Extension of Load Equivalency Factors for Various Pavement Conditions

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Analyses of the AASHO Road Test results to derive load equivalency factors (LEFs) as a function of pavement condition (present serviceability index [PSI]) were performed. The results of the analyses show that the LEFs are strongly dependent on the pavement condition, i.e., LEF values increase as the initial PSI decreases. This result may affect the analyses of legal load limits and special movement of heavy trucks. To adapt the above results to multiple loads, a framework based on limiting the strain at the top of the subgrade and various strain criteria for various PSI of the pavement were developed and are presented. A modification to the common Miner's law for damage accumulation was calibrated for tandem axle loads and suggested for use with multiple wheel axles. The use of the proposed framework is illustrated for single, tandem, and multiple axle loads. The results showed that the LEFs of all load configurations are dependent on the condition of the pavement.

State officials who have the responsibility for considering and changing the legal load limits are continuously faced with requests for increasing loads on a specific truck or for allowing heavier loads on new trucks. Based on economic analyses, these requests seem legitimate, in Israel, but there is a strong feeling that the formal result is misleading. The analyses are usually conducted using AASHO Road Test load equivalency factors (LEFs) obtained from newly constructed pavements, but the network on which these loads are applied is not new. In fact, in Israel the network is in rather bad condition, with an average pavement condition index lower than 50. Therefore, the validity of these analyses using AASHO LEFs is questionable. Increasing the LEFs reduces the optimal load to be allowed on the truck.

Highway agencies are also often asked to issue special permits to allow special tractors and trailers to haul very large and heavy machinery for power plants or structural components for bridges. To minimize the damage to pavements and bridges, these tractors and trailers are equipped with numerous axles and tires. Greer (*1*) and Terrel and Mahoney (*2*) analyzed pavements and evaluated their structural capacity to support trailer units with 192 wheels (12 axles with 16 wheels per axle). Kilareski (*3*) presented a study of the potential pavement damage caused by heavily loaded units with four and five axles. The approach to the problem of heavy loads and requests for increasing legal loads is straightforward. Analyses of stresses or strains that develop under these loads can be conducted to evaluate the number of allowable repetitions for the specific loading configuration (*4–6*). Alternatively, the pavement method currently used by the state can be used or adapted to design the road upgrading required to permit the movement of heavy loads (*1*).

In the case studies reported in the literature (*1,2*), although testing programs and analyses were conducted for the specific haul routes, the authors did not consider directly the pavement condition as a variable. If the effect of load magnitude on pavement performance depends on the condition of the pavement, it would imply that various LEFs correspond to various pavement conditions (for the same pavement structure). Such dependence could be attributed to the dynamic effect induced by roughness, resulting in relatively heavier loads as the pavement condition deteriorates, or to nonlinearity in the damage accumulation (Miner's law).

This paper presents a derivation of LEFs based on AASHO Road Test results. The analyses show clearly that LEFs are influenced by the pavement condition, i.e., on the serviceability index at the time of application of the load. A framework is presented to adapt the derivation of LEFs to accommodate heavier loads and a larger number of wheels than in the AASHO Road Test. It is based on computation of vertical strain at the top of the subgrade and calibration with AASHO Road Test results. An illustration of the approach is presented using several load magnitudes and wheel configurations.

DERIVATION OF LOAD EQUIVALENCY FACTORS USING AASHO ROAD TEST RESULTS

The derivation is based on the well-known definition of LEF and AASHO Road Test performance equations:

$$F_j = W_e/W_j \tag{1}$$

where

F_j = LEF for load L_j,
W_j = number of repetitions for load L_j, and
W_e = Equivalent number of repetitions of a reference load L_e (usually taken as 18 kip).

