AN INVESTIGATION OF CONCRETE PAVEMENT DISTRESS ON INTERSTATE 78

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"An Investigation of Concrete Pavement Distress on Interstate 78"

by

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Abstract

The concrete pavement on Interstate Route 78, Section 2B has a history of premature and progressive deterioration in the form of faulting, cracking and spalling. This study was undertaken to determine if the pavement distress, particularly the recently accelerated cracking and spalling in the vicinity of transverse joints, resulted from inadequate design of the Department's standard expansion joint.

Based on field and laboratory analyses of excavated pavement-joint specimens and underlying soil layers (subbase and subgrade), it is concluded that the general deterioration of the subject pavement was initiated by the pumping of fines. In particular, it is believed that the pumping phenomenon precipitated spalling by causing wear of the concrete surrounding the free side of the dowels, forming a channel through which pumped incompressibles entered and accumulated in the expansion space. This loss of expansion space resulted in the formation of cracks which subsequently manifested as surface spalls.

It is recommended that the present design for the Department's standard Type A expansion joint remain unchanged.

Key Words: faulting, cracking, spalling, expansion joint, subbase, permeability, subgrade, pumping, wear of concrete, dowels, incompressibles.
Acknowledgments

The writers wish to thank the following individuals for their assistance in accomplishing the work of this project:

The Bloomsbury maintenance yard crew under the direction of Elix Penyak removed the pavement-joint sections for the investigation.

Malcolm MacKenzie of the Bureau of Project Inspections performed the field density and moisture tests.

The Bureau of Quality Control performed all the necessary physical and chemical tests.
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PART I. INTRODUCTION

A. Objective and Scope of Study

A factor of particular concern in the history of premature pavement distress on Interstate 78, Section 2B, has been the recent accelerated cracking and spalling in the vicinity of the transverse expansion joints.

This report presents the results of a study undertaken in November 1971 to determine if the pavement deterioration at transverse joint locations is specifically attributable to inadequate joint design.

B. Pavement History

The subject portion of Route 78 is located in the rural area between Still Valley Road and the Central Railroad of New Jersey in Warren County and is of typical Interstate design, i.e., a divided highway with a median strip and limited access.

The original construction contract included the paving of two lanes in each direction for four miles. This mainline pavement was constructed by the single lane method in the summer of 1959. After eight years of service, inner lanes were added to complete the present six-lane highway.

The pavement conforms to the Department's typical design, that is, the reinforced concrete slabs are 9 inches thick, 12 feet wide and 78 feet 2 inches long. The Department's standard, Type A
transverse expansion joint is used for load transfer at slab ends (see Appendix A-1 for construction details). The pavement is underlaid with two 6-inch layers of subbase. Top and bottom course subbase are comprised of Type 1A* and 1C granular material, respectively. The shoulder section consists of 12 inches of subbase, and 7 inches of macadam base course topped with 2 inches of bituminous concrete.

C. Traffic History

When Section 2B was opened to traffic in October 1959, the two-way AADT was 11,820 vehicles per day. The most recent data available (1970) indicates that the volume has doubled to 23,670 vehicles per day.

Data from a loadometer station established for this section in 1964 indicated that at that time, about 37 percent of the total traffic was trucks. By 1970, the truck percentage decreased slightly to 33 percent. However, since the overall traffic volume has increased, the absolute number of trucks has also increased. In comparison to other main traffic arteries in New Jersey, the subject section has one of the highest percentage of trucks. While comparative data from other locations within the State as to the absolute number of daily truck loadings is not readily available, it appears that Section 2B would also rank quite high on this basis. Due to the high prevailing speeds and the access control, traffic flows uniformly on this road, with little reason for lane changes. Most of the heavy loading, therefore, is concentrated in the slow or truck lane.

*Approximately 22 percent of type 5A subbase was constructed as an alternate to the 1A top subbase on the project. However, none of the alternate material was encountered at study locations.
D. Deterioration History

Throughout the life of this pavement, numerous condition surveys were performed by representatives of the Bureau of Structures and Materials and the Soils Bureau. This general condition data was complemented by three faulting and four crack surveys performed by Research personnel.

Each of the condition surveys noted evidence of pumping on the project. As might be expected from the previously described traffic considerations, the earliest and subsequently most severe distress was observed to occur in the outer (truck) lanes.

While it is not precisely known when the pavement exhibited the initial signs of oncoming distress, as early as 1963—four years after the pavement was constructed—about one-fourth of the transverse joints in the east and westbound truck lanes were faulted* at least 1/8 inch.

Analysis of the data compiled in the various pavement surveys indicates that the pavement deteriorated progressively up to 1969. At that time, 34 percent of all pavement slabs on Section 2B were cracked, with 59 percent of the cracked slabs being located in the truck lanes.

