Evaluation of Heavy Load Damage Effect on Concrete Pavements Using Three-Dimensional, Nonlinear Dynamic Analysis

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A study on the effects of heavy loads on Indiana highways was conducted. Available load equivalency factor (LEF) concepts were found to be inadequate for the study. An analytical approach was used to develop LEFs for concrete pavement. These LEFs are based on the total surface deformation. A three-dimensional dynamic finite-element method (3D-DFEM) was used in the analysis. The 3D-DFEM was verified for static, linear elastic, and dynamic nonlinear analyses. The 3D-DFEM predictions were compared with actual field measurements. There was good agreement between predicted and measured pavement responses. A comparison was made between the AASHTO LEFs and the Purdue LEFs for conditions similar to those of the AASHO Road Test, and no significant difference was found. Purdue LEFs consider different load and cross-section parameters, whereas the AASHTO LEFs do not. Also, Purdue LEFs were developed on the basis of an analytical model that can be extended in the future to cover a wider range of pavement thicknesses, layer materials, and load variables.

Accommodation of mixed traffic with a wide variety of axle loads and configurations is a critical step in pavement design. During the last 50 years a number of load equivalency concepts have been used to transform complex load configurations into a single standard load that can be used for the design of concrete pavements. These load equivalency concepts include equivalent single wheel load, equivalent single axle load (ESAL), and equivalent single axle radius (*1*). ESAL is the most commonly applied concept for highway pavements and was introduced by AASHO in the 1972 interim design guide (*2*). Load equivalency factors (LEFs) that use the ESAL concept are based on equal loss of serviceability. Empirical/statistical LEF sets were developed from analysis of the AASHO Road Test (*3*) results.

Westergaard analysis (*4,5*) and two-dimensional finite-element analysis (2D-FEM) are widely used to predict the structural response of rigid pavements to loads. These types of analyses assume static loading conditions and linear elastic material properties (*6*). Realistically, pavements are subjected to moving loads. Also paving materials may be characterized as elastic, elastic-plastic, visco-elastic, or plastic. The inability of the Westergaard analysis and currently used 2D-FEM analysis to represent actual loading conditions and paving material characteristics is significant. This reflects on the predicted pavement response and hence on any LEFs based on these predictions.

A study was conducted at Purdue University to develop a procedure for permitting overloaded trucks in Indiana. Funding was provided by the Indiana Department of Transportation (INDOT) and FHWA. The study addresses the permissibility of overloaded trucks as well as recommendations of vehicle configuration for various load levels. Both bridges and pavements were considered. However, the pavement part of the study only is addressed in this paper.

A 1-year sample of overload permit applications was reviewed to determine the configurations of the trucks being permitted. This sample revealed that permits were requested for trucks with up to nine axles in one group as well as trucks with single axle loads of 72 kips (32,668 kg). The AASHO Road Test included only single and tandem axle loads of up to 30 kips (13,612 kg) and 48 kips (21,779 kg), respectively. LEFs on regression analysis of the AASHO Road Test results are valid only for these numbers of axles and load ranges. Simple extrapolation of such regression relations beyond the range of factors for which data have been collected is questionable unless there is a basis of realistic material and structural models. This appears to be a deficiency in the AASHTO *Guide for Design of Pavement Structures* (*7*), in which LEFs are presented for single and tandem axle loads higher than those in the Road Test as well as for tridem axles, which were not used at all. These extrapolations are made by using the original serviceability-based regression equations for performance.

In the Load Equivalency Workshop, sponsored by FHWA (*8*), Barenberg emphasized the importance of using validated mathematical models in predicting pavement response and developing LEFs. He said, "A validated mathematical model is a model that accurately predicts pavement response to load and environment." He defined pavement response as deformation and strain.

In this paper LEFs for concrete pavements are presented. These LEFs were developed for the study addressing the permissibility of overloads and are based on equal maximum surface deflection (the elastic deformation of all layers and the plastic deformation of the bound and unbound layers under the concrete slabs, if any). A three-dimensional dynamic finite-element method (3D-DFEM) was used to analyze concrete pavements (*9*). This 3D-DFEM has the capability to simulate truck loads moving at different speeds. Also, it can realistically model paving materials as elastic, elastic-plastic, plastic, and viscoelastic materials. The 3D-DFEM predicts both the elastic and plastic pavement responses for one or more load applications. The 3D-DFEM was verified in two steps: first for static linear elastic analysis and then for dynamic nonlinear analysis (*10*).

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AVAILABLE LEF APPROACHES

There are several approaches to evaluating the effect of loads on pavements, and therefore to determining LEFs. Two examples of LEF concepts for concrete pavement are equal loss of pavement serviceability and equal pavement distress (e.g., fatigue).

Loss of Serviceability Approach (AASHTO LEFs)

In 1959 and 1960 the AASHO Road Test was conducted in Ottawa, Ill. (*3*). Two types of truck loading were used: single and tandem axles. The results of the AASHO Road Test and the concept of present serviceability index (PSI) were used as a measure of pavement performance in the AASHTO Interim Design Guide (*2*). The PSI of concrete pavement is a function of pavement slope variance (roughness), cracking, and patching. Pavement failure was defined in terms of terminal serviceability instead of strict structural failure. Empirical relationshps were developed to correlate PSI, as a measure of pavement performance, to the number of load repetitions. On the basis of these two factors, PSI and load repetitions, the AASHTO LEFs were developed. In the AASHO Road Test the maximum axle loads were 30 and 48 kips (13,612 and 21,779 kg) for single and tandem axles, respectively. In the 1986 AASHTO design guide (*7*) LEFs for higher loads and tridem axle configuration are presented by using the same statistical models.