$$W_k = \zeta_k 10^{G/\beta_k} \tag{2a}$$

$$\zeta_k = \frac{10^{5.93}(SN+1)^{9.36} L_2^{4.33}}{(L_1+L_2)^{4.79}} \tag{2b}$$

$$G = \log\frac{4.2 - p_t}{4.2 - 1.5} \tag{2c}$$

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$$\beta_k = 0.40 + \frac{0.081\,(L_1 + L_2)^{3.23}}{(SN + 1)^{5.19}\,L_2^{3.23}} \tag{2d}$$

where

$SN = a_1d_1 + a_2d_2 + a_3d_3$,
L_1 = axle load, in kip,
L_2 = 1 for a single axle, 2 for a tandem axle,
SN = structural number,
p_t = terminal serviceability number,
a_1,a_2,a_3 = layer coefficients,
d_1,d_2,d_3 = layer thicknesses, in in., and
k = index (may be either e or j).

The well-known AASHO LEFs are obtained from the above equations for various terminal serviceability indices (p_t). These factors are related to the pavement damage that begins at a unique initial pavement serviceability index (PSI) of about 4.2 (average of all initial serviceability indices in the AASHO Road Test). If the load equivalency is influenced by the pavement condition, then the usual LEFs are average values for the service life of the pavement, from the time of its construction with an initial PSI of about 4.2 to the time it reaches its terminal PSI. To deal with various initial conditions, the LEF should be redefined as

$$F_j(p_i \rightarrow p_t) = W_e(p_i \rightarrow p_t) \,/\, W_j(p_i \rightarrow p_t) \tag{3}$$

where p_i is the initial serviceability index and $p_i \rightarrow p_t$ denotes pavement condition deterioration from p_i to p_t. When p_i = 4.2, Equation 3 simplifies Equation 1 corresponding to the usual definition of LEF. When p_i is a value other than 4.2, the number of repetitions should be computed from the performance equations (Equation 2) as follows:

$$W_k(p_i \rightarrow p_t) = W_k(4.2 \rightarrow p_t) - W_k(4.2 \rightarrow p_i) \tag{4}$$

where k may be either j or e.

Figure 1 shows the components of Equation 4 using the performance curve. When referring to an initial serviceability index (p_i) lower than 4.2, it is assumed that this lower p_i is because of deterioration from past traffic and not because of bad construction. Results of computations of LEFs for four different load magnitudes and configurations (SN = 3 and various p_i and p_t's) are shown in Figures 2 through 5. The dotted lines in the diagrams correspond to the usual LEFs with p_i = 4.2. From these figures, it is possible to get F_j for any p_i and any p_t. For example, from Figure 2 one can get $F_j(4.0 \rightarrow 2.5)$ = 17.0 as compared with $F_j(4.2 \rightarrow 2.5)$ = 8.0 or $F_j(3.5 \rightarrow 2.5)$ = 22.4. F_j values for lower p_i (for example $F_j[2.0 \rightarrow 1.5]$ = 35.3) are very large compared with the value of 10.4 obtained previously. This example emphasizes the importance of pavement condition on LEFs. It seems that F_j is influenced slightly by the structural number, moderately by the terminal serviceability, load magnitude, and configuration, and highly by the initial serviceability. The results of the AASHO Road Test support the hypothesis that LEFs depend on the initial serviceability.

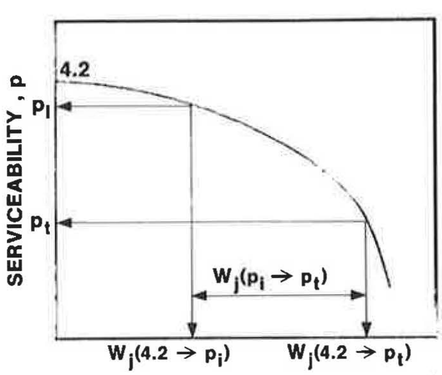

FIGURE 1 Schematic representation of load repetition increment on performance curve.

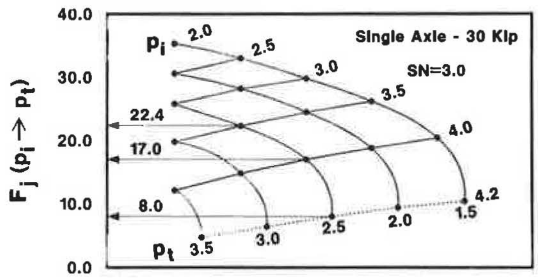

FIGURE 2 Load equivalency factors for 30-kip single axle load as function of PSI.

