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FINAL REPORT

PERMISSIBLE WEIGHT LIMITS FOR CONSTRUCTION VEHICLES ON FINISHED AND UNFINISHED PAVEMENTS

by

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(The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the sponsoring agencies.)

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SUMMARY

The report summarizes studies directed at developing guidelines for controlling the use of partially completed pavements by heavy construction equipment. It is shown that the damaging effects of flotation tires may be analyzed in the same manner as those of conventional dual tires, and that the AASHTO traffic equivalencies for such dual tires are applicable to the flotation tires. Tables of traffic equivalencies for various single-axle loads and the maximum permissible 18-kip (8,160-kg) equivalent repetitions for a variety of thickness indices are given as guidelines for field engineers. The development of specifications limiting the travel of construction equipment on various pavement layers was not considered to be an objective of the present study and is to be accomplished in an implementation phase.

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INTRODUCTION

In the construction of highways, the use of heavy vehicles by contractors has sometimes severely damaged the base and subbase layers. This damage has led to noticeable structural failures in semiconstructed pavements and to fatigue damage unobserved at the time of occurrence but which ultimately reduced the life of the road. On the other hand, denying the contractors the use of such vehicles on semiconstructed or fully constructed pavements increases their transportation and maintenance costs, and the increase in costs eventually is reflected in bid prices. It was thought necessary, therefore, to determine the limitations that should be placed on the weights of construction vehicles to be allowed on pavements and any restrictions needed on the numbers of trips such vehicles can make on the roadway.

PURPOSE AND SCOPE

Investigations, including the AASHTO Road Tests, have determined the impact of some construction vehicles on the pavement, but they have failed to generalize the conclusions reached or to suggest an approach to the design or evaluation of pavements that would incorporate consideration of the impact from the flotation tires used on construction vehicles.

The purpose of the present investigation was to remedy this deficiency and to develop mathematical solutions for determining the durability of pavements subjected to loadings of heavy construction vehicles. To achieve this purpose, the investigation was divided into the following tasks.

1. Compiling statistical information on tires from research reports and industrial sources to establish basic principles of the tire's involvement in the transmission and application of loads as a foundation for a mathematical analysis of the vehicle weights and load repetitions.

2. Carrying out field tests to validate the analytical approach taken.

3. Establishing methods for evaluating the techniques used in determining the influence of conventional tires on the durability of pavements, and looking into the suitability of using these techniques to determine the influence of flotation tires used on construction vehicles.

4. Equating construction vehicle loads to one standard such as the traffic equivalency factor (18-kip [8,160-kg] equivalent), or to a damage equivalency factor, to simplify the evaluation of the effects of the variable wheel loads of the construction equipment.

5. Developing guidelines for controlling the movement of heavy vehicles on finished and unfinished pavements.

ROLE OF THE TIRE IN TRANSMISSION AND APPLICATION OF LOADS

Types of Tires

Tire manufacturers classify tires as being: 1) conventional, or on-and off-the-road tires; and 2) off-the-road tires, commonly termed "flotation tires". Conventional tires are those normally used on trucks having single, dual, or tandem wheels and traveling at speeds of 50 to 70 mph (80 to 112 kph). These tires carry pressures ranging from 60 to 90 psi (413 to 620 k Pa). Off-the-road flotation tires usually carry low pressures of 30 to 55 psi (207 to 379 k Pa) to enable vehicles equipped with them to travel over construction sites providing poor ground support and at speeds ranging from 30 to 55 mph (48 to 88 kph). All flotation tires mentioned in this report as being used on construction vehicles are of this low-pressure type.

For both partially and fully constructed pavements the tire-related factors considered were wheel load, tire inflation pressure, transmitted tire pressure, and tire tread configuration.

For fully constructed pavements, the axle width as related to the lane width was considered. These factors are discussed below.

Wheel Load

For both conventional and flotation tires the heavier the wheel load, the greater the deflection, and the steeper the deflection basin imposing stresses and strains upon the pavement. Zube and Forsyth compared the effects of dual-wheel conventional tires (10.00 x 20.50 — 12 ply) against those of flotation tires (18.00 x 19.50 — 16 ply) carrying almost equal tire pressures.[1] Their investigation showed that the damage caused by a single-wheel carrying a 6-kip (2,720-kg) load and using a flotation tire equaled or exceeded the damage caused by dual-wheels with a 9-kip (4,080-kg) load on conventional tires.

Tire Pressure

In the discussion of the Zube and Forsyth study it was concluded that the pressure transmitted by flotation tires is probably greater than that transmitted by conventional tires, even when the inflation pressure of both tires is equal. The AASHTO Road Test, on two scrapers with low-pressure flotation tires, showed that the inflation pressure appeared to have little or no effect on the transmitted pressure.[2] That study showed that for flotation tires the tire-pavement contact area and the transmitted pressure increased as the load increased. The study, further, showed that, due to tread configuration, the actual contact area for low pressure tires on pavement surfaces approximates 30% to 40% of gross apparent contact area.[2] Hence, in mathematical analyses involving the use of flotation tires on completed pavements, contact areas should be assumed to be lower than would be the case for conventional tires. To equate with a 30% to 40% reduction in gross contact area, the assumed tire pressure will have to be two to three times the actual inflation pressure.

The field data obtained for flotation tires in this investigation, and discussed later in this report, also show that higher transmitted pressures should be assumed for the purpose of mathematical analyses involving flotation tires. Scala determined that the shape of the deflected basin changes with tire pressure, though the maximum deflection does not change,[3] and Freitag and Green have shown that with low tire pressures, the sidewalls of the tires play a large role in transmitting pressures.[4]

All these reported investigations and the field data obtained in the present investigation show that flotation tire pressures transmitted from the tire to the road should be assumed to be larger than the inflation pressure in the mathematical analysis if the results are to be correlated with field data. In the absence of data to the contrary, it is recommended that the transmitted pressure of low-pressure flotation tires be taken as 70 psi (482 k Pa). This transmitted pressure was chosen for the mathematical analyses because it is the standard used in the analysis of pavements in Virginia with 18-kip (8,160-kg) equivalent axle loads carried on conventional dual tires.

Wheel Configuration

Various axle and wheel configurations are used on vehicles to distribute loads over pavements and thus prevent excessive stresses, strains, and damage. In order to evaluate the overall effect of such configurations, an equivalent wheel load technique can be used to determine an equivalent single axle load for each configuration. This equivalent load would cause the same stresses, strains, and damage as the combined effect of all the wheels in the system. Various methods of determining the equivalent loads are available. One of the most convenient and popular methods employs computer programs such as that developed by the Chevron Corporation.[5]

Axle Width

The on-road configuration of the wheels of a vehicle and the weight limitations normally are defined by statute. In Virginia the maximum overall width of the vehicle is limited to 8 ft. (2.5 m) and no axle width (measured outside to outside of tire tread) is specified. The legal limit is exceeded under certain conditions through a special permit system. Depending on the number of axles, the axle width under special permit could vary from 8 to 12 ft. (2.5 to 3.7 m).[6] Manufacturers of vehicles try to keep within the limits of axle weights and loads to avoid the need for special permits for movement. For example, some truck cranes up to 35-ton (38.5-mt) capacity have a maximum overall vehicle width of 8 ft. (2.5 m) and single-axle equivalents of 20,000 lb. (9,066 kg). Larger truck cranes will normally require a special permit.

Off-the-road vehicles such as scrapers usually have widths larger than 8 ft. (2.5 m). Except for very small scrapers, the width is often about 12 ft. (3.7 m). Larger scrapers can be highly damaging to the pavement, because one wheel has to travel along the edge of the pavement or along the shoulder. The damage caused by such scrapers can be viewed as given in the following example.

In the case of a Caterpillar 641 scraper with a tire size of 37.5-39 and tire pressure of 55 psi (370 k Pa), the deflection, movements, and stresses at the edge of a 6 in. (15 cm) thick asphaltic concrete pavement, as read from influence charts by Pickett and Ray,[7] are about two and a half times the deflections, movements, and stresses in the center of the pavement. On the shoulder, the deflections could be enormously high, depending on the strength of the shoulder.

Investigations have shown that in Virginia 88% of the secondary roads and 18% of the primary roads have lane widths less than 10 ft. (3 m). Further, 96% of the secondary roads and 64% of the primary roads have lanes less than 12 ft. (3.7 m) wide.[8] Hence, the outer wheels of the wide vehicles usually will be on or near the edge of the pavement or on the shoulder. It would, therefore, appear advisable to consider pavement lane widths on the proposed route at the time permits are issued for the movement of scrapers.

In the case of unfinished pavements on construction sites, where lanes are not marked, the paths traveled by construction equipment are constantly changing such that the repetitive effect of wheel loads may not be directly cumulative. A good example of this is the work carried out by Sherman et al. which showed that on new cement-treated bases a limited number of heavy loads not exceeding 20,000 lb. (9,066 kg) per axle could be allowed.[9] They, however, recommended that such loads be restricted to the center portion of the pavement. This restriction could be adapted in Virginia.

EVALUATION OF FIELD DEFLECTION DATA

In the working plan for the present study it was proposed that Benkelman beam deflections taken on partially constructed pavements under construction traffic would be correlated with dynaflect deflections and results from a theoretical evaluation of those pavements. It was proposed that this work would be carried out for five to eight construction projects.

The execution of this part of the study became difficult because, in most cases, immediately after the earthwork was completed, the contractor would remove all the heavy equipment from the project. However, it was possible to collect data on some projects with the following equipment: a) a Caterpillar 621 scraper, b) a Caterpillar 631 scraper, c) a Ford 880 truck, and d) a Diamond Reo truck. It was not feasible to determine the actual axle loadings on the equipment as tested. However, since all the equipment seemed to have been loaded to design capacities, the weights could be obtained from the respective specification brochures.

In the case of scrapers and their tire configurations, the Benkelman beam could measure deflections no closer than about 18 in. (45 cm) from the center of the wheel. The Ford and Diamond Reo trucks were dual-tandem and a complete deflection basin for each was determined.

Table 1 gives the axle load, tire size, and tire pressure for the two scrapers and two construction trucks used for testing in this investigation. This table also shows the maximum Benkelman beam values, the dynaflect deflections, and deflection basin slopes determined from the field data. In this table, d_o is the maximum deflection obtained for trucks by placing the beam between the dual tires, and D_{18} is the deflection at 18 in. (45 cm) from the center of the flotation tires on the scraper. The slope is the change in deflection per in. (cm) width recorded by the Benkelman beam between 0 to 24 in. (0 to 60 cm)from the truck's tires and between 18 and 42 in. (45 to 105 cm) from the center of the scraper tire. The dynaflect deflections converted to equivalent Benkelman beam deflections for 18-kip (8,160-kg) axle loads at 0 and 18 in. (0 to 45 cm) and the corresponding slopes obtained from the dynaflect are also given in Table 1. Examples of the deflection data obtained for both the equipment and the dynaflect at serial numbers 1, 2, 5, and 8 are shown in Figures 1 through 4. The following deductions have been made from these data.

1. The data in Table 1 show that axle loads greater than 18 kip (8,160 kg) on a scraper produce higher deflections and steeper basins than obtained from dynaflect data equated to an 18-kip (8,160 kg) axle load. This occurs in spite of the low pressure (40 to 50 psi [276 to 345 K pa]) of the tires on the scraper. The magnitude of the deflections and the steepness of the deflection basins caused by the scraper loads were, also, somewhat higher than predicted from theoretical analyses for the actual tire pressures of 40 to 50 psi (276 to 345 K pa). However, theoretical analyses in which the same 26,000- to 48,000-lb. (11,800- to 21,700-K pa) axle loads were assumed to be carried on dual tires at 70 psi (480 K pa) pressures yielded results in good agreement with the field data. This

finding confirms that from other studies mentioned earlier that, due to the configuration of the tire tread, the effective pressure of flotation tires is somewhat higher than the inflation pressure.

2. For the Diamond Reo truck with tire pressures of 85 psi (586 K pa), a 14,000-lb. (6,350-kg) axle load caused very nearly the same magnitude of deflection as the dynaflect equated to an 18-kip (8,160-kg) axle load. The truck, however, caused a steeper basin than did the dynaflect. If a 70-psi (482-K pa) tire pressure was assumed for the 14,000-lb. (6,350-kg) axle load, the field data would be closer to the theoretical data. In the case of the Ford 880 truck with tire pressures of 80 psi (550 K pa), the 14,000-lb. (6,350-kg) axle load caused a lower deflection and a flatter deflection basin than the dynaflect equated to an 18-kip (8,160-kg) axle load. As a whole, however, results for the trucks equipped with tires carrying an 80-psi (550-K pa) pressure seemed to compare reasonably well with the dynaflect values.

It is concluded that when field data for low-pressure flotation tires are to be mathematically analyzed, high tire pressures should be assumed. The present technique of evaluating pavements assumes a wheel load of 18 kip (8,160 kg) at a transmitted pressure of 70 psi (482 K pa). In this investigation, a transmitted pressure of 70 psi (482 K pa) was adopted for flotation tires, irrespective of the actual inflation pressure.

In the following methods developed for determining the permissible weight limits for heavy construction vehicles, axles with flotation tires will be treated like any other axle with dual conventional tires at a 70-psi (482-K pa) tire pressure.

