 7/16 in., respectively, was found at two channel-dowel joints in pavement constructed in 1934. However, faulting at these joints was not due primarily to structural failure as a result of heavy traffic. Instead, the effectiveness of the dowels was found to have been destroyed by the infiltration of practically incompressible silty, sandy material which, having accumulated in the lower portion of the joint spaces and displaced the filler, offered sufficient localized resistance to closure of the joints during pavement expansion as to rupture the concrete supporting the dowels. This having occurred, the possibility of infiltrated material eventually destroying the effectiveness of load-transferring mechanisms is a phase of joint design which merits serious consideration. The destructive effects of infiltration are discussed at length in Mr. Giffin's paper.

On other routes, no faulting or defects due to pumping are as yet apparent at the channel-dowel joints, even in lanes constructed directly on native, clayey soil which have been carrying in excess of 3000 trucks per day for more than six years. Many of these joints are immediately adjacent to 3/4 in. round dowel joints in adjoining older lanes which have faulted badly.

In general, the performance of the channel-dowel joints from the standpoint of preventing faulting and diminishing pumping has been satisfactory. Very likely, where truck traffic is heavy and subgrades are clayey, there is some slight pumping and, in consequence, it is expected that eventually the pavement at the joints will become depressed. On the other hand, except where infiltration destroys the effectiveness of the dowels, no faulting is anticipated.

In comparison with the unsatisfactory performance of weaker joints, the benefits derived from the use of these joints are beyond question. Actually, the pavement in the immediate vicinity of most channel-dowel joints in heavy duty pavements appears to be in as good if not better condition than elsewhere because of more apparent deterioration at intermediate cracks.

In view of this, it would seem that especially in those locations where the cost of granular or other suitable subbase material is prohibitive, and where successful treatment of the native soil is in doubt, the possibility of at least minimizing pumping and at the same time completely preventing faulting by the utilization of sturdy joint structures, properly installed, is certainly worthy of serious consideration.

Drains

Because the test road demonstrated that sturdy joints would only retard and not completely prevent pumping under extremely severe conditions, longitudinal crushed stone drains, constructed immediately adjacent to the pavement edge, were specified in most contracts during the period 1934-1939. The effectiveness of these drains is not known due to the indeterminate influence of the improved joint structures and because even where drains were not included pumping at present appears to be no more pronounced than where they were. The stone drains were expensive and their use was abandoned in favor of subbase material.

Subbase

In view of the generally recognized benefits to be derived from the use of granular subbase material with respect to minimizing or preventing pumping, reducing damage due to frost action, and increasing the load bearing capacity of the pavement, practically all pavements constructed since 1939 are supported on a layer of bank-run sand, gravel, or cinders, at least 8 in. thick. Insufficient time has elapsed to justify any predictions as to the effectiveness of these materials on a long-range basis. But to date, where granular subbase materials have been used in conjunction with channel-dowel joints, the pavement has remained practically true to grade, cracks are few and far between, and there have been no indications of pumping, even under the most severe conditions imposed by heavy trucking. Although the use of properly graded subbase material may prevent pumping there is considerable doubt as to whether, in the absence of adequate load transfer at joints and cracks, it will also prevent faulting. Some of the older pavements in New Jersey that have carried heavy truck traffic for years were laid on old macadam roads that were scarified, regraded, and rolled. Although the joints in these pavements lacked sturdy load-transferring mechanisms they have not faulted appreciably. But this type of construction is, of course, exceptional. Where, in one location, the pavement constructed in 1923 was laid on a layer of granular materials the joints in one area have faulted to a maximum of 9/16 in. Whether faulting was due to pumping, further compaction, or progressive rearrangement of the underlying material is not known. Subbase Sample No. 354,111 was removed from under a joint in this area which had faulted 9/16 in. Another section of the same job on a different kind of subbase has no faulted joints. The subbase in this section is represented by Sample No. 354,110. This pavement was laid on a regraded gravel road which was about one year old, and the subbase materials are presumably the original gravel surfacing, scarified, regraded, and rolled. This too is an exceptional type of construction.

Unfortunately, there are no known locations in New Jersey where a layer of subbase material as it is usually thought of was utilized in conjunction with weak joint structures on roads which have been carrying heavy truck traffic. Consequently, it cannot be stated definitely whether the use of subbase material without the assistance of adequate load transfer at joints and cracks will or will not prevent faulting. However, if the performance of pavements constructed on native granular soils is any criterion, the chances are that subbase alone is very likely to prove inadequate. In New Jersey, the degree of faulting on granular soils has been very variable. On one route, short stretches laid on native somewhat coarse, sandy soil did not fault, and perhaps did not pump, even under extremely heavy wartime hauling of sufficient intensity to cause complete failure in other areas laid on clayey soil. But faulting of more than 1/2 in. occurred in other sandy areas on the same route. The best that can be said at this time is that faulting has, and has not, occurred on granular soils, all depending upon the influence of many factors. So far as is known, the presence of pumping is not necessarily essential to the development of faulting if the underlying materials are susceptible to further compaction or rearrangement.

Present Views Concerning Subgrades and the Process of Pumping

Until very recently, no attempt has been made to classify pumping and nonpumping types of soil on the basis of detailed physical and chemical analysis. The practice, rather, has been to judge subgrade soils primarily in terms of their permeability and susceptibility to erosion. Wherever pumping has occurred in New Jersey the subgrade soil and the adjacent shoulders are more or less impervious. In addition, the subgrade soil is susceptible to erosion.

The process appears to take place essentially as follows: Surface water infiltrates to the subgrade through leaking joints, and at other points affording entrance, and collects in whatever vacancies may exist between the bottom of the pavement and the subgrade surface. Apparently very little water is absorbed by the subgrade. The only part of the subgrade that seems to combine with the water to any extent and become susceptible to pumping is confined to a thin film on the upper surface of the subgrade, 1/4 in. or possibly less in thickness. The thin film of water-saturated subgrade frequently has the consistency of thick paint. If the film is scraped off, apparently stable unaffected subgrade is exposed. With the agitation of the entrapped water some of the finer particles in the water-saturated film mix with the water and are carried away. As the particles are progressively carried off new surfaces of unaffected subgrade are exposed which in turn are softened and carried away.

There appears to be little evidence to indicate that, in general, pumping is the result of a softening of the subgrade for any appreciable depth or that large masses of subgrade are expelled at any one time. If a considerable depth of soft, water-saturated subgrade were involved failure would, it seems, occur much more rapidly than has been observed. Therefore, for the time being at least, it is assumed that any soil that becomes practically impervious when compacted but which, nevertheless, remains susceptible to erosion when in contact with violently agitated water is, in all probability, a pumping type of soil. The suitability of subbase materials is also judged in certain respects from the same point of view.

At the present time, numerous samples of soil from pumping and non-pumping areas are being analyzed and it is admitted that the results may modify the foregoing views.