A recent examination of the project indicates that deterioration has accelerated considerably during the past two years, with significant increases in the incidence and severity of pavement cracking and spalling at transverse joint locations being observed. More specifically, the concrete at nearly half of the Type A joints is now spalled which corresponds to about a 70 percent increase in the number of affected

*A transverse fault is here defined as a difference in elevation between abutting slabs, in which the slab farthest from oncoming traffic is lower.
joints since 1969. Almost without exception, this spalling has occurred in the outer truck lane and on the slab nearest to oncoming traffic (which on this project coincides with the free side of the expansion joint). Approximately 50 to 60 percent of the spalls are located at the corners of the slabs.

Most of the pavement slabs in the truck lane presently have transverse cracks extending across their full width, 10-20 feet on each side of the joint. Historical records indicate that when this crack pair occurs, in 2/3 of the cases, the slab farthest from oncoming traffic cracks first.

It is interesting to note that the history of pavement distress on Section 2B in a number of respects parallels that of the adjacent 2J Section. Section 2J (Jugtown Mountain) was opened to traffic two years after Section 2B and has essentially the same traffic and subgrade conditions. The truck lanes of the Jugtown Mountain section have evidenced the pumping, cracking and faulting apparent on Section 2B, but not the extensive and severe spalling at transverse joint locations.

In a 1965 Research report, it was concluded that the basic cause of the 2J pavement distress was pumping. The pumping phenomenon in turn was attributed to the combination of a heavy truck loading, the availability of fine-grained material, the availability of water through an open pavement-shoulder joint, and the subsequent accumulation of this water immediately below the pavement due to relatively impervious sub-base and subgrade. Variations in slab/subbase contact due to volume changes in the subbase and subgrade were believed to have aggravated the pumping.
E. Method of Investigation

To identify the causes for the concrete deterioration at transverse expansion joints, samples were taken of the joint device (dowels), subbase (top and bottom layers), and earth subgrade. Both field and laboratory evaluations were performed on each sample.

Four locations were chosen for sample removal and investigation, all of which were in the truck lane. The pavement at each location selected for study showed varying degrees of spalling.

Removal of the pavement-joint specimens was accomplished with the aid of an air hammer and a concrete saw. Considerable attention was given to ensure that neither the removed dowels nor the dowel assembly was damaged in the removal operation. Due to limitations on the size and weight of the concrete specimens, a maximum of 4 dowels were removed at any one location.

As a complement to a field examination of eight dowels, two intact pavement-dowel specimens were examined in detail at the central laboratory after saw-cutting along the length of the dowel.

The program of soils testing consisted of in situ subbase moisture and density tests, together with standard laboratory tests on the subbase and subgrade to determine their suitability in terms of gradation, density, permeability, bearing capacity, and breakdown characteristics.
PART II. OBSERVATIONS AND TEST RESULTS

A. General Observations and Nature of Test Locations

Several conditions were common to all four test locations:

1. At each location, the truck lane exhibited localized settlement at the joint of from 3/8 to 3/4 of an inch with respect to the adjacent lane; spalling on the free side of the dowel; and on both sides of the joint, a full-width transverse crack at a distance of 10 to 20 feet from the joint.

2. The sealer in all pavement joints, both longitudinal and transverse, was no longer adhering to the concrete, thereby resulting in an open joint.

3. Extensive cracking existed in the bituminous shoulder adjacent to the concrete pavement. The shoulder was separated from the pavement by 1/8 to 1/2 inch forming a reservoir which was filled with fines.

4. All locations were on inclined profile grades and the outflow end of the subbase outlet drains nearest to each test location was in a dry condition.

Details of the test locations are as follows:

Location A (Westbound Roadway) - A three foot square, centered on the joint and immediately adjacent to the shoulder was removed. This specimen contained three dowels which joined slabs 595 and 596. As shown in Figure 1, the depth of this spall presented a severe hazard to traffic.

![FIGURE 1](image-url)

Appearance of test location A. Note cavity resulting from dislodged concrete.
At this location large pieces of concrete had been dislodged by traffic, thereby exposing portions of the mat reinforcement and the top of one dowel.

Location B (Westbound Roadway) - A 4x3' section joining slabs number 599 and 600 was removed. As shown in Figure 2, the sample was shifted from symmetry about the joint in order to include the entire spall and remove four dowels. This specimen contained the most extensive spalling investigated.

