NEW JERSEY COMPOSITE PAVEMENT PROJECT

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This paper reports on the 8-year performance of a 1,500-ft section of composite pavement constructed in 1963 on NJ-3, a major route carrying approximately 88,780 vehicles daily: Of those vehicles, 14.6 percent are trucks. The pavement consisted of a 5-component system involving a 6-in. crushed stone subbase, an 8-in. plain concrete base, a 3-in. layer of densely graded crushed stone, a 5-in. layer of dry-bound macadam, and a 3½-in. surface of bituminous concrete. The pavement was designed to provide long-term serviceability and to require an absolute minimum of maintenance, despite the unstable nature of the terrain and very heavy traffic. Cost was about 15 percent higher than that of New Jersey's standard design of reinforced concrete pavement but is expected to be offset by the estimated 23 percent longer service life. Performance data from Benkelman beam deflections, rut depths, roughometer measurements, cross-sectional and profile measurements, and condition surveys show that the composite pavement has performed as designed with minimal maintenance.

•THIS REPORT analyzes the performance of an experimental composite pavement on the westbound roadway of NJ-3 at the east approach to the Hackensack River bridge during the period from July 1963 to June 1971. It also includes condition surveys of similar pavement on the other approaches and of a somewhat thinner composite pavement on the NJ-3 connection to NJ-20 northbound.

The basic data analysis for the pavement on the westbound roadway consisted of the following: traffic data, including loadings; deflection measurements; settlement determinations; concrete base performance; and bituminous concrete surface performance.

DESIGN AND CONSTRUCTION

Several factors influenced the use of a composite design rather than a conventional design in the bridge approach sections:

1. A portion of the terrain over which this highway was constructed was meadowland, and the subsoils consisted of thin varied layers of silt, clay, and sand.
2. The high daily volume of vehicles and a large percentage of heavy truck traffic required that the pavement section be of an unusually sturdy construction in order to remain trouble-free and require minimum maintenance during a relatively long period of time.

The usual procedure for embankment construction in a meadowland area is to preconsolidate the subsoils with the use of sand drains and an overburden. That procedure, unfortunately, requires a minimum settlement time of several months. Because of the high volume of vehicles and the excessive deterioration of the old bridge, which was approaching a hazardous condition, the lengthy settlement period had to be eliminated. Accordingly, no preconsolidation was attempted. Construction proceeded with the removal of the organic peat and muck and replacement with an open-graded quarry-processed material. A normal earth embankment and pavement construction were then placed above that.

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New Jersey's standard design of reinforced concrete pavement (78-ft, 2-in. slab length having doweled expansion joints) did not appear suitable for the project because of the compressible nature of the underlying soil and the possibility of appreciable differential settlements causing serious cracking. The past performance of flexible pavements on major trucking routes in New Jersey meadowland raised doubts as to the adequacy of any conventional flexible pavement under the extreme traffic and unstable soil conditions in that area.

Past experience indicated that bituminous surfacing materials are capable of satisfactorily carrying heavy truck traffic, provided that the bituminous materials are on a highly stable foundation. Two rehabilitation projects that involved thick overlays placed on existing concrete pavement (US-22 between Somerville and Chimney Rock and US-130 in the vicinity of Deans) provided outstanding performance. That performance served as the prime basis for the selection of a composite pavement design for the Hackensack River site.

The objectives in using a composite pavement were

1. To maintain the structural integrity of surface despite an anticipated differential settlement resulting from the deep fill and compressible nature of the underlying soil;
2. To achieve the high load-carrying capacity of a rigid pavement necessitated by the large volume of heavy truck traffic; and
3. To achieve the continuity of surface of a flexible pavement and to minimize reflection cracking.

The specific design selected to achieve those objectives is shown in Figure 1. A 6-in. layer of quarry-processed stone was placed on the subgrade. An 8-in. plain portland cement concrete base (New Jersey class D mix, 1:2.25:4.5) was placed on the subbase and was overlaid with a 3-in. course of densely graded stone base and a 5-in. lift of dry-bound macadam base ($2\frac{1}{2}$-in. stone choked with stone screenings). The composite pavement section was completed with a $3\frac{1}{3}$-in. layer of bituminous concrete (FA-BC-2) consisting of a 2-in. bottom or binder course and a $1\frac{1}{2}$-in. surface course.

The pavement was constructed in July 1963. It is approximately 1,500 ft in length and is divided into two 12-ft lanes and two 13-ft lanes. A concrete curb abuts the edge of the inside lane. A bituminous shoulder abuts portions of the outside lane.