Some years ago an effort was made to devise an erosion test, primarily to determine the most erosion-resistant types of subbase, but the results were erratic and inconclusive. However, this test did tend to substantiate the opinion that all subgrade materials that are essentially impervious are susceptible to erosion, unless the grains are adequately cemented together by some agent.

Comments on Subbase

If the use of subbase is indicated, the ideal type of construction might be thought to be slabs resting upon a layer of porous granular material which in turn rests upon a more or less impervious subgrade. Theoretically, the infiltrated surface water will drain through the porous subbase to the subgrade surface and thence, percolating through the subbase, flow along the surface of the subgrade to an outlet of some sort. Apparently the desirable functioning of this type of construction would indicate that the subbase material be as "open" as possible and that the subgrade surface be true to grade so as to avoid the retention of water in depressions. But there appear to be limitations as to how near this ideal of "open" subbase may be approached.

In 1932, for a distance of several miles on one heavy trucking route, an attempt was made to end the pumping and faulting problem once and for all by constructing the pavement on 9 in. of 2½ in. crushed stone. No fine materials were used in combimation with the stone to fill the voids. Not long after being in service the pavement settled considerably in many places. An examination disclosed that the crushed stone had been pressed into the clayey subgrade during wet or thawing weather and that the clay in turn had worked up into the voids in the stone. After 12 years of service, the pavement is in much poorer condition than adjacent sections laid on native, pumping-type soil.

Sometime later, tests were made to determine whether or not, under conditions of pressure, moisture, or softening, clayey or silty subgrades would work up into the voids of granular materials consisting of relatively small particles, such as sand or sand gravel mixtures. Attempts were made to combine adjacent layers of soft clay and coarse building sand by the application of pressure and agitation in the presence of free water—without success. The most severe test made was performed as follows: A cylindrical sheet metal container 14 in. in diameter, 8 in. high, was filled to a depth of 4 in. with silty-clay soil which had been mixed with water to a soft, almost sloppy, buttery consistency. Four inches of fairly coarse, washed, white sand was then carefully placed on top of the clay and covered with a metal disk 13-3/4 in. in diameter to which pressure could be applied by means of a lever system. The container was then placed completely under water and kept submerged for two weeks. During this period a sudden load of not less than 2000 lbs. was applied to the disk approximately 12,000 times. The person who operated the lever was instructed to make every effort to combine the two materials. These efforts were unsuccessful. At the conclusion of the test a perfectly clean line of separation was found between the materials. There were no indications whatsoever that the clay had entered the voids in the sand. In spite of having been submerged for two weeks the clay was much more stable than before. The pressure had squeezed considerable water out of the clay and stabilized it. Similar tests with crushed stone, and open gravel, on the same consistency of clay resulted in complete combination of the materials under a single application of load.

Efforts were made recently to combine adjacent layers of soft, sloppy, clayey soil and sand by repeated applications of a pressure of 30,000 lbs. per sq. ft. without success.

In view of the foregoing it is believed that no subbase material containing large voids should be used unless particular care is exercised to completely fill the voids with sandy material even though this results in an appreciable lowering in the rate of percolation.

If sand or similar subbases are used it appears probable that:

1. No matter how much water is present, it is unlikely that subbase and subgrade will combine because of pressure alone.

2. If the subgrade should become soft or sloppy, from whatever cause, but has no lateral means of escape, it may possibly be stabilized by heavy wheel loads squeezing some of the water out of the subgrade into the subbase.

3. Even if there is free water in the lower portions of the subbase layer, but no free water at the upper surface, no pumping will occur.

4. The subbase acts as a protective coating to the subgrade tending to keep it stabilized, and preventing its erosion even though there may be free water on the surface of the subgrade.

This probably accounts in large measure for the success of pavements laid on a layer of sand or well-graded porous, granular material.

Although bank-run sands and gravels are usually thought to be more or less permeable numerous tests have shown that many of the bank-run materials available from many sources in New Jersey may be compacted into a practically impervious mass. Usually the impermeability of these materials is due to the presence of a very small percentage of clay or materials removable by elutriation. Future studies may indicate the desirability of processing most subbase materials.

At present very little is known as to the effect of the subbase materials used. It is very likely that their effectiveness is susceptible to considerable variation inasmuch as they vary from practically clean sand to bank-run gravel containing a fairly high percentage of clay. Some of the bank-run gravels are capable of being compacted into a very firm mass and their use perhaps increases the bearing capacity of the pavement considerably more than sand. On the other hand, material of that kind may be open to the objection that since it is likely to be impervious it may permit the accumulation of free water immediately under the pavement and thus eventually lead to failure by erosion. With this thought in mind, perhaps the most suitable subbase material may not necessarily be that which results in the greatest increase in bearing capacity but rather a material that has, in addition to sufficient bearing value, just enough porosity to prevent the accumulation of free water. The determination of the most suitable subbase materials to be used in conjunction with the construction of concrete pavements is regarded as one of the most important subjects for future study.

Wood Joint Filler

When the causes of pumping and faulting were investigated in 1931 it became apparent that the conventional bituminous fillers were unsatisfactory, in fact it was concluded that because of certain fundamental characteristics bituminous fillers could not satisfactorily exclude water and foreign material. Since then considerable study has been devoted to other materials and methods of sealing joints. Having as an objective the discovery or development of a filler that would fill the entire joint space at all times tests were made several years ago on various kinds of wood to determime the extent of recovery after compression. It was found that most woods after being compressed considerably will remain in a state of compression indefinitely, if kept dry, but that if soaked in water will swell a great deal. The possibility of using precompressed wood as a joint filler became apparent and, in consequence, many tests have been made, and are still in progress, to determine its behavior over a period of years. In addition, a few joints in service have been filled with precompressed wood.

The primary purpose of these tests has been to determine how long various kinds of wood retain their capacity to swell, under what conditions they might lose that capacity, and at what rate.

Since the efficacy of joint filling materials must of necessity be judged on the basis of long-range performance, and since there appears to be no reliable way to accelerate tests of this kind, nor even to subject the materials in the laboratory to conditions comparable to actual service, much still remains unanswered. However, information concerning the manner of testing and the behavior of the test specimens and installations to date may at least serve to stimulate interest in this material.

General Behavior of Wood

Most woods resist appreciable compression at pressures less than 500 lbs. per sq. in. At pressures somewhat greater, but usually less than 1000 lbs. per sq. in., depending upon the kind of wood and its grain structure, the elastic limit is exceeded and appreciable compression occurs with little additional pressure. For example, the general behavior of white pine is that its thickness is decreased only 5 per cent by a pressure of 700 lbs. per sq. in. but decreased 25 per cent at 1000 lbs. per sq. in. and 50 per cent at 1600 lbs. per sq. in. (All percentages are in terms of the initial uncompressed thickness of the wood.) If compression is continued, so-called "hard-bottom" is reached eventually at which point little additional compression occurs even under very high load. The amount of compression that occurs before hard-bottom is reached varies considerably with different kinds of woods. In general, the lower the specific gravity of the wood the more it may be compressed. White Cedar, for example, reached hard-bottom after being compressed about 65 per cent whereas Cypress may reach the same point at 50 per cent. (These percentages vary considerably for even the same kinds of wood, depending primarily upon the specific gravity and the grain direction.) Water-soaked wood is somewhat easier to compress.

