VEHICLE LOAD EFFECTS ON ALASKAN FLEXIBLE PAVEMENT STRUCTURES

by

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Vehicle-Load Effects on Alaskan Flexible Pavement Structures

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Falling Weight Deflectometer measurements from four Alaskan pavement section types and a reverse iterative technique, assuming a four-layer model pavement structure were used to calculate a dynamic elastic modulus for each layer as a function of load. The four-layer model pavement structures and fatigue criteria for the asphalt and base support layers were used to determine the relative damage potential for heavy loads on Alaskan pavements in terms of a comparative damage factor. For pavement categories and load ranges tested, the damage factor (ratio) between any two loads was found to be proportional to the axle weight ratio raised to the 2.2 power.

Calculate damage factors for tandem axle configurations were just twice as large as those for single axle configurations with the same per axle load. This indicates that Alaskan pavements are generally too thin to support a continuous deflection basin between multiple axles and that multiple axle loadings should be evaluated separately regardless of axle spacing.
ABSTRACT

Load deflection measurements from four Alaskan pavement section types and a reverse iterative technique, assuming a four layer model pavement structure, were used to calculate a dynamic elastic modulus for each layer as a function of load. The four layer model pavement structures and fatigue criteria for the asphalt and base support layers were used to determine the relative damage potential for heavy loads on Alaskan pavements in terms of a comparative damage factor (CDF). The damage factor was defined as

\[
CDF = \frac{N_T}{N_L},
\]

where \(N_T\) = the number of 18 kip single axle dual wheel loads to failure on a selected standard pavement, and \(N_L\) = the number of multiple axle dual wheel loads to failure with the total axle group load TL on a given strength pavement. Damage factors for an average strength Alaskan pavement caused by a given axle group can be represented by

\[
CDF = 3.5 \times 10^{-10} \ (TL)^{2.22} \ n/n^{2.22}
\]

where \(n\) is the number of equally loaded dual wheel axles.

Calculated damage factors for tandem axle configurations were just twice as large as those for single axle configurations with the same per axle load. This indicates that Alaskan pavements are generally too thin to support a continuous deflection basin between multiple axles, and that multiple axle loadings should be evaluated separately regardless of axle spacing.
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SUMMARY AND CONCLUSIONS

A method for evaluating the fatigue damage potential of heavy loads on Alaskan flexible pavements under late summer conditions was developed in this study. Pavement surface load-deflection measurements for four pavement strength categories, taken with a falling weight deflectometer, and a reverse iterative technique were used to estimate the dynamic elastic moduli for a four layer model pavement structure. Four pavement categories were selected from a deflection inventory of 13 roadway sites to represent strong, moderately strong, average strength and weak structural types. The model pavement structure consisted of a surface asphalt layer, base layer, sub-base layer and subgrade. The critical stresses and strains for the pavement structure at various applied wheel loads were calculated using a 4-layer elastic analysis incorporating the method of equivalent thicknesses. Fatigue criteria for the asphalt layer and the base support layers were used to determine the number of repetitions of a single or tandem axle dual wheel load with a total group load of TL before significant pavement damage occurred for each of the four pavement strength categories.

The relative damage produced by a given axle load on a given pavement to a reference axle load on a reference pavement was defined as

\[ \text{Comparative damage factor (CDF)} = \frac{N_r \text{ (18 kip, PA)}}{N_L \text{ (TL, PL)}} \]

where \( N_r \) = number of 18 kip single axle dual wheel loads on a selected standard pavement, PA, until fatigue damage occurs, and

\( N_L \) = number of multiple axle dual wheel loads, with a total axle group load of TL on a given strength pavement, PL, until fatigue damage occurs.
The following was concluded from the comparative damage factor analysis for heavy loads on Alaskan pavements:

1. For a given axle group loading and pavement, the number of loads to failure for the tandem axle configuration was estimated at virtually half the corresponding number for a single axle with a same total load; this was due to the very thin asphalt pavement.

2. Damage factors for the four pavement categories did not vary significantly between pavement types when \( PA = PL \) for each pavement category; these were well represented by the expression

\[
CDF = N_r \times \frac{18 \text{ kip, PL}}{N_L \times \text{TL, PL}} = 3.5 \times 10^{-10} \times (\text{TL})^{2.22} \times n^{0.22},
\]

where \( n \) is the number of axles in an axle group with total weight \( \text{TL} \).