METHOD FOR DETERMINING PERMISSIBLE REPETITIONS OF LOADS FOR A CONSTRUCTION VEHICLE

Two basic methods applicable in determining permissible repetitions of loads for conventional vehicles were used in seeking a solution to the present problem. These two methods are 1) the assessment of pavement durability based on a loss in the AASHTO serviceability index, and 2) the assessment of pavement durability based on analyses of fatigue failure. The development of these methods and their resultant combining for application to heavy construction vehicle loads are described below.

Table 1

Deflection Data for Construction Equipment

Serial No.	Equipment: Type	Equipment: Axle Load, lb.	Equipment: Tire Pressure, psi	Benkelman Beam: d_o, in.	Benkelman Beam: d_{18}, in.	Benkelman Beam: Slope per in., 10^{-4} in.	Dynaflect: d_o, in.	Dynaflect: d_{18}, in.	Dynaflect: Slope per in., 10^{-4} in.	Pavement Type
1	Scraper — Cat 621	48,000	45	—	.030	10.8	—	.0145	3.6	Subgrade
2	Scraper — Cat 613	26,000	50	—	.021	4.7	—	.015	3.6	A. C. + Agg. + Soil Cement
3				—	.020	5.7	—	.016	4.9	A. C. + Agg. + Soil Cement
4				—	.014	4.6	—	.014	3.3	A. C. + Agg. + Soil Cement
	Average				.018	5.0		.015	3.9	
5	Truck — Ford 880	14,000	80	.0215	—	5.0	.036	—	6.3	A. C. + Agg. + Soil Cement
6				.027	—	5.0	.048	—	9.2	A. C. + Agg. + Soil Cement
7				.055	—	7.5	.066	—	15.0	A. C. + Agg. + Soil Cement
	Average			.035		5.8	.050		9.8	
8	Truck — Diamond Reo	14,000	85	.035	—	7.5	.04	—	7.0	A. C. + Agg. + Soil Cement
9				.028	—	7.5	.033	—	5.8	A. C. + Agg. + Soil Cement
10				.045	—	10.0	.039	—	7.5	A. C. + Agg. + Soil Cement
	Average			.036		8.3	.037		6.8	

Note: d_o = Maximum deflection.

d_{18} = Deflection at 18 in.

Slope = Deflection per in. width for 0 to 24 in. or 18 to 42 in.

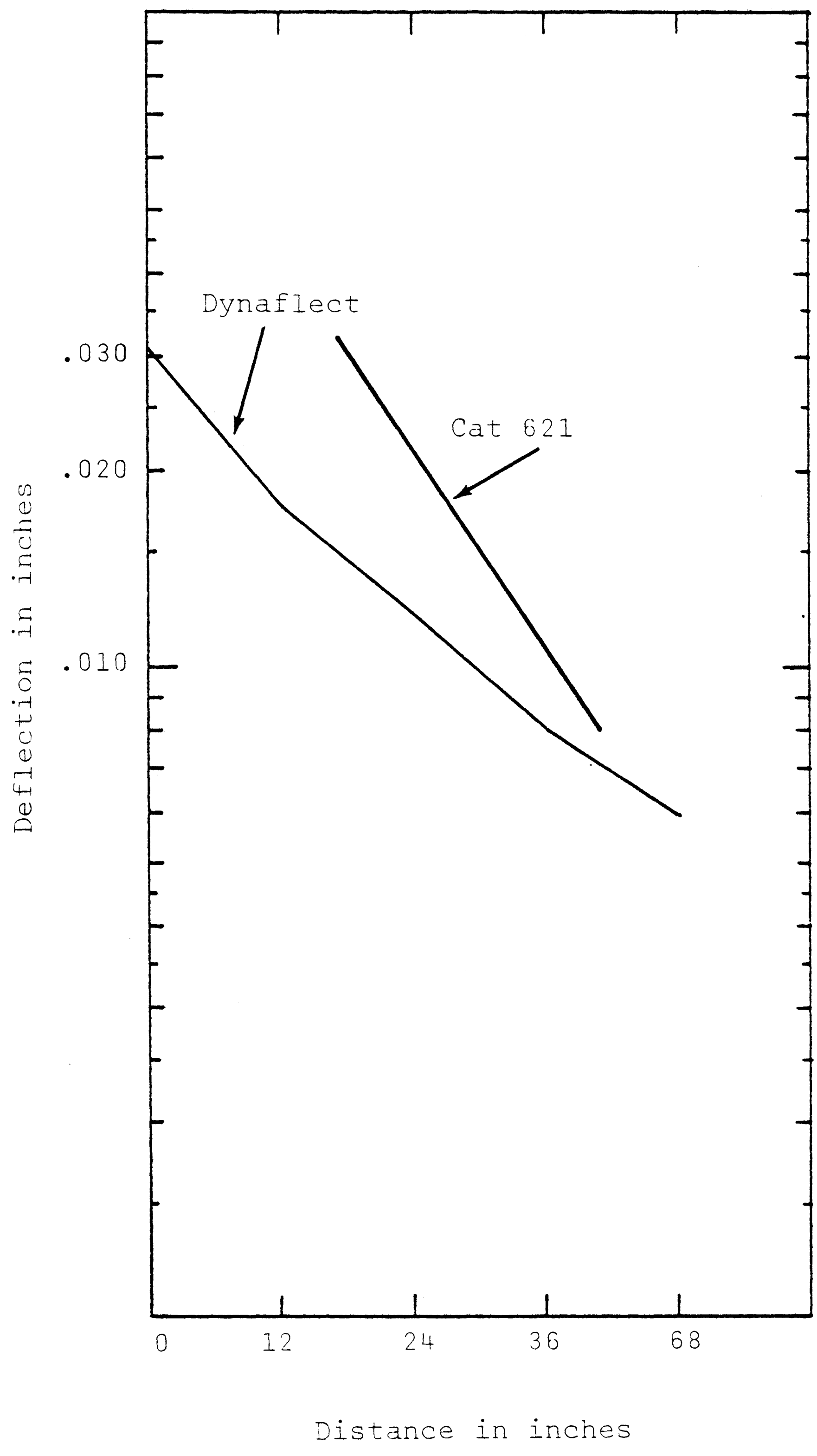


Figure 1. Deflection data for caterpillar 621 scraper on subgrade.
Project: 0254-007.
Conversion factor 1 in. = 2.54 cm.

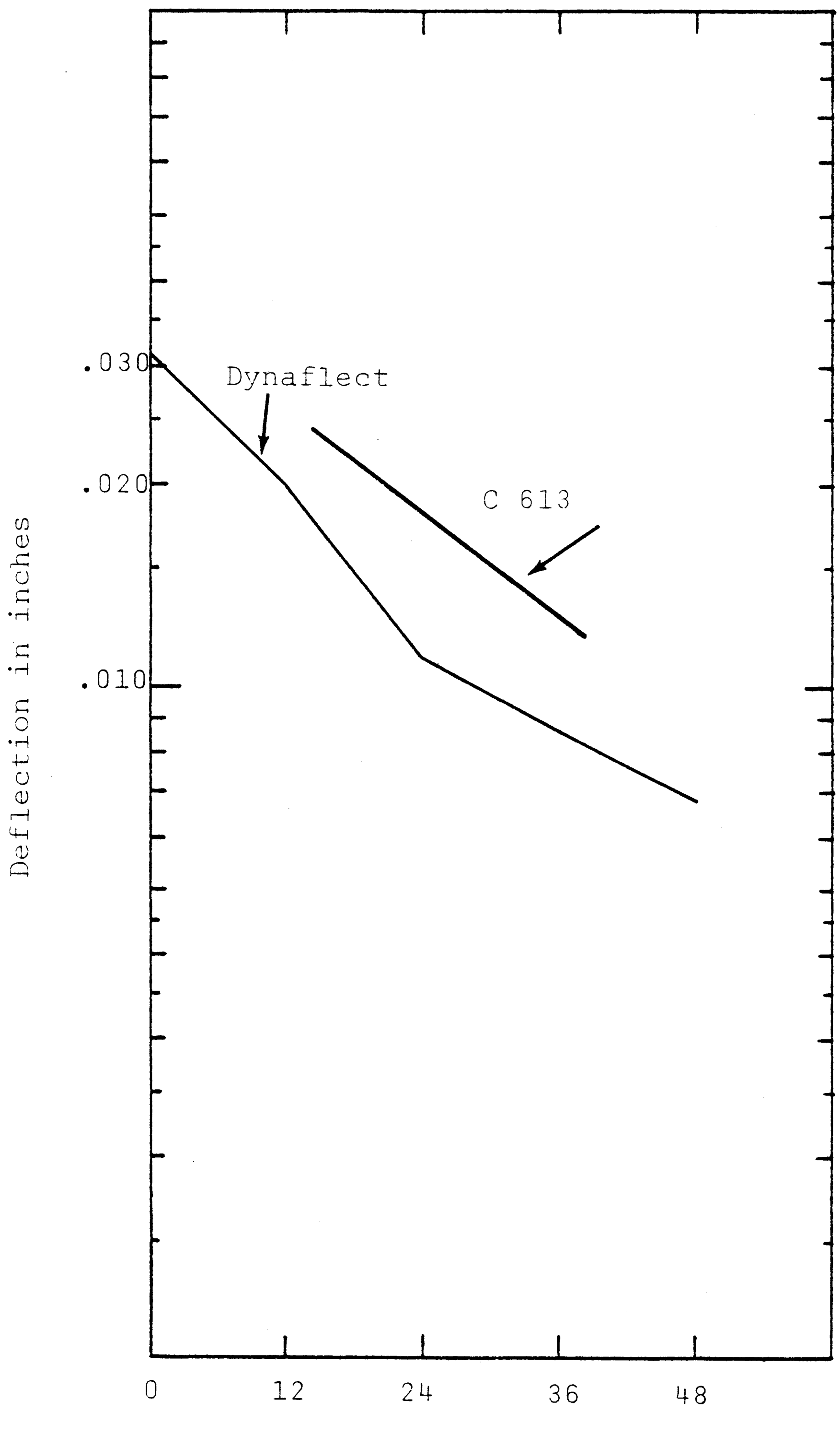


Figure 2. Deflection data for caterpillar 613 on asphaltic concrete layer. Project: 7460-073.
Conversion factor 1 in. = 2.54 cm.

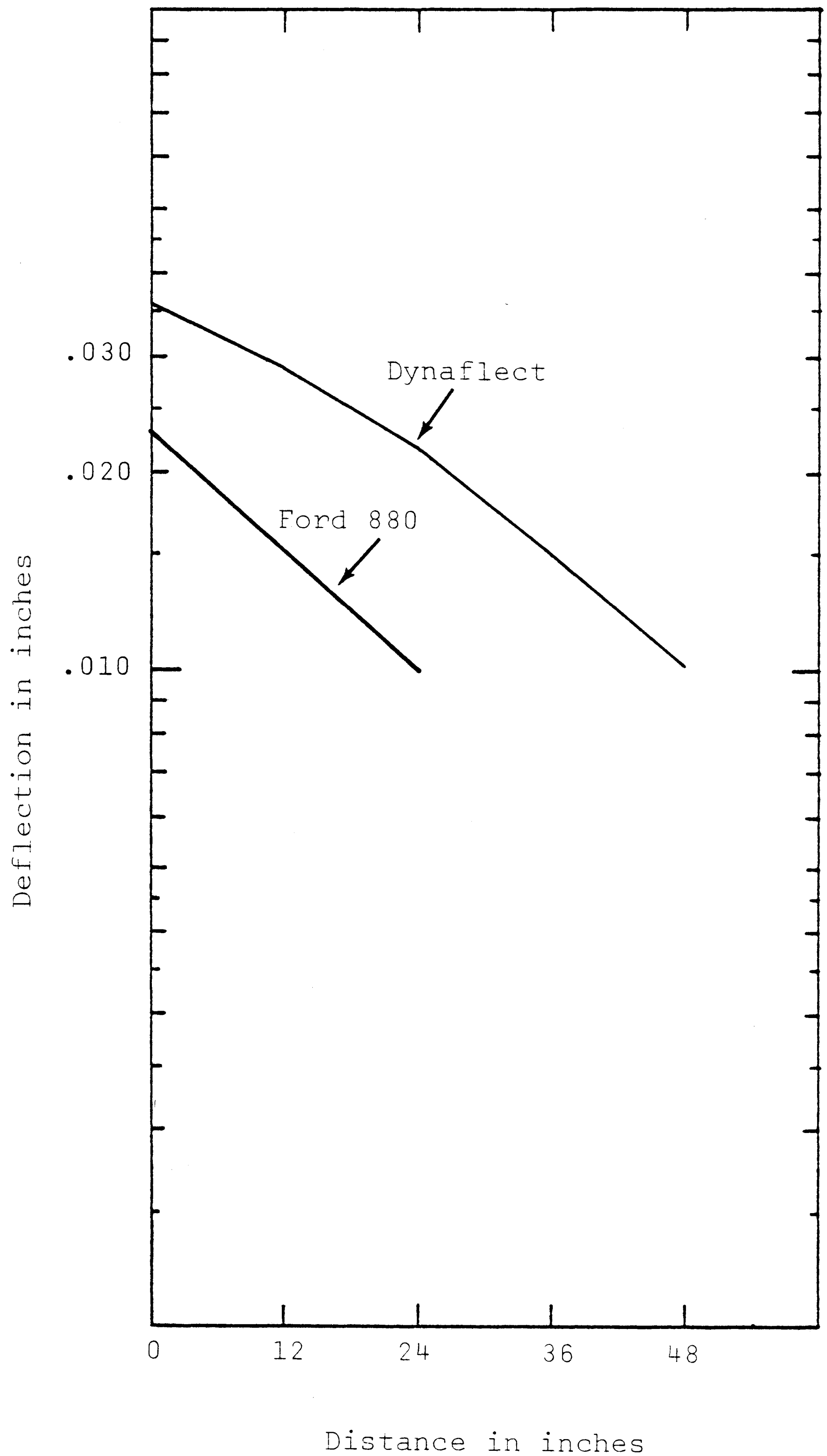


Figure 3. Deflection data for Ford 880 truck on 6 in. of stone over 6 in. of soil cement. Project: 171-147. Conversion factor 1 in. = 2.54 cm.

