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Pavement Design for a 3.5-Million-Pound Vehicle

WILBUR CHARLES GREER, JR.

The investigative and analysis procedures used to evaluate roadways for the movement of two nuclear reactor vessels from Knoxville, Tennessee, to the Tennessee Valley Authority's (TVA) Phipps Bend power generation plant are presented. The route included 74 km (46 miles) of state, county, and private roadways, both paved and unpaved. The total weight of each reactor pressure vessel and its transport trailers was approximately 13 345 kN (3 million lb). The five prime movers brought the total weight for each move to approximately 15 569 kN (3.5 million lb). The transport trailer was supported on 24 axle lines with 16 wheels/axle line. The methods of compiling existing construction data, field testing, and analysis procedures are discussed. The application of proofrolling to verify the results of the analyses is also presented. Deflection and density testing, both before and after the moves, were performed by the Tennessee Department of Transportation. An analysis of these data indicates no serious effects due to the two moves that occurred in 1980 and 1981. A comparison between the results of the U.S. Army Corps of Engineers procedures originally used to evaluate pavement thickness requirements and the results of layered elastic computer program analyses after the second move is presented. The results of the two methods compare favorably.

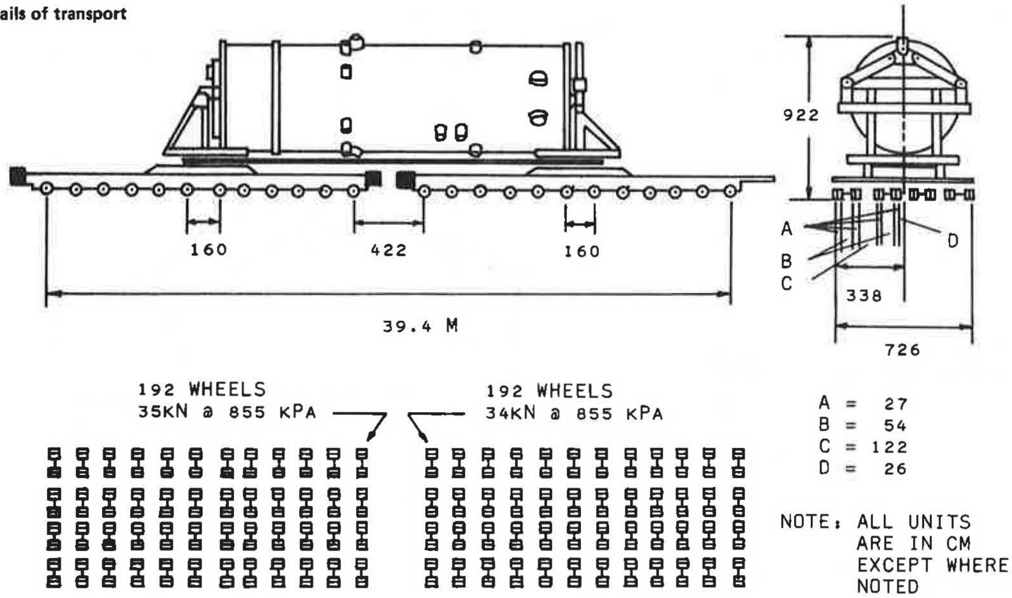
The construction of large industrial manufacturing plants and power-generation facilities may require that very heavy mechanical items be brought to the site in one piece. Items such as generators and nuclear reactor pressure vessels may weigh a few thousand kilonewtons (several hundred thousand pounds) to in excess of 9000 kN (2 millions lbs). Most projects may have only one or two such pieces of equipment to move. Where possible, these items can be brought to the site by water or rail transportation; however, the equipment has to be transported across paved roads to many sites. The vehicles used to move the equipment often have many axles and numerous tires per axle in order to spread the load and reduce individual tire loads to acceptable levels. The high axle loads, numerous tires, and very low number of applications complicate the analysis of the pavement structures.