The NJ-20 ramp was of similar construction, except that a 6-in. plain concrete base was used instead of an 8-in. base. The plain concrete base at both locations was constructed with undoweled contraction joints at 15-ft intervals and no expansion joints. Longitudinal joints between lanes were of the plain butt type without tie bars.

The 3-in. base of densely graded stone was placed between the concrete base and the 5-in. macadam base in order to act as a buffer to prevent the reflecting of cracks from the concrete base into the bituminous surface pavement.

COST

The construction bid price of the composite pavement was \$12.39/yd^2, which was much higher than originally anticipated. The cost difference was attributed to the contractor's inexperience with that type of pavement. The relatively short length of the section was also a factor. In subsequent contracts for the other bridge approaches, the bid price was reduced to \$9.85/yd^2. As a comparison, the bid price of a high type of bituminous concrete pavement and a reinforced portland cement concrete pavement in New Jersey, at that time, was approximately \$6 and \$9/yd^2 respectively.

The present construction bid price of the composite pavement (8-in. concrete base), as determined by current averaged weighted quantity prices for New Jersey, is approximately \$17.30/yd^2. In comparison, the same source indicates that a high type of bituminous concrete pavement and a reinforced portland cement concrete pavement cost approximately \$8 or \$15/yd^2 respectively.

The current construction bid price of a 6-in. plain concrete base is approximately 25 percent less than that for the 8-in. plain concrete base. The overall construction bid price of a composite pavement using the 6-in. plain concrete base design is approximately 15 percent less than that of one using an 8-in. concrete base and is essentially the same as the cost for reinforced portland cement concrete pavement.

TRAFFIC DENSITY

The 1970 AADT at the approaches to the NJ-3 Hackensack River bridge in both directions was 88,780 vehicles. In the westbound roadway, trucks accounted for 14.6 percent of the traffic and were distributed per lane as follows (lane 4 is inside left):

Lane	Percent
1	19
2	43
3	32
4	6

From July 2, 1963, to May 27, 1964, the present westbound roadway was used for both eastbound and westbound traffic during the construction of the present eastbound approaches to the Hackensack River bridge and rehabilitation of the southerly bridge. At that time the present westbound roadway was divided into 5 lanes, each 10 ft wide, and was subjected to 3,505,056 equivalent 18-kip axle repetitions.

After May 28, 1964, the roadway was used exclusively for 4 lanes of westbound traffic. The accumulated 18-kip equivalent axle repetitions up to and including December 31, 1970, were as follows:

Lane	Repetitions
1	2,785,704
2	6,427,714
3	4,723,743
4	904,258

ESTIMATION OF PAVEMENT LIFE

Estimates of the remaining service life of existing pavement are based on the findings of the AASHO Road Test (1). According to the Road Test findings, pavement performance is directly related to pavement structure and axle loads. The AASHO Road Test developed equations to estimate pavement performance in terms of the number of axle load applications. Those equations evolved into a simplified procedure for estimating the remaining equivalent 18-kip axle load applications for pavement service life.

The flexible pavement service life nomograph shown in Figure 2 (1) and developed from the AASHO Road Test equations was used to estimate the accumulated number of equivalent 18-kip single-axle load applications from the time that the pavement is placed in service to the time that it requires resurfacing or reconstruction (ELA_p). The calculated structural number $\overline{SN}$ of the pavement was based on the AASHO design equation ($SN = a_1d_1 + a_2d_2 + a_3d_3$), where a is the material strength coefficient and d is the layer thickness. In accordance with those equations, the structural number of the composite pavement section was estimated and is given in Table 1.

The ELA_p was estimated from the nomograph (Fig. 2) by the known soil support value ss of 6.75 for the NJ-3 location and the derived structural number of 7.62 for this pavement. However, the nomograph does not go beyond a structural number of 6.0. With that structural number, a conservative estimated service life of 65 million 18-kip applications was obtained. The composite pavement has already experienced 14.8 million 18-kip single-axle load applications. Therefore, the estimated remaining service-life was 50.2 million 18-kip axle applications.

The remaining service life after December 31, 1970, was conservatively estimated to be 30 years. The total service life, from the time of construction until overlay or replacement of the original bituminous surfacing is required, was estimated to be 38 years. Of course, this theoretical concept does not take into account deterioration of the bituminous surface course through age hardening.