EXTENSION OF THE LOAD EQUIVALENCY FACTOR TO MULTIPLE WHEEL LOADS

The approach for computing LEFs from AASHO performance equations is restricted to the single and double axle loads that traveled on the test sections. To extend the approach to various axle configurations, a semirational (semiempirical) framework is suggested. This framework is less restrictive than the empirical one. It consists of

1. Stress-strain computation and material characterization;
2. Calibration of a failure criteria using all the available test results; and
3. Validation and implementation of the framework to the unusual loads.

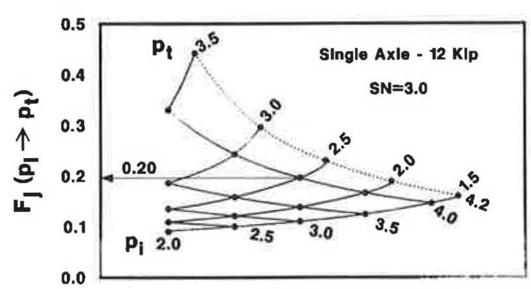

FIGURE 3 Load equivalency factors for 12-kip single axle load as function of PSI.

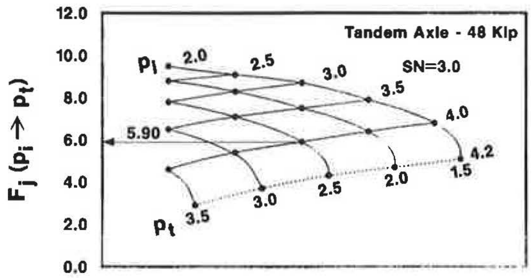

FIGURE 4 Load equivalency factors for 48-kip tandem axle load as function of PSI.

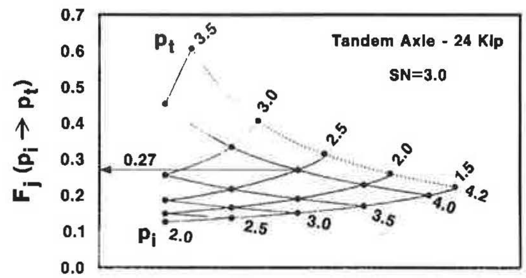

FIGURE 5 Load equivalency factors for 24-kip tandem axle load as function of PSI.

Unless nonlinear material characterization is used, the stress level induced by the unusual loads should not exceed (or should exceed only slightly) the stress level that developed in the test. For simplicity, the proposed framework is based on the linear elastic theory. It could be upgraded by including nonlinear material characterization and analyses.

Development of the Framework

The steps for developing the semirational framework are as follows:

1. *Pavements*—Seven pavement sections of the AASHO Road Test are chosen for analyses (Table 1). The ranges of the layer thicknesses and structural numbers embrace a wide variety of pavements serving from light to heavy traffic loads.
2. *Loads*—All five single axle loads (6, 12, 18, 22.4, and 30 kip) that traveled in the test sections are included in the calibration analyses. In this case, each axle is equipped with two dual wheels on each side; the distance between dual wheels is 14 in., and the contact pressure is 70 psi.
3. *Material properties*—The moduli of elasticity of the subgrade and the asphalt concrete are assumed constant and equal to 5,700 psi (corresponding to a CBR value of 3–4) and 450,000 psi, respectively. The modulus of elasticity of the granular subbase and base material is assumed to depend on both the layer thickness and modulus of the underlying layer. The equations used are those of the USACE granular material characterization for roads (see Smith and Witczak [7]).

$$E_{sub} = E_{sg}\,(1 + 1.5\,H_{sub}/20.0) \tag{5a}$$

$$E_b = E_{sub}\,(1 + H_b/6.0) \tag{5b}$$

where

E_b, E_{sub}, and E_{sg} = the base, subbase, and subgrade layer moduli, respectively, and
H_b and H_{sub} = the base and subbase layer thicknesses, in in.