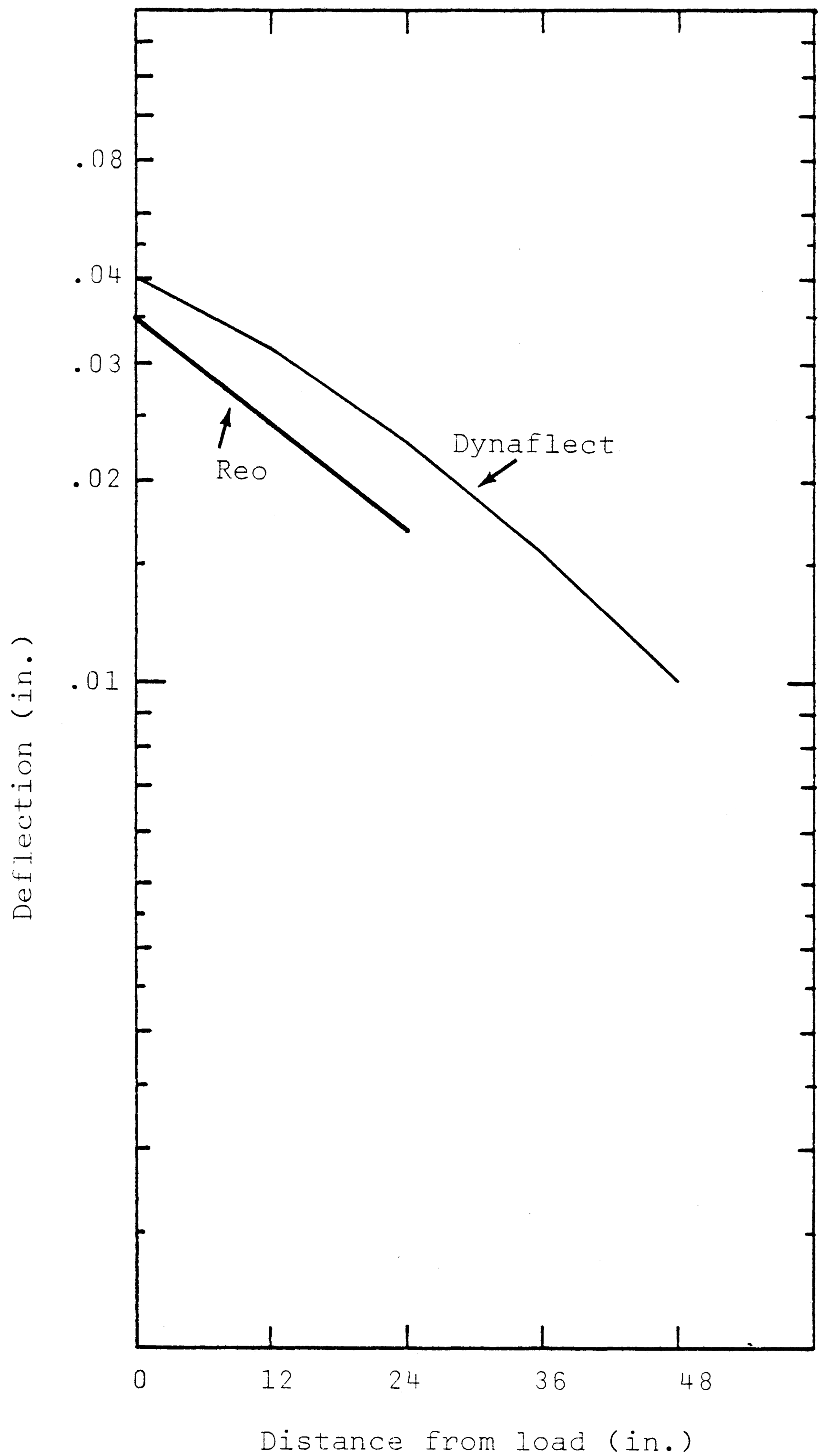


Figure 4. Deflection data for Diamond Reo truck on asphaltic concrete layer. Project: 171-147.
Conversion factor 1 in. = 2.54 cm.

Permissible Repetitions Based on Loss in AASHTO Serviceability Index

The loss in serviceability index of a pavement depends on 1) the structural strength of the pavement, hereinafter described in terms of the thickness index of the pavement, D;[10] 2) the 18-kip (8,160-kg) equivalent axle load capacity of the pavement; and 3) the number of repeated applications of the load, N. Thus, N_{18} denotes the number of applications of an 18-kip (8,160-kg) equivalent axle load.

A relationship between the serviceability index and the above three factors has been developed by the authors from AASHTO Road Test data and is shown in Figure 5.[11,12] The loss in serviceability as a function of cumulative axle loads can be determined from this figure.

Repetitive loads cause repetitive strains and, ultimately, cracking at the bottom of the top layer of the pavement. Cracks progress upwards toward the surface under additional load repetitions. AASHTO, as a result of the analysis of their road test data, developed equation (1) relating pavement strength to the cumulative 18-kip (8,160-kg) load applications needed to develop class 2 cracks.[12] Class 2 cracks are defined by AASHTO as those which are visible at a distance of 15 ft. (4.5 cm) and are less than 1/4 in. (0.6 cm) wide. In the AASHTO equation

$$\log N_{18} = 5.484 + 7.275 \log (0.33\, h_1 + 0.10\, h_2 + 0.08\, h_3 + 1)$$
$$+ 2.947 \log L_2 - 3.136 \log (L_1 + L_2), \; \ldots\ldots\ldots \; (1)$$

where

N_{18} = Number of 18-kip (8,160-kg) single-axle equivalent loads, termed simply the cumulative 18-kip (8,160-kg) equivalent;

h_1, h_2, and h_3 = the thicknesses, respectively, of the surface, base, and subbase in in. (2.54 cm);

L_1 = Nominal axle load = 18 for an 18-kip (8,160-kg) single-axle load or = 32 for 32-kip (14,500-kg) tandem-axle load; and

L_2 = 1 for single-axle configuration and = 2 for tandem-axle configuration.

From the AASHTO Road Test Results

$$D = 0.44\ h_1 + 0.14\ h_2 + 0.13\ h_3. \qquad (2)$$

Hence, in equation (1), $0.33\ h_1 + 0.10\ h_2 + 0.08\ h_3 = 0.75\ D.$[12]

For an 18-kip (8,160-kg) single-axle load, $L_1 = 18$ and $L_2 = 1$, and equation 1 reduces to

$$\text{Log } N_{18} = 1.474 + 7.275 \log (0.75\ D + 1). \qquad (3)$$

The empirical equations (1) and (3) cover a wide range of values of each variable. The values of L varied from a minimum of a 2-kip (906-kg) single-axle load or 24-kip (10,900-kg) tandem-axle load to a maximum of a 30-kip (13,600-kg) single-axle load or 48-kip (21,800-kg) tandem-axle load.[12] The tire size varied from a minimum of 6.7 x 15 in. (17 x 38 cm) to a maximum of 12 x 24 in. (30.5 x 61 cm). None of these tires were flotation or wide based tires, but conventional tires commonly used as on-road tires. The inflation pressure varied from 24 psi (166 K pa) for a 6.7 x 15 in. (17 x 38 cm) tire to 80 psi (550 K pa) for a 12.0 x 24 in. (30.5 x 61 cm) tire. The asphaltic concrete thickness varied from 1 to 6 in. (2.5 to 15 cm), the stone base thickness from 0 to 9 in. (0 to 23 cm), and the subbase thickness from 0 to 16 in. (0 to 41 cm). Thus, it is evident that equations (1) and (3) could be applied to most conceivable combinations of conventional tires and pavement strengths.

Based on equation (3), Figure 6 has been drawn to correlate the AASHTO pavement thickness index (D) with N_{18}, the number of load repetitions that cause class 2 cracks.

To develop a relationship compatible with the Virginia pavement design approach, the Virginia thickness index of asphaltic concrete is taken as 1 as compared to 0.44 in the AASHTO Road Tests. Based on this ratio, Figure 7 has been drawn to correlate the Virginia thickness index with the cumulative 18-kip (8,160-kg) loads necessary to cause class 2 cracks.

The permissible limit of loading of pavements by construction equipment should be less than that projected to initiate class 2 cracks. To determine the loss in serviceability when class 2 cracks develop, the values of AASHTO D versus the cumulative 18-kip (18,160-kg) loads needed to initiate class 2 cracking (Figure 6) were superimposed on the curves in Figure 5 to correlate the AASHTO D with the cumulative 18-kip (8,160-kg) loads and the AASHTO serviceability index. This superimposition is shown in Figure 8.

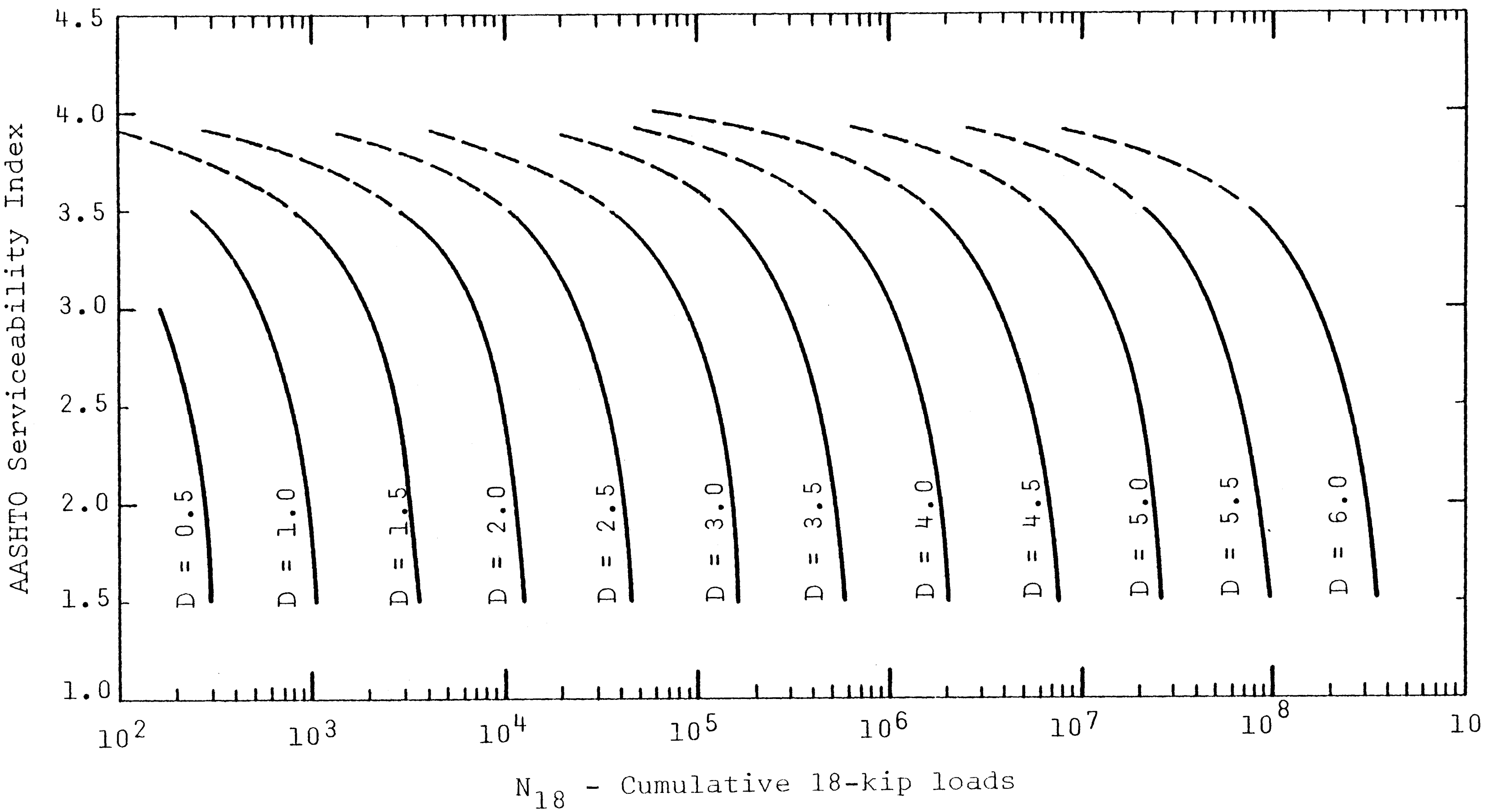


Figure 5. Relationship between cumulative traffic and AASHTO serviceability index for pavements having varying thickness indices. (Conversion factor: 18 kip = 8,160 kg)

