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Abridgment

Pavement Failure Investigation: Case Study

VISHNU A. DIYALJEE

ABSTRACT

An investigation of pavement distress occurring along a major two-lane roadway 5 years after its construction is presented. The primary objective of the study was to determine the probable cause or causes of the pavement distress. The investigation involved a condition survey and an examination of the pavement structure and subgrade through soil borings. The condition survey showed that outer wheel path rutting and associated cracks were severe on both lanes and covered about 68 percent of the overall length of the roadway. The soils investigation revealed that the bank gravel subbase was saturated and the bituminous base course had deteriorated to a virtually cohesionless material that could be easily removed with the fingers. Distinct rapid seepage of water was observed at the interface of the base and subbase layers and within the subbase. On the basis of the findings of the investigation, it was concluded that the major factor causing distress was free water trapped within the pavement structure. This water, it was reasoned, infiltrated the pavement through cracks and a porous surface but because of the poor drainability of the subbase was unable to leave the pavement through the shoulders. This situation resulted in the pavement existing in a "bathtub" condition.

Most, if not all, flexible pavement structures undergo some form of distress during their design life. Investigation of the cause or causes of distress is required for successful pavement rehabilitation and to provide data for improving or modifying design methods, construction techniques, and job specifications.

An investigation undertaken to determine the probable cause or causes of continually occurring pavement distress along a major two-lane roadway is described.

BACKGROUND

The roadway investigated is located in Trinidad, West Indies, an island with a uniform average yearly temperature of 26° C (79° F) and annual rainfall of

1.5 to 3.0 m (5 to 10 ft). This roadway, situated in the Southern Basin, was constructed in 1975 as a connector road between a major four-lane divided highway and a secondary road at the southern extremity of the four-lane highway.

The cross section of the connector road consisted of two 3.66 m (12 ft) travel lanes with 1.22-m (4-ft) shoulders. The design pavement thickness was 267 mm (10.5 in.) under the travel lanes and consisted of 127 mm (5 in.) of asphaltic concrete pavement overlying 140 mm (5.5 in.) of down-graded bank gravel subbase with a maximum size of 38 mm (1.5 in.) and 25 percent passing the 0.074 mm (No. 200) mesh. The shoulder design consisted of 89 mm (3.5 in.) of chip-sealed asphaltic concrete overlying the bank gravel subbase.

Outer wheel path ruts, longitudinal cracks, and pavement distortions along both lanes of the roadway prompted an investigation to be undertaken in June 1980. The investigation included a pavement condition survey and a subsoil investigation. Before this investigation, isolated areas had been overlaid but the previous distress recurred.

CONDITION SURVEY

This survey, done by the guidelines outlined in the Manual for Condition Rating of Flexible Pavements (1), indicated that the major pavement distress manifestations were

1. Outer wheel track rutting and associated fatigue cracks,
2. Pavement distortion, and
3. Longitudinal cracks and depressions along the shoulders.

These distress manifestations were predominant along the eastbound lane where only about 6 percent of the pavement surface was free from major defects.

SUBSOIL INVESTIGATION

Test Holes

Five boreholes, including two groundwater observation holes, and three test pits were sunk at locations shown in Figure 1. During the drilling, the following were observed:

1. Rapid seepage of water at the interface of the asphaltic base course and bank gravel layers and through the bank gravel itself. The quantity of flow was measured as roughly 76 cm^3 (0.003 ft^3) per second in Borehole 1.
2. Seepage or "bleeding" of water through cracks in the pavement surface.
3. Deterioration of the asphaltic concrete base course. This layer was found to be deficient in asphalt and extremely brittle. In Borehole 2, for example, the base course had deteriorated into an almost cohesionless material that could be easily removed with the fingers.

Groundwater Conditions

No groundwater was encountered in the test holes or in the two observation boreholes sunk off the edge of the shoulder. The observation holes were drilled to a depth of 5 m (15 ft) relative to the elevation of the carriageway at the test boring locations and observed over a period of 2 weeks following the site investigation.

Soil Profile

The profile deduced from the boreholes consisted of structural pavement layers overlying a silty clay subgrade soil. The thickness of the structural pavement varied between 300 and 584 mm (12 and 23 in.) and consisted of 120 to 406 mm (5 to 16 in.) of asphaltic concrete pavement. The subbase course consisted of bank gravel varying in thickness between 127 and 180 mm (5 and 7 in.).

