Pavement Strains Induced by Spent-Fuel Transportation Trucks

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Four types of vehicles are being considered for the transportation of spent-fuel casks to the high-level nuclear waste repository that is to be located in Yucca Mountain, Nevada. The use of a finite-layer moving-load model to compute the pavement strains is described. Pavement strains are required to compare the relative pavement damage caused by each of the spent-fuel trucks and to estimate the increased cost associated with the increase in maintenance and rehabilitation on pavements caused by the spent-fuel trucks. The strain response induced by the spent-fuel trucks for a site near Reno, Nevada, is reported. The asphalt concrete layer and the unbound materials are assumed viscoelastic and elastic, respectively. Pavement material properties were deduced from falling-weight deflectometer (FWD) testing. The study reveals that the strain response is affected strongly by the axle configuration and by the speed of the vehicle. Increased vehicle speed reduces the pavement strains substantially; longitudinal strains in the asphalt concrete layer decrease by as much as 33 percent when the speed of the vehicle increases from 30 to 60 km/hr. A substantial compressive strain component is also present when tandem and tridem axle loading are considered. The difference in contribution to pavement distress between the two legal-weight trucks and between the two overweight trucks is minimal. Laboratory fatigue and cyclic triaxial tests are being evaluated to compare the effects of legal-weight and overweight axle loading.

The Yucca Mountain area, located in the state of Nevada, is the only site being considered for storage of high-level nuclear spent fuel. Numerous studies are under way to assess the suitability of the site to store high-level nuclear spent fuel permanently. If the studies reveal no serious concerns, the repository is scheduled to receive spent-fuel elements from nuclear power plants early in the next century. A number of transportation modes are being appraised for transporting spent fuel from nuclear power plants to the site (1). One mode would be to use existing highways and specially designed trucks. Some of these trucks are overweight (heavier than the legal weight).

Both construction of the repository and transportation of spent fuel to the repository are expected to contribute to the deterioration of the existing highway pavements in Nevada. One objective of a study now under way at the University of Nevada, Reno, is to estimate the cost of maintaining pavements that might be affected by repository-related traffic to Yucca Mountain. In an attempt to quantify the cost, it is necessary to estimate the contribution of the repository-related traffic to the increase in maintenance and rehabilitation required to keep the pavement serviceable above some critical level.

Repository-related traffic's contribution to pavement distress or deterioration may be predicted using either empirical or mechanistic analyses (2-4). Empirical methods are based on AASHO road tests carried out in the 1960s. Mechanistic methods are based on pavement performance models that require traffic-induced pavement response (mainly strains) to predict pavement distress. The mechanistic approach, which uses fundamental pavement material properties, is considered a more rational approach and therefore has been adopted for the University of Nevada, Reno, study. Major modes of pavement distress are fatigue cracking, rutting, low-temperature cracking, roughness, and debonding. Widely used pavement distress models relate fatigue cracking to the horizontal tensile strain at the bottom of the asphalt concrete (AC) layer and the rutting to the vertical compressive strains at the top of the subgrade (3-5).

The distress or performance indicators just identified are often computed using static loading conditions, assuming either linear or nonlinear material characterization. Recently, Zafir (6-8) outlined an efficient finite-layer moving-load model that can predict pavement strains. The model accounts for the important dynamic effects of the moving load, such as inertia, damping (material and radiation), and wave reflection, and also presents the rate-dependent (viscoelastic material) properties. The AC layer is treated as viscoelastic, whereas the base and the subgrade are treated as linear elastic.

This paper describes the application of the aforementioned moving-load model to compute pavement strains caused by four types of trucks that are to be used in spent-fuel transportation. One approach to deducing viscoelastic material (required for the model) from falling-weight deflectometer (FWD) data is also presented.

EXISTING METHODS FOR PAVEMENT STRAINS

Moving traffic wheel loads exert dynamic forces on pavement; therefore, effects such as inertia, damping (material and radiation), and resonance become important. The response of deposits to moving surface pressure loading has been studied by a number of researchers. Excellent summaries of the description and applicability of the methods have been presented by Werkle and Waas (9) and Siddharthan et al. (10).

Widely used pavement response computer models such as ELSYM5 and BISAR are based on layered-elastic theory. These models assume static loading conditions and single or multiple circular loaded areas that are fixed in location. This means that moving-load (dynamic) effects and the rate-dependent material properties have been neglected.