![Figure 2](image)

**FIGURE 2**
Appearance of test location B

Location C (Westbound Roadway) - Only two dowels were included in the two foot square removed between slabs 660 and 661. The sample was taken from the center of the transverse joint, where the concrete surface displayed the most distress. As shown in Figure 3, a crack extends from the spall toward the adjacent lane, following a line which coincides with the location of the dowel ends.
Location D (Eastbound Roadway) - The specimen obtained between slabs number 492 and 493 differed from the others in that it was in a fill area and in the eastbound roadway. The small (1'x2') section removed from this location contained only the dowel closest to the shoulder. The removed sample thus contained only a part of the large spall shown in Figure 4.
B. Investigation of Removed Joint Specimens

1. Field Investigation

At each study location, the filler paper was reasonably intact and pliable at the top of the joint. At two locations, however, the filler paper at the bottom of the joint was deteriorated and replaced by a coarse granular material. Based on a visual comparison to other materials present on the site and to abrasives stockpiled in a nearby maintenance yard, it was concluded that this intruded material was some of the maintenance mix spread during winter deicing operations.

The concrete was fractured and subsequently broken away from the free end of eight dowels. In each case, there was a combination of fine grained soil and what seemed to be pulverized concrete collected along the dowel. Most of the cap was missing, through wear and/or corrosion. Having fulfilled its purpose (i.e. protecting and holding the cork in place during construction), the cap was no longer necessary for the joint assembly to function properly. The material found in the cork space appeared to be composed of incompressibles rather than cork.* The stainless steel sleeve on the dowel was only slightly corroded, causing discoloration but not pitting. The end of the sleeve was deformed and is pictured in Figure 5.

![FIGURE 5](image)

Close-up of deformed sleeve end

*As described later, laboratory tests indicated that this material was in fact, principally a fine soil.
The sleeve-end appeared to be forced toward the joint forming a bulge on the bottom edge of the dowel. The severity of the deformation varied from dowel to dowel. In every case the reinforcing mat was nearly resting on the dowel, but there was no noticeable mat corrosion.

2. Laboratory Investigation

The fixed end of each of the two dowels that were investigated in the laboratory was firmly bonded to the surrounding concrete. The concrete on the free side of the dowel was worn, appearing ovular in cross section (the dowel was movable in the vertical but not the horizontal direction). The fixed end of the dowel was removed by breaking off the surrounding concrete and the free end was pulled out by hand with considerable effort.

After each concrete specimen was sawn longitudinally in half, the dowel cavity was measured. At the cork end the measurement was 1-5/16", but near the joint edge it was 1-9/16" (nominal dowel diameter: 1-1/4"). In Figure 6 the dowel was replaced to illustrate this funnel shape.

The concrete was well-compacted around the dowel, indicating that any void in this area was caused by wear rather than by improper consolidation during construction.

FIGURE 6
Photograph illustrating funnel shape of dowel cavity.
The inside of the dowel cavity was irregular and rough, i.e., aggregate was exposed and the surrounding mortar was worn away (Figure 7).

FIGURE 7
Photograph of dowel cavity interior
Note roughness and small cracks at bottom left

There were fine cracks at the bottom of the cavity near the joint. On one of the samples the area around the end of the cork space remained in place after sawing. Small cracks were noticed extending from the cork space. A close-up of one of these cracks is shown in Figure 8.

FIGURE 8
Close-up of crack extending from cork space through aggregate
Since each of the observed cracks extends through the aggregate, it is believed that they were created by loading stresses. In contrast, cracks that develop during the period immediately following placement are generally confined to the mortar matrix.

As in the field examinations, the sawed specimens contained about one inch of tightly packed, soil-like material in the cork space (See Figure 9). This material was sampled for testing. Part of the cap and tie wires were found to be still in place, and the uncorroded reinforcing mat was nearly resting on the dowel.
As shown in Figure 10, the removed dowel was found to be straight and free of corrosion. As required in the specifications, the carbon steel end measured a constant average diameter of 1-1/4". The end covered with the stainless steel sleeve had a nominal diameter of 1-5/16". There was no corrosion on the sleeve, but there was a small deformation on the end.

![FIGURE 10](image)

Photograph indicating the condition and straightness of dowel

3. **Summary of Dowel Investigations**

The Type A transverse expansion joint used by the Department did not have any of the generally accepted symptoms of failure. The dowels were not rusted or worn and the stainless steel sleeve was only slightly corroded. Any of these conditions could have caused seizure had they been present. Further, the dowels were not bent, indicating an adequate transfer of loads.
On examining the dowel assemblies, the only condition found which would serve to inhibit proper joint performance was the existence of incompressible material at the free end of the dowels. For this reason it is believed that the spalling of the investigated joints occurred through the following chain of events: the intrusion of incompressible fines into the cork space permitted the creation of expansion stresses sufficient to initiate cracks which, upon reaching the surface, manifested as spalls. When compared to other Departmental studies of joint assemblies, this accumulation of fines in the cork space is both unique and unexpected.