4. *Analyses*—Vertical strain at the top of the subgrade is computed for all seven pavement sections, applying all five loads. The computations are made using a microcomputer program for the linear elastic multilayer system (*8*).
5. *Strain criteria*—The number of load applications is computed using Equation 2 for the above-mentioned pavement sections and loads, at three different serviceability indices p_t of 1.5, 2.5, and 3.5. The relationships between the number of load repetitions and computed vertical strain are shown in Figure 6. Various strain criteria are obtained for the various terminal serviceabilities. In addition, when the number of load repetitions is lower than 20,000, the relationship between vertical strain and load repetitions is independent of p_t and begins to curve. By using the number of load repetitions computed from Equation 2 rather than the actual number of load applications, the scatter in Figure 6 is reduced substantially. This reduction is because most of the randomness of the number of load applications is eliminated or filtered out in the regression of Equation 2. The lines in Figure 6 are represented by the following equations (for more than 20,000 load applications):

$$\varepsilon_v = 5.00 \times 10^{-2}\, W_j^{-0.36} \tag{6a}$$

$$\varepsilon_v = 1.99 \times 10^{-2}\, W_j^{-0.261} \tag{6b}$$

$$\varepsilon_v = 1.47 \times 10^{-2}\, W_j^{-0.228} \tag{6c}$$

for p_t = 3.5, 2.5, and 1.5, respectively, where ε_v is the amount of maximum vertical strain at the top of the subgrade and W_j is the number of load applications causing deterioration of the pavement from p_i = 4.2 to p_t. Only single axle loads are

TABLE 1 DESCRIPTION OF PAVEMENT SECTIONS USED IN THE ANALYSES

Pavement	Thickness of (in.)			Structural
No.	Asphalt	Base	Subbase	Number
1	3	0	8	2.20
2	3	6	4	2.60
3	4	3	8	3.06
4	4	6	12	3.92
5	5	6	12	4.36
6	5	9	12	4.78
7	6	9	12	5.22

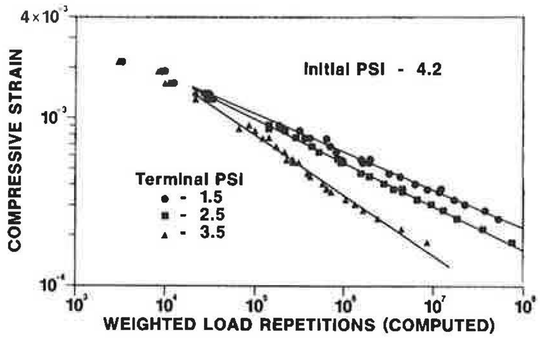

FIGURE 6 Vertical strain at top of subgrade as function of number of load repetitions and PSI for single axles.

used in the development of the above relationships (calibration stage). These relationships are valid only with the material characterization and analyses mentioned above.

The use of the framework for computing LEFs for a single axle is simple: $W_j(4.2 \rightarrow p_t)$ and $W_j(4.2 \rightarrow p_i)$, which are required in Equation 4, may be obtained from Figure 6. For example, to compute the LEF for a pavement with SN = 4.78, 30-kip single axle, p_i = 3.5, and p_t = 2.5, the following steps are necessary: (a) Compute the vertical strain under the load (under a 30-kip load, one obtains ε_v = 629 microstrains); (b) At this strain level, obtain from Figure 6 or Equation 6 the terms $W_j(4.2 \rightarrow 2.5)$ and $W_j(4.2 \rightarrow 3.5)$. At 629 microstrains, $W_j(4.2 \rightarrow 2.5)$ = 559,600 and $W_j(4.2 \rightarrow 3.5)$ = 190,000; (c) Compute $W_j(3.5 \rightarrow 2.5)$ from Equation 4: 559,600 − 190,000 = 369,600; (d) Repeat steps 1 through 3 with the standard load 18 kip to get $W_e(3.5 \rightarrow 2.5)$ = 3,178,600; (e) Compute the LEF using Equation 3 as 3,178,600/369,600 = 8.6. This value is in good agreement with the value of 9.5, obtained from Equation 2.