The Tennessee Valley Authority's (TVA) Phipps Bend nuclear power generation plant is located in northeast Tennessee. The plant, currently under construction, will have two nuclear units when completed. The steel reactor pressure vessel (RVP) for each unit was fabricated in Memphis nearly 692 km (430 miles) away. Each RPV weighs 9190 kN (1033 tons) and thus just cannot be loaded on a normal truck and hauled to the plant site. Plans for the transport of the RPVs to the plant site called for each RPV to be barged to Knoxville and then hauled over 74 km (46 miles) of state, county, and private roadways to the plant site. These are believed to be some of the heaviest, if not the heaviest, loads ever moved over U.S. roads.

The VSL Corporation, the transport contractor, authorized an investigation and evaluation of the 74 km (46 miles) of roadways to be traversed relative to each pavement's ability to withstand the loads imposed through the transport trailers. The objectives of the investigation were to investigate and develop recommendations with regard to pavements and to the geotechnical aspects of the travel route, of planned detours, and of planned road widenings. The basic criteria for a successful move were defined as the following:

1. Prevent, as economically as possible, the transport trailers from becoming stuck; stoppage could result in enormous delay costs; and
2. Minimize visible and measurable damage to the roadways that could result in significant damage charges against the contractor.

Figure 1. Details of transport assembly.



The investigation concentrated on identifying and evaluating potential critical pavement sections along the route that might impede the transport of the RPVs or that might experience intolerable damage due to the transportation effort. The first RPV was moved successfully in July 1980, and the second RPV was moved successfully in July 1981.

TRANSPORT ASSEMBLY

The transport assembly for each RPV was comprised of two main trailer sections. Each main trailer section was approximately 19.5-21.3 m (64-70 ft) long and 7.6 m (25 ft) wide. When assembled, the transport assembly was 47 m (154 ft) long with 24 axle lines and 16 tires/axle line (384 tires in total).

The transport trailers and RPV together weighed 13 247 kN (1489 tons). This load was transferred to the pavement surface through the 384 tires. The inflation pressure of each tire was 855 kPa (124 lb•f/in²). The load on each tire on the front trailer was approximately 34 kN (7590 lb•f) and the load on each tire on the rear trailer was approximately 35 kN (7890 lb•f). Details of the transport assembly and tire spacings are shown in Figure 1.

The transport trailers were pulled by three tractors and pushed by two more tractors. The typical tractor type (Michigan) had four wheels and weighed 436 kN (49 tons). The tires had an inflation pressure of 234 kPa (34 lb•f/in²). The axle configuration was single axle-single wheel. The wheel spacing was 274 cm (9 ft) and the axle spacing was 330 cm (10 ft 10 in). The speed of the transport assembly was a maximum of 4.8 km/h (3 mph).

INVESTIGATIVE PROCEDURES

In order to evaluate the existing pavements, it was necessary to determine the existing thickness of each pavement and the subgrade support conditions. Because of the relatively long total length of roadways to be investigated, typical pavement thickness profile data for both the traveled way and shoulders were compiled for each roadway by reviewing the available construction plan drawings from the files of the agencies responsible for each roadway.

The thickness profiles were supplemented by drilling 49 soil test borings along the proposed route to verify that the in-place thicknesses for each pavement section generally corresponded to the construction plan drawings. The borings generally were extended to a depth of 3 m (10 ft) below the ground surface. The borings also were used to develop pavement thickness profiles where little or no information from construction plan drawings was available.

A total of 26 different pavement sections were identified along the proposed route. The pavement thickness profiles ranged from as little as 2.5 cm (1-in) of bituminous concrete over 10 cm (4 in) of crushed stone on county roads to as much as 38 cm (15 in) of bituminous concrete over 30 cm (12 in) of crushed stone on the state routes. The state routes had been overlaid numerous times in the past.

Field California bearing ratio (CBR) tests were performed near the top of the subgrade in 42 of the borings to determine the in-place CBR values.

THICKNESS ANALYSIS PROCEDURES

The pavement thickness design procedures considered in the premove analysis are discussed elsewhere (1-6). The procedures for flexible pavements based on the U.S. Army Corps of Engineers design equation (1, Equation 1) were chosen as the basis for the analysis and evaluation:

$$t = [(0.23 \log C) + 0.15] \sqrt{P\{[1/8.1(CBR)] - (1/p\pi)\}} \quad (1)$$

where

t = thickness of total pavement structure (in),
C = a measure of traffic called coverages,
P = equivalent single wheel load (lb),
CBR = a measure of soil strength or support capability, and
p = tire contact pressure (lb•f/in²).