C. Soils Investigations

1. Testing and Sampling Procedure

After each pavement test specimen was removed, nuclear density and moisture measurements were taken at various levels throughout the full (12") subbase depth. For comparison, one sand cone density test was performed on the top course subbase in the east bound roadway and two in the westbound roadway.

Samples of subgrade and subbase material were obtained from each roadway. Since the boundary between the two subbase courses could not be distinguished, the top and bottom course samples were selected from only the upper and lower 4 inches of the 12 inch thickness.

At each location, the presence of considerable oversize subbase material (i.e., greater than 4") interfered with the sampling and field testing.

To determine their soil classifications, tests for grain size distribution and Atterberg limits were performed on individual samples from
each roadway (10 lbs. subgrade; 80 lbs. top subbase and 80 lbs. bottom). After combining subbase samples from each roadway to give one sample of each course, the following test characteristics were determined: maximum density, permeability, California bearing ratio, and resistance to breakdown of particle size in the freeze-thaw and compaction tests.

Four samples of the material present in the cork space were subjected to a series of physical and chemical tests to determine composition.

2. Top Course Subbase Test Results

The samples of top course subbase are classified as HRB Type A-1-a soil. The Unified classification is GM, a silty gravel.

As shown in Table 1, this material exceeds the Department's 7 percent specification maximum for material passing the No. 200 sieve.

<table>
<thead>
<tr>
<th>Sieve</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Required</td>
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<tr>
<td>4&quot;</td>
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</tr>
<tr>
<td>2&quot;</td>
<td>70-100</td>
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<td>10-25</td>
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<tr>
<td>No. 200</td>
<td>0-7</td>
</tr>
</tbody>
</table>

*Oversize material removed prior to test.
Several possible reasons for this excess of fines can be offered:

Since the time of construction, there may have been a breakdown of coarse aggregate by either freeze-thaw cycles or traffic loading. This theory is substantiated by the noticeable breakdown of particles during both the Proctor and freeze-thaw tests.*

The pumping phenomenon offers another possible explanation for excessive fines at the locations sampled. That is, the movement of water under pressure could conceivably cause a migration of fine particles from the same or underlying soil layers to the joint.

Finally, non-compliance of present samples could mean that at the time of construction a portion of the project material was non-conforming. Construction materials records show that the majority of samples were near the maximum limit of 7 percent. Statistical theory indicates that data of this type, when enough samples are taken, should generate a normal frequency distribution. This statistical distribution is represented by the familiar bell curve in which all possible (test) values are symmetrical about the mean value. Since the peak or mean of the distribution of construction acceptance samples is near the upper specification limit, it is very possible that a portion of the as-constructed 1A material did not comply with the fines limitation.

While it is not definitely known which of the above described reasons was primarily responsible for the excess of fines, it is evident that at least on an overall basis, the subbase did not conform to required gradation at the time of construction. That is, each test excavation revealed considerable subbase aggregate having a least width of over 4 inches and in several cases over 9 inches. In certain cases, the oversize material was up to 14 to 17 inches long. Figure 11 is a photo of a few of the oversize specimens.

*See Appendix A-2 for soil test results not contained in text.
FIGURE 11

Photograph of large aggregate removed from subbase. The black square indicates the specified maximum allowable size (4 inches).

The original compaction that could have been achieved with this large size aggregate is questionable since the subbase was specified to be placed in only 6-inch lifts. After twelve years of traffic loading, however, the average sand cone* density was adequate, being only slightly less than the Proctor maximum density (136 vs 138 lb/ft³).

At the maximum Proctor density, a CBR of 97 was obtained. After four days of soaking, the moisture content increased from the optimum 8 percent level by only 0.4 percent and there was no detectable swell. However,

*The nuclear density results are not reported because confidence could not be established in the validity of the variable readings.
the bearing ratio of the soaked sample decreased by 31 percent to a value of 67. While the significant decrease in support shown for this soil is certainly undesirable, it is believed that both the soaked and unsoaked bearing ratios may be considered "adequate"; the Department's current design procedures for concrete pavement assumes a minimum CBR of approximately 50 for top course subbase. Further, the considerably more liberal Portland Cement Association design criteria indicates that for the design conditions on this project, bearing capacity should not be a factor in failure if a CBR of more than 9 is obtained.

The average variable-head permeability for the top subbase is extremely low: $3 \times 10^{-6}$ cm/sec. In comparison, any soil with permeability less than $10^{-4}$ cm/sec. is considered to have poor drainage properties and can, in fact, be used for impervious sections of earth dams. When in place then, the subbase does not permit water entering through the unsealed joints to drain. This suggests that below the pavement there would be variable moisture conditions and the attendant variable pavement support as indicated by the CBR tests.