Calibration and Verification

The extension of the framework to tandem axles requires some clarification of the rational approach based on cumulative damage. For common pavement thicknesses, the strain distribution shows two clear peaks. These two peaks usually are considered as two separate repetitions, equivalent to dividing by two the number of repetitions obtained from Figure 6. However, this is correct only for thin pavements (or for large spacing between axles of the tandem) when the strain between the axles decreases to zero (full reversal of the strain), or practically to less than 10 percent of the peak strain. For thick pavements (or small spacing between axles of the tandem), the strain distribution may show only one peak. In this case, or practically when the strain between the axles is greater than 90 percent of the peak strain, each pass of the tandem is considered as one application of the peak strain. In reality, the strain between axles (located 60 in. apart) is nearly zero for thin pavements and reaches about half the peak value for commonly thick pavements. This situation calls for a correction of the Miner's damage accumulation law, especially for thick pavements. Such a correction is proposed below based on the AASHO Road Test results for tandem axle loads. For thin pavements, it was found that the strain between axles is near zero, suggesting that no correction is needed. Moreover the computations show that the peak strain under one axle of the tandem is lower than the peak strain under one single axle. The second dual-wheel axle in the tandem induces negative strain at large distances from the load. Such beneficial effect is questionable in flexible pavements because of the presence of highly nonlinear granular material.

To quantify the effect of the tandem axle, the pavement sections listed in Table 1 are analyzed under tandem axle loads (two dual wheels on each side of the axles) of 24, 32, 40, and 48 kip. The vertical strain at the top of the subgrade is computed under these loads, and the number of tandem axle load repetitions is obtained using the AASHO Road Test performance equations (Equation 2). The results are presented in Figure 7, superimposed on the strain criteria of the single axle. The dotted lines represent the strain criteria for tandem axles. It is seen that each pass of the tandem is equivalent to 1.6 passes of a single axle, except in the low range of number of repetitions in which each pass of the tandem is equivalent to about two passes of the single axle.

In view of the above, the following procedure for the analysis of tandem axles is suggested. The maximum strain and the strain between axles of the tandem are evaluated. When the strain between the axles is less than 10 percent of the maximum strain, the tandem axle is considered as two separate applications of the peak strain. When the strain between axles is greater than 90 percent of the maximum strain, the tandem axle pass is considered as only one application of the peak strain. In addition, when the peak strain computed with two dual wheels (on one side of the tandem axle) under or near one axle is smaller than the peak strain computed with one dual wheel (on one side of the single axle), the tandem axle pass is considered as two separate applications of one single axle. In other words, the peak strain obtained for one axle of the tandem, without the "beneficial effect" of the second axle is used to get W_j from Figure 6, and this number is divided by two. When the peak strain, computed with two dual wheels under or near one axle, is equal to or larger than the peak strain computed with one dual wheel (and the strain between the axles is greater than 10 percent and smaller than 90 percent of the peak strain), the tandem axle pass is considered as 1.6 applications. In other words, the peak strain under the two dual wheels is used to get W_j from Figure 6,

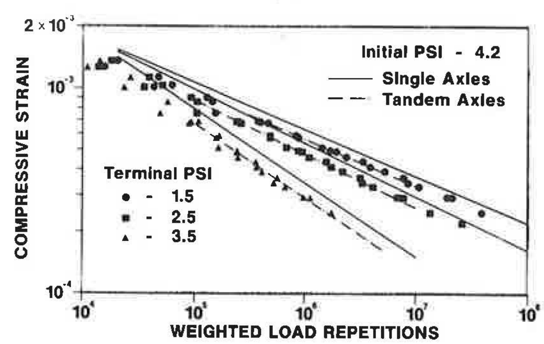

FIGURE 7 Vertical strain at top of subgrade as function of number of load repetitions and PSI for tandem axles.

and this number is divided by 1.6. LEF values computed using the above proposed approach are shown in Figure 8 and compared with those computed from AASHO Road Test-based equations (Equation 2). It is seen that the results are dispersed within a maximum deviation of 20 percent and that they agree very well.