Pavement engineers have known the importance of the dynamic loading caused by the moving load for some time (3,4). A recent extensive field testing program sponsored by FHWA at the Pennsylvania State University test track has clearly indicated the influ-
ence of the dynamic nature of the wheel loading (11,12). In these studies two new pavement sections, representing a thin (0.15 m of AC over 0.2 m of crushed aggregate) and a thick (0.25 m of AC over 0.25 m of crushed aggregate) section were subjected to moving traffic loads.

The experimental program modeled a wide range of pavement response histories, such as vertical deflection and longitudinal strain at the bottom of the AC layer under moving truck loads (Figure 1). Testing revealed that pavement responses are influenced strongly by the speed of a vehicle and by its wheel load acting on the pavement. There was a reduction of as much as 70 percent in the maximum tensile strain in the AC layer when the speed of the vehicle increased from 32 to 80 km/hr. Furthermore, the study clearly demonstrated that the strain history response is a result of the complex dynamic interaction between adjacent wheel loadings (in the case of tandem configuration), resulting in a substantial compressive strain (as much as 57 percent of the maximum tensile strain) component in the AC layer. Widely used computer models, such as ELSYM5 and BISAR, cannot predict the observed responses. A dynamic moving-load model that accounts for the rate-dependent material properties is required for this purpose.

The most recent study to predict the entire strain history response is based on the three-dimensional finite-element model (13). Sousa and his colleagues at the University of California, Berkeley, are also working on the development of a moving-load model that is also based on the finite-element method (J. B. Sousa, personal communication, 1993). The rate-dependent material properties can be accounted for in these models. To obtain accurate results, the finite-element techniques require a relatively fine finite-element mesh to accommodate large strain gradients, and the discretization should include a substantial lateral extent to model the moving load. There is no doubt that the computational effort associated with such an undertaking will be substantial.

Sousa et al. reported on an analytical model for computing pavement strains subjected to stationary circular loaded plates (14,3). They developed a computer model, SAPSI, to compute the response of a viscoelastic or elastic-layered system in the frequency domain. The input of the layer properties are Young's modulus, Poisson's ratio, and damping ratio as a function of the excitation frequency. The load-time history on the loaded areas varies as a function of the velocity of the moving vehicle. Their approach is identical to one proposed by Sebaaly and Mamlouk (15).

However, these methods suffer from a major limitation in that they do not account for the true nature of the moving load. The assumption that a uniform pressure is present on the entire stationary loaded areas, even when the tire occupies only a part of the loaded area, is questionable. The assumption becomes more important, especially in the case of pavements, because the size of the loaded area and thickness of the AC layer are on the same order of magnitude.

**BRIEF OUTLINE OF MOVING-LOAD MODEL**

Zafr and Siddharthan (6) and Zafr et al. (8) reported on the formulation of a continuum-based, "finite layer" model to evaluate dynamic pavement strains subject to moving traffic load. Complex surface loadings, such as multiple loads and nonuniform tire-pavement interaction pressures, can be handled relatively easily because the method uses Fourier transform technique. The pavement layer system may be characterized as consisting of a number of viscoelastic or elastic horizontal layers (as many as necessary) with each layer characterized using a set of uniform properties. For viscoelastic materials, the Young's modulus, the Poisson's ratio, and the material damping vary as a function of excitation frequency. Laboratory tests to produce such relationships are available (3,14). Currently, the three-dimensional effects of the wheel loading are taken into account by the use of two special viscous boundaries (front and back) connected to the two-dimensional strip model shown in Figure 2. More details on the formulation of the model have been presented elsewhere (6,8).

**GOVERNING EQUATIONS AND SOLUTION SCHEME**

Figure 2 shows a horizontally layered pavement subjected to a moving traffic load at the surface. It depicts a modified plane strain model proposed by Lysmer and his coworkers (16,17). Forces acting on an element in the x direction are also shown. The element has a width B in the y direction equal to the width of the loading. The equations of motion at any point in the x and z directions can be written as follows:

\[
\begin{align*}
\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xz}}{\partial z} + \frac{2pV_x}{B} \frac{\partial u}{\partial t} &= -\rho \frac{\partial^2 u}{\partial t^2} \\
\frac{\partial \sigma_z}{\partial z} + \frac{\partial \tau_{xz}}{\partial x} + \frac{2pV_z}{B} \frac{\partial w}{\partial t} &= -\rho \frac{\partial^2 w}{\partial t^2}
\end{align*}
\]

where

- \(\sigma_x, \sigma_z\) = normal stresses (compressive) in the x and z directions, respectively;
- \(\tau_{xz}\) = shear stress;
- \(\rho\) = mass density;
- \(u, w\) = displacements in the x and z directions, respectively; and
- \(V_x, V_z\) = shear wave velocity.