3. **Bottom Course Subbase Test Results**

   Like the top, the bottom course subbase is a HRB Type A-1-a soil. The Unified classification is well-graded, silty gravel (GW-GM).

   As shown in Table 2, the sampled bottom subbase also exceeds the Department's specification allowance for the material fraction passing the No. 200 sieve.
TABLE 2

GRADATION ANALYSIS

TYPE 1C BOTTOM COURSE SUBBASE

<table>
<thead>
<tr>
<th>Sieve</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Required</td>
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<tr>
<td>4&quot;</td>
<td>100</td>
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<tr>
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<td>60-100</td>
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<td>No. 4</td>
<td>30-100</td>
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<td>5-35</td>
</tr>
<tr>
<td>No. 200</td>
<td>0-5</td>
</tr>
</tbody>
</table>

*Oversize material removed prior to test.

The possible reasons offered for excessive fines in the top course also apply to the bottom course: particle breakdown, migration, or non-compliance at the time of construction.

Again, which reason or combination of reasons was primarily responsible for the excess of fines cannot be precisely determined. In this case, however, it seems most likely that the as-constructed material was non-compliant. That is, during construction, one-third of the original bottom course samples exceeded the fines specification. Further, of the approved (check) samples, 80 percent were exactly on the upper specification limit. Using the same statistical reasoning presented for the top course, i.e., in a normal distribution, actual values range on either side of the mean test value,
it is concluded that some (possibly very substantial) fraction of the as-constructed subbase contained excess fines.

Type 1C subbase has the same maximum size limit as Type 1A. However, like the top course, it contained considerable oversize aggregate. Therefore, the same possibility for compaction problems existed for both layers. The laboratory density for both courses is nearly the same, as is their lack of permeability.

The relative bearing capacity of the bottom course subbase is significantly lower than that of the top course, the CBR values obtained at both optimum moisture content and after soaking being only about 1/4 that of the top course. Specifically, a CBR of 23 was obtained at optimum moisture content (8.5%) and a 16 after four days soaking (moisture increase: 0.6%). While this latter CBR represents a significant decrease for only a slight increase in moisture, it is above the minimum value (9) which the PCA criteria indicates as providing "adequate" support. The CBR's for this course are, however, less than the Department's desired value of 30.

4. Subgrade Test Results

As previously described, a study of the abutting pavement Section 2J indicated that the subgrade contributed to the observed pumping problem due principally to its impermeability and volume change when subjected to wetting and drying.

While the lack of a sufficiently large sample prevented performing certain of the tests (i.e. permeability and CBR) necessary to fully characterize the nature of the subgrade on the present project, the observed similarities indicate that the subgrade information reported for Section 2J can be substantially extended to apply to Section 2B.
One indication that the subgrade on the present section had at least the same influence on pavement behavior as the 2J subgrade can be judged by the comparative soil classifications: the 2J subgrade is a silty sand (SM) while the 2B subgrade is an inorganic clay (CL).

The grain size distribution and Atterberg limits for subgrade on this section are shown compared to those of Section 2J in Table 3.

### TABLE 3

**COMPARATIVE SUBGRADE SOIL TEST RESULTS**

**I-78 SECTION 2J and I-78 SECTION 2B**

<table>
<thead>
<tr>
<th>Gradation</th>
<th>% Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sieve Size</td>
<td>Section 2J</td>
</tr>
<tr>
<td>4&quot;</td>
<td>100</td>
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<tr>
<td>2&quot;</td>
<td>98</td>
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<td>3/4&quot;</td>
<td>94</td>
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<tr>
<td>No. 200</td>
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<table>
<thead>
<tr>
<th>Atterberg Limits</th>
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</thead>
<tbody>
<tr>
<td>Liquid Limit</td>
</tr>
<tr>
<td>Plasticity Index</td>
</tr>
<tr>
<td>Shrinkage Limit</td>
</tr>
</tbody>
</table>
As can be seen from this tabulation, the principal difference in gradation is that the 2B samples have a greater percentage passing the No. 200 sieve. While the two subgrades have an approximately equal liquid limit, the 2B section has about a 3 units higher plasticity index. Given this equal liquid limit, the higher plasticity index for Section 2B is indicative of relatively lesser permeability.

This soil type is susceptible to frost-heave. Even though the subgrade is 21 inches below the surface, frost can penetrate 33 to 38 inches in this area.

D. Other Test Results

1. Material in Dowel Cork Space

The material in the cork space of four dowels was subjected to a series of physical and chemical tests. In two cases, the sample was obtained in the field, while in the other two, the material was removed from sawed specimens.