In general, if dry wood is compressed to 50 per cent of its initial thickness, in a device that prevents transverse spreading, it will recover to about 65 per cent and remain practically at that thickness indefinitely, if kept dry. Humid atmosphere does not cause appreciable further recovery. The first time it is soaked in water it will swell to within at least 94 per cent of its initial thickness, and some woods swell to more than 100 per cent. If recompressed in a clamp to 50 per cent, and kept clamped at 50 per cent until dry, it will shrink to about 48 per cent - that is, it will eventually become loose in the clamp. If again soaked, it will not swell quite as much as it did the first time. Repetitions of clamping at 50 per cent, drying while clamped, followed by soaking, cause a further reduction in "swelled thickness". This progressive loss has been termed "compression-shrinkage". The rate of loss is not constant, however, inasmuch as it diminishes as the process is repeated, until, at normal temperatures, a point is reached where the "swelled thickness" remains more or less stable. The conditions under which the wood is kept clamped have a marked effect upon the rate of loss. Oven-drying causes the highest rate of loss, and the lowest rate occurs if the wood is kept wet at all times. Loss due to air-drying at normal temperatures is intermediate between oven-drying and constant soaking.

Most of the tests were made as described in the foregoing, and some of the results are shown in Table 1. For example, Cypress Specimen 165D was compressed to 50 per cent of its initial thickness on September 1, 1942 (3220 lbs. per sq. in. pressure). It recovered to 63.3 per cent, and had swelled to 94.7 per cent during soaking for 24 hours. After one cycle of being clamped at 50 per cent and oven-dried it swelled to 87 per cent, and after 5 cycles it swelled to 81 per cent. As will be noted, its least "swelled to" thickness was 65.3 per cent on May 1, 1943, after 71 cycles. After May 1, 1943, oven-drying was discontinued and the specimen was air-dried. From then on the "swelled to" thickness increased somewhat and appears now to have become stabilized at about 70 per cent. During the 27 additional cycles between August 1, 1943 and August 1, 1944 no further loss in swelled thickness occurred. Cypress Specimen 165W (part of the same piece of lumber as 165D) has been kept under water continually and, as may be noted, its "swelled to" thickness exceeds that of Specimen 165D. Both of these specimens, and those that are described in the following, have lost an indefinite amount of thickness due to some progressive transverse spreading in the clamps.

The relative behavior of various kinds of wood is indicated by Specimens No. 272, 273, 274, and 275. All of these specimens were subjected to identical treatment. During the first 12 cycles they were oven-dried at 100°F, air-dried thereafter. As indicated, Redwood suffered the greatest loss.

It is desirable to mention here that the "swelling range" of the wood is considered to be the difference between the thickness shown in the "Swelled to" columns and the "Recovered to" column. It is assumed that the wood will be installed at its "Recovered to" thickness and that consequently the greater the difference between its "Recovered to" thickness and its "Swelled to" thickness the more likely it will be to always fill the joint space. Therefore, while the "Swelled to" thickness of White Pine Specimen No. 275 is 86.5 per cent as compared to 77 per cent for Cypress Specimen No. 272 its actual swelling range, due to its greater "recovered thickness", is only 1.8 per cent greater.

Other specimens are being tested to determine the loss in swelling capacity due to being kept in a protracted state of compression such as would likely occur in service during the summer when the wood may be under considerable compression for several months. These specimens are Nos. 172B, 173, and 174B. They have been stored in damp sand continually, clamped at one-half their initial thickness, since September 1942 except that every three months they are released and soaked for 24 hours, measured, reclamped, and returned to storage. As will be noted, these specimens now swell less than any others. Whether or not they will eventually fail to swell at all is not known. It seems problematical as to whether these protracted compression tests are indicative of what may be expected in service inasmuch as daily as well as seasonal variations in joint width will provide some relief, and may tend to keep the wood "alive", so to speak, whereas long-continued compression at a constant dimension may cause the wood to acquire a so-called "set".

Cypress Specimen 172A has been kept clamped at one-half thickness and stored continually out-of-doors exposed to sun, rain, freezing, and thawing. Every three months it is released and its swelling recorded.

Due to restrictions imposed by the war only a few joints in actual service have been filled with precompressed wood. These installations were made more than two years ago and the performance to date has been satisfactory. During dry weather some shrinkage apparently occurs in the wood near the pavement surface permitting the infiltration of fine material to a depth of one or two inches. Below this depth, however, the wood appears to remain damp and in a swelled condition, and has so far been found to completely fill the joint space at all times.

Until the merits of precompressed wood are more definitely determined, and until the manufacturing process is worked out, the intention is to use uncompressed, clear, heartwood Cypress specially fabricated so that the grain direction of the main body of the wood is vertical in the joint space. Because this ordinary wood is not expected to completely fill the joint space in the winter time an extra strip has been attached to the bottom to exclude as much surface water as possible. As mentioned in Mr. Giffin's paper, it is supposed that infiltrated material will progressively accumulate in whatever vacancies occur and eventually compress the wood causing it to function in time somewhat like precompressed wood.

Two of the important merits of wood joint filler appear to be:

1. Unlike the conventional bituminous fillers, wood will not extrude, regardless of the extent of joint closure or infiltration. This applies only to wood with the grain direction installed vertically and only if restraining means such as dowels are provided to avoid lateral extrusion at the edges of the pavement.

2. Unlike other fillers, the wood is expected to retain sufficient swelling capacity and resiliency to prevent the detrimental accumulation and distribution of infiltrated material in the joint spaces which, in many locations, has caused rupturing of the concrete.

Recent Widening and Faulting of Cracks

As mentioned previously, no faulted cracks were found in 1931, in fact not until very recently has the faulting of cracks been of any consequence in New Jersey. Early in 1945, however, investigations were started to determine the cause of wide cracks which were recently observed to be increasing in number in a few locations in New Jersey. Due to a lack of time and sufficient personnel these investigations are not as complete as desired and, of necessity, the causes in certain instances are still not conclusive. However, the findings to date are believed to be of sufficient importance to be brought to the attention of the Committee, and to merit the attention of all others who are concerned with the design of joints, for the following reasons:

Number	Species	Manner of Testing	Compression Data: Uncomp Thick	Compression Data: Spec Grav	Compression Data: Date	Compressed To: Meas.	Compressed To: %	Load P.S.I.	Recovered To: Meas	Recovered To: %	Swelled To: Meas.	Swelled To: %	Date First Clamped	Clamped Thickness: Meas	Clamped Thickness: %	1 Cycle Swell: Meas	1 Cycle Swell: %	5 Cycle Swell: Meas	5 Cycle Swell: %	Nov. 1, 1942: No of Cycles	Nov. 1, 1942: Swelled to Meas	Nov. 1, 1942: Swelled to %	Feb 1, 1943: No of Cycles	Feb 1, 1943: Swelled to Meas	Feb 1, 1943: Swelled to %
165 D	Cypress	Wet & Dry Cycles	1.50"		Sept 1, 1942	0.75"	50	3220	0.95"	63.3	1.42"	94.7	Sept 3, 1942	0.75"	50	1.30	87	1.22	81	23	1.03"	68.7	50	0.99"	66.0
165 W	Cypress	Wet Cycles	1.50"		Sept 1, 1942	0.75"	50	3220	0.95"	63.3	1.42"	94.7	Sept 9, 1942	0.75"	50	1.38	92	1.30	87	23	1.26"	84.0	49	1.20"	80.0
172 A	Cypress	Weathering	1.50"		Sept 1, 1942	0.75"	50	2740	0.95"	63.3			Sept. 28 1942	0.75"	50					1	1.34"	89.4	2	1.25"	83.3
172 B	Cypress	Damp Sand	1.50"		Sept 1, 1942	0.75"	50	2740	0.95"	63.3			Sept 28, 1942	0.75"	50					1	1.31"	87.3	2	1.22"	81.4
173	Douglas Fir	Damp Sand	1.50"		Sept 1, 1942	0.75"	50	3720	0.91"	60.7			Sept 28, 1942	0.75"	50					1	1.19"	79.4	2	1.14"	76.0
174 B	White Cedar	Damp Sand	1.50"		Sept. 28, 1942	0.750"	50	1300	0.95"	63.3			Sept 28, 1942	0.75"	50					1	1.11"	74.0	2	1.00"	66.7
272	Cypress (Heartwood)	Wet & Dry Cycles	1.47"	.595	Sept. 30, 1942	0.735"	50	5780	0.952"	64.6	1.43"	97.0	Oct 4, 1943	0.75"	50.8	1.25	85								
273	Douglas Fir	Wet & Dry Cycles	1.49"	.566	Sept 30, 1942	0.744"	49.7	4880	0.942"	63.4	1.46"	98.0	Oct 4, 1943	0.75"	50.3	1.28	86								
274	Redwood	Wet & Dry Cycles	1.49"	.405	Sept. 30, 1942	0.745"	50	1846	0.931"	62.4	1.40"	94.0	Oct 4, 1943	0.75"	50.2	1.07	72								
275	White Pine	Wet & Dry Cycles	1.48"	.413	Sept. 30, 1942	0.740"	50	1580	1.07"	72.3	1.52"	103.0	Oct 4, 1943	0.75"	50.7	1.37	93								

Number	Species	Manner of Testing	May 1, 1943: No of Cycles	May 1, 1943: Swelled to Meas	May 1, 1943: Swelled to %	Aug 1, 1943: No. of Cycles	Aug 1, 1943: Swelled to Meas.	Aug 1, 1943: Swelled to %	Nov. 1, 1943: No. of Cycles	Nov. 1, 1943: Swelled to Meas	Nov. 1, 1943: Swelled to %	Feb 1, 1944: No. of Cycles	Feb 1, 1944: Swelled to Meas	Feb 1, 1944: Swelled to %	Mar 1, 1944: No of Cycles	Mar 1, 1944: Swelled to Meas.	Mar 1, 1944: Swelled to %	May 1, 1944: No of Cycles	May 1, 1944: Swelled to Meas	May 1, 1944: Swelled to %	Aug 1, 1944: No. of Cycles	Aug 1, 1944: Swelled to Meas	Aug 1, 1944: Swelled to %
165 D	Cypress	Wet & Dry Cycles	71	0.98"	65.3	77	1.08"	72.0	85	1.06"	71.6	94	1.06"	71.6	96	1.05	70.0	101	1.05"	70.0	104	1.09"	73.0
165 W	Cypress	Wet Cycles	76	1.19"	79.4	86	1.19"	79.4	95	1.14"	76.0	105	1.16"	77.3	107	1.15	76.8	113	1.17"	78.0	118	1.15"	77.0
172 A	Cypress	Weathering	3	1.25"	83.3	4	1.19"	79.7	5	1.07"	71.7	6	1.09"	73.0				7	1.12"	75.0	8	1.15"	77.0
172 B	Cypress	Damp Sand	3	1.15"	76.6	4	1.14	76.0	5	1.04"	69.4	6	1.02"	68.0				7	1.02"	68.3	8	0.99"	66.0
173	Douglas Fir	Damp Sand	3	0.98"	65.4	4	0.97	64.7	5	0.88"	58.6	6	0.87"	58.0				7	0.88"	58.5	8	0.87"	58.0
174 B	White Cedar	Damp Sand	3	0.95"	63.4	4	0.96	64.0	5	0.88"	58.6	6	0.88"	58.6				7	0.89"	59.0	8	0.88"	58.5
272	Cypress (Heartwood)	Wet & Dry Cycles							6	1.13"	76.6	15	1.08"	73.4	17	1.08	73.4	22	1.08"	73.7	23	1.13"	77.0
273	Douglas Fir	Wet & Dry Cycles							6	1.15"	77.2	15	1.09"	73.2	17	1.10	74.0	22	1.09"	73.0	23	1.13"	76.0
274	Redwood	Wet & Dry Cycles							6	0.99"	66.3	15	0.97"	64.6	17	0.97	64.6	22	0.96"	64.5	23	1.00"	67.0
275	White Pine	Wet & Dry Cycles							6	1.29"	87.1	15	1.21"	81.7	17	1.21	81.7	22	1.20"	81.0	23	1.28"	86.5

Note – Percentages shown are per cent of Uncompressed Thickness

TABLE 1. PRECOMPRESSED WOOD TESTS

1. Highway engineers in general should be informed as to the conditions under which these wide cracks occurred in order that adequate provisions may be made to avoid their occurrence in future work.

2. In an effort to prevent the faulting of joints in future work it appears likely that many engineers may be planning to install larger dowels, or a greater number of dowels, without giving due recognition to the influence of certain important factors (notably the effects of rusting of the dowels or other metallic load-transferring mechanisms) which if ignored may soon lead to the development of serious defects.

The following is not to be regarded as an exhaustive report on the causes of wide cracks but, instead, primarily as a word of caution to those whose responsibility it may be to evaluate the merits of various types of joints and to decide upon which shall be used.

Apparently most of the cracks which have recently widened in heavy duty pavements in New Jersey occurred during early life, due to differential frost-heaving or subgrade settlement. For some years the reinforcing steel evidently was capable of maintaining the cracks at hair-line width. In recent years, however, some of these cracks have widened considerably, present width averages about 3/8 in. — maximum width 7/8 in. Where there is heavy trucking the wider cracks have faulted, some as much as 5/8 in. In practically all instances where the crack width is 1/4 in. or more the longitudinal reinforcing steel has broken.