Analytically Based LEFs

As another approach to determining LEFs, Westergaard analysis (*4,5*) or a 2D-FEM analysis (*11*) was used to predict the elastic pavement response for different load parameters, such as axle load and spacing. The pavement damage owing to different load parameters is estimated by using correlations of various types of distress, such as cracking, and pavement response, such as tensile stress. Several relationships are available to correlate fatigue failure to maximum tensile stress. A frequently used relationship was developed by Vesic and Saxena (*12*):

$$N_{2.5} = 225{,}000\left(\frac{M_R}{\sigma}\right)^4$$

where

$N_{2.5}$ = load repetitions to a serviceability index of 2.5,
M_R = modulus of rupture of concrete (lb/in.^{2}), and
σ = tensile stress (lb/in.^{2}).

In an analysis reported by Hallin et al. (*13*), tensile stress was taken as the combined load and warping stresses. The load stress was determined by using ILLI-SLAB, a two-dimensional finite-element program (*11*), whereas the warping stress was based on regression equations developed by Darter (*14*). The combined stress was determined at the maximum load-related stress position. The LEF for load (i) is represented as (*13*)

$$(\text{LEF})_i = \frac{[N_f]_{18}}{[N_f]_i}$$

where $[N_f]_{18}$ and $[N_f]_i$ are the number of repetitions of the standard 18-kip (8,167-kg) load and any load (i), respectively, resulting in a serviceability index of 2.5.

COMPARISON BETWEEN DIFFERENT LEF METHODS

A comparison between the AASHTO LEFs (*2*) and LEFs based on fatigue analysis (*13*) for a single axle load configuration is presented in Figure 1. LEFs based on fatigue analysis are significantly different from the AASHTO LEFs. The fatigue analysis LEFs were found to underestimate the pavement damage caused by any axle load. This was expected because the fatigue analysis LEFs are based on the elastic response of pavements, whereas the AASHTO LEFs are based on slope variance (roughness), which is a function of permanent pavement deformation or loss of support. Not accounting for or incorrectly accounting for other factors such as temperature curling or moisture warping may also contribute to the poor correlation of fatigue analysis.

THREE DIMENSIONAL-DYNAMIC FINITE ELEMENT ANALYSIS

Features of Finite-Element Model

A jointed reinforced concrete pavement cross section similar to that of the AASHO Road Test was modeled in the present analysis as two 12-ft (365.76-cm) lanes plus 8-ft (243.84-cm) shoulders on either side. The pavement structure consists of three layers: concrete slab, granular subbase, and subgrade. Granular shoulders were used in the analysis to be consistent with the AASHO Road Test. Three-dimensional finite-element meshes (3D-FEMs) with variable openings were created to model the pavement structures. Meshes with variable size openings were used to reduce the computer memory requirements and computational time. A smaller mesh spacing was used to provide detailed response predictions when they were needed. Pavement structures were modeled as a

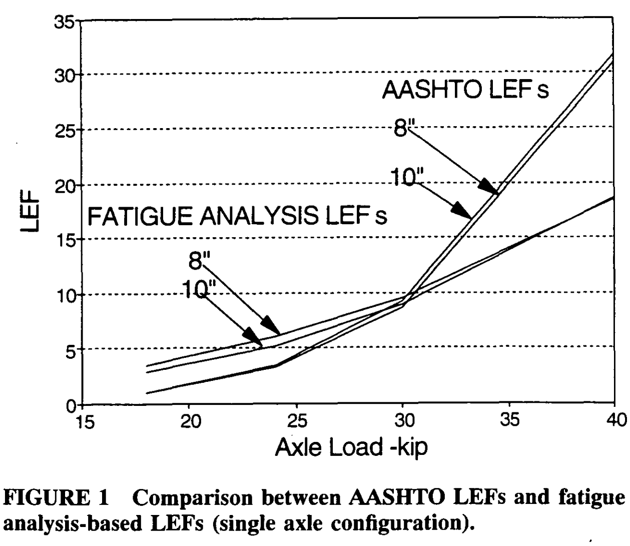


FIGURE 1 Comparison between AASHTO LEFs and fatigue analysis-based LEFs (single axle configuration).

set of layers. Shown in Figure 2 is one of the 3D-FEMs used in the analysis. In this example the subgrade thickness was represented by five elements; the concrete slab and the granular subbase thicknesses were represented by single elements. Longitudinal and transverse joints were modeled by using gap elements with an initial opening of 3/8 in. (9.53 mm). Depending on the deformed shape of the slabs after loading, the slabs might come into contact and develop friction. Dowel bars were modeled and located in the midthickness of the slab. The bond stress of one-half of the dowel bar was set equal to zero. Details of the finite-element features used in the analysis have been reported previously (*12,14*). Loads were sequentially applied at surface nodes. The time rate of loading from one node to the next simulated vehicle speeds. From previous studies (*10,15*), vehicle speed was found to have a significant effect on pavement response. Therefore, a speed similar to the average speed of the AASHO Road Test, 35 mph (50 km/hr) (*16*), was used in developing the LEFs.

Material Models

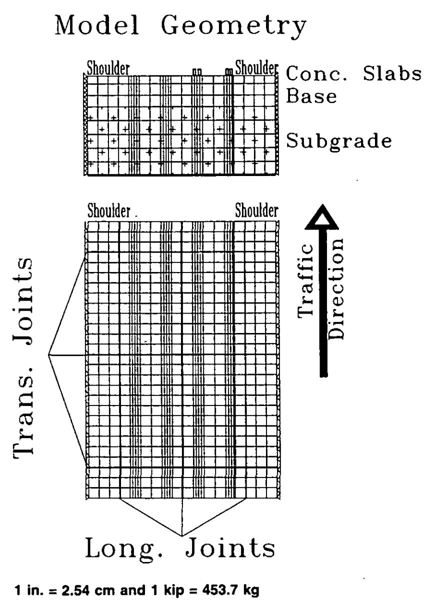
In the analysis pavement materials were divided into three groups: portland cement concrete, granular materials, and cohesive soils. Details of these material models were reported by Zaghloul and White (*17*).