Because the surface load moves at a constant speed c and the pavement layer properties do not vary in the horizontal direction,
any response, say, for example, the displacement \( u \) using Fourier transform, can be written as

\[
u = u(x - ct) = \text{Re} \sum_{n=0}^{N} U_n \exp[i \lambda_n (x - ct)]
\]

where

\( U_n \) = variation in \( u \) with only \( z \) for the \( n \)th harmonic,
\( \lambda_n \) = wave number,
\( N \) = number of harmonics considered, and
\( i = \sqrt{-1} \).

When responses are written in the form indicated in Equation 3, the derivatives with respect to \( x \) and \( t \) are simply

\[
\frac{\partial u}{\partial x} = i \lambda_n u, \quad \frac{\partial u}{\partial t} = -i \lambda_n u
\]

(4)

After expansion of the stresses \( \sigma_x, \sigma_z, \) and \( \tau_{xz} \) in terms of the displacements \( u \) and \( w \), and subsequent use of the simplification presented in Equation 4, it is possible to derive the following equation for \( U_n \) from Equations 1 and 2:

\[
\frac{d^4 U_n}{dz^4} + \left[ -2\lambda_n^2 + \frac{\bar{\rho} \lambda_n^2 (3 - 4\nu)}{2(1 - \nu)G} \right] \frac{d^2 U_n}{dz^2} + \left[ \lambda_n^4 - \frac{\bar{\rho} \lambda_n^4 (3 - 4\nu)}{2(1 - \nu)G} + \frac{\bar{\rho} \lambda_n^4 (1 - 2\nu)}{2(1 - \nu)G^2} \right] U_n = 0
\]

(5)

where \( G \) is the shear modulus, \( \nu \) is the Poisson’s ratio, and \( \bar{\rho} \) is given by

\[
\bar{\rho} = \rho + \frac{2i\sqrt{\pi p}}{\lambda_n B}
\]

The solution to the fourth-order ordinary differential equation above can be obtained using the method of characteristics.

The boundary conditions are as follows: First, at the surface, \( \sigma_z \) equals the applied traffic load, and \( \tau_{xz} \) equals zero. Second, at the bottom boundary, the displacements \( u \) and \( w \) are zero. At the interfaces between the layers, continuity relations in terms of displacement \( u, w, \sigma_z, \) and \( \tau_{xz} \) need to be satisfied. Finally, the responses from all of the harmonics are algebraically added to get the complete response.

A computer code DYNPAVE has been developed incorporating the steps above. This program can handle any number of layers with any type of load distribution at the surface. The higher the number of layers, the larger the computational effort. At present, the code is capable of incorporating frequency-dependent properties for the AC layer at the same time that the base and subgrade are treated as linear-elastic layers. There is no practical limit to the number of horizontal layers that can be considered by the program. The material characterization used for the AC layer in the proposed study is presented subsequently.

It was not necessary to consider all of the harmonics, because the contribution of the harmonics with large wave numbers \( (\lambda_n) \) is quite small. The computational effort associated with the proposed analysis can be reduced substantially by specifying a cutoff wave number above which the computation of the response is not required.

**PAVEMENT STRAINS FOR SPENT-FUEL TRUCKS**

**Configuration of Spent-Fuel Trucks**

Spent-fuel elements currently are stored near nuclear power plants, but they will need to be transported to the Yucca Mountain repository. Plans are to enclose the fuel elements in a cask and mount the cask on a flatbed truck. A proposed cask-trailer system is presented in Figure 3. A detailed study has identified several types of trucks, both typical and atypical, that might possibly be
used in the spent-fuel transportation to the repository (18). The four types of vehicles under consideration are

- Existing legal-weight truck (LWT1),
- Future legal-weight truck (LWT2),
- Existing overweight truck (OWT1), and
- Future overweight truck (OWT2).

Table 1 shows the payload, gross vehicle weight, axle configuration, and axle load distribution for each vehicle. The overweight trucks use a tridem axle, whereas the legal-weight trucks use a tandem axle. It may be noted that if overweight trucks are used, there will be fewer trips to the repository site. On the other hand, overweight trucks' speed on state routes and through mountainous terrain will be slower.