SOIL CHARACTERISTICS

Atterberg limits, shear strength, and standard Proctor compaction results for the subgrade soil are shown in Figures 2 and 3, and California bearing ratio (CBR) test results are summarized in Table 1.

The subgrade soil was mainly of the CH type and exhibited very high potential expansiveness (Figure 4) as determined from Williams' chart (2).

Although swelling of the subgrade soil can contribute to pavement failure, there was no consistent evidence from the condition survey and the soils investigation that the distress along the carriageway was caused by subgrade volume change.

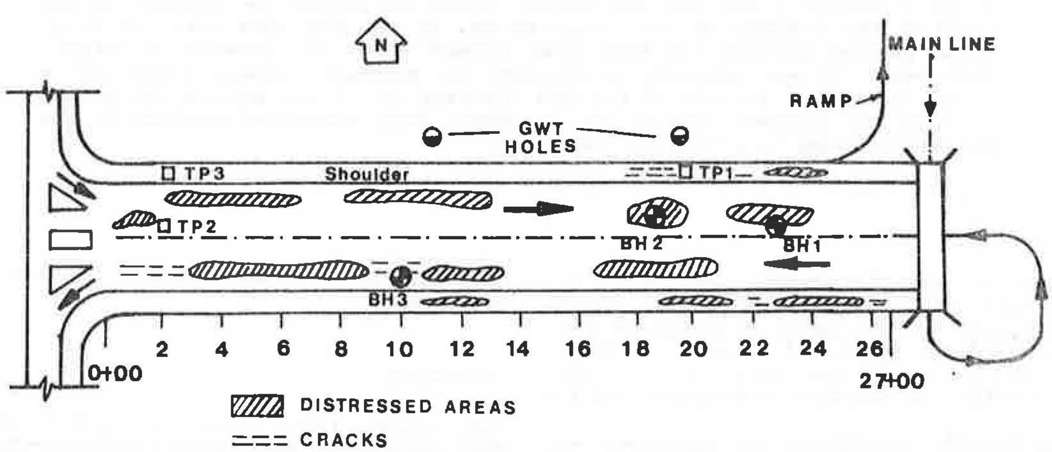

FIGURE 1 Plan of connector road.

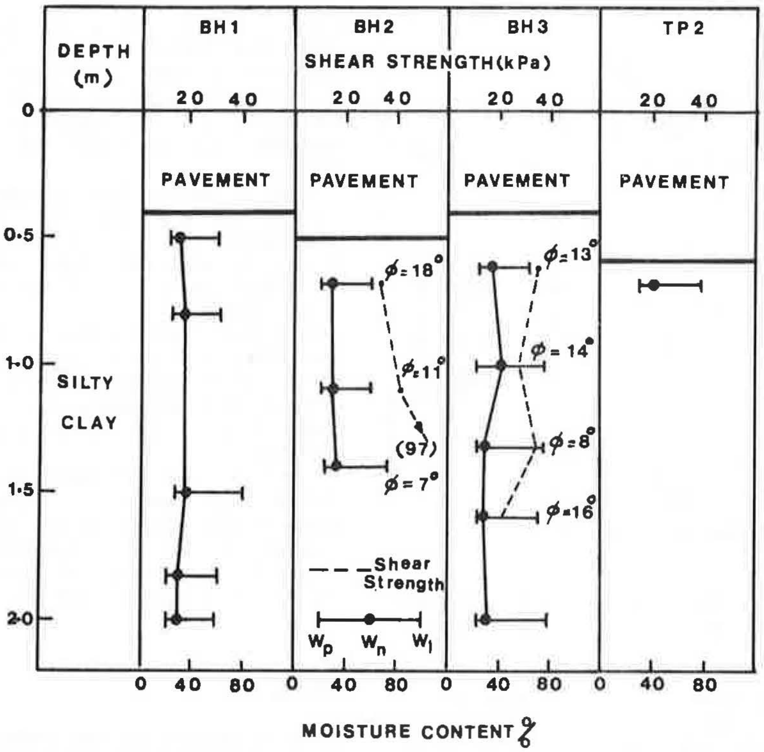

FIGURE 2 Moisture content and shear strength versus depth.