The equation shown is not the latest one available but, as will be discussed later, it provided acceptable and more economical pavement sections than did the latest equation.

Subgrade Strength (CBR)

As would be expected along a number of different

Figure 2. Frequency histogram of in-place CBR values.

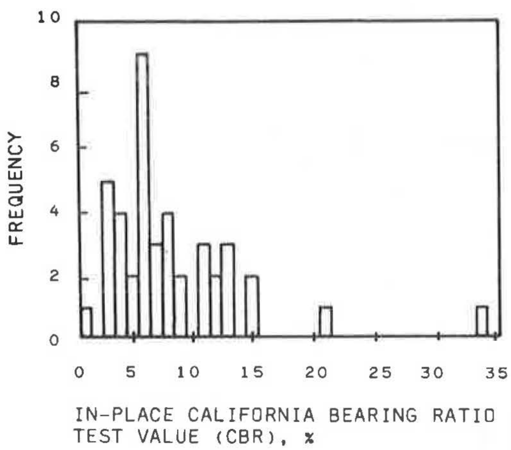
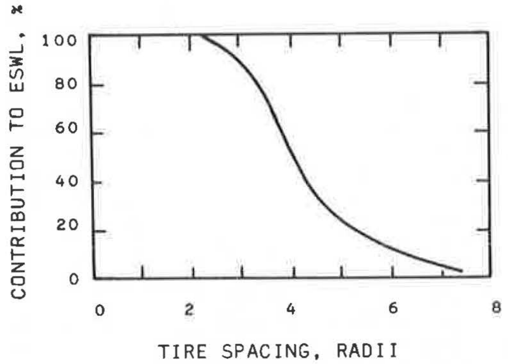


Figure 3. Contribution of individual wheel loads to ESWL.



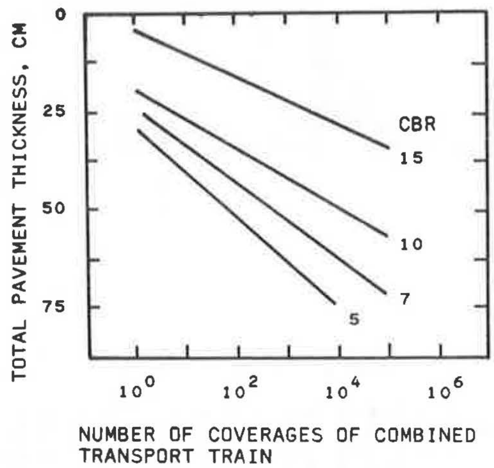
roadway pavement sections, particularly for the length in this project, a wide range of in-place CBR values were obtained. Figure 2 shows the frequency and range of CBR values.

Because of the wide range in the CBR values and that only one or two CBR values generally were available for each of the 26 sections of roadway, a design CBR value based on the data for the complete length of roadway was chosen for evaluation rather than trying to develop a design CBR value for each section. It was thought that the variation in CBR values for the whole route would be indicative of the variation in CBR values within any of the 26 sections of roadway. A CBR value for which 75 percent of the test values was equal to or greater than that value was chosen as the design value. The 75 percent figure is less conservative than the 85-90 percent figure normally used by most designers (6). However, I felt justified in using the less conservative CBR value because the entire route was to be proofrolled with one simulated axle line in order to detect exceptionally soft areas. Based on these criteria, a design CBR value of 5 was selected.

Equivalent Single Wheel Load

Equivalent single wheel loads (ESWLs) were calculated for the push-pull vehicles and the interior and edge wheels of the main trailers based on the tire loads, spacings, and pressures previously discussed and the influence chart in Figure 3 (1). The edge wheels of the main trailer were evaluated to

Figure 4. Coverages of transport train versus total pavement thickness.



determine shoulder requirements should the vehicle inadvertently be steered onto the shoulder. The calculated ESWLs are given in the table below (note: 1 lb•f = 4448 kN; 1 lb•f/in² = 2143 kPa).