As shown in Appendix A-3, the field cork space samples contained no cork, only soil. A comparison of the test results for this material to the results of identical tests on samples of top course subbase strongly suggests that the soil in the cork space was top course material that had pumped along the funnel-shaped passageway surrounding the dowel.

The material found in the cork space of those dowels opened in the laboratory consisted of a combination of soil and cork. The amount of cork could not be precisely determined. When these samples were tested in layers, it was found that the farthest area from the dowel end generally contained the most cork. The amount of cork progressively decreased toward the dowel. This suggests that the soil gradually works its way along the dowel and into the cork space.
2. **Concrete Quality**

Based on a review of the original pavement cores obtained on this project, it is concluded that the quality and thickness of the concrete was not a factor in the distress of the pavement.

The compressive strength of the project cores ranged from 3,290 to 7,340 psi and averaged over 5,000 psi.

The average air content (3.6%) was within the presently specified 3 to 6 percent range.

The indicated average pavement thickness was approximately 9-1/4 inches.
PART III. SUMMARY OF FINDINGS

1. Traffic Considerations: Interstate 78, Section 2B has an AADT of nearly 24,000 vehicles per day, of which approximately 1/3 are trucks. In comparison to other main traffic arteries in New Jersey, the subject section apparently can be placed in one of the higher categories with respect to both the percentage of truck traffic and the absolute number of daily truck loadings. Significantly, due to the geometrics and rural setting of the road, this heavy truck loading is predominantly confined to one (outer) lane in each direction of travel.

2. Deterioration History: The studied section has a history of premature and progressive pavement distress, beginning with faulting and pumping observed shortly after the project was opened to traffic. As might be expected from the prevailing traffic conditions, the earliest and subsequently most severe distress was observed to occur in the outer (truck) lanes. While the order of incidence of distress varied for individual truck lane slabs, an overall pattern of deterioration can be described. As shown in Figure 12, pavement distress generally progressed from joint faulting to the formation of a transverse crack pair approximately centered about the joint, followed by differential settlement with respect to the adjacent lane at the joint and spalling on the free side of the joint device. The incidence of spalling has increased by about 2/3 over the past two years (i.e., between the 10th and 12th years of service) to the point that 40 percent of all Type A joints in the truck lanes are now affected.
Stage 1. Faulting of joint.  
(After four years service, 1/4 of slabs had reached this condition. Pumping observed shortly after opening to traffic.)

Stage 2. Initial crack development.  
(In 2/3 of cases, slab farthest from traffic cracks first.)

(After 10 years service, 3/4 of slabs were cracked. A crack on the free side was usually evident before a spall.)

Stage 4. Spalls evident on free side of joint; differential settlement at the joint.  
(25% of joints spalled after 10 years service, 40% after 12 years.)
3. **Condition of Joint Sealant**: The hot poured sealant in the longitudinal and transverse joints on this project in general, and at each of the studied four test sites in particular, was no longer bonded to the concrete, thereby resulting in an open joint.

Apart from providing free entrance of water to sustain pumping, at two locations, the unsealed joint condition permitted the entrance of incompressibles. At these locations, the coarse granular material which had intruded into the expansion space was visually identified as some of the maintenance mix spread during deicing operations.

4. **Condition of Removed Dowels**: None of the ten dowels examined in this study displayed the generally accepted symptoms of failure (i.e., significant corrosion or bending). Corrosion and the associated potential for joint seizure was limited to a slight discoloration of the stainless steel sleeve. The absence of bending is indicative of adequate load transfer. The only apparent changes occurring during 12 years service was the development of a slight deformation of the sleeve end on the free side of the dowel and the loss of the cap through wear and/or corrosion.

5. **Nature and Condition of the Dowel Cavity**: Removal of dowels from the concrete on the free side of the joint revealed an oval rather than a round cavity. At the face of the joint the oval had a vertical diameter of 1-9/16 inches while at the cork end the cavity measured 1-5/16 inches. This overall funnel shape of the dowel cavity undoubtedly resulted from wear between the dowel and concrete under the action of moving loads.
On first (visual) examination, the material in the cork space appeared to be an incompressible soil-like material. Subsequent laboratory testing confirmed that this cork space material was, in fact, either partially (2 of 4 samples) or exclusively composed of soil rather than cork. More specifically, it is believed that these incompressibles deposited in the cork space are top course subbase fines which entered through the widened dowel cavity by the pumping mechanism.

In one of the intact pavement-joint specimens sawed in the laboratory, fine cracks were observed to radiate from the end of the cork space toward the top and bottom surfaces of the concrete slab. The formation of these cracks apparently represents the beginning stage of spalling.