3. The comparative damage factors for the four pavement types referenced to the average pavement depend on relative pavement strength as well as relative load level and can be expressed as:

(a) \( CDF = 1.95 \times 10^{-10} \times (\text{TL})^{2.041} \times n^{0.041} \) for strong pavements,

(b) \( CDF = 1.59 \times 10^{-9} \times (\text{TL})^{2.20} \times n^{2.20} \) for moderately strong pavements,

(c) \( CDF = 3.5 \times 10^{-10} \times (\text{TL})^{2.22} \times n^{0.22} \) for average strength pavements,

(d) \( CDF = 2.38 \times 10^{-11} \times (\text{TL})^{2.63} \times n^{2.63} \) for weak pavements.

Current practice does not differentiate between different strength pavements when determining the relative pavement damage caused by heavy loads. A damage factor estimate analogous to that used in other literature sources can be made by using the CDF for average pavements (Equation c) to describe relative damage without regard to pavement strength characteristics.
RECOMMENDATIONS

Damage factor calculations are primarily a function of the assumed pavement structure and fatigue damage failure criteria. Since the damage factor is a relative measure of fatigue damage, it is not necessary to have extremely accurate information about the pavement or its failure process. The analysis done in this study was based on estimates of pavement strains derived from field measurements and performance criteria developed from AASHTO road tests. The method can be used to provide preliminary information about the relative damage effects of heavy loads on Alaskan pavements during late summer conditions. Additional tests and analyses are needed to determine how the choice of performance criteria, pavement structure and environmental conditions affect damage factor calculations. These factors can be investigated by:

1. conducting field and laboratory tests to determine appropriate pavement failure criteria in terms of cycles of stress or strain, with a given period, to failure for a large variety of highway construction materials actually used in Alaskan roads,

2. conducting an analytical analysis of the relationship between pavement asphalt thickness and damage factors for selected base support structures,

3. conducting a damage factor analysis similar to that conducted in this study, using load-deflection data and an appropriate fatigue damage criteria, for spring thaw conditions.
VEHICLE LOAD EFFECTS ON ALASKAN FLEXIBLE PAVEMENT STRUCTURES

Introduction

Flexible road pavements are constructed to provide a functional surface for vehicular traffic, within given load limits, during the anticipated service life of the pavement. When vehicle loads exceed the design limits of the pavement, premature surface damage may result (premature implies that damage occurs before the projected life span of the roadway is reached). In Alaska, vehicles carrying extremely heavy loads have been allowed limited access to the highways during the late summer months over the past few years. These have, in some cases, resulted in single axle dual wheel loads in excess of 50,000 lbs acting on normal pavement surfaces. Available fatigue damage analyses for flexible pavements under heavy loads do not address such extreme loadings. Also, available analyses often assume pavement structures the same as those used in the American Association of Highway Transportation officials (AASHTO) or similar road tests which are not necessarily representative of Alaskan materials and construction (Havens et al., 1979; Uzan and Wiseman, 1979). The objective of this study was to use load-deflection measurements obtained from Alaskan pavements during late summer, in conjunction with an accepted mechanistic analytical model and AASHTO derived damage criteria for the pavement structures to develop a method for assessing the damage potential of heavy loads on Alaskan roads.

The assessment method was designed to provide Department of Transportation personnel (DOT/PF) with a means of evaluating the damage potential of heavy loads as compared to a standard single axle dual wheel loading of 18,000 lbs. This comparison was done by defining two damage factors.
The first was a measure of the relative damage potential of a given load to the standard load on the same pavement. The second compared given loads on different pavements to the standard load on a standard pavement. The second damage factor has not been used previously, but provides important information about the relative damage potential of heavy loads on different strength pavements compared to a selected standard pavement type.