Vehicle	Wheel	Actual Tire Load (lb•f)	Tire Pressure (lb•f/in²)	ESWL (lb•f)
Michigan push-pull	NA	24 000	34	25 325
Main trailer	Interior	7 890	124	19 830
Main trailer	Edge	7 890	124	18 720

The ESWL varies with depth and thickness profile of the pavement as well as with the criteria (equal surface deflections, strains, stresses or volumes of deflection basin) for the ESWL determination. However, because of the large number of greatly different pavement sections, the simplified procedures based on Figure 3 for determining an ESWL were considered reasonable, particularly when one considers that a doubling or tripling of the pavement thickness may only change the ESWL by 20-25 percent. This 20-25 percent change in ESWL will only change the thickness requirement for the pavement by approximately 10 percent.

Thickness Requirements Versus Coverages

Graphs of thickness versus allowable coverages of each type of ESWL were developed for the design CBR value of 5. Trial and error procedures then were used to determine the thicknesses necessary to allow various coverages of the transport assembly and prime movers. One pass of the transport assembly and prime movers included 24 coverages of the main trailer ESWL and 10 coverages (5 vehicles, 2 axles) of the Michigan ESWL. Figure 4 presents the required total pavement thickness versus coverages of the transport train for various CBR values.

REQUIRED PAVEMENT THICKNESS AND COMPOSITION

For new pavement construction (detours and widenings), county and private roads, and shoulders, the required pavement thickness was determined based on three passes (factor of safety = 1.5 for the two actual passes) of the transport trailer and prime movers. For state routes, it was assumed that pavement thicknesses capable of carrying many passes (10 or more, factor of safety = 5 for the two actual passes) of the transport trailers and prime moves were necessary because of the large volume of traffic they would have to carry once the moves of the RPVs were completed. The required total thicknesses determined for a CBR value of 5 are presented in the table below (note: 1 in = 2.54 cm).

Roadway Type	Required Total Pavement Thickness (in)	
	Traveled Way	Shoulder
County, private, detours, and widenings	14	14
State	16	14

The U.S. Army Corps of Engineers has updated Equation 1; however, an analysis of the project with this updated procedure resulted in pavement thicknesses 5-8 cm (2-3 in) greater than those for the older procedure when the number of passes of the combined transport train was less than 100. When the number of passes exceeded 100, the pavement thicknesses with updated procedure were less than those for the older procedure. If the updated procedure had been used the contractor would have had to expend money to build pavements thicker than necessary.

The U.S. Army Corps of Engineers procedure also yields the total thickness of the cover required over the subgrade regardless of the type of materials used as cover. Based on my practical experience, it was concluded that the cover could consist of well-compacted, well-graded crushed stone, provided the crushed stone was overlain by a minimum of 10 cm (4 in) of well-compacted bituminous concrete. This results in basic pavement thickness profiles of 10 cm of bituminous concrete over 25 cm (10 in) of crushed stone and 10 cm of bituminous concrete over 30 cm (12 in) of crushed stone for the two types of roadways. The relatively small difference in thickness is due to the fact that the required thickness is a function of the log of the traffic. The bituminous concrete was recommended in order to minimize slippage when the tractor wheels started from a dead stop. However, several temporary detour areas were crossed by the transport train, where the pavement consisted of only crushed stone and no problems were encountered.

For those pavements evaluated that had more than the minimum of 10 cm (4 in) of bituminous concrete, it was judged that the total thickness requirement (i.e., crushed stone thickness) could be reduced. Based on my practical experience, it was decided to count 2.5 cm (1 in) of bituminous concrete as 5 cm (2 in) of crushed stone when calculating equivalent pavement thickness. The required equivalent pavement thicknesses are presented in Table 1. The bituminous concrete to crushed stone conversion was not considered valid when the thickness of crushed stone was less than 15 cm (6 in).

It was recommended that all roads that had actual thicknesses or equivalent thicknesses less than those indicated in Table 1 be overlaid to bring them up to the necessary thickness requirements. This involved only the county and private roadways. The state routes generally had thicknesses far in excess of those deemed necessary. Shoulder thicknesses were less than required, and the contractor was cautioned not to steer the vehicle onto shoulders.