Portland cement concrete behavior was divided into three stages: elastic, plastic, and after-failure stages. The stress-strain curve used to model portland cement concrete is shown in Figure 3. In this model if the concrete slab is subjected to a stress level less than its yield stress it will behave as an elastic material. When the stress level exceeds the yield stress of the concrete, behavior is elastic-plastic until the stress reaches the failure limit. At that point the after-failure stage starts (*18*).

Granular materials, base, subbase and subgrade in some cases were modeled by using the Drucker-Prager model (*20,21*). This is an elastic-plastic model in which granular materials are assumed to behave elastically for low stress levels. When the stress reaches a certain yield stress the material will subsequently behave as an elastic-plastic material. The assumed stress-strain curve for a granular material is shown in Figure 3*a*.

The Cam-Clay model (*18,21,22*) was used to model clays. This model uses a strain rate decompositoin in which the rate of deformation of the clay is decomposed additively into an elastic and a plastic part. The assumed soil response in pure compression is shown in Figure 3*b*.

Other material and layer characteristics required in the analysis include modulus of elasticity, Poisson's ratio, damping coefficient, and bulk density. Listed in Table 1 are the material properties used in the analysis.



1 in. = 2.54 cm and 1 kip = 453.7 kg

FIGURE 2 Example of 3D-FEMs used in analysis.

Finite-Element Model Verification

The 3D-DFEM was verified for static, linear elastic analysis as well as for dynamic, nonlinear analysis.

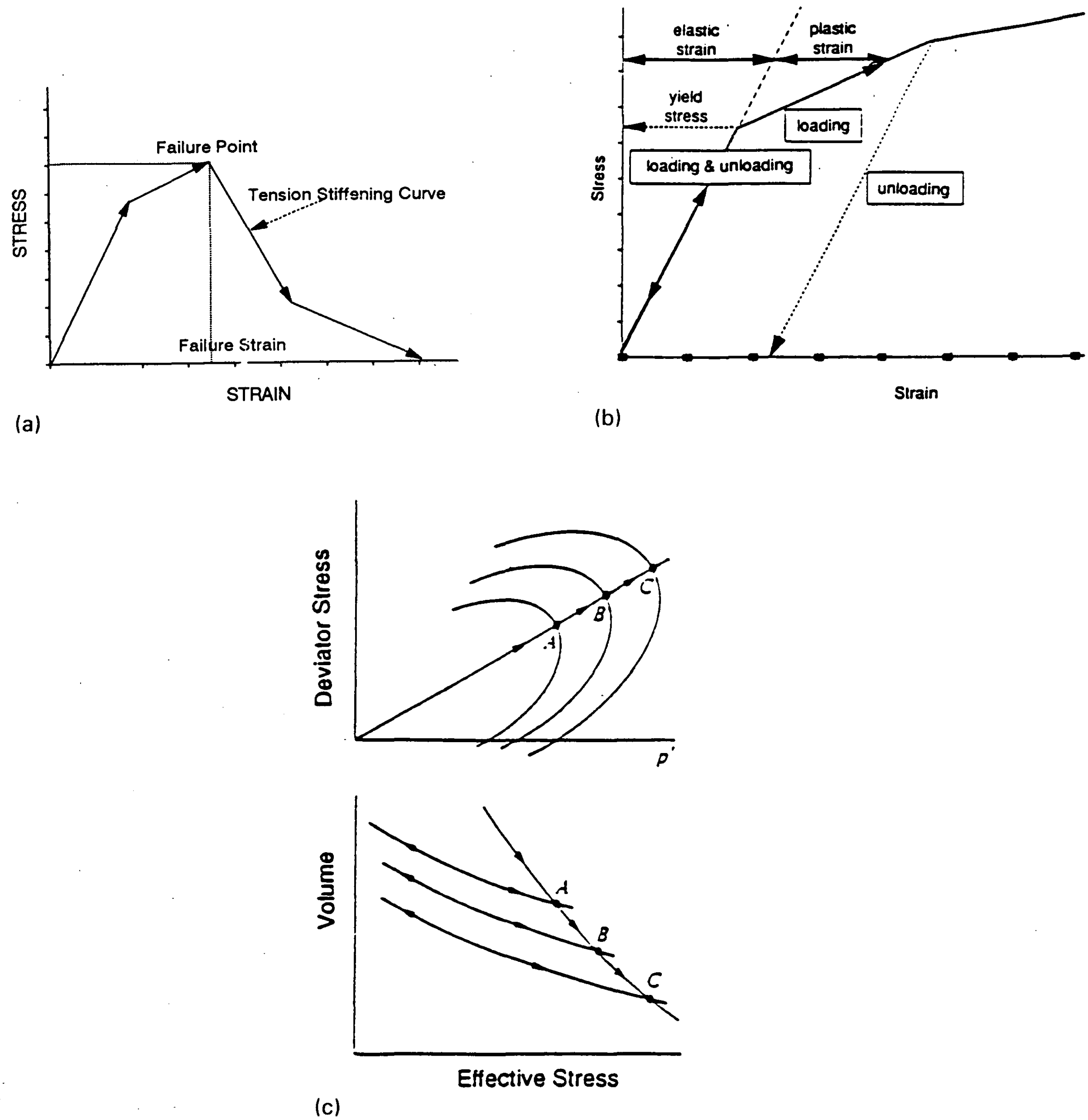
Static Analysis Verifications

A design of experiment (DOE) was developed to determine if the 3D-DFEM predictions of pavement response agree with those calculated by using the Westergaard equations (*4,5*). Three factors were included in the DOE: slab thickness, subgrade type, and load position. Three levels for the slab thickness, 6, 10, and 14 in. (15.24, 25.4, and 35.56 cm), and two subgrade types, sand and clay, were included in the analysis. Values of modulus of elasticity (*E*) and modulus of subgrade reaction (*k*) were selected on the basis of the correlation of soil type with the Unified Soil Classification system (*23*). Assuming a static loading condition and linear elastic material properties, the maximum tensile stress in the concrete slab for three loading positions, center, edge, and corner, were predicted by using the 3D-DFEM and the Westergaard equations (*4,5*). An analysis of variance of the results was made to test if there was a linear correlation between the deflections predicted by the 3D-DFEM and those calculated from the Westergaard equations. It was found that there is a very high linear correlation between the pavement responses predicted by the 3D-DFEM and those calculated from the Westergaard equations (R^2 = 97.7 percent).