METHODS

Damage Factor Analysis

Flexible pavement designs for heavy loads are primarily a function of traffic volume, material characteristics and the relative damage caused by different loads and load configurations. Traffic volume and material characteristics are generally known for a pavement section. The relative damage caused by heavy loads would, however, need to be determined to fully understand pavement behavior. Several different methods have been used in the past to estimate relative damage. These all begin by first determining the elastic behavior characteristics of the pavement. This includes knowing the number of layers that constitute the pavement structure, the dynamic elastic modulus and the Poisson's ratio for each layer. Stresses and strains in each pavement layer and at the interfaces between the layers due to a given load are then calculated using an elastic multi-layered pavement model. Once the strains are known throughout the pavement structure one or more damage criterion are used to determine the damage potential for the load. The procedure is repeated for different load configurations to find the relative "damage factors" for a given pavement. The definition for the damage factor depends somewhat on how fatigue damage criteria are
chosen. Haven's et al., (1979) used the concept of strain energy density to estimate fatigue damage. They calculated the average work strain at the various locations at the base of the top asphaltic layer for various load configurations. The computed work strain for a single axle dual tire group with an 18,000 lb single axle load was used as the base value. The damage factor was defined as the ratio of work strain at any given load to work strain for the 18,000 lb axle load. Uzan and Wiseman (1979) used the maximum shear stress at the top of the pavement subgrade as a damage criterion. The relative damage ratio was determined by translating a given load configuration into its equivalent single wheel load (ESWL), which causes a magnitude of shear stress equal to the original configuration. The ratio of the ESWL at a given load to a base ESWL value was used as the relative damage factor. Other fatigue damage criteria based on some function of stress, strain or deflection have also been used.

In this study a four layer elastic model was used to describe the pavement structure. An asphalt layer, base layer, subbase layer and subgrade of semi-infinite thickness were used. It was assumed that failure could occur in any one of the four layers. Consequently, two different damage criteria were used. The fatigue relationship for the asphalt layer was given by

\[(1) \quad \varepsilon < 0.00228 N^{-0.178},\]

and for the unbound materials in the base, sub-base and subgrade by

\[(2) \quad \sigma_z < 7.07 N^{-0.307} E/160 \quad \text{for } E > 160 \text{ or}\]

\[(3) \quad \sigma_z < 7.07 N^{-0.307} (E/160)^{1.16} \quad \text{for } E < 160,\]

3
where \( \epsilon \) is the tensile strain, \( \sigma_z \) is the normal stress, \( E \) is the dynamic elastic modulus and \( N \) is the number of cycles. The stress and modulus values are given in MPa so that \( \sigma_z = \sigma_z'/1.45 \times 10^2 \)

where \( \sigma_z' \) is in psi and \( E = E'/1.45 \times 10^2 \)

where \( E' \) is in psi. The allowable tensile strain for common asphalt concrete materials was determined from laboratory results and the allowable normal stress from an analysis of AASHTO road test and other subsequent data (Stubstad, 1983; Ullidtz, 1977).

A damage factor was defined in the following manner. The stresses and strains in the pavement were calculated for a given axle load configuration. Equations 1, 2 and 3 were then used to calculate the number of cycles until the allowable strain or stress accumulation in each layer was exceeded. The layer for which the number of cycles was least was taken as the critical layer. The number of cycles for this layer was then defined as the number of loads to failure for the given load configuration and pavement structure. A comparative damage factor was then defined as the ratio of the number of strain or stress cycles to failure for a single axle dual wheel 18,000 lb load to the number of strain or stress cycles to failure for a given axle group load, TL, on a given road, PL.

\[
(4) \quad CDF = \frac{N_r (18 \text{ Kip, PA})}{N_L (\text{TL, PL})}
\]

The number of 18,000 lb single axle dual wheel loads that can pass over an average strength pavement before fatigue failure occurs is \( N_r \) (18 Kip, PA). The number of loads to failure for a given load configuration is \( N_L \) (TL, PL). Equation 4 is a completely general damage factor definition and allows for comparisons between different load configurations and pavement structures.
Relative Damage Factors for Alaskan Pavements

Previously, damage factor analyses have been conducted on roadway pavement structures patterned after well-known pavement test programs, for example the AASHTO tests. The objective of this study was to develop damage factors that were specifically representative of Alaskan pavements. This was accomplished by measuring load-deflection basins for 14 different pavement sections in the Fairbanks area. The load-deflection data was used to determine the properties for a four-layered model pavement structure. Finally, the stresses and strains in the model pavement were calculated for given vehicle wheel load configurations and equations 1, 2, 3 and 4 used to determine damage factors. The calculations of the stresses and strains for given pavement structure and wheel load configurations and the number of loads to failure were done by Dynatest Consulting, Inc.

Seven different loads were applied at each test location using a falling weight deflectometer to generate pavement deflection basin curves. The FWD was designed to simulate the loading conditions of a dual wheel load. The theory and operation of the FWD have been described in detail by Ullidtz (1974) and will not be repeated here.