6. Subbase and Subgrade Quality: The top and bottom course subbase sampled in this study failed to meet gradation requirements with respect to both oversize material and fines. While the oversize (plus 4") material encountered could only be the result of non-compliance at the time of construction, alternate possibilities exist for the occurrence of excessive fines, including breakdown of large size particles and migration of fines to the joint through the pumping mechanism. Although the source of the excessive fines is not definitely known, the application of statistical theory to the original project materials records strongly suggests that at least a portion of the as-constructed subbase was non-compliant with respect to fines as well as oversize material.

Permeability tests indicate that both subbase courses have poor drainage properties. By way of comparison, the subject test results
are on the same order of magnitude as those required for soil used in the impervious sections of earth dams. Since one of the prime objectives in placing subbase is to provide free drainage, it is apparent that this material did not completely fulfill its design function.

The California Bearing Ratio of both courses decreases by about one-third when subjected to soaking indicating that pavement support varies with moisture content, thereby aggravating pumping. In view of this subbase characteristic, the observed unsealed condition of the joints is particularly undesirable.

A previous study of pavement deterioration on the abutting Section 2J indicated that the subgrade contributed to the observed pumping problem due to its impermeability and volume change when subjected to wetting and drying. Based on comparative test data, it is concluded that the clay subgrade on the present project influenced pavement behavior at least to the same extent as the silty sand subgrade of Section 2J.
PART IV. CONCLUSIONS

It is concluded that the pavement distress on Interstate 78, Section 2B is not attributable to any design inadequacy of the standard Type A expansion joint. Thus, no modification of the Department's existing joint design need be contemplated as a result of the observed performance of this pavement section.

The underlying cause of the general deterioration of this section of pavement is believed to be pumping. In particular, it is proposed that the pumping phenomenon precipitated spalling through the following chain of events:

A. Cyclic vertical movement of the pumping slabs caused the unbonded portion of the dowel to abrade the surrounding concrete.

B. Pumped fines, entering through this channel worn in the concrete, eventually accumulated in the cork space.

C. The loss of expansion space permitted stress build-up at the end of the dowels.

D. The resulting cracks subsequently manifested as pavement spalls.

The findings of this study emphasize the importance of the Department specifying and obtaining subbase materials having desired quality characteristics with respect to gradation and permeability.

In this connection, it is to be realized that compliance with gradation requirements does not necessarily ensure desirable drainage characteristics (an untested property in our present specification).
For example, test results for a number of (Type I) subbase samples passing gradation requirements indicated permeability coefficients on the order of $10^{-4}$ to $10^{-5}$ cm/sec. In more common units, these values correspond to seepage rates of less than one inch per day.

Obviously, it would be unrealistic to expect joint sealant placed by Construction or Maintenance forces to completely compensate for subbase materials with poor drainage properties. Further, it would appear that Maintenance forces, who must deal with priority work on a day-to-day basis, do not presently have the capability to maintain pavement joints on all projects in a sealed condition.
PART V. RECOMMENDATIONS

1. The present design for the Type A expansion joint should remain unchanged.

2. Experience has shown that the entrance and subsequent entrapment of water in the soil layers underlying concrete pavements is a relatively common occurrence. Permeability tests in general have shown that the ability of subbase to quickly carry off this water may be questionable. A number of studies of deteriorated pavement in particular have shown this lack of subbase drainage to be a factor in the observed distress.

   It is therefore recommended that a high priority be assigned to initiating previously proposed research into subbase permeability. Such research would be designed to determine the desirability and feasibility of improving subsurface drainage through such means as improving material quality (e.g., by specifying minimum permeability requirements) or by providing alternate drainage schemes (e.g., by carrying the subbase laterally through the berm to the slope edges, so-called "daylighting"). It is worth noting that it may be possible to establish this latter ("daylighting") procedure as a workable solution without a major verification effort.

3. Additional sampling of concrete pavements exhibiting various degrees of distress should be made in order to determine if the accumulation of incompressible fines in the dowel cork space observed on this project is in fact a unique phenomenon.
4. The pavement on this project is in need of repairs in order to minimize further deterioration. First consideration should be given to the following limited corrective measures:

   a. Restore the most seriously settled pavement areas to grade and cross section by slabjacking.
   b. Clean out and seal pavement joints (transverse and longitudinal) and wide cracks which presently permit the entrance of surface water into the subbase. Seal the pavement shoulder joint in those areas where a significant separation exists.