Table 1. Equivalent pavement thickness requirements when bituminous concrete thickness exceeds 4 in.

Required Total Pavement Thickness	Required Equivalent Pavement Thickness (in)	
	Bituminous Concrete	Crushed Stone
16 in—state roadways	4	12
	5	10
	6	8
	7	6
14 in—county, private, detours, and widenings and shoulders	4	10
	5	8
	6	6

Note: 1 in = 2.54 cm.

PROOFROLLING

In order to evaluate the analyses and recommendations from a practical standpoint and to identify potential localized problem areas, it was recommended to the contractor that the entire length of roadway be proofrolled with a simulation of at least one axle line. This was considered to be mandatory because of the uncertainties involved with the design analyses and the limited amount of data for the roadways involved. The entire route was proofrolled with a simulated axle line prior to any upgrading of roads. The visible deflection response of the pavements was visually observed and documented. The use of a single axle was justified because the analysis procedure indicated no significant interaction between the axles.

The deflections of the surface of the state routes generally were not observable with the unaided eye. The county roads consisted of only 2.5 cm (1 in) of asphalt and 7.5-10 cm (3-4 in) of crushed stone and often pumped and began to break up under just one pass of the one simulated axle. These observations were used to develop the final recommendations for the project.

FINAL THICKNESS RECOMMENDATIONS

The county roads were to be upgraded to 7.5 cm (3 in) of bituminous concrete and 25 cm (10 in) of stone. The reduction to 33 cm (13 in) of total pavement thickness reduced the theoretical number of passes of the transport assembly to approximately two. After the second move, 2.5 cm (1 in) of bituminous concrete were to be placed to smooth out any areas of county roads that might be slightly damaged by the two moves. The recommended thickness requirements for the state routes were left unchanged.

THICKNESS BY TENNESSEE DEPARTMENT OF TRANSPORTATION

As part of the permitting process, the Tennessee Department of Transportation (TDOT) performed its own testing at 15 locations. This testing consisted of the following:

1. Transverse elevation cross sections,
2. Benkelman beam deflections,
3. Determination of the in-place density of the asphalt pavement, and
4. Determination of the thicknesses of the asphalt and base course.

This testing was done prior to each move and items 1, 2, and 3 were repeated shortly after each transport move. TDOT also measured pavement surface temperatures during the move at several locations.

Benkelman Beam Deflections

An analysis of the TDOT reported data was performed to evaluate the effects of the move of the RPV. Of the 15 test locations selected by TDOT, all were

Figure 5. After-move Benkelman beam deflection versus before-move Benkelman beam deflections, second move.

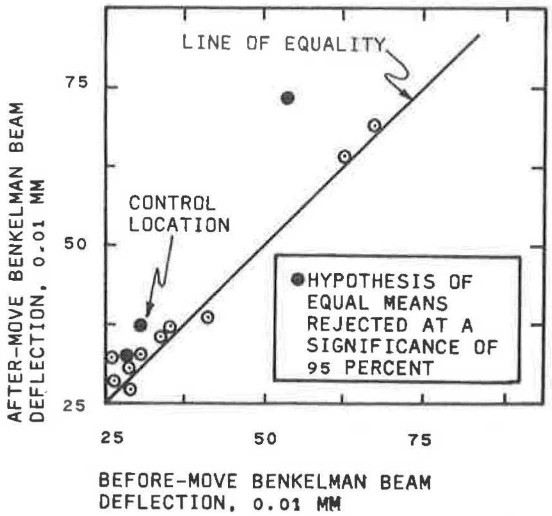


Table 2. Input parameters for ELSYM5 analysis.