These conditions were noted in the fall of 1944 on U.S. Route 22 in the vicinity of North Plainfield. This route carries heavy truck traffic. In this location 9 faulted cracks, ranging from 1/4 in. to 7/8 in. in width, were found in a stretch of pavement 2000 ft. long. All of these cracks are located within the middle third portion of the slabs. No cracks of any width were found at or within less than 9 ft. from the transverse joints. At all of these 9 cracks the reinforcing steel had failed. The pavement was constructed during the summer of 1938, as follows:

Thickness: 10 in. Uniform. Width of Slab: 10 ft. Length of Slab: 53 ft. Longitudinal Reinforcement: Sixteen 3/8 in. Diameter bars. Dummy or Contraction Joints: None. Subgrade: Essentially silty clay — no subbase. Load transfer at Expansion Joints: Twelve 2 in. channel dowels, (7/8 in. flange and 1/2 in. web). Prior to installation the dowels were given two coats of paint — white lead followed by red lead. Immediately prior to embedment in the concrete the dowels were given an additional coating of mineral oil. The coatings were applied the entire length of the dowel.

These particular coatings were used because comprehensive tests made in 1935 indicated that the bituminous coatings previously used on 3/4 in. round and 2 in. channel dowels did not prevent excessive resistance to sliding. These tests indicated that the white lead, red lead, and oil coatings, if applied in accordance with the specifications, would facilitate free sliding of a 2 in. channel dowel (10 in. embedment) at considerably less than 1000 lbs. These coatings were applied primarily to facilitate free sliding during early life inasmuch as it was expected that if freedom of slippage had been established initially the subsequent opening and closing of the joints would cause a continuing decrease in sliding resistance. In the light of present knowledge, these expectations were erroneous.

Inasmuch as measurements of the periodic variations in width at various types of experimental joints installed twelve years ago in another route indicated that for some unknown reason the sliding resistance of dowels might possibly increase materially in time it was decided to install a series of measuring plugs adjacent to all joints and cracks in this 2000-ft. stretch of pavement to determine whether the joints were opening and closing normally, and to measure the variations in width occurring at the cracks. Measurements taken immediately before and after a considerable drop in pavement temperature indicated that practically none of the joints in this location opened to any extent and that contraction of the pavement was accounted for principally by an appreciable opening of the cracks at which the reinforcing steel had failed. As a result of these measurements, and on the basis of an assumed subgrade resistance, it was estimated that the sliding resistance per dowel was at least 3000 lbs. This was later confirmed by removing the concrete on one side of a so-called "frozen" joint and, with a special dowel-pulling device, ascertaining the force required to cause slippage. The sliding resistance of 5 dowels was determined. The average resistance was found to be approximately 18,000 lbs. per dowel. Of these five, the dowel which offered the greatest resistance remained practically immovable at 15,000 lbs. When subjected to a constant pull of 24,500 lbs., it slid at the rate of .0075 in. in 7 minutes. More or less "free" sliding occurred at 25,000 lbs.

It is evident from the foregoing that, contrary to what might be supposed, the sliding resistance of these dowels has increased very materially within the past 6½ years.

Three of these dowels were entirely withdrawn from the concrete for detailed inspection. Considerable rusting of the dowel surfaces was found, especially for several inches each side of the joint space. (See Figure 1). The remaining 6 in. or so in the vicinity of the ends of the dowels was comparatively free from rust although some rust was found along the entire bottom surface of the channels and along the bottom surface of the top flange. The dowels were surrounded by very dense, well consolidated concrete. The indications are that moisture entered at the joint faces and progressed inwardly for several inches. As the dowels were withdrawn, pieces of concrete at the joint face which evidently adhered to the rusted portions of the dowels were broken out.

In so far as this particular joint is concerned there appears to be no doubt that corrosion of the dowel metal is the primary cause of the resistance to sliding. And there appears to be no reason to doubt that corrosion has similarly affected the rest of the joints in this and many other locations. That corrosion is the primary cause of restraint appears to be substantiated by (a) appreciable rusting and pitting of the dowel surfaces and (b) the fact that as soon as the pulling had advanced to the point where the rusted portion of the dowels had been withdrawn from the concrete the force required to continue sliding was considerably less.

So far as is known, corrosion causes restraint to sliding for the following reasons:

1. When steel corrodes the volume of the products of corrosion is several times the volume of the metal destroyed.

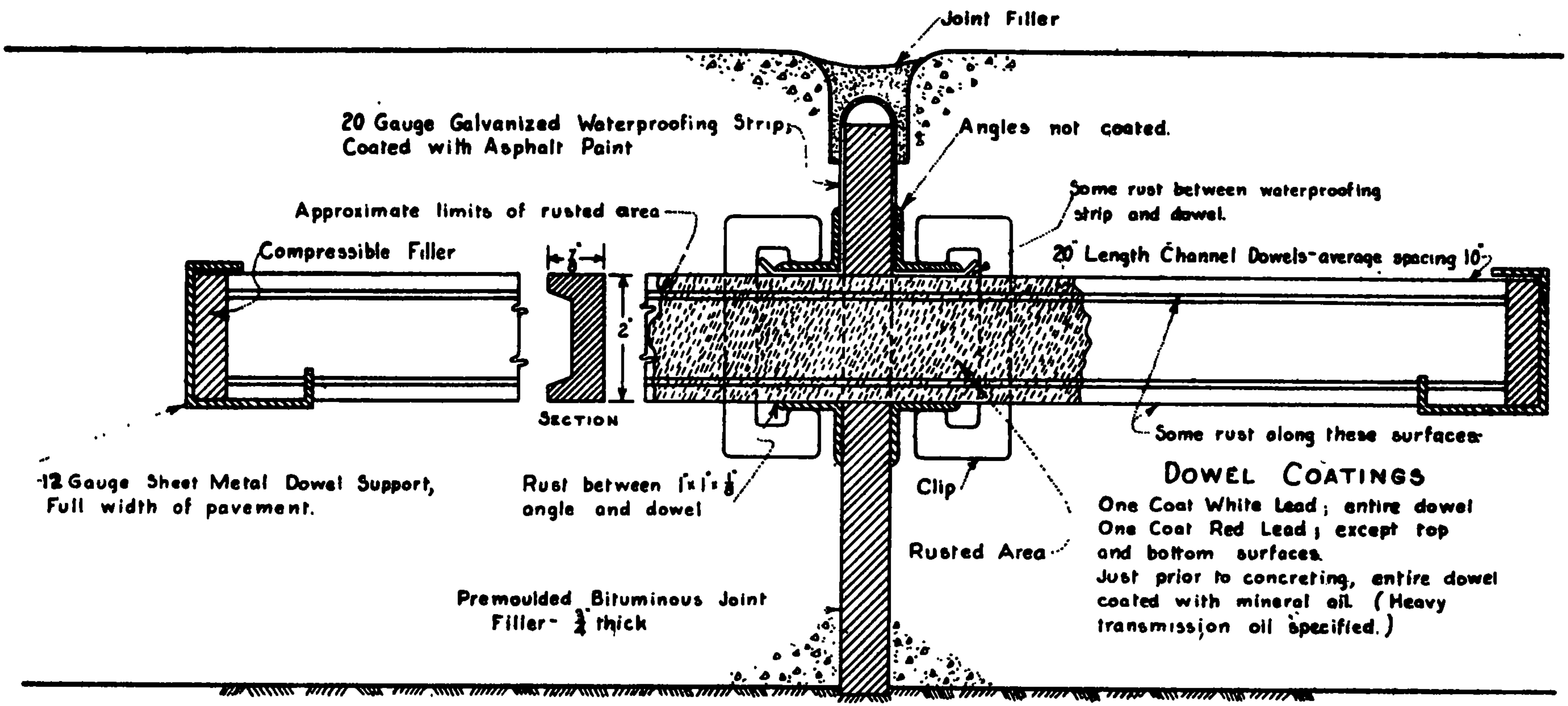

FIGURE 1. SECTION THROUGH CHANNEL-DOWEL JOINT SHOWING RUSTED AREAS.
SCALE: 1/2" = 1"

2. If corrosion occurs in a state of confinement the products of corrosion exert a considerable expansive effort which, under certain conditions, may be as much as 2000 lbs., per sq. in.

3. If corrosion occurs in immediate association with concrete an exceptionally voluminous form of rust may occur.

(The foregoing information regarding the behavior of the corrosion products is according to Ulick R. Evans, M.A., Sc. D., in his comprehensive treatise "Metallic Corrosion Passivity and Protection", Edward Arnold & Co., London.)

It appears, therefore, that as the rust forms its tendency to expand imposes a progressively increasing squeezing or gripping action on the dowels which gradually builds up sliding restraint. Apparently sliding of the dowels back and forth in the concrete as normally occurs initially is insufficient to counteract the gripping tendency induced by corrosion.

Evidently the magnitude of the restraint is a function of the area of the corroded surface and that for equal percentages of corrosion the larger the surface area of the dowel the greater the magnitude of the restraint. This is not to imply, however, that corrosion is of no consequence in the case of small dowels. As will be brought out later, the first wide, faulted cracks to be observed in New Jersey occurred two or three years ago on U.S. Route 1 in pavements in which joints with twelve 3/4 in. round dowels were installed. However, this has not as yet been definitely determined to be due to the same cause.

In any event, there is no question but that any material restraint to dowel sliding, from whatever cause, will sooner or later prove detrimental, especially to those pavements which carry heavy truck traffic, mainly for the following reasons:

1. During periods when the pavement temperature is decreasing the sliding resistance of the dowels induces tension in the pavement by tending to prevent its contraction. The tension thus induced is additive to whatever tension may be present due to other causes.

2. The tension induced by the sliding resistance of the dowels imposes an additional strain on the reinforcing steel spanning transverse cracks and thus tends to elongate the steel and widen the cracks.

Where cracks have recently widened in heavy duty pavements excessive tension was apparently responsible for starting the process of widening. As soon as a small amount of widening had taken place, however, the influence of heavy trucking appears to have accelerated the process. As mentioned previously, many of these cracks apparently occurred during the early life of the pavement due to causes other than contraction. Evidently for some years, during which time dowel restraint was perhaps more or less negligible, they remained at practically hair-line width. In later years, however, as dowel corrosion progressed and sliding resistance increased, the additional tension caused some further widening. So long as the reinforcing steel maintained the cracks at hair-line width it is probable that efficient load transfer was effected through aggregate interlock. With further widening, however, the effectiveness of the interlocking action was correspondingly decreased and some independent vertical movement of the abutting pavement sections at the cracks became possible. When sufficient independence of movement had thus been permitted the reforcing steel became subject to some flexing with each passage of load. As a result of thousands of flexings under heavy traffic within a relatively short time the steel became fatigued and failed, apparently while the cracks were still quite narrow. The cracks probably did not exceed 1/8 in. in width at the time the steel failed. After the steel failed, however, the rate of widening probably increased.

That the process of crack widening on Route 22 took place as described appears to be substantiated by the following observations:

1. No definite necking down of the fractured reinforcing steel was found. What appears to be necking down, however, has been noted at wide cracks in some very light traffic pavements in which 2 in. channel dowels were installed; apparently, in these instances, due to direct tension resulting from excessive dowel restraint.

2. In some of the cracks which are badly ravelled in the vicinity of the pavement surface, pieces of broken longitudinal reinforcing steel about 1¼ in. long have been found in the broken up concrete in the crack space.

It appears, therefore, that in so far as heavy duty pavements are concerned there is a critical crack width which if exceeded will permit excessive independent vertical movement of the abutting pavement sections. If the independence of movement is excessive in conjunction with frequent repetitions of load the reinforcing steel may soon fail due to bending fatigue while the cracks are apparently of harmless width, even though the ultimate strength of the steel in direct tension may have at no time been approached. The minimum amount of steel necessary to prevent cracks from widening in excess of a critical maximum under any given conditions, even with complete absence of restraint to joint opening, is as yet uncertain. But inasmuch as the faulting of cracks appears to be by no means infrequent in other locations as well as in New Jersey the relationship of critical crack width to traffic intensity is a phase of design which evidently merits further study.

As mentioned previously, a number of wide, faulted cracks have occurred in a section of U.S. Route 1. This section is between Trenton and Penns Neck and was constructed in 1932, as follows:

Thickness: 9 in. Uniform. Width of Slab: 10 ft. 6 in. Length of Slab: Approximately 69 ft. Longitudinal Reinforcement: Sixteen 3/8 in. Diameter bars. Dummy or Contraction Joints: None. Subgrade: Essentially silty clay — no subbase. Load Transfer at Expansion Joints: Twelve 3/4 in. round dowels, 20 in. long, sliding ends coated with cut-back tar, grade U.C., or K.P., 4 in. or 6 in. length cardboard sleeves with compression caps at ends.

Wide, faulted cracks in this pavement were first noted about two years ago and they have become more numerous since then. Most of the wide cracks are located approximately midway between joints, but a few are within 12 to 15 ft. from the joints. At the present time it is not possible to make any statement as to the cause of these wide cracks, for the following reasons:

1. None of the dowels has as yet been removed to determine whether there is any corrosion present.

2. Practically all of the joints have faulted from 1/4 to 1/2 in. and it appears entirely possible that even though corrosion may exist its effects at present may be substantially counteracted by the faulting. With faulting there is inevitably some bending of the dowels and enlargement of the dowel holes in the vicinity of the joint space. Furthermore, at faulted joints, independent slab

action under passage of load probably induces a small amount of dowel sliding, and the whole process of faulting involves a general loosening of the dowels. Whether or not there was a time prior to the faulting of the joints that the dowels, due to corrosion, offered appreciable restraint to sliding and thus induced the initial widening of the cracks, and heavy truck traffic then completed the process, is not known. But whatever the cause, this instance is mentioned simply to point out that wide cracks have occurred in conjunction with 3/4 in. round dowels as well as in conjunction with 2 in. channel dowels and that corrosion might have been influential in this instance as well.