### Site and Material Characterization

The University of Nevada, Reno, and the Nevada Department of Transportation collected extensive FWD data for at least 4 years and during all four seasons on a variety of pavements located within the state. The data base consists of FWD measurements taken at 27 sites at 15.2-m intervals covering each test section, 305 m long. FWD testing using the Dynatest FWD model 8000 was carried out at four load levels, varying from 27 kN to as much as 90 kN (19).

Deflection basins obtained in the FWD tests were used with the MODULUS program to backcalculate layer moduli (20). Note that when using the MODULUS program the thickness of the subgrade layer is not required because the variable is treated as an additional unknown along with layer modulus values. The program is much more efficient than other backcalculation programs and yields reasonable results. Results from the program have been used to construct a resilient-modulus data base for all sites and all four seasons (19,21).

From this extensive data base, only the results corresponding to Site 24, which is located near Reno, Nevada, have been selected for the site-specific study reported in this paper. Pavement layer thicknesses obtained from coring and the average pavement layer resilient modulus values for summer were extracted from the data base and are shown in Figure 4 (19,21).

It may be noted that the proposed dynamic pavement response model is capable of handling viscoelastic characterization for the layers. Because only the AC layer exhibits strong frequency-dependent behavior, the viscoelastic layer characterization is used for the AC layer, and the base and the subgrade layers are assumed to be elastic. For the AC layer, the frequency-dependent resilient modulus must be deduced from the resilient moduli (elastic) back-calculated from FWD tests. A procedure adopted to achieve this is presented below.

Figure 5 indicates the load pulses applied during FWD testing. Two pulses are presented that have been normalized so that the shapes of the pulses can be seen clearly. The normalized Fourier transforms obtained for these pulses are represented in Figure 6. It is clear from Figures 5 and 6 that the dominant frequency range associated with the pulses is quite wide (up to 30 Hz). In other words, the backcalculated AC resilient modulus from FWD testing is, in fact, a representative value for this wide range of dominant frequencies.

Sousa and Monismith (22) studied the effects of different parameters on the resilient modulus of AC. The AC samples were tested under three different temperatures, 11, 25, and 40°C, and

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**TABLE 1 Cask and Vehicle Types and Weight Distribution**

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>Loaded Cask Weight, kN</th>
<th>Gross Vehicle Weight (GVW), kN</th>
<th>Axle Configuration and Loads, kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>LWT1: Legal Weight</td>
<td>200.0</td>
<td>331.4</td>
<td>Single-Tandem-Tandem 44.5 - 144.6 - 142.3</td>
</tr>
<tr>
<td>LWT2: Future Legal Weight</td>
<td>224.6</td>
<td>351.4</td>
<td>Single-Tandem-Tandem 48.9 - 151.2 - 151.2</td>
</tr>
<tr>
<td>OWT1: Current overweight</td>
<td>349.6</td>
<td>493.7</td>
<td>Single-Tridem-Tridem 53.4 - 209.1 - 231.3</td>
</tr>
<tr>
<td>OWT2: Future Overweight</td>
<td>355.8</td>
<td>516.0</td>
<td>Single-Tridem-Tridem 48.9 - 226.8 - 240.2</td>
</tr>
</tbody>
</table>
were subjected to sinusoidal cyclic axial and torsional loading with varying frequencies. They found that for AC the amplitude of the dynamic complex Young's modulus \(|E^*|\) is a function of temperature and frequency of the loading (Figure 7). Figure 7 illustrates that the dynamic Young's modulus for AC increases with an increase in frequency and decreases with an increase in temperature.

The steps employed to arrive at the frequency-dependent AC modulus can be summarized as follows:

1. Assume that the curves, \(|E^*|\) versus frequency \((f)\), given in Figure 7 are master curves. This means that the variation in \(\log|E^*|\) in summer \((T = 25^\circ C)\) at Site 24 will vary linearly with \(\log(f)\) as shown in Figure 8. Therefore, if \(E^*_1\), which is the value of \(|E^*|\) at \(f = 1\) Hz, is known, the entire variation of \(|E^*|\) with frequency can be defined. Then an equation for \(|E^*|\) at 25°C and at a frequency \(f\), (Figure 7) is given by the following:

\[
\log|E^*| = \log(E^*_1) + 0.165 \log(f) \tag{7}
\]

2. Assume a value for \(E^*_1\), and compute the axial strain \(\epsilon_a\) as...
where $P(t)$ and $p_n$ are the pulse loading (Figure 5) and the corresponding Fourier amplitude.