Parameter	Case 1	Case 2	Case 3
Layer thickness (in)			
Bituminous concrete	4	4	3
Crushed stone	10	12	10
Subgrade	Infinite	Infinite	Infinite
Main trailer wheel loads			
Wheel load (lb·f)	7 890	7 890	7 890
Tire contact pressure (lb·f/in^2)	124	124	124
Tire contact radius (in)	4.50	4.50	4.50
Michigan tractor wheel loads			
Wheel load (lb·f)	24 500	24 500	24 500
Tire contact pressure (lb·f/in^2)	34	34	34
Tire contact radius (in)	15.14	15.14	15.14
Elastic modulus (lb·f/in^2)			
Bituminous concrete	100 000	100 000	100 000
Crushed stone	15 000	15 000	15 000
Subgrade	7 500	7 500	7 500
Poisson's ratio			
Bituminous concrete	0.35	0.35	0.35
Crushed stone	0.35	0.35	0.35
Subgrade	0.40	0.40	0.40

Note: 1 in = 2.54 cm; 1 lb·f = 4448 kN; 1 lb·f/in^2 = 2143 kPA.

traversed by the transport assembly except location 12. This location was used as a control section when it was learned that it would not be traversed. The surface deflection of a pavement is accepted as an indicator of the pavement's strength and integrity. Therefore, it was thought that if the transport move did any significant damage, then the effect would be to weaken the pavements, which would result in higher Benkelmen beam deflections.

A plot of the average after-2nd-move (A2M) Benkelman beam deflections versus the before-2nd-move (B2M) deflections at each of the 13 test locations for which temperature-corrected deflection data are available is shown in Figure 5. This plot shows that 11 of the 13 average A2M deflection readings were greater than the average B2M deflection readings. The average A2M deflection readings increased from only slightly to 0.18 mm (0.007 in) over the average B2M deflection readings. All but one of the locations exhibited increases in Benkelman beam deflections of less than 0.08 mm (0.003 in) and two locations actually exhibited a decrease in deflection. The average A2M deflection readings at location 12 (which was not traversed) increased by 0.05 mm (0.002 in) over the B2M deflection readings. The overall average increase was 0.03 mm (0.001 in) for all 13 locations.

In order to evaluate the B2M and A2M deflection readings, statistical procedures concerning hypotheses of equal means by using paired observations were used to analyze the data (7). The t-statistics for paired observations (individual B2M deflection paired with individual A2M deflection at each measurement location) were calculated for the 13 test sections. Of the 13 test locations analyzed, only 3 were found to have A2M average deflections that were statistically different from the B2M average deflections at a 99 percent confidence level. One of these 3 locations included the control section that was not traversed by the transport vehicle.

An analysis of the paired average B2M and average A2M deflections for all 13 test locations indicated that the hypothesis that the average A2M deflections were equal to the B2M deflections would not be rejected at a 99 percent level of significance. Thus, the second move did not appear to weaken the pavements at the test locations to the point where significantly greater deflections would be observed under the standard load.

Asphalt Density

With the exception of two locations, the average asphalt density generally increased from 16 to 32 kg/m^3 (1-2 lb/ft^3). The average B2M and A2M asphalt density data were analyzed on a paired basis similar to that for the deflection readings. The hypothesis of equal asphalt densities before and after the second move was rejected at a confidence level of 99 percent.

Transverse Cross Sections

The cross section data [surveyed to 3 mm (0.01 ft)] generally indicated changes in cross section elevation ranging up to 12 mm (0.04 ft). The changes in cross section readings indicated a general settlement rather than shear failure with resultant heave. The changes in cross section elevations are probably a result of consolidation in both the subgrade and the asphalt layers. Sufficient information is not available to allow an allocation of the settlement between consolidation of the asphalt and consolidation of the subgrade.

POST-MOVE ANALYSIS WITH LAYERED ELASTIC THEORY

As a follow-up evaluation after the completion of the second move, analyses were made of the recommended pavement thickness sections by using a layered elastic computer program. The program used was ELSYM5, and it was run by ARE, Inc. (8).

Selection of Input Parameters

An elastic modulus of 52 MPa (7500 lb·f/in^2) was chosen for the subgrade based on the widely accepted relation of subgrade modulus (lb·f/in^2) = 1500 x CBR and a design CBR of 5. For the crushed stone base course, an elastic modulus of 103 MPa (15 000 lb·f/in^2) was chosen. This value is slightly less than the 138-172 MPa (20 000-25 000 lb·f/in^2) that is recommended in some published correlations; however, it is considered reasonable (9,10). An elastic modulus of 690 MPa (100 000 lb·f/in^2) was used for the bituminous concrete. This value is based on a relation by Barker and others (9) and a pavement surface temperature in the range of 38°-49°C (100°-120°F). Similar values are reported by

Shook and Kallas (11). The TDOT reported that temperature of the pavement surface at selected locations during the moves ranged from 27° to 66°C (80° to 150°F); most of the values were in the 35°-49°C (95°-120°F) range. Due to the wide range in pavement sections and temperature conditions, the assumed elastic moduli are considered representative of an average condition during the moves.