It appears desirable to add that an indication of the possibility of a progressive increase in dowel sliding resistance was found 2½ years ago. In 1932, for experimental purposes, various types of joints were installed in series in some sections of a widening lane then under construction on U.S. Route 1. The dowels in some of these joints are 3 in. I-Beams (4, and in some cases 2, per joint, average length 21 in.) painted with cut-back tar. During construction Monel metal gauge plugs were installed each side of all the joints in the experimental sections in order that subsequent variations in joint width might be accurately determined. In 1942 it was noted that most of the joints with four I-Beams had considerably decreased in width during the preceding 10 years whereas those joints that had no dowels or mechanisms to restrain movement had compensated for this decrease by opening a corresponding amount. This is also true, but to a lesser degree, in the case of some of the joints with two I-Beams. Measurements taken in December, 1932 indicate that all of the I-Beam joints opened more or less normally (slightly less than completely unrestrained joints) during the first winter following construction. Recent measurements indicate, however, that the maximum annual variation in width of a number of these I-Beam joints is less than .03 in. whereas the immediately adjacent unrestrained joints have an average annual variation of about .45 in.

When this was first observed it was concluded that for some unknown reason the sliding resistance had progressively increased during the preceding ten years — but the cause was only surmised. However, the possibility that restraint might be due to corrosion of the dowels was suspected, but not investigated — in fact no investigations have as yet been made to definitely determine why no slippage takes place at these I-Beam joints.

Prior to recently determining that corrosion quite definitely can be the cause of excessive dowel restraint it was the intention to specify 1 3/4 in. x 1 in. x 20 in. length hot-rolled steel dowels spaced 12 in. c. to c., in future work and to facilitate their free sliding by greasing and encasing the sliding ends in sheet metal sleeves. However, in the light of recent investigations the efficacy of these measures in permanently facilitating free sliding is indeed questionable. Sleeves, due to some looseness of fit, might actually permit freer access of moisture to the dowel surfaces than coatings. There are also serious doubts, considering past experiences, as to whether coatings of any kind such as oils, greases, paints, bituminous compounds, rust-inhibiting preparations, synthetic resin finishes, galvanizing, cadmium plating, or any purely surface treatment can be relied upon to provide any more than temporary protection against corrosion. Without any question whatever, white lead, red lead, and oil coatings applied to ordinary hot-rolled steel dowels cannot be depended upon to prevent corrosion and eventual excessive restraint. This may hold true for any paint or dip coat.

For the foregoing reasons consideration is being given to the possible use of dowels made of materials that are corrosion-resistant. At present intensive studies are in progress to determine the relative corrosion-resistance of various kinds of metals, including several grades of stainless steel, under conditions tending to promote rapid corrosion. Inasmuch as metals that are corrosion-resistant are generally more expensive than ordinary steel studies are being made to determine the minimum dowel size, in terms of weight of material utilized, that tests indicate will meet requirements. Heretofore there has been some hesitancy about using structural shapes such as small I-Beams, mainly because of the possibility that their relatively thin webs or flanges might rust out completely within a few years. If, however, the dowel material is such that it may be relied upon to resist corrosion indefinitely the dowels can probably be of the most efficient shape such as to provide maximum strength, in conjunction with sufficient area of bearing surface, per pound of metal utilized. Some further decrease in the size of section may also be possible if the dowel material has a higher elastic limit than materials heretofore employed.

Present Views on Faulting, and Future Joint Design

In terms of past experience in New Jersey it appears essential that some substantial form of connection be provided at all points of interruption in the continuity of the pavement in order to counteract the persistent tendency of heavy truck traffic to cause faulting. At cracks this is presumably accomplished by the interlocking of the irregular surfaces of fracture, or so-called aggregate interlock, provided the amount of longitudinal reinforcing steel is sufficient to maintain the cracks at hair-line width. At joints various kinds of connections or means to counteract faulting are of course available. But, in so far as New Jersey is concerned, the employment of a series of substantial dowels has proved to be the most direct, dependable means of accomplishing this purpose. In view of this, and notwithstanding the difficulties that have recently become apparent in some locations due to dowel corrosion, it is the present intention to continue the utilization of substantial dowels. This intention is believed to be justified for the following reasons:

1. Three-fourths inch round dowels spaced 10 in. apart have proved to be incapable of preventing serious joint faulting in pavements carrying heavy truck traffic. The faulting of joints with 3/4 in. round dowels is due primarily to the fact that these dowels are deficient in stiffness. This deficiency in stiffness permits the development of excessive bearing pressures of the dowels on the concrete in the immediate vicinity of the joint space. These excessive pressures repeated hundreds of thousands and even millions of times in the course of a few years on heavy trucking routes cause a progressive crushing or wearing away of the concrete supporting the midsection of the dowels. This action necessarily enlarges the dowel holes for some distance each side of the joint space and soon renders the dowels non-effective.

2. In the case of dowels with greater bearing area and stiffness there is a correspondingly lesser tendency for the bearing capacity of the concrete to be exceeded. Increased bearing area and increased stiffness each contribute to a lessening of the unit pressures on the concrete. Tests now in progress have as one objective the determination of the most efficient dowel section and size, in terms of bearing area and bending resistance, consistent with the holding capacity of the concrete.

3. The more efficiently the load-transferring device prevents independent vertical movement of the adjacent slab ends the lower will be the magnitude of the forces and actions tending to cause pumping and faulting.

4. Notwithstanding the recognized value of granular subbase materials, and procedures having as an objective the stabilization of subgrades, the construction of subgrades and subbases has not yet reached the degree of dependability necessary to guarantee the prevention of faulting at joints which lack substantial load-transferring devices. Considering the numerous complex factors that tend to promote faulting, the magnitude of the forces involved, and the degree of stability it is necessary to provide, it appears to be uneconomical, and less dependable, to endeavor to counteract faulting by omitting load-transferring devices and resorting to special methods of subgrade stabilization alone. Instead, in the interests of economy and dependability, it appears preferable to utilize substantial load-transferring devices in conjunction with ordinary methods of subgrade stabilization.

5. Due to the absence of adequate load-transfer, should faulting occur on granular subbase because of greater compaction or rearrangement of the materials under the "receiving ends", not only will the pavement suffer but mudjacking might impair the functioning of the subbase.

6. During 1942, and since, several miles of pavement have been constructed in which no dowels or load-transferring devices were installed at the joints. Instead, the slab ends are supported on 4 in. thickness plain concrete sills (5 ft. wide across practically the entire width of the slabs). Even though these sill joints are in service in comparatively light traffic roads their performance to date has not been entirely satisfactory. There has been some cracking of the pavement at the edges of the sills. During the past winter differential heaving of the adjacent slab ends occurred at many of these joints even though these pavements were laid on 12 in. of apparently good quality granular subbase material. At a number of joints the differential amounted to 1/4 in., the maximum observed being 1/2 in. At present (April, 1945) some of these raised ends have not returned to their normal position. Where frost heaving occurs sills are apparently unsuitable.