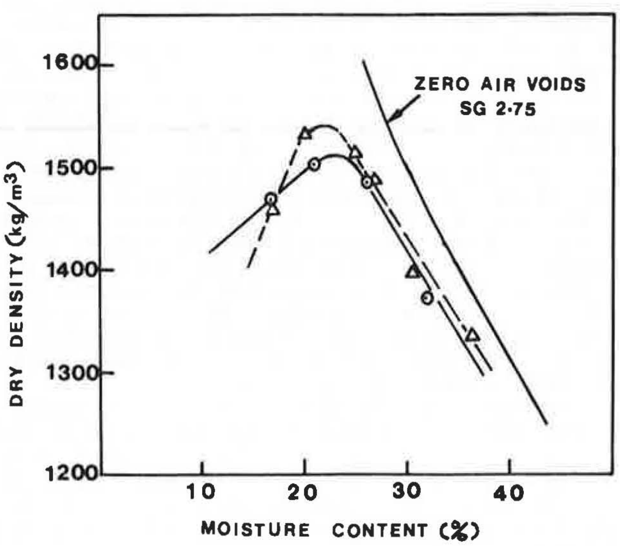

FIGURE 3 Moisture-density relationship of subgrade soil.

SUBGRADE STRENGTH

For pavement design, the strength of the subgrade in tropical climates is normally assessed in terms of the California bearing ratio. Using shear strength parameters, c = 36 kPa (5.22 psi) and ϕ = 14 degrees, and average of values within the top 0.6 m (2 ft) of subgrade in Boreholes 2 and 3, an estimated in situ CBR value of 6.3 percent was obtained (3). This value compares favorably with 6.6 percent obtained from laboratory tests on an in situ sample, (Table 1).

Although several criteria have been proposed for determining the soil strength to be used for design

TABLE 1 Summary of California Bearing Ratio Test Results

Test Pit	Bulk Density in kg/m³ (lb/ft³)	Dry Density in kg/m³ (lb/ft³)	MC (%)	CBR[a] (%)	Swell (%)	Remarks
1	1902 (119)	1514 (94)	25.6	6.8		Remolded
		1538 (96)	22			Standard Proctor optimum
1		–	–	3.2	2.5	4-day soak
2	1891 (118)	1458 (91)	32	6.6		In situ
3	1869 (117)	1506 (94)	24	7.1		Remolded
		1510 (94)	23			Standard Proctor optimum
3				2.5	2.6	4-day soak

[a]Average of CBR values obtained from testing both top and bottom of specimen.

purposes (4-6), the 4-day soaking period is considered the most appropriate for soils exhibiting appreciable swell and for climates in which annual rainfall exceeds 245 mm (9.6 in.) (6). Therefore, the CBR of 2.9 percent rounded to 3 percent was considered appropriate for design.

DESIRABLE PAVEMENT THICKNESS

As a first step in pinpointing the cause of the pavement failures, the adequacy of the pavement design of the connector road was checked using the National Crushed Stone Association method of design (7). This method was chosen in the absence of actual traffic data.

Using a CBR of 3 percent, pavement thicknesses of 533 mm (21 in.), 610 mm (24 in.), 660 mm (26 in.), and 762 mm (30 in.) were obtained for four categories of traffic loadings--medium, medium heavy,

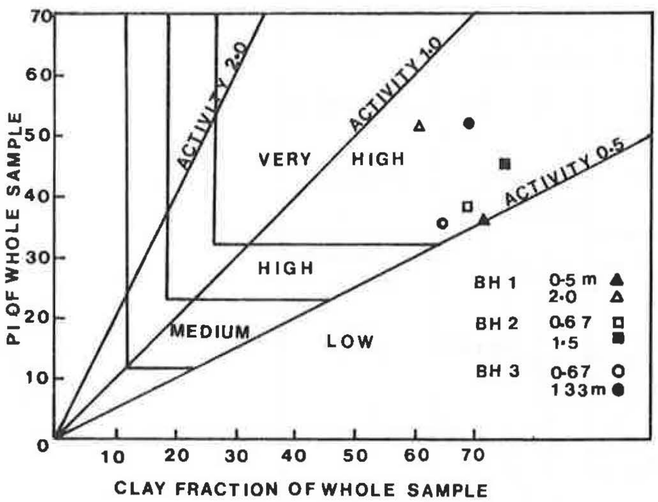

FIGURE 4 Plasticity index versus minus 2μ fraction.

heavy, and very heavy. These pavement thicknesses are considerably greater than is the design pavement thickness of the roadway.

Except for the design pavement thickness of 267 mm (10.5 in.) for the connector road, no information could be found on the "thickness design" of this roadway. However, for the adjoining main-line (four-lane) highway the design pavement thickness was 394 mm (15.5 in.). This thickness was determined using the Asphalt Institute's method of design (8), a design CBR of 2.5 percent, design traffic number of 82, and a 20-year design life (1970 to 1990). This pavement was made up of 234 mm (9.2 in.) of asphaltic concrete pavement overlying 140 mm (5.5 in.) of downgraded bank gravel.