   At some time in the future an in-depth study will be necessary to determine the specific nature of a complete pavement rehabilitation program. Apart from the design of an overlay in such a study, particular consideration should be given to means of providing for additional subsurface drainage and methods for eliminating or reducing the occurrence of reflection cracks in the overlay.
APPENDICES

Appendix A-1 New Jersey Type A Expansion Joint
Appendix A-2 Miscellaneous Subbase Soil Test Results
Appendix A-3 Comparative Test Results: Project Subbase and Materials in Cork Space
# APPENDIX A-2

## MISCELLANEOUS SUBBASE SOIL TEST RESULTS

<table>
<thead>
<tr>
<th>TEST PROPERTY AND METHOD</th>
<th>TOP COURSE SUBBASE RESULTS</th>
<th>BOTTOM COURSE SUBBASE RESULTS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SOIL CLASSIFICATION</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HRB System</td>
<td>A-1-a</td>
<td>A-1-a</td>
</tr>
<tr>
<td>Unified System</td>
<td>GM</td>
<td>GW-GM</td>
</tr>
<tr>
<td><strong>PROCTOR COMPACTION</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(ASTM D-698, METHOD C)</td>
<td>Maximum Density</td>
<td>138 lb/ft$^3$*</td>
</tr>
<tr>
<td></td>
<td>Optimum Moisture Content</td>
<td>8.0%</td>
</tr>
<tr>
<td><strong>FIELD MOISTURE/DENSITY</strong></td>
<td>Average Sand Cone Density</td>
<td>135.8 lb/ft$^3$</td>
</tr>
<tr>
<td></td>
<td>Average Moisture Content</td>
<td>7.4%</td>
</tr>
<tr>
<td><strong>CALIFORNIA BEARING RATIO</strong></td>
<td>CBR At Proctor Values</td>
<td>97</td>
</tr>
<tr>
<td></td>
<td>CBR After 4 Days Soaking</td>
<td>67</td>
</tr>
<tr>
<td></td>
<td>Moisture Content After Soaking</td>
<td>8.4%</td>
</tr>
<tr>
<td></td>
<td>Swell After Soaking</td>
<td>None</td>
</tr>
<tr>
<td><strong>FALLING HEAD PERMEABILITY</strong></td>
<td>K for 24&quot; to 18&quot; Head</td>
<td>3.3x10$^{-6}$ cm/sec</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.43x10$^{-6}$ cm/sec</td>
</tr>
<tr>
<td><strong>PARTICLE BREAKDOWN BY</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>FREEZE-THAW TEST</strong></td>
<td>(AASHO T-103, PROC.&quot;A&quot;)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loss of Fine Aggregate,22Cycles</td>
<td>0%</td>
</tr>
<tr>
<td></td>
<td>Loss of Coarse Aggregate,22Cycles</td>
<td>1.0%</td>
</tr>
<tr>
<td><strong>PARTICLE BREAKDOWN BY</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>COMPACTION TEST</strong></td>
<td>(5 POINT PROCTOR)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Before/After % Passing</td>
<td></td>
</tr>
<tr>
<td></td>
<td>No. 4</td>
<td>47.2/53.1</td>
</tr>
<tr>
<td></td>
<td>No. 50</td>
<td>19.0/22.1</td>
</tr>
<tr>
<td></td>
<td>No. 200</td>
<td>12.2/13.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>39.6/48.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>14.0/18.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8.0/9.2</td>
</tr>
</tbody>
</table>

*Particle breakdown observed during test.
# APPENDIX A-3

## COMPARATIVE TEST RESULTS: PROJECT SUBBASE AND MATERIALS IN CORK SPACE

<table>
<thead>
<tr>
<th></th>
<th>Cork Space Samples</th>
<th>Subbase Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Field Specimens</td>
<td>Laboratory Sawed Specimens</td>
</tr>
<tr>
<td></td>
<td>Location C</td>
<td>Location A</td>
</tr>
<tr>
<td>SiO₂%</td>
<td>70.30</td>
<td>64.00</td>
</tr>
<tr>
<td>IGNITION LOSS%</td>
<td>10.51</td>
<td>10.61</td>
</tr>
<tr>
<td>MICROSCOPIC ANALYSIS</td>
<td>no cork present</td>
<td>no cork present</td>
</tr>
<tr>
<td>Al₂O₃%</td>
<td>1.74</td>
<td>2.10</td>
</tr>
<tr>
<td>Fe₂O₃%</td>
<td>8.66</td>
<td>13.88</td>
</tr>
<tr>
<td>CaO%</td>
<td>6.35</td>
<td>5.93</td>
</tr>
<tr>
<td>MgO%</td>
<td>2.03</td>
<td>2.01</td>
</tr>
<tr>
<td>SO₃%</td>
<td>0.36</td>
<td>0.3</td>
</tr>
<tr>
<td>Na₂O%</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>K₂O%</td>
<td>0.06</td>
<td>0.07</td>
</tr>
</tbody>
</table>

*The expected ignition loss for a sample which is exclusively cork is nearly 100% and is about 10-12% for a sample of subbase soil. Any intermediate value is thus indicative of a mixture of soil and cork.*