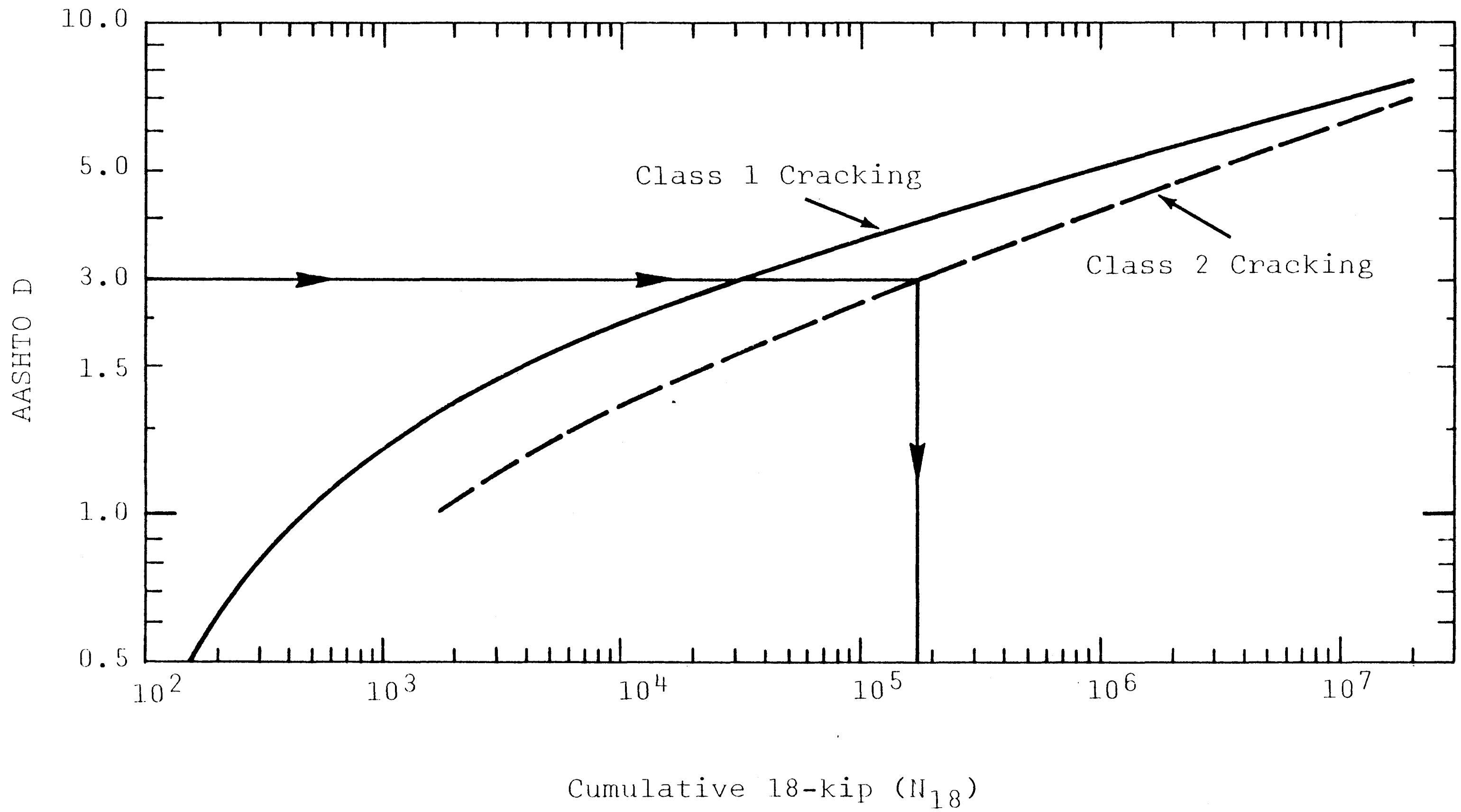


Figure 6. Fatigue curves for class 2 cracks correlating the AASHTO D with N_{18}.

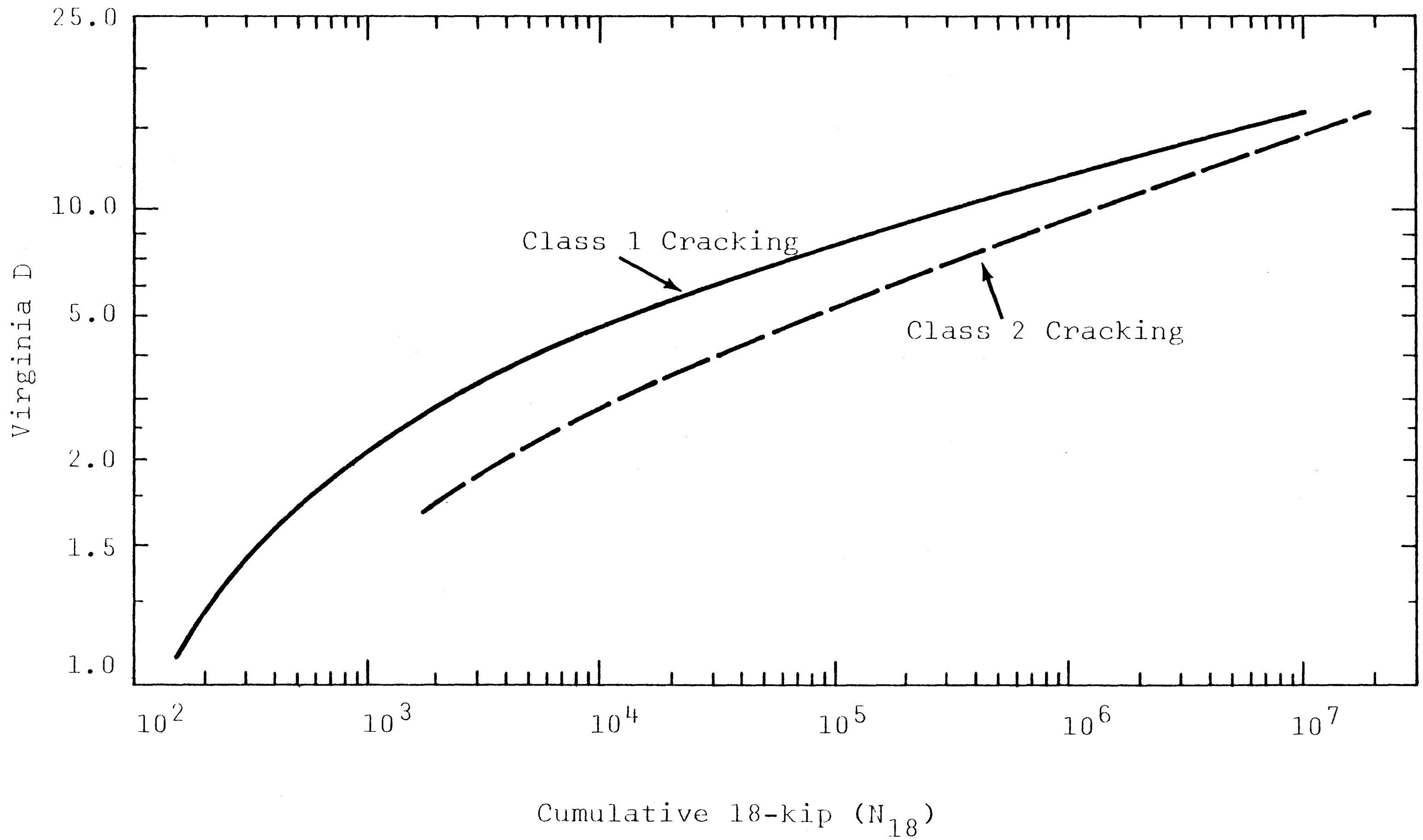


Figure 7. Fatigue curves for class 2 cracks correlating the Virginia D with N_{18}.

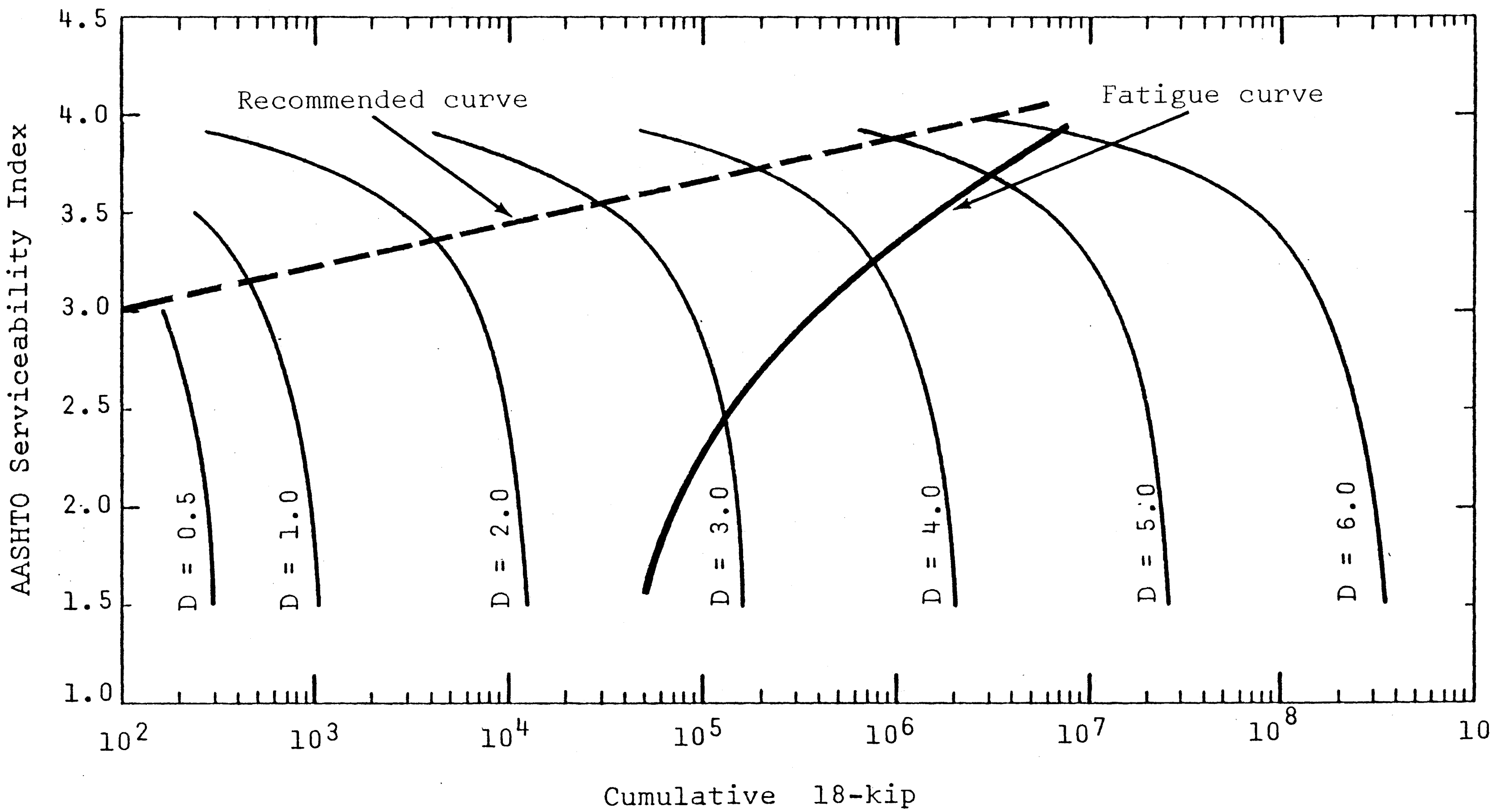


Figure 8. Curves for class 2 and class 1 cracks superimposed on the AASHTO serviceability curve.
Conversion factor: 18-kip = 8,160 kg.

Figure 8 shows that pavements with high strengths (high D values) retain high serviceability indices, even though they are considered to have failed due to class 2 cracks, while pavements with lower strengths have completely failed from the serviceability index point of view before they develop class 2 cracks. In other words, the development of class 2 cracks in a low strength pavement is preceded or accompanied by complete failure of the pavement.

It is, therefore, necessary that the allowable repetitions of 18-kip (8,160-kg) loads be less than the number which causes a heavy loss in the serviceability index or results in class 2 cracks. The maximum permissible limit in cracking should be limited to class 1 cracks, which have been defined by AASHTO as fine cracks not visible under dry surface conditions to a person with good vision standing at a distance of 15 ft. (5 m).

The AASHTO Road Test has shown that for new pavements the reduction in the serviceability index at the time class 2 cracks develop is from 1 to 2.6 units.[12] To prevent heavy construction vehicles from causing class 1 cracks, a reduction in the serviceability index of not more than 0.2 for high type pavements and not more than 0.5 for low type pavements is recommended; and in no case (including pavements in use) should the serviceability index be less than 3.0. A curve recommended to satisfy this requirement is shown in Figures 6 and 8 superimposed on the pavement serviceability curves for various AASHTO thickness indices and cumulative 18-kip (8,160-kg) equivalents. The same curve has also been drawn in Figure 7 to correlate the Virginia D with the maximum permissible cumulative 18-kip (8,160-kg) equivalent for heavy construction equipment.

Thus, if the traffic equivalency (18-kip [8,160-kg] equivalent) of a construction vehicle is known, the number of permissible loadings by that vehicle can be determined. The method of determining the traffic equivalency of a construction vehicle is described later in this report. An example of the use of Figures 6 and 7 to establish the maximum permissible loading is given below.