Table 2 presents the three thickness cases analyzed and the input parameters for the main trailer calculations and the Michigan tractor calculations. Cases 1 and 2 represent the original thickness recommendations and case 3 represents the slightly reduced final thickness recommended for the upgrading of county roads.

Results of ELSYM5 Analysis

Figure 6 presents the typical results of computer program analysis in graphical form for a single-wheel load on the main trailer for case 1. The deflection of the pavement surface, the tensile strain at the bottom of the bituminous concrete (ε_t), and the vertical compressive strain on the top of the subgrade (ε_c) are presented. These two strains are generally recognized as the controlling strains in the thickness design of flexible pavements. To determine the maximum deflections and strains, the principle of superposition was used. The maximums for the main trailer were found to occur beneath the center point between either set of the center sets of dual wheels. The strains beneath one wheel of the Michigan tractor were basically unaffected by the other wheels on the tractor. Table 3 presents the maximum strains and deflections calculated for the combined wheel loads for the three cases analyzed.

As can be seen from Figure 6, the tensile strain at the bottom of the bituminous concrete (ε_t) and the vertical compressive strain on top of the subgrade (ε_c) dissipates to virtually nothing at a distance of 50-75 cm (20-30 in: 4.4 to 6.7 radii) from the center of the loaded areas for the transport trailer wheels. However, the deflection of the pavement surface is affected to a distance in excess of 254 cm (100 in; 22.2 radii) from the center of the loaded area. This indicates that the extent of effect of one wheel on the strains in the pavement is similar to that in Figure 3.

The maximum strains calculated and presented in Table 3 were compared with published correlations for strain and repetitions to failure. Figure 7 presents typical curves for ε_t versus number of strain applications to failure (12,13). Figure 8 presents typical curves for ε_c versus number of strain applications to failure (9,14). The theoretical number of repetitions for the combined transport train are presented in the table below for both the elastic analysis and the U.S. Army Corps of Engineers procedures.

	Case		
Combined Transport Train	1	2	3
No. of theoretical repetitions of transport train to failure based on ε_v (critical criteria)	7-15	10-28	3-5
No. of theoretical repetitions of transport train to failure based on Corps of Engineers' procedure	3	10	2

Figure 6. ELSYM5 strains and deflections, main trailer, case 1: H1 = 4 in and H2 = 10 in.

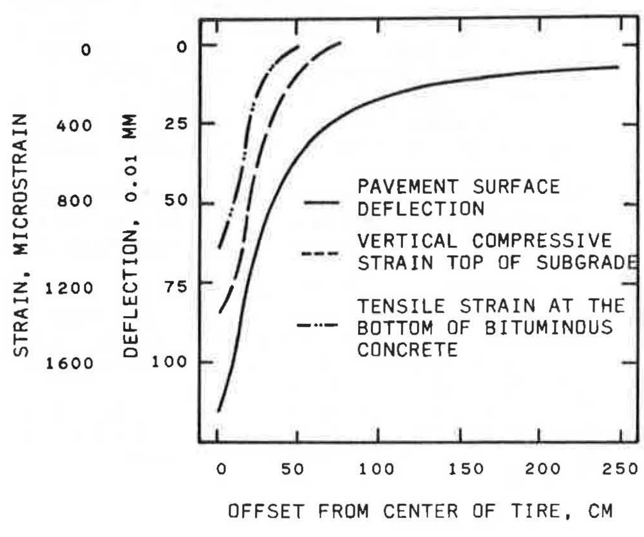


Table 3. Maximum strains and deflections for main trailer and Michigan tractor.