7. In New Jersey, no evidence has been found to indicate that strong dowels materially restrain warping, or cause warping cracks.

Subject to minor modifications, New Jersey's heavy-duty concrete pavements to be constructed during the immediate post-war period are being planned essentially as follows:

Pavement Thickness: 9 or 10 in., uniform. Slab Length: Approximately 60 ft. Dummy or Contraction Joints: None. Joint Filler: Cypress, 1 1/8 in. thick, fabricated, grain direction vertical, sealing strip at bottom. Longitudinal Reinforcement: Single or double line of 3/8 in. diameter bars 7 1/2 in. c. to c., or equivalent in welded mats. Longitudinal Joints: Tongue and groove, or tie bars. Subbase: Sand, sand-gravel, stone-sand, or cinders — minimum thickness 8 in. under all pavements on impervious, erodible, or frost-susceptible soils. Load Transfer at Joints: Dowels of rectangular solid bars, or of structural shape, which have a bending resistance within the elastic limit of not less than 7500 inch pounds — 12 in. center to center. The dowels probably will consist of corrosion-resistant materials, or the sliding ends will be provided with positive means to prevent corrosion and to guarantee continued free slippage.

On the basis of present knowledge, the dowel problem appears to be essentially as follows:

1. If the dowels are relatively small the sliding restraint due to corrosion may be inconsequential. However, small members not only lack sufficient strength to transfer heavy truck loads and counteract faulting but are also susceptible to rusting out completely.

2. If, on the other hand, the dowels are of adequate size to fulfill their intended function the effects of corrosion with respect to materially restricting their freedom to slide may have serious consequences.

It is evident, therefore, that to efficiently counteract the faulting of joints in pavements which will be subject to heavy trucking the dowels must be substantially large. But it is also evident, particularly in conjunction with dowels that have a large surface area, that positive means must be provided to insure free slippage indefinitely. To this may be added that it is probable that the more efficiently the joint structure transfers load the more susceptible it is to being restrained by corrosion — due to its greater rigidity and resistance to a loosening effect under traffic.

War-Time Hauling

From December 1943 to August 1944 extremely heavy war-time hauling was done on 21 miles of concrete pavement between Perth Amboy and Colts Neck. The average weight of the loaded trucks was approximately 60,000 lbs. Based on counts, the estimated average daily number of these units passing over the pavement during this period was 212. No serious damage seems to have occurred anywhere during the winter months. But, during a rainy, thawing period early in the spring of 1944, serious failure occurred in several areas where the pavement was laid on silty-clay subgrade. Pumping was extremely violent in these areas, and the slabs soon became cracked at very close intervals. Failure rapidly progressed to such an extent that the pavement in some locations had to be reconstructed. The destruction of the pavements on adverse subgrades continued until the latter part of June when, due to the setting in of a long period of drought, pumping diminished and finally ceased. On the other hand, similar pavements on granular soils were affected much less severely, and none needed reconstruction. Faulting in varying degrees was common at all joints with six 3/4 in. round dowels except in some comparatively short stretches where the native granular soil appears to be of exceptionally good quality. In these exceptional locations, no defects of any kind are apparent.

Samples of the soils supporting these pavements were taken at various places along the trucking route, and the general condition of the pavement was noted. Information relative to the kind of joints, year constructed, condition of the pavement, and the characteristics of the soils in various locations is shown in Table 2. These locations are shown in miles from Perth Amboy.

It is desirable to point out that the pavement on Route 4 and 35, and Route 4, has channel-dowel joints. Route 4 and 35 was laid on native sandy soil, and Route 4 was laid on 8 in. of granular subbase. As noted under "Comments", the pavement on Route 4 and 35 is cracked immediately adjacent to some joints due to excessive sliding resistance of the dowels, but no cracking due to traffic is apparent. On Route 4, of the 225 slabs involved, only 4 have visible cracks, and these are at transitions from cut to fill. The pavement in this 3 mile stretch of roadway does not appear to have been damaged in any way by the heavy trucking.

TABLE 3. NEW JERSEY - 1940 TRAFFIC COUNT - INCLUDING WEIGHTS OF TRUCKS

Route	Location	Type	Average Daily Traffic	% of trucks in daily traffic	Average Daily Truck Traffic	Number of Trucks of Various Weights - Daily Average: Less than 5 tons	5 to 10 tons	10 to 15 tons	15 to 20 tons	20 to 25 tons	25 to 30 tons	30 to 35 tons
35	South Amboy	Dual	7599(South-bound)	9.2	699 (South-bound)	438	164	60	22	9	6	
4	From Route 35 Southerly	Dual	5537(South-bound)	7.2	398 (South-bound)	250	93	34	13	5	3	
4	North of Matawan	2 Lanes	3627(South-bound)	7.2	261 (South-bound)	165	61	22	8	3	2	
34	South of Matawan	2 Lanes	2475(South-bound)	9.2	228 (South-bound)	143	53	20	7	3	2	
25	North of Camden	Dual	9148(North-bound)	15.6	1427 (North bound	488	404	200	208	116	7	4
25	South of Bordentown	Dual	5667(North-bound)	18.5	1048 (North bound	259	299	163	202	89	31	5
25	South of New Brunswick	Dual	15,077 (Total)	20.7	3120 (Total)	917	977	568	353	215	84	6
25	North of New Brunswick	Dual	19,843 (Total)	20.7	4107 (Total)	1207	1286	747	464	284	111	8
25	At Newark Airport	4 Lanes	62,184 (Total)	16.0	9949 (Total)	2756	2845	1751	1671	667	189	70
26	North of Trenton	Dual	12,477 (Total)	20.7	2582 (Total)	759	808	470	292	178	70	5
29	Somerville	3 Lanes	11,344 (Total)	17.6	1997 (Total)	581	607	268	318	177	42	4
6	East of Hackettstown	2 Lanes	6000 (Total)	9.3	558 (Total)	155	160	98	93	37	11	4

With respect to the grading and characteristics of the various soils shown in the tabulation, it is desirable to add that most of these samples were taken in cuts and that every effort was made to obtain samples truly representative of the soil supporting the pavement. However, due to the extremely variable types of soil encountered, and particularly to the presence of a thick undulating layer of heavy, black, greasy, silty-clay soil which was frequently found under only parts of a slab, the described condition of the pavement in some locations may appear, and perhaps is, inconsistent with what might be inferred from the characteristics of the samples.

As a matter of general information, a tabulation (lower part of Table 2) has been added showing the characteristics of some of the soils which have contributed to severe pumping and faulting on other routes. Two of the most unsatisfactory subgrade soils in New Jersey, with respect to pumping and faulting, are represented by Samples No. 354,381 and 354,253.

Traffic

The Table 3 shows traffic counts taken in 1940 at points on several trucking routes, and the various weights of the trucks.