Example — It is necessary to move premix concrete trucks with a 34,000-lb. (15,400-kg) loading on a tandem rear axle and 12,000-lb. (5,440-kg) load on a single front axle over a 6-in. (150-mm) soil cement subgrade. The traffic equivalency for a 34,000-lb. (15,400-kg) tandem axle load from the AASHTO traffic equivalency values is 1.11, and that for the 12,000-lb. (5,440-kg) front axle is 0.2.[13] Thus, the total 18-kip (8,160-kg) equivalent for each truck is 1.11 + 0.2 = 1.31. The Virginia thickness index for 6 in. (150 mm) of soil cement is 2.4.[10] Hence, from Figure 7 it is found that the soil cement subgrade can carry 500 premix trucks in a specified lane before it develops class 1 cracks. On four-lane divided highways traffic could be permitted in the inner lanes up to the maximum number of trucks for class 2 cracks, which in this case would be 2,000 trucks. The reason for allowing more construction traffic on the inner lanes is that after the road is built, only a small percentage of trucks would use these lanes.

This method can also be used for temporary roads which are called upon to carry very heavy traffic for the first few years with a considerable reduction in traffic later.

Permissible Repetition Based on Fatigue Failure Techniques

The durability equation based on fatigue is

$$\log N = A + B \log R, \qquad (4)$$

where

N is the number of repetitions of a given axle load; and

R = critical elastic response, which could be in terms of elastic stresses or strains at the bottom of the top pavement layer; and A and B = constants.

The fatigue limits, or the number of repetitions at which the pavement is considered to have failed, are determined from the characteristics of the paving materials. The fatigue limits of materials commonly used in pavement construction are discussed below.

Asphaltic Concrete and Other Stabilized Layers

Cement or lime stabilized layers with a low Poisson's ratio and which fail by rupture may be evaluated in terms of the modulus of rupture as a criterion for determining fatigue life. However, determining the modulus of rupture would be an expensive and time-consuming process and would be unacceptable to the resident or materials engineers who may be called upon to recommend a permissible limit of load repetitions for a construction vehicle. An alternative, therefore, is to depend upon the radial strains at the bottom of the treated layer. Such a procedure also is applicable to the asphaltic concrete layer in the pavement system.

Treybig et al. developed an equation by applying AASHTO equation (1) to 27 AASHTO Road Test sections.[14] Their equation reduces to

$$N = 9.7255 \times 10^{-15} \left(\frac{1}{e_{rl}}\right)^{5.16}, \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

which may be written as

$$\text{Log } N = 5.16 \log \left(\frac{1}{e_{rl}}\right) - 14.012, \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

where

N is the number of load repetitions sustained by a pavement before the appearance of class 2 cracks; and

e_{rl} = transverse strain at the bottom of the top layer.

The relationship represented by the above equation is graphed in Figure 9 and typical values are given in Table 2.

Another type of fatigue-related pavement failure is permanent deformation of the subgrade. So far no suitable fatigue relationship based on permanent deformation of the subgrade has been supported by field data. Two equations of subgrade fatigue are given by the TRB Task Force on Fatigue Failure[15] and by Shell Oil.[16] Both equations have been graphed in Figure 9.

As can be seen in Figure 9, the TRB Task Force recommendation would provide a design criterion very similar to that given in equation (6). The fatigue relationship recommended by Shell precludes permanent deformation and, as can be seen in Figure 9, the relationship permits significantly higher strains than either equation (6) or the TRB recommendation. To design against fatigue failures one could assume that the allowable compressive strain values lie somewhere between the subgrade strain values recommended by Shell and the radial strain values given by equation (6). A relationship lying between the two limiting relationships has been graphed in Figure 9 and given in Table 2. This latter relationship for subgrade failure is

$$N = 5.16 \log \frac{1}{e_{z2}} - 12.5, \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

where N is defined in equation (6) and e_{z2} is the permissible compressive strain at the top of the subgrade. This relationship may be used in limiting load repetitions to asphaltic concrete or stabilized layers lying directly on àn unstabilized subgrade.

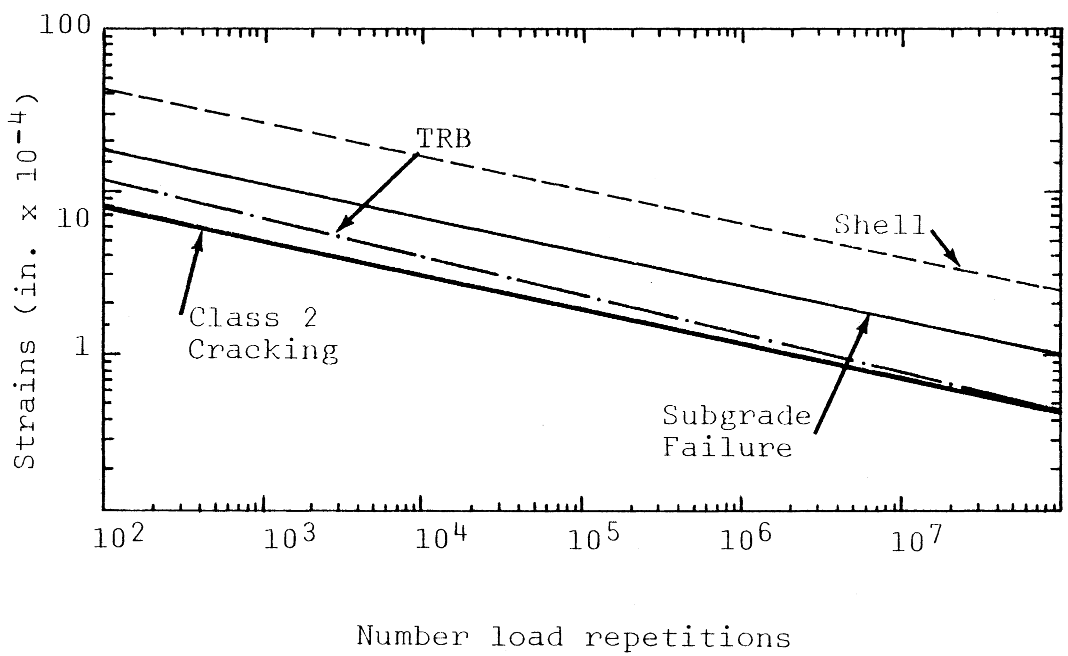


Figure 9. Comparison of fatigue curves for subgrade and bottom of top layer.
Conversion factor 1 in. = 2.54 cm.

Table 2

Recommended Design Values
for Fatigue Failure

Cumulative 18 Kip	Strains	
	e_{z2}	e_{r1}
10	0.00240	0.00005
100	0.00158	0.00079
1,000	0.00100	0.00050
10,000	0.00064	0.00032
100,000	0.00042	0.00021
1,000,000	0.00026	0.00013
10,000,000	0.00016	0.00008
100,000,000	0.00010	0.00005

Untreated Granular Layers

Often an untreated granular layer resting directly on an unstabilized subgrade fails due to excessive permanent deformation of the subgrade. In such cases the same criterion (equation 7) as given for asphaltic concrete layers on an untreated subgrade can be used to limit load repetitions.

Untreated granular materials overlying stabilized granular materials or soil cement subgrades typically fail due to failure of the stabilized layer or by rutting and shoving of the granular layer itself. Criteria for failures of the latter type have not been developed to the authors' knowledge.

Two approaches to estimating the permissible cumulative 18-kip (8,160-kg) equivalent axle loadings for untreated granular materials overlying a stabilized layer are discussed below.

The pavement may be taken as a two-layer system in which a) the top layer is an untreated granular material and the underlying layer is composed of a stabilized material and a raw subgrade of semi-infinite depth; or b) the top layer is composed of the untreated aggregate over stabilized material, while the bottom layer is a raw subgrade of semi-infinite depth. If one can then determine e_{z2} for the top of the underlying layer for both a) and b) and e_{r1} for the bottom of the top layer for case b), the minimum value of the cumulative 18-kip (8,160-kg) equivalent axle loading determined to first induce failure can be considered the maximum allowable for the pavement system.

Method for Determining e_{z2} and e_{r1}

For the determination of e_{z2} at the top of the subgrade or e_{r1} at the bottom of the top layer, the following information is needed.

1. Axle load or wheel load. The design wheel load of the vehicle must be used if the actual load is not known.

2. Tire pressure. Based on the investigations carried out by others and on the evaluation of the field data in the present study, tires with low inflation pressures should be assumed to have a tire pressure of 70 psi (482 k Pa). For tires with inflation pressures greater than 70 psi (482 k Pa) the actual tire pressure should be used.

3. Elastic properties of the pavement materials. The elastic properties of the materials in the pavement can be assumed based on some knowledge of the materials, or they can be determined. In Virginia, the elastic properties of the materials in the pavement system may be approximated as given in Table 3.

The elastic properties of the materials in a given pavement also may be determined from dynaflect data taken on it. Three methods of determining the moduli of the materials based on two-layer elastic theory have been developed by Vaswani.[18,19,20] One method, based on the ratios of deflections in the deflected basin, gives the modulus of the top layer and the average modulus of the underlying layers.[20] This method is considered applicable to the present investigation and is summarized as follows.

a. From the dynaflect data of a given pavement determine d_{max},

$\frac{d_1}{d_{max}}$, $\frac{d_2}{d_{max}}$ and Spreadability, where

d_{max} = equivalent Benkelman beam deflection for 9,000-lb. (8,160-kg) wheel load and is obtained by multiplying 28.6 by the maximum dynaflect deflection in in. (2.54 cm);

d_1/d_{max} = ratio of the dynaflect deflection at 12 in. (30.5 cm) from the load center to the maximum dynaflect deflection;

d_2/d_{max} = ratio of the dynaflect deflection at 24 in. (61 cm) from the load center to the maximum dynaflect deflection; and

Spreadability = (average of the sum of the five dynaflect deflections obtained in the deflected basin ÷ maximum dynaflect deflection) x 100.

b. Determine the average value of Ep/Es from Figures 10, 11, and 12, where Ep is the pavement modulus and Es is the subgrade modulus.

c. For the given wheel load and tire pressure, p, compute the radius, a, of the circular contact area.

d. Determine deflection coefficient F_w from Figure 13.

e. Determine Es from the equation given in Figure 13.

f. Determine Ep from the average value of Ep/Es.

g. Feed the data so obtained into the Chevron program or a similar program for a two-layer system, and determine e_{z2} and e_{r1} for any given wheel load.

The values of Poisson's ratio to be used in the computer program should be as follows:

For asphaltic concrete — 0.04 to 0.47

For materials treated with cement or lime — 0.13 to 0.17

For untreated aggregates — 0.45 to 0.5

For subgrade soil — 0.5

To enable field personnel to avoid the use of computer programs, Figures 14 and 15 have been developed so that approximate values of e_{z2} and e_{r1} can be obtained for use in evaluating pavements for heavy construction vehicles.

h. Given e_{z2} or e_{r1}, one can use Figure 9 to determine the number of load repetitions by a given vehicle that cause class 2 cracking.

The above two methods of determining permissible repetitions of a given axle weight of a construction vehicle could be used independently. The first method, based on the loss in serviceability index, could be used if the permissible loss in serviceability is known. The permissible loss in serviceability in this investigation was based on the development of class 1 cracks for the traffic lane and on the development of class 2 cracks for the other lanes. The second method, based on fatigue, limits load repetitions to fewer than the number that cause the development of class 2 cracks.

Table 3

Approximate Elastic Properties of Pavement Materials

Material	Elastic Modulus, psi	Basis
1. Asphaltic concrete (S-5 or B-3)	300,000 to 400,000 at 70°F (or 100,000 to 1,000,000 depending on temperature)	Laboratory investigation[a]
2. Cement treated subgrade	100,000	Field investigation[a]
3. Cement treated aggregate	150,000	Field investigation[b]
4. Untreated aggregate	50,000-90,000	Field investigation[b]

[a]From reference 17.

[b]From reference 18.

CAUTION: a. Do not use for $\frac{d_1}{d_{max}}$ greater than 80.

b. Do not extrapolate.