3. Compute the axial strain $\varepsilon_{\text{ax}}$ that corresponds to the resilient modulus derived from FWD analysis as

$$\varepsilon_{\text{ax}} = \frac{1}{E_{\text{AC}}}$$

where $E_{\text{AC}}$ is the resilient modulus given for AC by the FWD backcalculation. Note that unit pressure was used in the computations of $\varepsilon_f$ and $\varepsilon_{\text{ax}}$.

4. If the difference between $\varepsilon_{\text{ax}}$ and $\varepsilon_f$ is not within an acceptable limit (say, 5 µ), repeat Steps 2 through 4 with a new $E^*_s$ until convergence is achieved.

5. Once the convergence has been reached, the value of $E^*_s$ substituted into Equation 7 gives the variation of $|E^*|$ with the frequency for the AC layer at any site.

Convergence was reached for Site 24 (during summer) for $E^*_s = 8.3 \times 10^5$ kPa. Other material properties, such as a dynamic Poisson's ratio and the material damping, are assumed to be those reported by Sousa and Monismith (22).

**Results of Pavement Strains**

In pavement design, the contact area is determined by dividing the load on each tire by the contact pressure. In the literature the tire-pavement contact area often has been approximated by a rectangle ($0.4L \times 0.6L$) and two semicircles with a radius of 0.3$L$ as shown in Figure 9(a), in which $L$ is the total length of the loaded area. $L$ can be obtained following Yoder and Witczak (2) or Huang (3) as follows:

$$L = \sqrt{\frac{A_c}{0.5227}}$$

where $A_c$ is the contact area. Because the proposed approach can handle only a rectangular loaded area, one must arrive at an equivalent rectangular loaded area. The proposed equivalent rectangular area is indicated by dotted lines in Figure 9(a). The equivalent loaded area was obtained by selecting the length of the rectangle, such that the area of the rectangle equals $A_c$. Dual tires may be modeled by combining the two rectangular loaded areas as indicated in Figure 9(b). The tire pressure in all of the results reported here was assumed to be 861 kPa.

Although the program DYNPAVE can compute strain, stress, displacement, and acceleration at any point, only longitudinal strain response at the bottom of the AC layer is reported in this paper. A typical computed time history response of longitudinal strain for a tridem axle traveling at a speed of 60 km/hr is shown in Figure 10. Each tire in the axle was assumed to carry 19.3 kN, giving a total of 232 kN for the axle. There are three tensile axial strain peaks representing the three loaded axles. The maximum tensile and compressive strains are 435 and 168 µ.

Two observations similar to those made by Sebaaly et al. (11,12) in the field can be made: (a) the strain response has both tensile and a substantial compressive strain, and (b) the strain response is a result of complex interaction between adjacent wheels.

Figure 11 shows the maximum tensile and compressive AC strain and the vertical compressive subgrade strain induced by all four vehicles traveling at 60 km/hr. The tensile AC strains vary between 434 and 439 µ, whereas the compressive AC strains vary between 168 and 175 µ. The small variation in AC strains induced...
by the vehicles can be attributed to similar maximum load and
tire values for the vehicles (see Table 1). The maximum com-
pressive strain in the AC is as much as 40 percent of the maximum
tensile strain. Note that, even though the maximum AC strains
(tensile and compressive) may not be substantially different be-
tween tridem and tandem axle configurations, three and two strain
pulses are caused by these configurations, respectively. The influence
of the difference in the number of pulses on pavement dam-
age can be quite important, and it can be evaluated readily in the
laboratory using fatigue beam tests.

Vertical compressive strain in the subgrade varies between 698
and 720µ for all the vehicles. Higher strains are induced by over-
weight trucks. Unlike the AC strains, the axle configuration has
a somewhat higher influence on the magnitude of the subgrade
strain. However, because of the limited number of spent-fuel casks
to be transported, the increase in the magnitude of the subgrade
strain associated with the overweight trucks is not considered crit-
ical. However, given the different characteristics of the pulses gen-
erated by the tandem and tridem vehicles, their effect on the per-
manent deformation of the subgrade may be dissimilar.

The effect of vehicle speed on maximum tensile strain induced
in the AC layer is illustrated in Figure 12. Reduction in strain is
quite substantial in all cases (i.e., a decrease from 538 to 361µ
—a reduction of 33 percent—when the speed of the LWT1 ve-
icle increased from 30 to 90 km/hr).