In another study (*24*), the 3D-DFEM predictions, assuming linear elastic material properties and static loads, were compared with the predictions of a multilayer analysis by using the computer program Bitumen Structures Analysis in Roads (BISAR) (*25*). There was good agreement between the predictions of deflection by the two models at different depths and offset distances from the loaded area (R^2 = 96.0 percent).

Dynamic Analysis Verification

To evaluate the dynamic analysis capabilities of the 3D-DFEM, a comparison was made of its predictions with actual pavement deflections measured under moving trucks. In a study by the Portland Cement Association (*26*), a field testing program was conducted at six sites; three of these sites were located in Wisconsin and the other three sites were located in Pennsylvania. The surface



1 in. = 2.54 cm and 1 kip = 453.7 kg

FIGURE 3 Material models used in analysis: (*a*) concrete model (*18*), (*b*) Drucker-Prager Model (*18*), and (*c*) Cam-Clay model (*18,19*).

deflection was measured at these sites under single and tandem axles moving at creep speed, 2 mph (3 km/hr). The three pavement sections located in Wisconsin were incorporated into the verification study. Finite-element meshes were created to match these cross sections, and reasonable material properties were assumed. Moving axle loads similar to those used in the field test were considered in the analysis, and the total surface deflection from these loads was predicted by using the 3D-DFEM. The measured and predicted pavement deflections are shown in Figure 4. A linear correlation analysis between the measured and predicted deflections showed an excellent correlation between the 3D-DFEM predictions and the field measurements (R^2 = 99.64 percent).

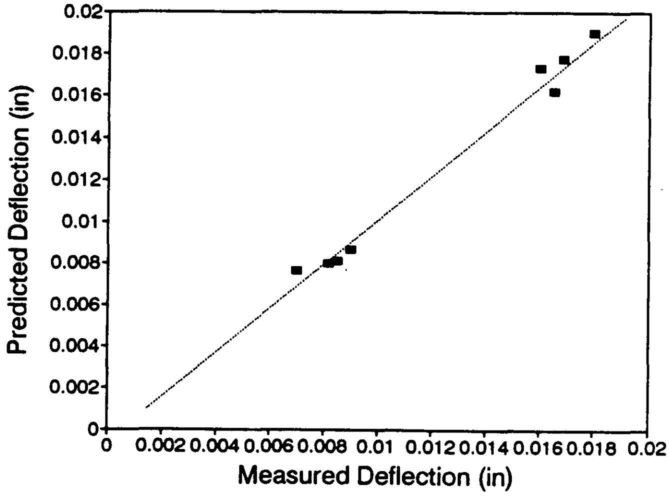
This dynamic verification study was conducted for pavement response to loads moving at creep speed, 2 mph (3 km/hr). No field data for concrete pavement response to loads moving at higher speeds were available at the time of the study. However, other dynamic verification studies were conducted by using the 3D-DFEM for asphalt pavements. These studies included comparisons of pavement response to loads moving at speeds of 4 and 35 mph (6 and 50 km/hr) (*24*) and comparison of pavement response to falling-weight deflectometer loading (*15*).

TABLE 1 Example of Material Properties Used in Analysis

Material Name	Material Property	Typical Value
Concrete Slabs	Modulus of Elasticity - psi(GPa)	4,000,000 (27.62)
	Poisson's Ratio	0.15
	Initial Yield Stress - psi (MPa)	2670 (18.4)
	Failure Plastic Strain	1.3E-03
	Density - pcf (gm/cm^3)	150 (2.403)
	Damping Coefficient (%)	5
Granular Subbase	Modulus of Elasticity - psi (GPa)	40,000 (0.276)
	Poisson's Ratio	0.3
	Initial Yield Stress - psi (MPa)	19.29 (0.133)
	Initial Plastic Strain	0.0
	Angle of Friction - degree	33
	Density - pcf (gm/cm^3)	135 (2.1625)
	Damping Coefficient (%)	5
Lean Clay (CL) Subgrade	Shear Modulus - psi (MPa)	2750 (18.964)
	Poisson's Ratio	0.3
	Logarithmic Hardening Modulus	0.174
	Initial Overconsolidation Parameter - psi (KPa)	8.455 (58.306)
	Permeability - ft/sec (cm/sec)	0.000021 (0.00064)
	Initial Void Ratio (%)	8
	Initial Stress psi (MPa)	weight of the pavement layers
	Density - pcf (gm/cm^3)	130 (2.0824)
	Damping Coefficient (%)	5

PURDUE LEFs

An LEF set was developed for concrete pavement with granular subbase (Purdue LEFs) on the basis of equal maximum surface deflection (MSD). MSD deflection consists of the elastic deformation of different layers and the plastic deformation of the unbound layers under the concrete slabs, if any. For a given load and speed the plastic deformation in the unbound layers increases with the number of load applications until an asymptotic value is reached. This asymptotic value and the rate of deflection increase with the number of load applications and are a function of load magnitude, speed, and slab thickness. The elastic deformation of the slab increases because of reduced support from the accumulating permanent deformation. Shown in Figure 5 is the effect of load repetitions on an unbound layer with a permanent deformation and on the total surface deflection for two slab thicknesses, 8 and 14 in. (20.32 and 35.56 cm).