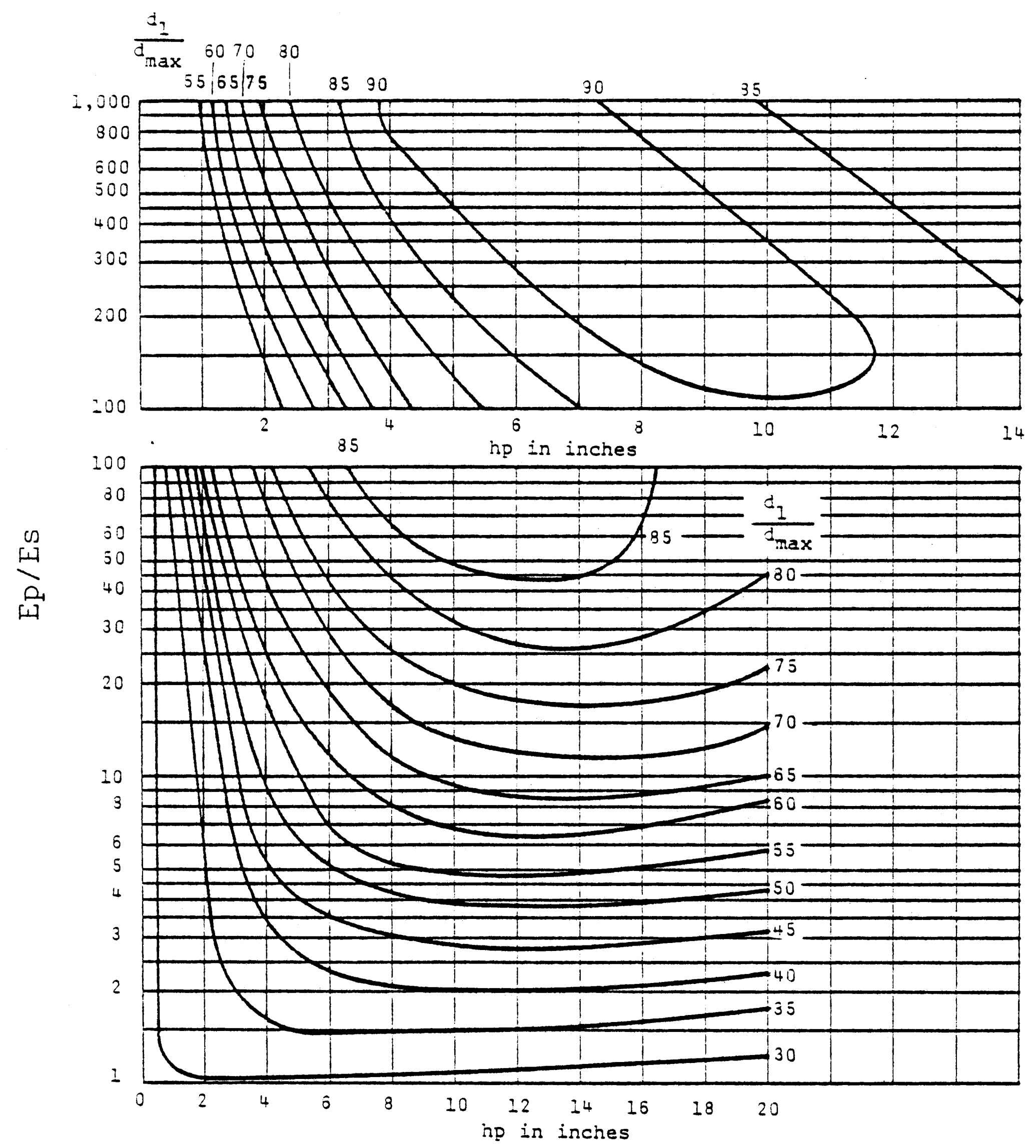


Figure 10. $\frac{d_1}{d_{max}}$ versus $\frac{Ep}{Es}$ versus hp.

(Wheel load = 9,000 lb; tire pressure = 70 psi; tire contact radius = 6.4 in.)

Conversion factors: 1 in. = 2.54 cm

1 lb. = 0.45 kg

1 psi = 0.69 kN/m^2

CAUTION: a. Do not use for $\frac{d_2}{d_{max}}$ greater than 80.

b. Do not extrapolate.

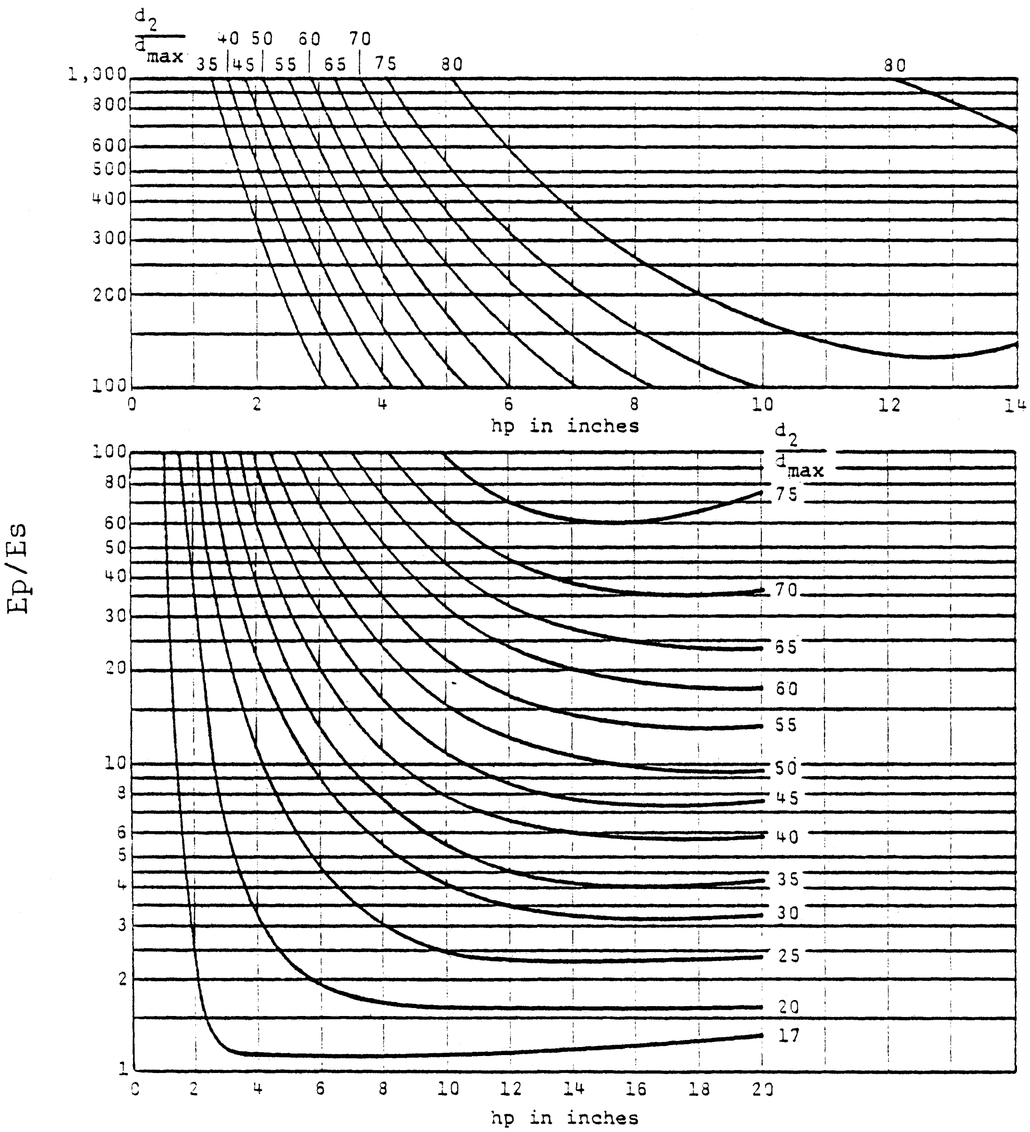


Figure 11. $\frac{d_2}{d_{max}}$ versus $\frac{Ep}{Es}$ versus hp.

(Wheel load = 9,000 lb; tire pressure = 70 psi; tire contact radius = 6.4 in.)

Conversion factors: 1 in. = 2.54 cm

1 lb. = 0.45 kg

1 psi = 0.69 kN/m^2

CAUTION: a. Do not use for values greater than 80.
b. Do not extrapolate.

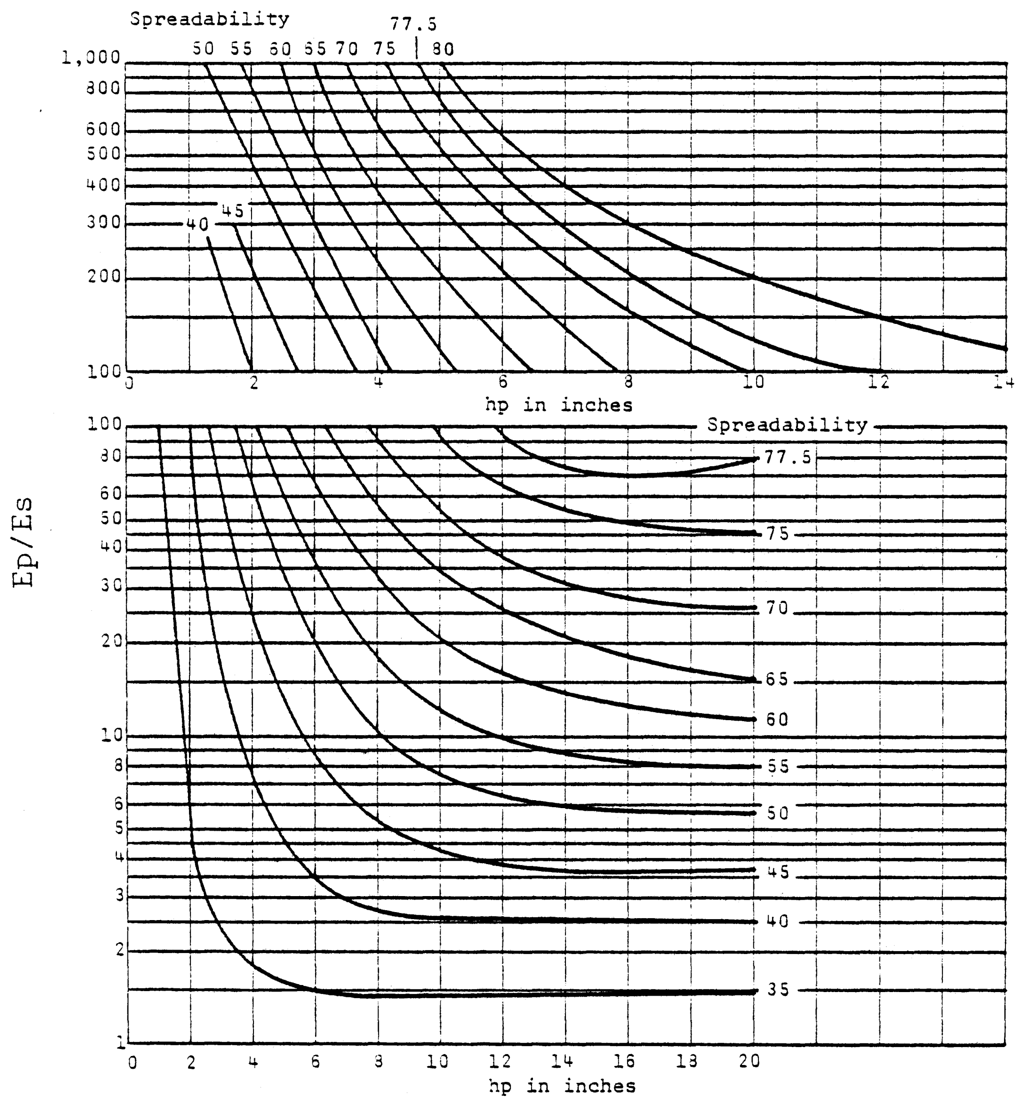


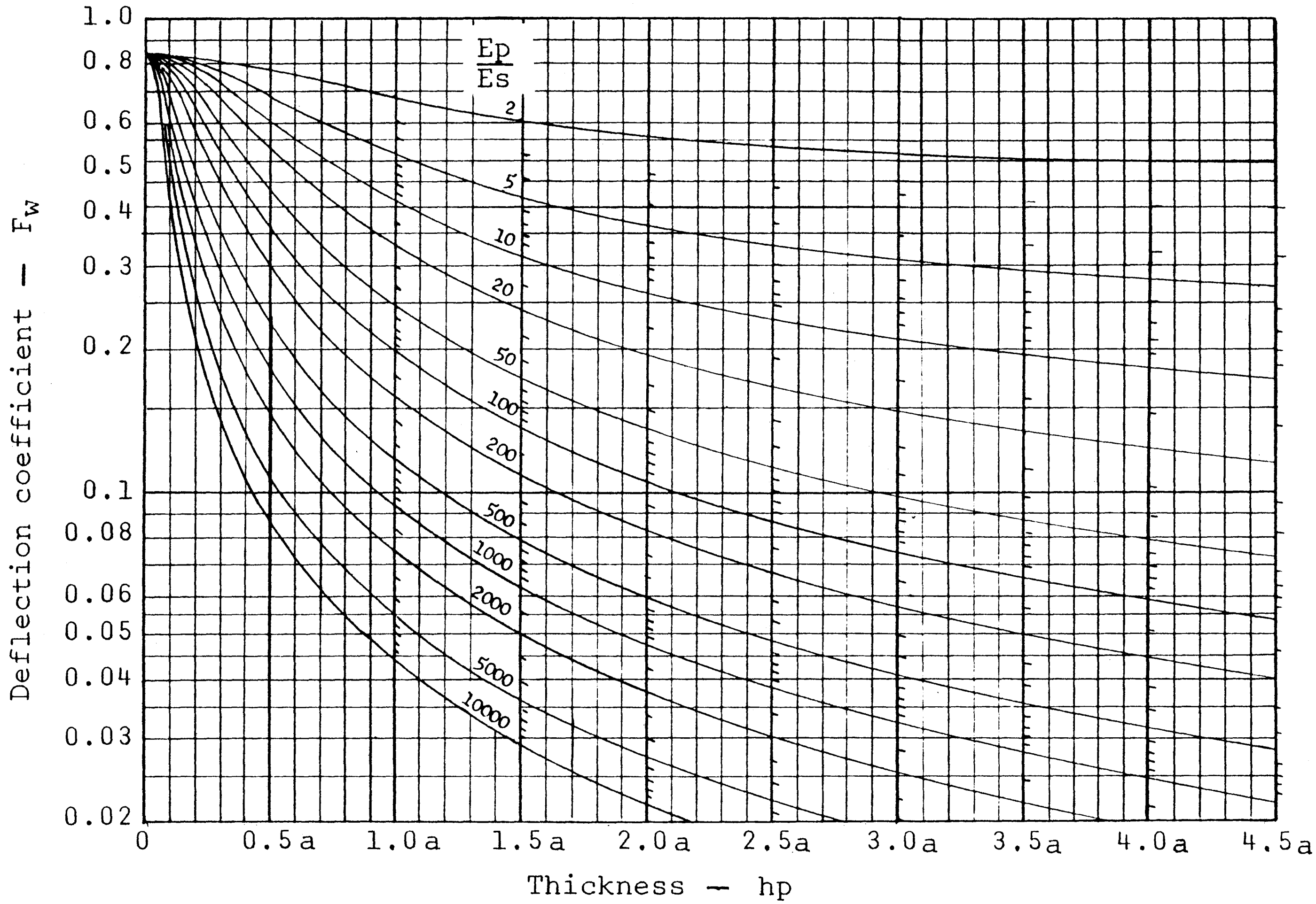
Figure 12. Spreadability versus $\frac{Ep}{Es}$ versus hp.