The pavement thickness of the connector road was considerably less than that of the main line. However, when the in situ thickness of the connector road at the time of investigation is considered, the average pavement thickness as well as asphalt pavement thickness far exceeded the design thickness of the main line.

Because the connector road was only 5 years old at the time of this investigation, the design thickness of 267 mm (10.5 in.) should have been adequate for a design traffic intensity similar to that of the main line using planned stage construction for a specified period (8).

It is of interest to note, however, that in a recent 20-year design (1980 to 2000) of the south extension of the main line, a pavement thickness of 851 mm (33.5 in.) was recommended. This design was based on a design traffic of 2 to 6 million standard axles and a subgrade CBR of 2 percent. The design pavement was to be made up of 76 mm (3 in.) of asphaltic concrete surface and leveling courses, 150 mm (6 in.) of asphaltic concrete base course, 200 mm (8 in.) of crushed rock base course, and 425 mm (17 in.) of stone and sand-free clay surface coatings.

Compared to the 1970 and 1975 main-line and connector road designs, this recent design suggests a large escalation in axle loads that could probably not have been anticipated during the previous designs. This increase in traffic was due to the increased off-shore exploration along the south coast of Trinidad and the increasing sales of commercial and passenger vehicles caused by a booming economy--the spin-off of increased world market oil prices in the mid 1970s. In 1979, for example, the waiting period for a new vehicle was about 3 years.

On the basis of the foregoing, it can be readily concluded that both the connector and the main line were seriously underdesigned. On the other hand, because the connector road with an in situ asphaltic concrete pavement of 406 mm (16 in.) and an overall pavement thickness of 583 mm (23 in.) still suffered failures, it is questionable whether inadequate pavement thickness was the principal cause of distress.

Corroboration that inadequate pavement thickness was not the principal factor was provided by an investigation of the northbound carriageway of the main line in areas where seepage through the pavement surface was noted. In such areas the only external evidence of pavement distress was short discontinuous longitudinal cracks concentrated along the inner and outer wheel paths. Investigation of these areas showed pavement seepage and deterioration of the asphaltic base course similar to that found during the investigation of the connector roadway.

On the basis of these observations, it was concluded that the major factor influencing distress along travel lanes of the connector road was water trapped within the pavement section. Inadequate pavement thickness, although identified, was not considered a significant factor.

DISCUSSION

It is surmised, in the absence of detailed information on the history of construction or postconstruction performance of this roadway, that this water entered the pavement from surface infiltration through cracks and a porous asphalt surface. This conclusion was reached because there was no evidence of groundwater seepage in any of the test holes or groundwater observation holes. That the roadway section investigated was entirely on fill ruled out the possibility of lateral seepage as well. It should be noted that, up to the time of this investigation, crack sealing was not normally carried out in Trinidad as part of the routine maintenance of roadways.

The loss of serviceability of the base course as a result of the presence of water is readily appreciated on examination of the base course. As mentioned previously, this layer has been reduced to an almost cohesionless material. It is the author's opinion that the drainage of the entrapped water occurs only by "bleeding" through the pavement surface as noted during the investigation.

Bleeding occurs through cracks in the pavement surface and is encouraged by repeated traffic loadings and thermal changes. Bleeding was noted to occur principally during the warmer portion of the day. The effect of increased temperature is to heat the air entrapped in the pavement and thereby give the water a "lift."

Good lateral drainage of the pavement structure, a factor considered essential to pavement longevity, could only have occurred through the bank gravel subbase course. However, the bank gravel used was moderately plastic (liquid limit 29 percent, plastic limit 18 percent). In addition, properly compacted bank gravel has a low permeability due to the high percentage of minus 0.074 mm (No. 200) fraction. Low drainability of the subbase was substantiated by the following observations:

- The amount of water observed to be contained within the carriageway on removal of the asphaltic layers and
- Water standing for a day or two on the surface of the subbase material after a period of rainfall.

The combination of the poor drainability of the subbase and the presence of water caused the pavement section to exist in a "bathtub" condition (9).

CONCLUSION

An investigation of the cause of pavement distress along a major two-lane roadway has been described. The results of this study have shown that the major factor affecting pavement performance was free water trapped within the pavement section. Overall, this study demonstrates the influence of water on pavement performance and the necessity for providing good drainage within the pavement structure in climates with moderate to heavy rainfall.

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