1 in. = 2.54 cm and 1 kip = 453.7 kg

FIGURE 4 Dynamic analysis verification.

Use of MSD was arrived at after evaluating the results of a number of sensitivity studies (*10*). There is a logic as to why the MSD of concrete pavements would correlate so effectively with the AASHO Road Test serviceability concept. Fatigue of concrete pavements is related to elastic defection, whereas roughness is related to permanent deformation. Therefore, it takes the combined MSD to provide a scale for rigid pavement serviceability. In application, the LEF of any load j is the number of 18-kip (8,167-kg) single axle loads required to develop the same MSD developed by one pass of the road j on the same pavement cross section.

Two statistical models were developed for Purdue LEFs. The first model predicts the MSD developed by one pass of any axle load configuration, including the 18-kip (8,167-kg) single axle load, on a range of rigid pavement cross sections. The second model predicts the MSD owing to repetitions of the 18-kip (8,167-kg) single axle load. LEFs are developed by using the two models. The first model predicts the MSD owing to load j on cross section i, and then the predicted MSD is used as an input to the second model to estimate the number of the 18-kip (8,167-kg) single axle load repetitions required to develop the same MSD in cross section i, which is LEF_{ij}. Figure 6 provides a graphical representation of the Purdue LEF concept for rigid pavements.

Design of Experiments

Two DOEs were implemented to develop the rigid pavement LEFs. The factors in Table 2 were included in the first DOE (DOE1). In a study concerned with developing LEFs for flexible pavements (*27*), the subgrade type was found to be insignificant for flexible pavement LEFs at a speed of 35 mph (50 km/hr). On the basis of this experience and because of the surface rigidity of the concrete pavement, a decision was made not to include subgrade type in DOE1. A 4-in. (10.16-cm) granular subbase was assumed for all cross sections included in this analysis. The sample of overload permit applications showed that the average axle spacing is 4 ft (121.92 cm); therefore, a 4-ft (121.92-cm) axle spacing was assumed for n axle configurations.

A partial factorial design was used to develop the first model. Different load-cross section combinations were analyzed by using the 3D-DFEM. An analysis of variance was used to test the significance of different factors included in DOE1. The significant main effects and two-way interactions were used to develop a regression model to predict the MSD for various axle load configurations. From this analysis it was found that speed is a significant factor. The MSDs for high speeds [>20 mph (32 km/hr)] are small compared with those for low speeds [<20 mph (32 km/hr)]; therefore, two regression models were developed to predict MSD, one model for low speeds and the other for high speeds. Both models showed high correlations, R^2 = 98.4 and 99.5 percent, respectively.

Low-speed model (LSM) (R^2 = 98.4 percent):

$$MSD = 5.6E\text{-}6 \cdot D^4 + 18.402 \cdot N - 1.036 \cdot T - 2.416 \cdot N^2$$

where

D = axle load (kip per single axle),
T = slab thickness (in.), and
N = number per axles.

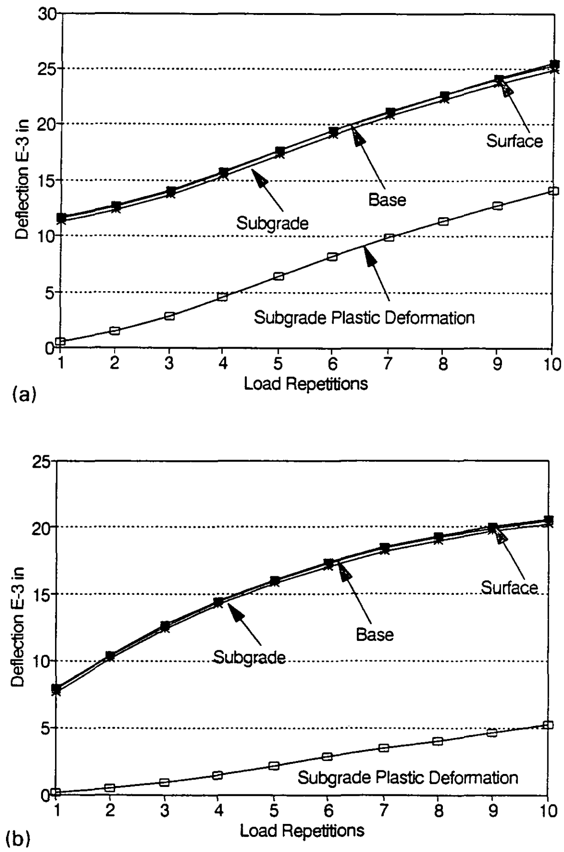


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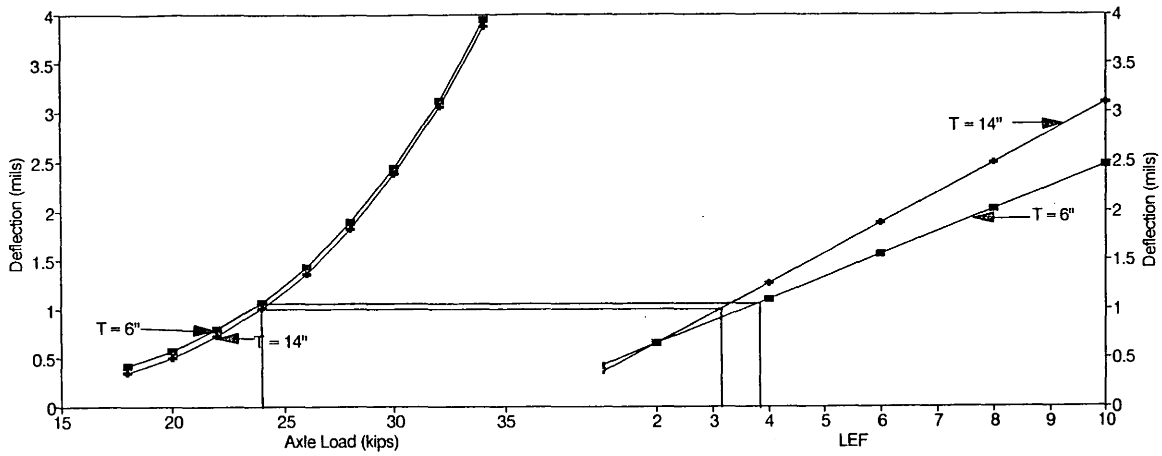
FIGURE 5 Effect of load repetitions on total surface deformation: (*a*) 8-in. concrete slab and (*b*) 14-in. concrete slab.