(Wheel load = 9,000 lb; tire pressure = 70 psi; tire contact radius = 6.4 in.)

Conversion factors: 1 in. = 2.54 cm

1 lb. = 0.45 kg

1 psi = 0.69 kN/m^2



hp

Figure 13. Burmister's influence curves of the deflection coefficient for the two-layer system.

$$E_s = \frac{1.5\ paF_w}{d_{max}}$$

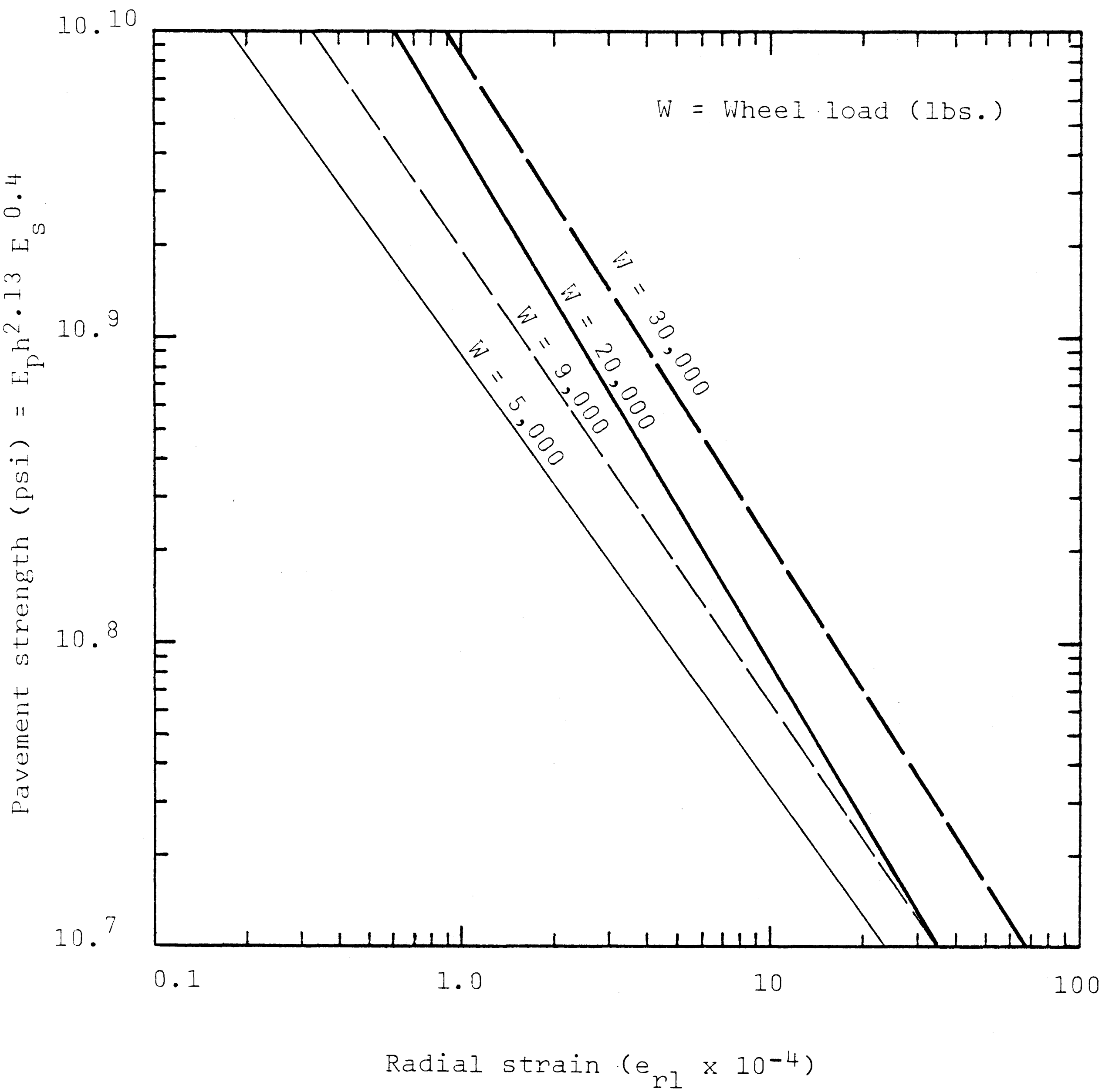


Figure 14. Relationship between the wheel load, pavement structural strength, and radial strength at the bottom of the top layer.
Conversion factor 1 lb. = 0.454 kg.

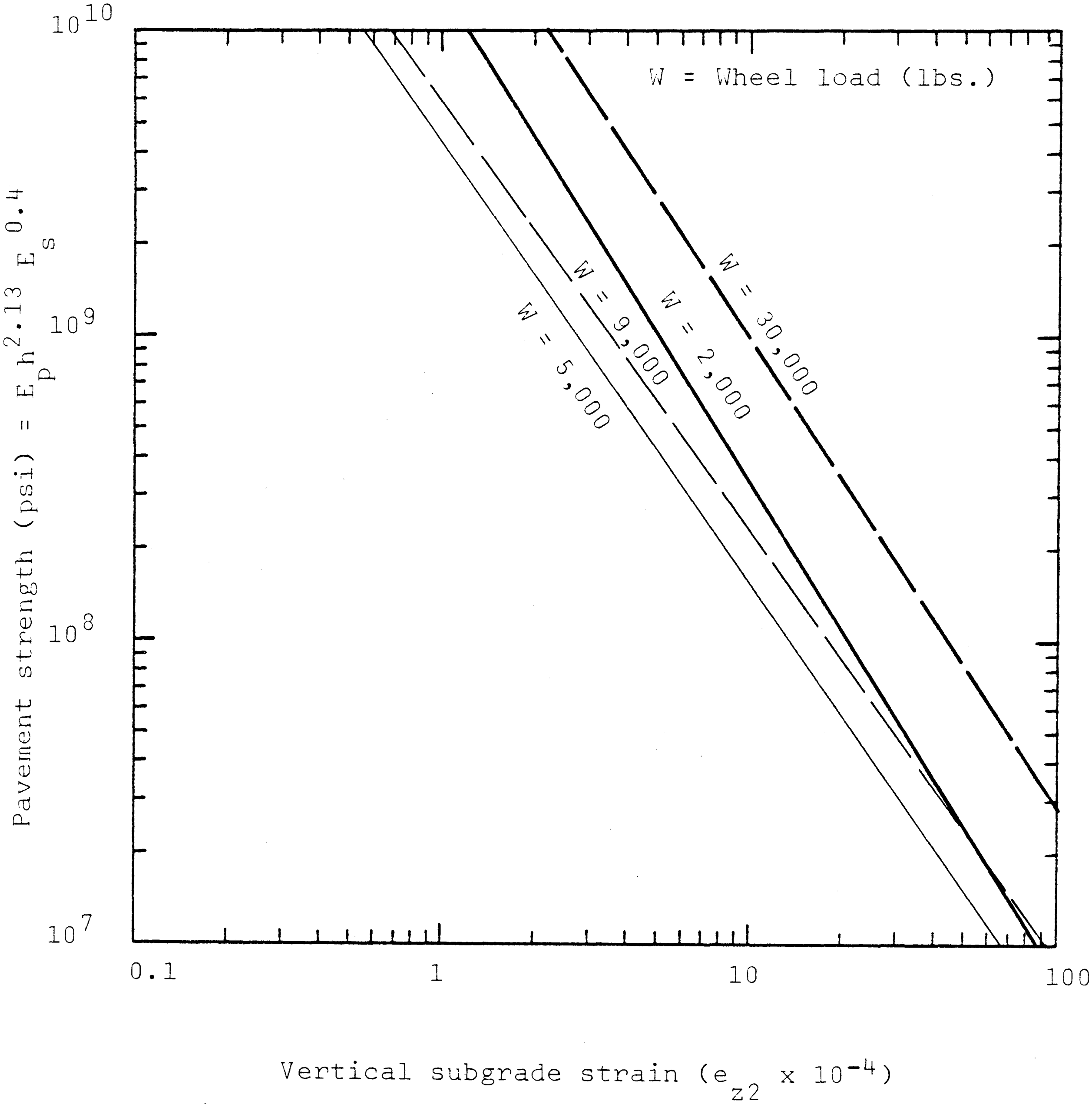


Figure 15. Relationship between the wheel load, pavement structural strength, and vertical subgrade strength. Conversion factor 1 lb. = 0.454 kg.

Traffic Equivalency Factors

For both the above methods, traffic equivalencies for the various loads need to be determined. An approach to developing this information is described in the following paragraphs.

The traffic equivalency factor of a given axle load describes the damage potential of that load as a ratio of the damage potential of an 18-kip (8,160-kg) axle load, and is a function of the strength of the pavement for which the ratio is determined. Thus, for a given pavement, a traffic equivalency of 10 indicates an axle loading ten times as destructive as one 18-kip (8,160-kg) axle load.

The AASHTO Committee on Design gives traffic equivalencies for given axle loads and pavement structural numbers.(13) Further, the AASHTO Interim Guide states that the 18-kip (8,160-kg) equivalent for pavements having an AASHTO structural number (or AASHTO thickness index, equal to 3 normally gives traffic equivalency factors which are sufficiently accurate for design purposes, even though the actual structural number is substantially different.(21) Based on the data given by the AASHTO Committee,(13) a relationship between axle loads and 18-kip equivalents was determined for a structural number of 3 and terminal serviceability of 2.5. This relationship is shown in Figure 16 and is given by the following equation.

$$\log(\text{traffic equivalency}) = 3.8 \log(\text{SAL in lb.}) - 16.17 \quad (8)$$

having correlation coefficient $R = 0.9995$ and $SE = .012$,

where

SAL = a single-axle load.

The AASHTO Interim Guide has tabulated equivalency factors up to a maximum of 40-kip (18,140-kg) single-axle loads. However, by means of equation (8), values beyond this limit could be extrapolated.

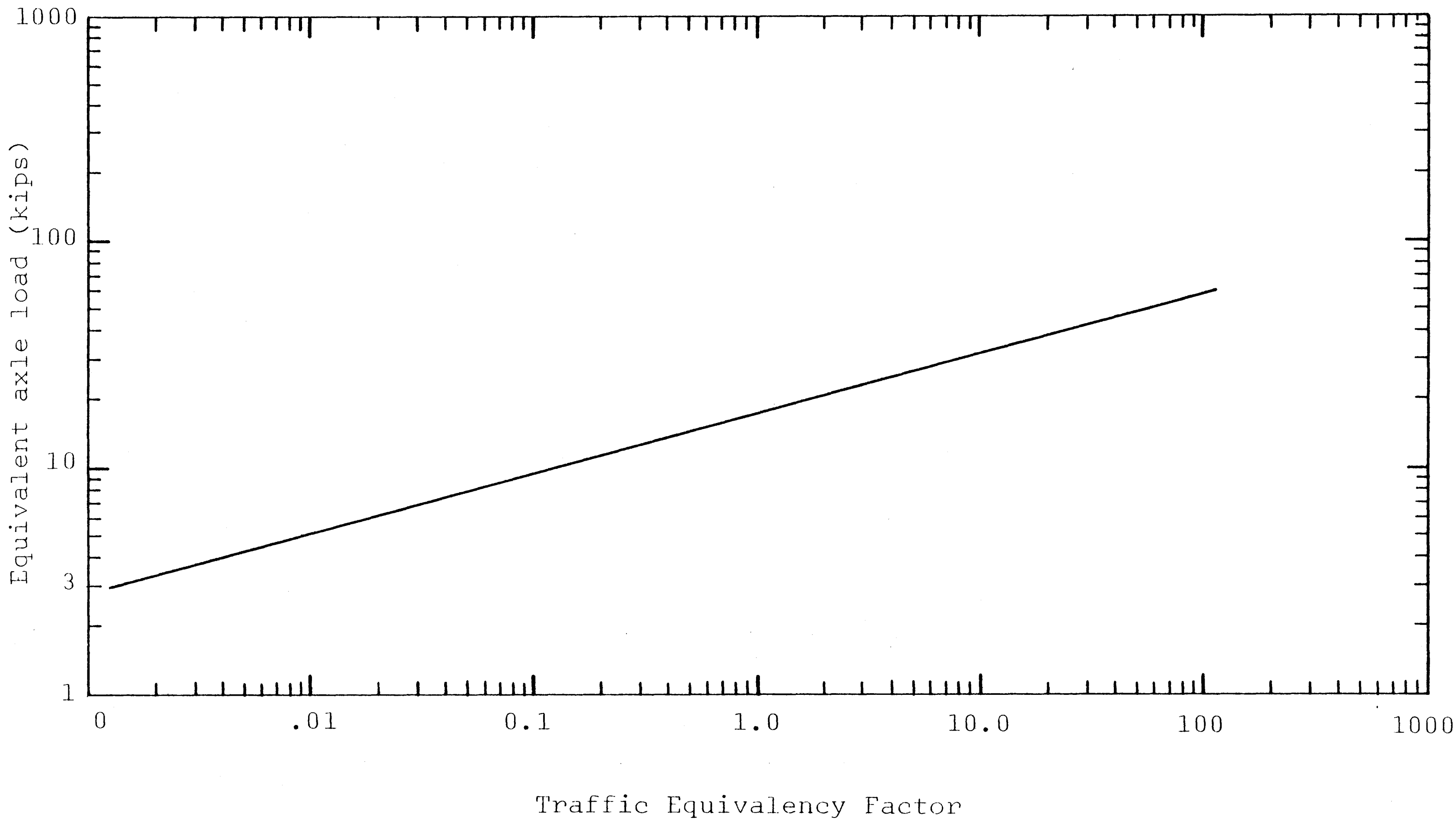


Figure 16. Relationship between the axle load and 18-kip equivalent for an AASHTO terminal serviceability of 2.5 and AASHTO S.N. of 3 or Virginia D of 6.8. Conversion factor 1 lb. = 0.454 kg.