ROUGHNESS SURVEY

The fourth consecutive roughometer survey, in which the New Jersey Department of Transportation roughometer was used, was completed in March 1971. For readings prior to January 1968, the BPR roughometer was used.

Table 2 gives the roughometer measurements made to date. There was little variation in the measured riding quality of the pavement in spite of the fact that data were collected with 3 separate roughometer units. There are also no discernible differences due to seasonal changes in either the pavement surface or the equipment.

A commonly applied criterion for assessing roughness data is that developed by the Federal Highway Administration from a 580-mile survey of new rural Interstate mileage in 17 states. According to that criterion, lane 1 and lane 4 rated poor in riding quality. Lanes 2 and 3 rated fair. That rating is, of course, subjective. Because of the process of extrapolating data from the actual 0.3-mile length to the indicated inches of roughness per mile, the actual roughness of the measured section tends to be exaggerated.

DEFLECTIONS

Pavement deflections were measured with a Benkelman beam in the right wheelpath of lane 1. Nine Benkelman beam surveys were conducted since the inception of the project. A 7,500-lb wheel load was used in the first 2 surveys, and a 9,000-lb wheel load in all the others. That loading corresponds to the 18-kip axle loading used in flexible pavement design. Readings were taken at the contraction joints (aggregate interlock load transfer), construction joints (no load transfer), and midpoints of the concrete base slabs.

A summary of the deflections, recoveries, and residuals to November 1970 is given in Table 3. The deflections measured to date are well within acceptable limits and do not appear to indicate a substantial increase from the original 9,000-lb load deflection and recovery readings. The upper limit (3) of acceptable values is approximately 0.020 in. for bituminous pavement.

Negative residuals recorded in the 1965 and 1966 readings were probably caused by preloading of the test slab because of the close proximity of the weighted vehicle. That indicates that the load transfer by aggregate interlocking action was functioning properly at the time. The measurements in cool weather (Oct. 1965) had lower negative values as would be expected. The absence of negative values in subsequent surveys may be due to that fact that measurements were made in cool weather or that the effective load transfer across the joints lessened.

In previous studies, Benkelman beam readings fluctuated somewhat with higher ambient temperatures and exposure to direct sunlight. Because measurements on this section were so small, the effect of those fluctuations may introduce the apparent negative residuals observed.

SETTLEMENT

Periodic close interval profile and cross-sectional readings were taken on the pavement surface in the outside and inside lanes by means of a level and rod; the readings were taken to the nearest estimated 0.001 ft. The profile readings, taken at 2-ft intervals in the outer wheelpath of the outside lane and the inner wheelpath of the inside lane, extend from station 0+00 (the junction of the pavement with the east abutment of the Hackensack River bridge) eastward to station 3+00. That was the approximate area where subsoils consisted of unstable materials.

Cross-sectional readings were taken at 1-ft intervals at each even station for the entire length of the project. The original cross sections extended across the entire roadway. However, all subsequent surveys were limited to the inside and outside lanes in an effort to minimize interference with traffic and provide for the safety of personnel.

Figure 3 shows the average profile settlement for both the inside and outside lanes. Figure 4 shows a typical cross-sectional plot for stations 1+00 and 3+00. The average profile settlement of both the outside and inside lanes has esstentially stopped; most

Figure 1. Experimental composite pavement section.

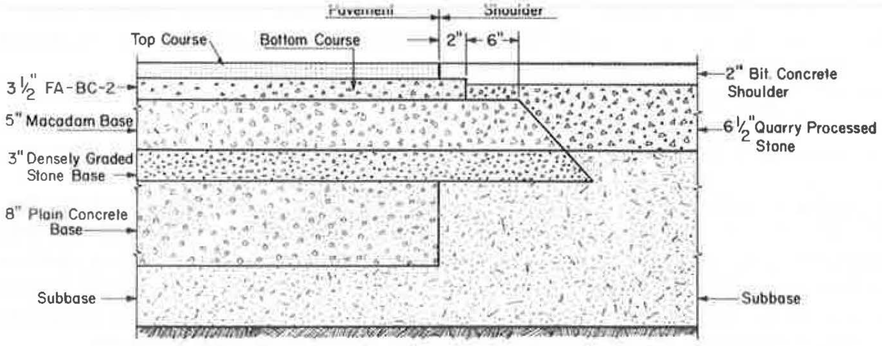

Figure 2. Flexible pavement service life nomograph (psi = 2.5).