	Case		
Condition	1	2	3
Main trailer			
Maximum surface deflection (in)	0.170	0.169	0.178
Maximum tensile strain at bottom of bituminous concrete, microstrain	1400	1380	1560
Maximum vertical compressive strain on top of subgrade, microstrain	2280	2000	2880
Michigan tractor			
Maximum surface deflection (in)	0.112	0.110	0.117
Maximum tensile strain at bottom of bituminous concrete, microstrain	260	250	280
Maximum vertical compressive strain on top of subgrade, microstrain	2150	1930	2340

Note: 1 in = 2.54 cm.

Figure 7. Strain repetitions to failure versus tensile strain at bottom of bituminous concrete.

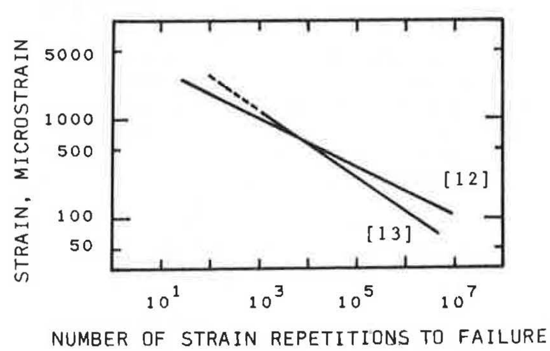
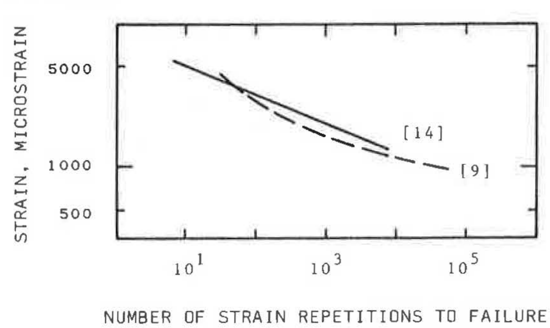


Figure 8. Strain repetitions to failure versus vertical compressive strain on top of subgrade.



There was also a reasonably good agreement between the CBR equation procedure and the strain procedure based on the elastic analysis, particularly when one considers that a slight error in the strain curves (log-log plot) can greatly affect the number of allowable strain repetitions.

SUMMARY

The movement of the RPVs can be considered a success in terms of the basic criteria established at the start of the project. The transport trailer did not become stuck nor did it experience any delays due to inadequate pavement sections on roadways I evaluated.

As for visible or measurable damage, the move of the RPVs did not appear to affect the roads significantly where TDOT performed tests. The increase in asphalt density could be construed to mean that the move actually improved the pavement structure since higher densities normally mean higher strength and stability for a given mix. The only detrimental effect of this increase in density would be if the density increased to the point where the surface was bleeding asphalt and reducing skid resistance. However, this bleeding has not been evident. The major visible damage done by the move occurred by inadvertent travel onto inadequate shoulder sections. Localized scarring of the pavement surface occurred due to slippage of the tires on the prime movers when they started from a dead stop.

An overlay patch on a road in an industrial park was peeled up during the first day of the first move. The area affected was approximately 46 m (150 ft) long. The overlay was only 2.5 cm (1 in) thick and the bond between the overlay and underlying layer was observed to be very poor due to water and dirt at the interface. There was no failure in the underlying asphalt layers. A patch of asphalt of apparently similar age several hundred meters down the road appeared to be unaffected by the move.

There was reasonably good agreement between the thickness requirements for the U.S. Army Corps of Engineers procedure (relatively simplistic) and the layered elastic theory (relatively sophisticated). However, the close agreement achieved in the subject case may have been fortuitous and may not occur in other cases. Also note that the CBR procedure and strain procedure were developed in different manners. It would appear that practicing engineers who do not have access to sophisticated computer programs can use the U.S. Army Corps of Engineers procedures (1) to evaluate heavy vehicle moves and the results can be expected to be reasonable.

This project also indicates that if the vehicle for heavy transports is properly designed in terms of numbers of wheels and axles then flexible pavements of moderate thickness can be used. Most high-traffic state and Interstate routes (flexible pavements) can probably handle a few repetitions of a properly designed transport vehicle without suffering significant damage.

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