ILLUSTRATION OF THE PROPOSED APPROACH TO MULTIPLE WHEEL LOADS

The extension of the proposed approach to multiple wheel loads is based on the assumption that the "failure" criterion derived for two axles holds also for more than two axles. In the case of multiple axle loads, calibration factors of 0.8 for leading axles and 0.6 for internal axles are suggested. These factors are found as follows. (a) The symmetric strain distribution under a tandem axle that corresponds to 1.6 repetitions of a single axle can be separated into two incomplete cycles (see Figure 9). Because a tandem axle is composed of two leading axles, each axle induces 0.8 (1.6/2) of the damage caused by a single axle. Therefore a calibration factor of 0.8 corresponds to each leading axle. (b) The strain distribution under a tandem axle can be viewed as composed of one complete cycle and one incomplete cycle in the central portion between peaks of the strain distribution (see Figure 10). This central part contributes 0.6 (1.6–1.0) of the damage caused by a tandem axle. Hence a calibration factor of 0.6 is suggested for each internal axle. At this time, no test results exist for calibrating the above factors. However, the above procedure is similar to the Curvature Method presented by Treybig (*5*).

The proposed procedure for computing W_j for multiple axle loads is summarized as follows. When the peak strain computed with all dual-wheels of the n – axles is smaller than the peak strain computed with only one dual-wheel, the multiple axle pass is considered as n separate applications of one single axle. The peak strain obtained for one axle (without the "beneficial effect" of the other axles) is used to get W_j from Figure 6, and this number is divided by n. When the peak strain computed with n dual wheels is equal to or larger than the peak strain computed with one dual wheel, then W corresponding to each peak strain is obtained from Figure 6. These numbers are divided by either 0.8 or 0.6, depending

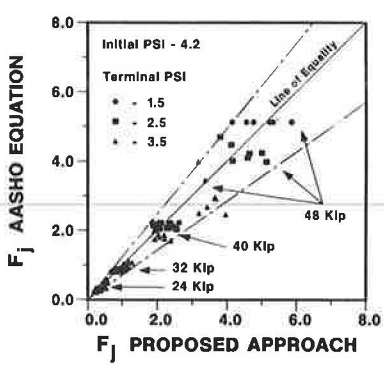

FIGURE 8 Comparison of LEFs obtained using the proposed procedure and AASHO equations for 24, 32, 40, and 48-kip tandem axle loads.

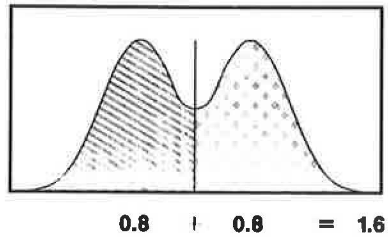

FIGURE 9 Separation of strain distribution of a tandem into two incomplete cycles.

on the position of the peak in the strain distribution. Then an average number of applications is computed on the basis of equal damage.

Tables 2 through 4 summarize results of computation for a very heavy tandem axle (79.2 kip), a four-axle load (with 8.5 kip per wheel) described by Kilareski (*3*), and 24-wheel trailer (with 13.86 kip per wheel), respectively. The 24-wheel configuration, which is composed of six axles with four wheels 36 to 40 in. apart, is different from the other configurations that have dual wheels (two close wheels on each side of the axle). In the cases of heavy loads, only pavements with a structural number larger than 3.5 are considered. The results of the heavy tandem (Table 2) are also shown in Figure 11 and compared with those obtained using Equation 2 (extrapolation of the AASHO Road Test results). It is seen that in all cases, the LEF values increase as the initial serviceability decreases. The LEF values at p_i of 2.5 are about twice as large as the usual LEF values at p_i of 4.2. The effect of pavement conditions under heavy loads on LEF values is similar to, but milder than, the effect obtained with loads that traveled in the AASHO Road Test.