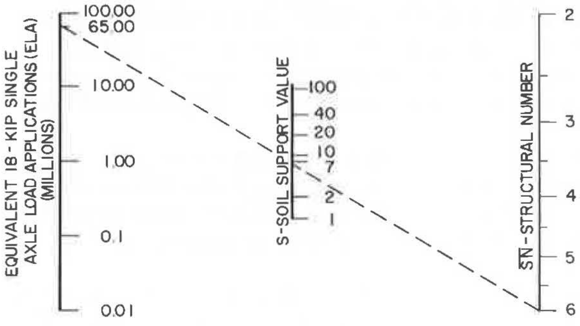

Table 1. Structural number of pavement section.

Thickness (in.)	Material	Coefficient	$\overline{SN}$
3½	Bituminous concrete	0.44	1.54
5	Macadam base	0.20	1.00
3	Densely graded stone	0.14	0.42
8	Plain concrete	0.50	4.00
6	Subbase	0.11	0.66
Total			7.62

Table 2. Road roughness survey results.

	Roughness Index (in./mile)			
Date	Lane 1	Lane 2	Lane 3	Lane 4
Nov. 1964	103	108	102	110
May 1965	110	107	105	110
Nov. 1965	110	103	103	107
Nov. 1966	106	92	101	107
Jan. 1968	116	103	110	121
Dec. 1968	100	91	96	117
Feb. 1970	115	95	102	124
March 1971	115	91	101	128

settlement occurred the first year after construction. The measured initial settlement was originally anticipated on the project. The typical cross-sectional plots show the maximum settlement in the wheelpaths. Settlement appears more prominent at station 1+00 in the proximity of the Hackensack River bridge, where as much as 0.10 ft of settlement occurred. A lesser amount of settlement occurred at station 3+00, 200 ft farther from the bridge.

RUT DEPTHS

Rut depths were taken at 50-ft intervals in both wheelpaths in lanes 4 and 1 for the entire length of the project. Rut depths were recorded as the measured distance from the bottom of an 8-ft straightedge placed across 1 wheelpath to the bottom of the rut. As with the profiles and cross sections, the rut-depth measurements were limited to the inside and outside lanes in the interest of traffic and safety. Five rut-depth surveys have been conducted to date. The average measurements are given in Table 4.

Little increase in rutting in the left wheelpath of lane 1 was noted since the original survey. However, rut-depth measurements in the right wheelpath show an increase of 0.15 in., most of which occurred between the December 1969 and November 1970 surveys. That rutting is mostly in the area of localized cracking near Grace Street and will be discussed later.

Essentially no increase in rutting in lane 4 was noted since the original survey. The large increase in rutting measured during the December 1968 survey was due to the improper selection of the wheelpath area. The relatively large rut-depth measurements in the left wheelpath are attributed to inadequate initial compaction close to the curb at the time of construction.

By the way of comparison, a survey on a standard section of flexible pavement on I-80 (constructed with a 2-in. bituminous top, 4-in. bituminous-stabilized base, 6-in. dry-bound macadam, 6-in. quarry-processed stone, and 12-in. subbase) having a comparable age but lighter traffic volumes shows average rut depths of 0.20 to 0.40 in. for the left wheelpath and 0.30 to 0.45 in. for the right wheelpath of the outer lane. In spite of the fact that NJ-3 received a greater volume of traffic, the rutting is not greater than the standard section of I-80.

CONDITION SURVEY

A detailed condition survey of the pavement surface was made in March 1971. A few fine inconsequential cracks were observed close to the catch basins and along the pavement edge but did not appear to have progressed since observed in the 1970 survey. No reflection cracks were found over the joints in the underlying concrete base.

A large crack, $\frac{1}{4}$ in. wide by 82 ft long, at the Grace Street exit is located over and parallel to a 24-in. water main. It is possible that the longitudinal cracking may be caused by differential settlement in the area of the water main.

A condition survey of the visual condition of the pavement surfaces of sections 1E, 1F, and 4G was conducted in July 1971. Those 3 approaches were partially constructed with the identical composite pavement design. However, some areas were constructed on existing concrete pavement with various thicknesses of bituminous materials.

Where the pavement was constructed with the composite pavement design, several fine reflection cracks were noted. However, at least 90 percent of the underlying joints on main-line pavement of sections 1F and 4G and section 1E do not show any reflection cracking. Insignificant patches and indentation or scarring were noted on all 3 approaches. No evidence of raveling or shoving was noted.