On the basis of the results of the DYNPAVE analyses presented
in Figures 11 and 12, it can be concluded that axle configuration
and vehicle speed are the most critical factors. Various vehicles
considered here apply strain pulses with similar maximum amplitu-
dy yet different characteristics. Vehicle speed, however, is highly
significant in determining the level of strain induced on the pave-
ment. Therefore, in order to clearly identify the effects of various
types of trucks on pavement performance, the influence of these
factors must be considered.

It is evident from the results presented that the difference in
contribution to pavement damage by the two legal-weight trucks,
LWT1 and LWT2, is insignificant. This is also true for the over-
weight trucks, OWT1 and OWT2. It was pointed out earlier that
the overweight trucks with heavier loads and tridem axle config-
uration will be traveling more slowly than the legal-weight trucks.
Furthermore, it is customary for state departments of transporta-
tion to assign a speed limit to overweight trucks. On the other
hand, the overweight trucks will carry heavier loads, which will
reduce the number of trips they must make to the repository.
Therefore, comparison of the effects of the overweight and legal-
weight trucks is basically a choice between lower speed, more
pulses, and fewer trips and higher speed, fewer pulses, and more
trips. An effort to quantify these differences is under way at the
University of Nevada, Reno.

**SUMMARY AND CONCLUSIONS**

Four types of vehicles were identified as vehicles that may be
used to transport spent-fuel casks to Yucca Mountain in Nevada.
The increase in traffic from spent-fuel trucks traveling to the re-
pository will accelerate pavement deterioration in the state. One
of the major goals of a study under way at the University of
Nevada, Reno, is an estimation of the cost associated with the
additional maintenance and rehabilitation of roads serving reposi-
tory traffic.

To use mechanistic methods to evaluate pavement distress,
strains induced on the pavement must be measured. This paper
describes the use of a newly developed "finite-layer" moving-
load model to compute pavement strains. In the model, a pave-
ment layer system may be characterized as viscoelastic or as hav-
ing elastic layers. The related computer program, DYNPAVE,
can handle any number of layers with any type of load distribution at
the surface.

A site near Reno, Nevada, was tested, and the response of the
longitudinal strain in the AC layer and the vertical compressive
strain in the subgrade caused by all four types of spent-fuel trucks
were reported. The frequency-dependent material properties (vis-
coelastic) of the AC layer were deduced from the Fourier trans-
form of the FWD load pulse, based on the assumption that the
amplitude of the resilient modulus varies linearly with the loga-
rithm of the frequency. Dynamic tests on triaxial samples support
this assumption. The base and the subgrade were considered elas-
tic, and their resilient modulus values were deduced from the data
base of backcalculated moduli derived from FWD measurements.

The AC strain history results are similar to those reported by
Sebaaly et al. (11,12), who measured AC strains under a moving
semitrailer. The results reported in this paper indicate that (a)
the strain response is a result of a complex interaction between adja-
cent wheel loads, (b) a substantial compressive strain component
is present (as much as 40 percent of the tensile strain), and (c)
the strain response is affected strongly by the speed of the vehicle.

The maximum tensile strain in the AC layer induced by the
four trucks traveling at 60 km/hr varies between 434 and 439µ.
The study reveals that the magnitudes of two important pavement
strains (tensile strain in the AC and compressive strain in the sub-
grade) are similar for legal-weight vehicles considered in the
study. Therefore, similar pavement deterioration may be expected
from legal-weight trucks. Overweight trucks indicated the identi-
cal result. On the other hand, the characteristics of the strain
pulses generated by the overweight trucks will be quite different
from those generated by the legal-weight trucks because different
axle configurations are used in the vehicles. Laboratory fatigue
tests on AC beams and cyclic tests on subgrade soils are being
considered to quantify the deterioration associated with the strain
histories generated by tridem and tandem axle loading.

An increase in vehicle speed reduced the longitudinal strain in
the AC layer by as much as 33 percent when the speed increased

![FIGURE 12 Variation of maximum tensile strain in AC with speed of vehicle.](image-url)
from 30 to 60 km/hr. This result has serious implications. Legal-weight trucks may travel faster, and thus induce smaller strains. However, they would need to make more trips to the repository to deliver the same number of spent-fuel tanks. Research to quantify these factors and to evaluate trucks different effects on pavement deterioration is under way.

REFERENCES


Publication of this paper sponsored by Committee on Strength and Deformation Characteristics of Pavement Sections.