SUMMARY AND CONCLUSIONS

The decision whether or not to change the legal load limits and to issue special permits for extra heavy loads is based on several factors and an economic analysis that considers all costs and benefits that are affected by such a change in legal load limits or special movements. For evaluation of construction and maintenance costs, the use of LEFs is well established. These factors are related to newly constructed pavements, whereas any change in the legal limits takes place at a given pavement condition of the network. The paper presents analyses of the AASHO Road Test results and derives LEFs as function of the pavement condition (PSI). The results of the analyses show that the LEFs are strongly affected by

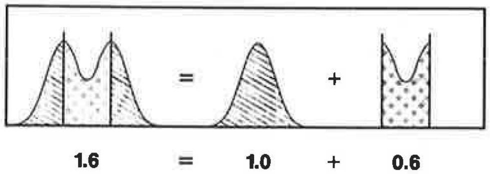

FIGURE 10 Separation of strain distribution of a tandem into one complete cycle and one incomplete cycle.

TABLE 2 RESULTS OF LEF COMPUTATIONS FOR 79.2-KIP TANDEM AXLE

	Initial Serviceability					
	4.2			3.5		2.5
Terminal Serviceability	3.5	2.5	1.5	2.5	1.5	1.5
SN=3.92	15.5	33.8	50.6	65.4	84.0	107.2
SN=4.36	15.9	35.1	52.8	56.9	74.9	94.9
SN=4.78	13.6	30.6	46.7	44.2	59.8	75.2
SN=5.22	14.3	33.0	50.8	44.9	61.9	77.3

TABLE 3 RESULTS OF LEF COMPUTATIONS FOR FOUR-AXLE LOAD

	Initial Serviceability					
	4.2			3.5		2.5
Terminal Serviceability	3.5	2.5	1.5	2.5	1.5	1.5
SN=3.92	22.7	43.9	61.7	71.0	88.4	108.4
SN=4.36	22.8	44.3	62.4	63.3	80.1	97.4
SN=4.78	18.1	37.1	53.6	49.7	65.0	79.2
SN=5.22	20.3	43.4	64.3	56.4	75.8	92.6

TABLE 4 RESULTS OF LEF COMPUTATIONS FOR 24 WHEELS

	Initial Serviceability					
	4.2			3.5		2.5
Terminal Serviceability	3.5	2.5	1.5	2.5	1.5	1.5
SN=3.92	19.4	35.5	48.4	53.6	65.6	79.0
SN=4.36	22.4	43.1	60.4	61.1	77.2	93.6
SN=4.78	29.3	62.6	92.6	86.5	114.9	141.9
SN=5.22	35.5	81.7	125.7	110.9	152.8	190.7

the pavement condition, i.e., LEF values increase as the PSI decreases.

A framework for extending the above results to multiple loads is proposed. This framework is based on computation of the strain at the top of the subgrade and strain criteria for various PSI of the pavement. The use of the framework for single axles is illustrated. For tandem axles, it is shown that the damage accumulation should be modified to take into account the longitudinal strain profile. Finally, heavy loads and multiple wheel load configurations are analyzed and LEFs are computed. The results show that the dependence of LEF on pavement conditions is similar to and milder for multiple wheels than for single and tandem axles.

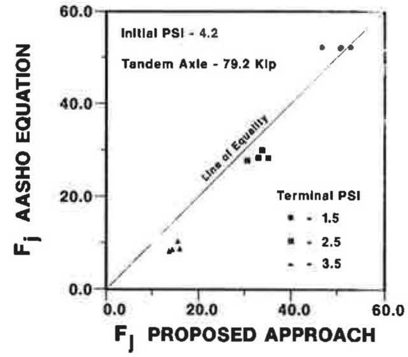

FIGURE 11 Comparison of LEFs obtained using the proposed procedure and AASHO equations for the 79.2-kip tandem axle load.

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