1 in. = 2.54 cm and 1 kip = 453.7 kg

FIGURE 6 Purdue LEFs.

TABLE 2 Factors in DOE1

Factor	Levels		
	1	2	3
Axle Load - kip (kg)	18 (8,167)	24 (10,889)	36 (16,334)
No. of Axles	1	2	4
Slab Thickness - in (cm)	6.0 (15.24)	12.0 (30.48)	18.0 (45.72)
Speed - mph (km/h)	1.75 (2.6)	20.0 (32.0)	40.0 (65.0)

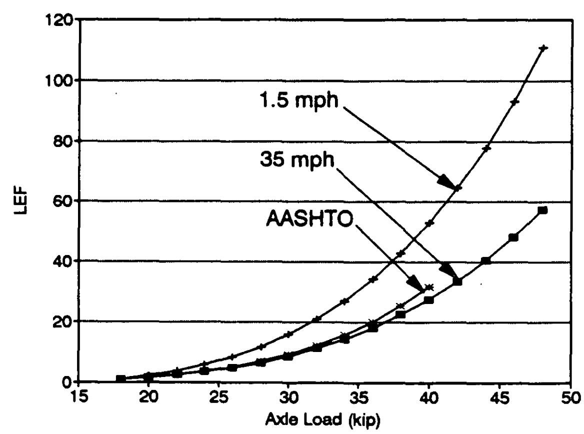
High-speed model (HSM) (R^2 = 99.5 percent):

$$\text{MSD} = 2.98E\text{-}6 \cdot D^4 + 0.3051 \cdot N + 0.0088 \cdot T - 0.0057 \cdot S \cdot N - 3.3E\text{-}9 \cdot S \cdot D^4 + 5E\text{-}5 \cdot S^2$$

where S is speed (in mph).

Two LEF sets were developed, one for low and one for high speeds. A comparison between these two sets and the AASHTO LEFs is presented in Figure 7. As can be seen from Figure 7, LEFs based on HSM agree with AASHTO LEFs. This is because the mean speed at the AASHO Road Test was approximately 35 mph (50 km/hr) (*16*). Subsequent analysis is made by using HSM. To extend the validity of HSM more cases were analyzed to cover a wider range of factor levels, and the results were compared with the extrapolated predictions of this model.

A second DOE (DOE2) was implemented to consider the effect of 18-kip (98,167-kg) single axle load repetitions. Two factors were included in DOE2, slab thickness (T) and number of 18-kip (8,167-kg) single axle load repetitions (C). Three levels for slab



1 in. = 2.54 cm and 1 kip = 453.7 kg

FIGURE 7 Effect of speed on Purdue LEFs.

thickness 6 in. (15.24 cm), 12 in. (30.48 cm), and 18 in. (45.72 cm), were included in the analysis, and the 18-kip (8,167-kg) single axle load was repeated up to 30 times. A regression analysis was made on the results obtained by using the 3D-DFEM, and the following regression model was developed (R^2 = 97.8 percent):

$$\mathrm{MSD} = (0.17 + 0.0096 \cdot T) \cdot C$$

Comparison Between Purdue LEFs and AASHTO LEFs

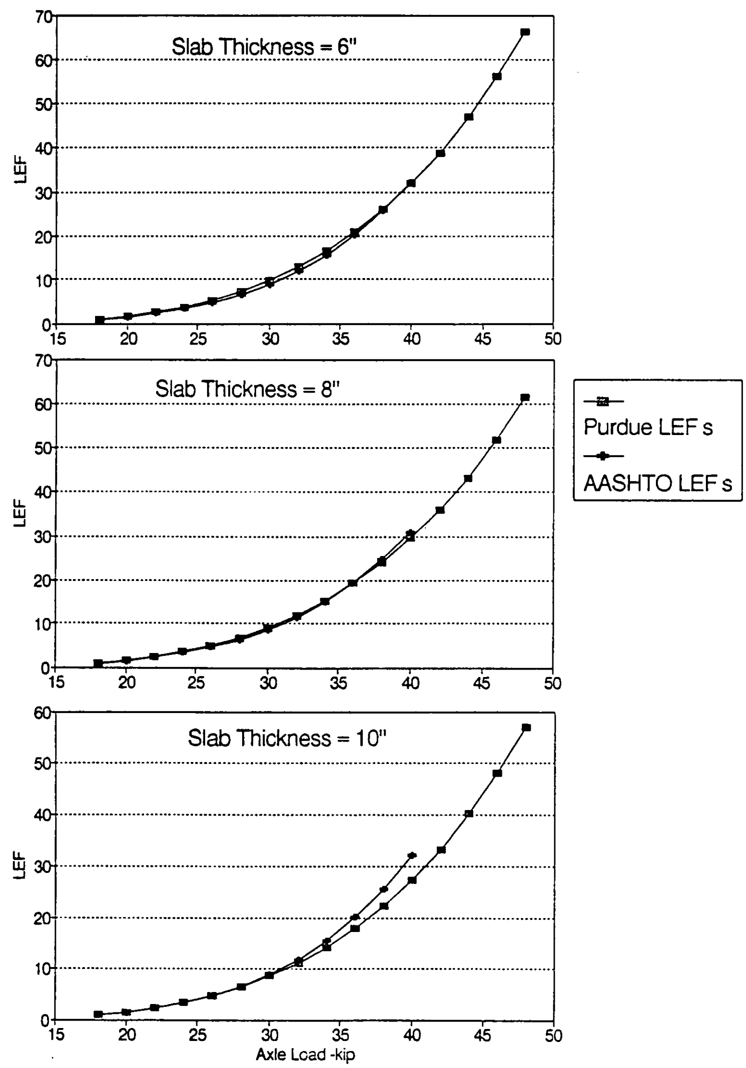
Purdue LEFs for different slab thicknesses were compared with the corresponding AASHTO LEFs for single and tandem axle configurations. Figures 8 and 9 show the results of this comparison. Purdue LEFs were found to agree with the AASHTO LEFs for both single and tandem axle configurations.