The NJ-20 ramp from NJ-3, part of section 1E, is constructed of a similar pavement design with the exception of a 6-in. concrete base rather than the 8-in. base as on the main-line pavement. This pavement was not constructed on a subgrade similar to that of the main-line pavement. In the 1971 condition survey of that ramp, approximately 15 percent of the underlying joints showed reflection cracks approximately $\frac{1}{8}$ to $\frac{1}{4}$ in. wide. Those cracks are mainly in the area where NJ-3 eastbound connects with NJ-20 northbound. The distress may be attributed to a localized depression.

Table 3. Benkelman beam survey results.

Date	Avg Value at Joints (0.001 in.)			Avg Value at Midpoints (0.001 in.)		
	Deflection	Recovery	Residual	Deflection	Recovery	Residual
Dec. 1963	5	5	0	5	5	0
May 1965	4	7	-3	4	7	-3
Oct. 1965	7	7	0	6	6	0
June 1966	4	7	-3	4	7	-3
Nov. 1966	7	7	0	6	6	0
Nov. 1967	6	6	0	6	6	0
Nov. 1968	7	7	0	7	6	1
Oct. 1969	7	6	1	6	5	1
Nov. 1970	7	7	0	7	7	0

Figure 3. Average profile settlement.

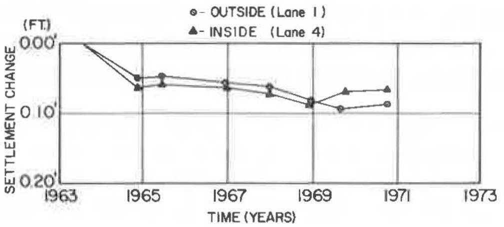

Figure 4. Typical cross section.

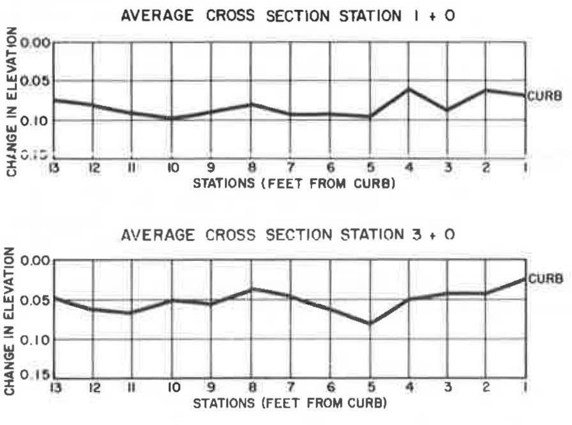

Table 4. Rut-depth measurements.

Date	Avg Value in Lane 4 (in.)		Avg Value in Lane 1 (in.)	
	Left Wheel-path	Right Wheel-path	Left Wheel-path	Right Wheel-path
Dec. 1966	0.45	0.15	0.30	0.20
Dec. 1967	0.40	0.15	0.30	0.20
Dec. 1968	0.45	0.45	0.20	0.25
Dec. 1969	0.45	0.20	0.20	0.20
Nov. 1970	0.40	0.20	0.25	0.35

In general, the composite pavement of the 3 approaches and the NJ-20 ramp discussed here appear to be in good condition.

CONCLUSION

The conclusions are based primarily on the performance of the westbound lanes of the east approach where complete data were collected. The composite pavement has functioned in accordance with design objectives with no interference with traffic for required maintenance or repairs. After 8 years of satisfactory service, the composite pavement in the westbound lanes of the east approach was subjected to more than 14.8 million 18-kip equivalent axle repetitions and gave no indication of progressive deterioration. No localized differential settlement was observed. A high load-carrying capacity was achieved as evidenced by the low deflection measurements.

The latest condition survey indicates no reflection cracking and no pavement deficiencies. Roughometer measurements and rutting measurements are within acceptable limits. Riding qualities of the pavement have not deteriorated to any significant extent.

The estimated service life of the 8-in. base composite pavement was calculated by the AASHO system for remaining service life of flexible pavement. Although the composite pavement is not a true flexible pavement, it exhibits many flexible pavement characteristics such as rutting and surface flexibility. A conservative estimate of the remaining service life of the composite pavement is 30 years or a total service life of at least 38 years. The cost of the 8-in. base composite pavement is approximately 15 percent greater than that of the reinforced portland cement concrete pavement. However, the increased expenditure produces approximately 23 percent greater service life.

The 6-in. plain concrete base pavement on the NJ-20 ramp also performed well. That ramp, which experiences comparatively light traffic, did not show patched areas or evidence of a need for maintenance.

ACKNOWLEDGMENTS

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