DEVELOPMENT OF CONSTRUCTION GUIDELINES

A general method for determining the traffic equivalency factor (i.e. 18-kip [8,160-kg] equivalent) and the permissible repetitions of loads for a given construction vehicle have been described previously. Guidelines which would give the maximum permissible repetitions of a given weight vehicle could be used for the guidance of construction personnel. The development of such guidelines has been divided into two steps as follows: 1) The development of traffic equivalency factors for construction vehicles of known weights or axle weights, and 2) the determination of maximum permissible repetitions of 18-kip (8,160-kg) equivalent axle loads for pavements of a given strength. The steps are described below.

Traffic Equivalency Factors for Construction Vehicles

Based on the AASHTO Road Test results, Figures 6 and 7 were drawn to correlate the thickness index and N_{18}. This relationship is for conventional vehicles only and does not account for changes in subgrade strength. For heavy vehicles, the effect of the subgrade strength needs to be determined.

By means of the Chevron program and multi-regression analysis it was determined that the total pavement strength and the subgrade strength could be related by the equation

$$\text{Pavement strength} = Ep\ h^{2.13}\ Es^{0.4} \quad \text{(14)}$$

Using the Chevron program, a relationship was developed between the pavement strength and pavement strains for wheel loads ranging from 5,000 to 30,000 lb. (2,300 to 13,700 kg). This relationship has been superimposed on Figures 14 and 15.

As described before, the modulus of asphaltic concrete in Virginia has been found to vary between 300,000 and 400,000 psi (2.07 to 2.78 m Pa). A 1 in. (2.54 cm) thick asphaltic concrete layer with an Ep = 400,000 psi (2.78 m Pa) is taken in Virginia to have a thickness index equal to 1. Pavement strength values for hp = 1 in. through 20 in. (25 - 500 mm) and Ep = 400,000 psi (2.78 K Pa) were determined from equation (14) for Es = 5,000 and 10,000 psi (34,500 and 69,000 k Pa). Based on this development, the Virginia thickness index could be related to the pavement strength and correlated with the load repetitions as shown in Figure 17. Eight such relationships for four wheel loads on subgrade moduli of 5,000 to 10,000 psi (34,500 and 69,000 k Pa) were developed and are shown in Figure 17. Assuming the traffic

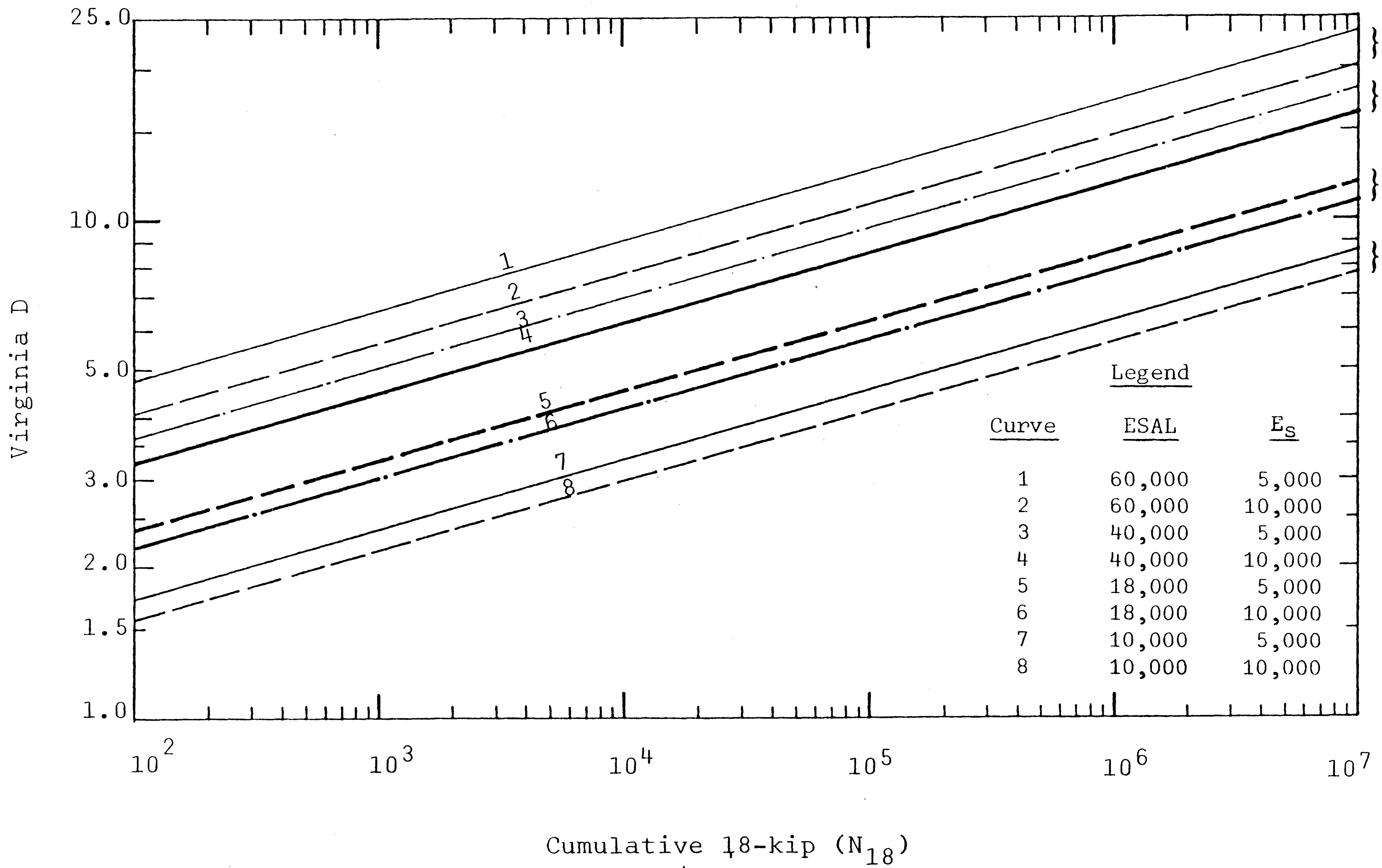


Figure 17. Relation between Virginia D and load repetition based on fatigue failure. Conversion factor 1 lb. = 0.454 kg and 1 psi = 0.68 k Pa.

equivalency of a 9,000-lb. (4,080-kg) wheel load is 1, i.e. for an axle load of 18,000 lb. (8,160 kg), traffic equivalency factors for single-axle loads of 10,000, 40,000, and 60,000 lb. (4,500, 18,200, and 27,300 kg) were determined from Figure 17. The average value so obtained for each axle load is given in Table 4. For comparison, the traffic equivalency values for the same axle loads as given by the AASHTO Committee(13) on Design are also given in this table. This table shows that the AASHTO traffic equivalency values are between the traffic equivalency values for ES = 5,000 psi (34,500 k Pa) and Es = 10,000 psi (69,000 k Pa). Subgrade modulus values of soils in Virginia usually are in the 5,000 to 10,000 psi (34,500 and 69,000 k Pa) range. Thus, it is evident that the traffic equivalency values recommended by the AASHTO Committee could be used for the construction vehicles on Virginia pavements.

It is, therefore,recommended that the traffic equivalency values recommended by the AASHTO Committee and graphically shown in Figure 16 or as given by equation (8) be adopted for determining maximum permissible load repetitions for construction vehicles in Virginia. The AASHTO values are, therefore, given in Table 5 and may be adopted as a part of any needed guidelines.

Maximum Permissible Repetitions for Construction Vehicles

As may be recalled, Figure 7 gives N_{18} versus the Virginia index D for class 1 and class 2 cracks on the AASHTO Road Test results. This figure shows that the load repetitions for class 2 cracks are about five times the load repetitions for class 1 cracks. The load repetitions due to fatigue failure in Figure 17 are also based on an equation which considers fatigue failure when class 2 cracks develop. One-fifth of the load repetitions obtained from Figure 17 could, therefore, be considered to cause class 1 cracks.

The minimum number of load repetitions needed to cause the development of class 1 and class 2 cracks for a range of thickness indices are given in Table 6. As discussed before, the load repetitions which would cause class 1 cracks are considered as the maximum permissible for traffic lanes, and the load repetitions which would cause class 2 cracks are considered as the maximum permissible on partially constructed pavement within the proposed median strips. The use of Tables 5 and 6 combined would give the maximum permissible repetitions for a construction vehicle with known axle loads on a pavement of a given Virginia thickness index. For pavements in use, the accumulated 18-kip (8,160-kg) equivalent (N_{18}) prior to the use by construction vehicles should be deducted from the permissible limits given in Table 6.

Table 4

Comparison of Traffic Equivalency Values from AASHTO and from Fatigue Analysis

Single-Axle Load	Traffic Equivalencies		
	AASHTO	Fatigue Test	
		Es = 5,000	Es = 10,000
10,000	0.11	0.11	0.1
18,000	1.0	1.0	1.0
40,000	20.8	22.5	17.5
60,000	97.0	150.0	89.0

Table 5

Traffic Equivalency Values
for Construction Vehicles
(AASHTO)

Single- Axle Load (Kips)	10	18	20	22	24	30	35	40	45	50	55	60	65	70
Traffic Equivalency	0.1	1.0	1.5	2.1	3.0	7.0	12.5	20.8	32.5	48.5	69.7	97	131	174

Table 6

Maximum Permissible 18-Kip Equivalent Axle Loads for Construction Vehicles

Va. D / Lane	2	3	4	5	6	7	8	9	10	12	14	15	16	18
	LOAD REPETITIONS													
Traffic Lane	8	140	1,200	5,000	15,000	34,000	70,000	150,000	300,000	620,000	2.2×10^6	6×10^6	10×10^6	20×10^6
Other Lanes	40	700	5,600	26,000	80,000	180,000	380,000	680,000	1.3×10^6	2.4×10^6	5.6×10^6	13×10^6	20×10^6	40×10^6

Note:

Maximum permissible load repetitions for the construction vehicle is the maximum permissible cumulative 18-kip equivalent (N_{18}) or truck load divided by the traffic equivalency of the respective axle or truck.

CONCLUSIONS

1. For mathematical analyses of wheel loads transmitted through low-pressure flotation tires, transmitted pressures higher than the inflation pressures should be assumed. A minimum of 70 psi (482 k Pa) for the transmitted pressure is reasonable.

2. Pavement damage by multi-wheel and multi-axle vehicles can be analyzed by the traffic equivalency technique.

3. For vehicles wider than 8 ft. (2.5 m), the damaging effect of the wheel over the pavement edge or over a weak shoulder should be considered.

4. The damaging effect of a construction vehicle should be limited to class 1 cracks for traffic lanes and class 2 cracks for median lanes under construction, and, in no case, should the loss in serviceability index exceed 0.5 nor should the serviceability index decrease below 3.0.

5. Either of the two methods for determining the permissible repetitions of a construction vehicle developed in this investigation could be used. The method based on the loss in serviceability index does not require the use of a computer program. In the fatigue failure method, the failure of untreated aggregate should be based on the vertical compressive strength of the pavement.

6. The traffic equivalency of an axle load of a construction vehicle as determined from pavement fatigue analysis approximates the traffic equivalencies as recommended by the AASHTO Committee on Design for a terminal serviceability of 2.5 and pavement thickness index of 3.0. Hence the traffic equivalencies given by the AASHTO Committee could be applied to heavy construction vehicles.

7. The traffic equivalencies for construction vehicles given in Tables 4 and 5 and the permissible 18-kip (8,160-kg) load repetitions for given pavement thickness indices given in Table 6 could be used as guidelines by field engineers.

ACKNOWLEDGEMENT

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