Examples

A permit request is made for an overloaded truck having two single axles in addition to the steering axle. Each single axle will carry a load of 72 kips (32,668 kg). The LEF of the 72-kip single axle is determined for a pavement section consisting of a 10-in. (25.4-cm) concrete slab and a 4-in. (10.16-cm) granular base course. The truck was assumed to travel at a speed of 30 mph (50 km/hr).

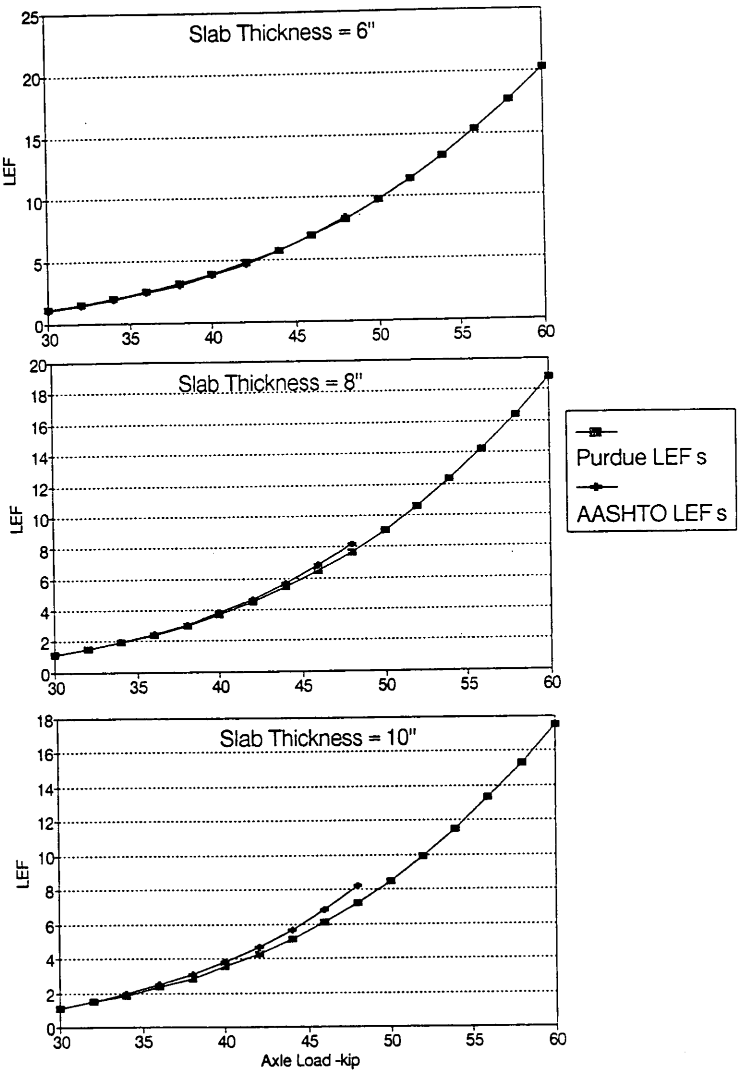
1. Total surface deformation for the 72-kip (32,668-kg) single axle load j and the 18-kip (8,167-kg) single axle load on section i:

$$\begin{aligned} \mathrm{TSD}_{ij} &= 2.98\mathrm{E}\text{-}6(72^4) + 0.3051(1) - 0.00877(10) \\ &\quad - 0.005678(30)(1) - 3.3\mathrm{E}\text{-}9(30)(72^4) \\ &\quad + 5.5\mathrm{E}\text{-}5(30)(30) \\ &= 77.52014 \text{ mils } (1.968 \text{ mm}). \end{aligned}$$



1 in. = 2.54 cm and 1 kip = 453.7 kg

FIGURE 8 Comparison between AASHTO LEFs and Purdue LEFs for single axle configuration.



1 in. = 2.54 cm and 1 kip = 453.7 kg

FIGURE 9 Comparison between AASHTO LEFs and Purdue LEFs for tandem axle configuration.

$$\begin{aligned}TSD_{18} &= 2.98\text{E-}6(18^4) + 0.3051(1) - 0.00877(10)\\ &\quad - 0.005678(30)(1) - 3.3\text{E-}9(30)(18^4) + 5.5\text{E-}5(30)(30)\\ &= 0.398996 \text{ mils } (0.0101 \text{ mm}).\end{aligned}$$

2. Number of 18-kip (8,167-kg) single axle loads required to develop the same total surface deformation from one pass of load j on cross section i (LEF_{ij}):

$$LEF_{72} = 1 + \frac{77.52014 - 0.398996}{0.17 + 0.00959(10)} = 454.654$$

The LEF of a 30-kip (13,612-kg) single axle load on the same pavement section is as follows:

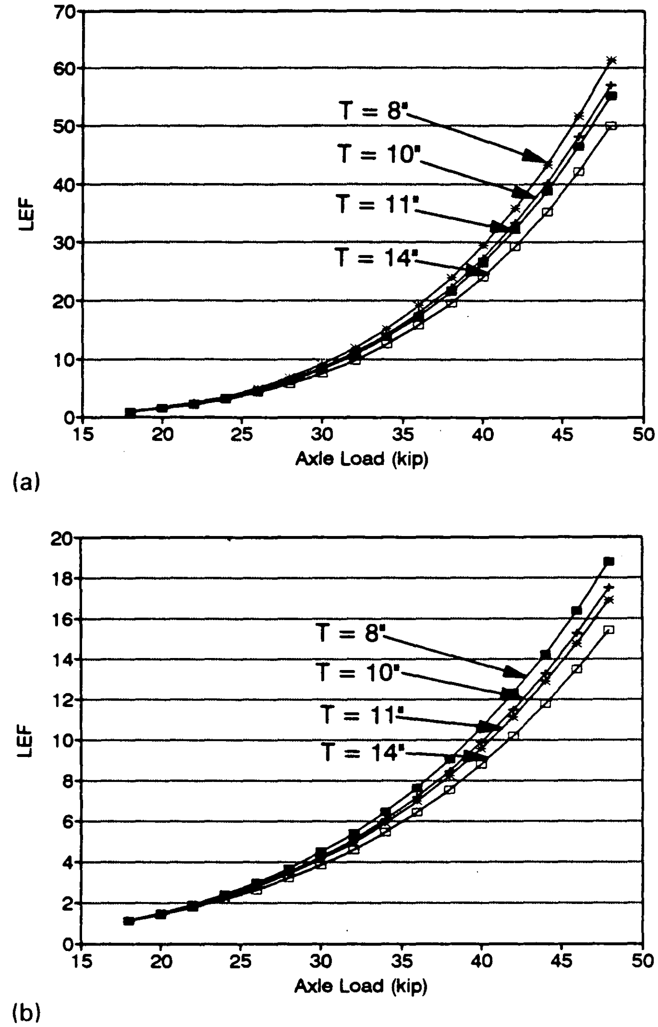
$$\begin{aligned}TSD_{30} &= 2.98\text{E-}6(30^4) + 0.3051(1) - 0.00877(10)\\ &\quad - 0.005678(30)(1) - 3.3\text{E-}9(30)(30^4) + 5.5\text{E-}5(30)(30)\\ &= 2.43017 \text{ mils } (0.0617 \text{ mm})\end{aligned}$$

$$LEF_{30} = 1 + \frac{2.43017 - 0.398996}{0.17 + 0.00959(10)} = 12.9$$

The corresponding AASHTO LEF is 12.17.

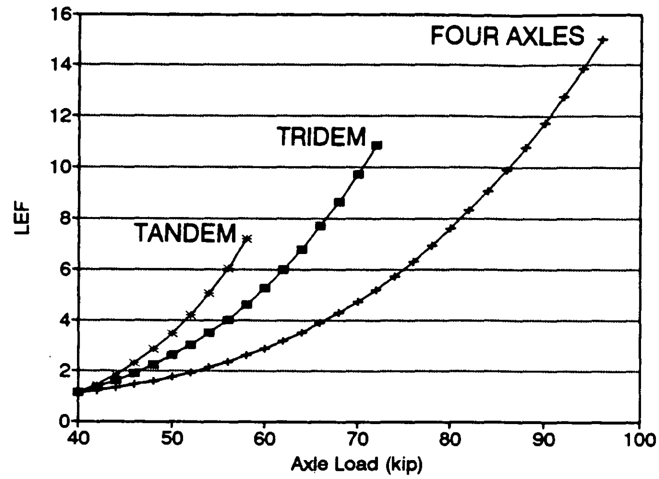
Sensitivity Analysis

The effect of slab thickness on LEFs is shown in Figure 10 for single and tandem axle configurations. With an increase in slab thickness the pavement damage because of the loads decreases and therefore the LEFs decrease. As can be seen from Figure 10 the LEFs decrease as slab thickness increases, as expected. The same is true for the AASHTO LEFs for thin slabs [6 to 8 in. (15.24 to 20.32 cm)]. For thicker slabs [>8 in. (20.32 cm)] the AASHTO LEFs increase with an increase in slab thickness. This could be associated with surface defects such as spalling and joint faulting.



1 in. = 2.54 cm and 1 kip = 453.7 kg

FIGURE 10 Effect of slab thickness on Purdue LEFs: (*a*) single axle configuration and (*b*) tandem axle configuration.



1 in. = 2.54 cm and 1 kip = 453.7 kg

FIGURE 11 Effect of number of axles on Purdue LEFs.

The effect of the number of axles on LEFs is shown in Figure 11. As expected for the same axle group load magnitude, the LEFs decrease with the increase in the number of axles.

Advantages of Purdue LEFs

Purdue LEFs can be considered serviceability-based LEFs. The reason there is such a good correlation between the AASHTO LEFs and Purdue LEFs is that subgrade permanent deformation is related to the potential accumulation of roughness, and roughness is a large component of serviceability. Purdue LEFs have the following advantages:

1. Purdue LEFs are based on dynamic analysis in which moving loads at different speeds were considered. Also realistic material properties and models were inlcuded in the analysis.
2. The 3D-DFEM analysis used to develop these LEFs has been verified for static, linear elastic analysis and dynamic, nonlinear analysis.
3. The concept presented here is different from those concepts that assume that the pavement response is a linear function in the number of load applications.
4. Both elastic and plastic deformations were included in the analysis. This cannot be done using a closed-form solution, which predicts the elastic pavement response only. Regardless of how accurately the elastic response is predicted, these methods do not provide a comprehensive measure of pavement performance.
5. Purdue LEFs are based on an analytical model. As a result they can be updated or extended to cover other factors not already included.

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