The load-deflection data were separated into four groups typified by sections 2, 3, 8 and 11 with each representing a different category of pavement strength. These were determined by visual examination of load-deflection basins and the pavement bulk dynamic modulus-load profiles. The bulk dynamic modulus was defined as the peak of the force impulse divided by the peak of the deflection response. This can be taken as a measure of the pavement stiffness which is more or less an average for the low frequency range (Ullidtz, 1974). Figures A1 through A4 in Appendix I show the deflection basins, bulk dynamic moduli and peak deformation
over the FWD load range 3,100 lbs to 23,400 lbs for the four pavement categories. The Geist road was strong, the Parks was moderately strong, the Parks 1 was of average strength and the Old Steese was weak.

Load-deflection basins are directly controlled by the pavement structure. Deflections immediately beneath the loading platten are due primarily to the combined compression and bending of the asphalt layer and all lower granular layers, while deflections further away are the result of deeper displacements. These FWD induced deflections may be modelled in a quasistatic fashion on which elastic layered solutions are based. Modelling the pavement structure as a layered elastic media provides a straightforward way to estimate the material properties for the pavement in terms of a dynamic elastic modulus. Once the material properties for a pavement are known, then the stresses and strains throughout the structure can be calculated and used to determine damage factors. Table I shows the structure for the pavement test sections. Test sections 2, 3, 8 and 11 and a reverse, iterative computer program describing a four layered pavement were used to calculate modulus values for each layer (Dynatest's ELMOD program). Modulus values for the surface asphalt layers were assigned since these are very difficult to calculate for thin layers (< 3 inches) using any reverse iterative technique. The modulus of the pavement sub-grade (the subgrade is the material upon which pavement structural layers are built) was determined using the deflection basin data and a relationship describing the non-linear elastic behavior of cohesive soils. The ratio between the moduli of the base and sub-base layers was held constant at 3. Assigning a constant modulus ratio between layer 2 and 3 greatly simplifies calculations and was found not to adversely affect the results. A few of the preliminary calculations of moduli values using the ELMOD pro-
<table>
<thead>
<tr>
<th>Test Section</th>
<th>Site Location</th>
<th>Surface Layer Thickness in Inches</th>
<th>Base</th>
<th>sub-base</th>
<th>Pavement strength category</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Farmers Loop Rd. by University of Alaska</td>
<td>3 in. AC**</td>
<td>3</td>
<td>6</td>
<td>average</td>
</tr>
<tr>
<td>2</td>
<td>Geist Road</td>
<td>3/4 BST*</td>
<td>6</td>
<td>6</td>
<td>strong</td>
</tr>
<tr>
<td>3</td>
<td>Parks Hwy. by Gold Hill</td>
<td>2 AC</td>
<td>6</td>
<td>8</td>
<td>average</td>
</tr>
<tr>
<td>4</td>
<td>Steese Highway Mile Post 10</td>
<td>1 1/2 AC</td>
<td>4 1/2</td>
<td>6</td>
<td>average</td>
</tr>
<tr>
<td>5 and 6</td>
<td>Elliot Highway Mile Post 27 and Chatanika River</td>
<td>2 AC</td>
<td>6</td>
<td>4</td>
<td>moderately strong</td>
</tr>
<tr>
<td>7</td>
<td>Richardson Highway</td>
<td>1 1/2 AC</td>
<td>4 1/2</td>
<td>6</td>
<td>strong</td>
</tr>
<tr>
<td>8 and 9</td>
<td>Old Steese, Parks Highway between Mileposts 8 and 9</td>
<td>3/4 BST*</td>
<td>6</td>
<td>6</td>
<td>weak</td>
</tr>
<tr>
<td>10 and 11</td>
<td>Parks Highway between Mile Post 292-293, 281-282</td>
<td>2 1/2 AC</td>
<td>6</td>
<td>6</td>
<td>moderately strong</td>
</tr>
<tr>
<td>12 and 13</td>
<td>Parks Highway Mile Post 262.4 - 245</td>
<td>1 1/2 AC</td>
<td>4 1/2</td>
<td>6</td>
<td>moderately strong</td>
</tr>
<tr>
<td>14</td>
<td>Parks Highway Milepost 231.7</td>
<td>1 1/2 AC</td>
<td>3</td>
<td>6</td>
<td>moderately strong</td>
</tr>
</tbody>
</table>

*BST - Bituminous surface treatment  
**AC - Asphalt concrete
gram were checked using a computer program that utilized the Cheveron n-
layer pavement model to calculate Young's modulus. These agreed with the
simplified four layer model technique primarily used in this study (Stubstad,
1983). The moduli values calculated using the ELMOD program are shown in
Table II. The base and sub-base modulus values generally increased with
increasing load. This stress sensitivity of the modulus values can be
described by the non-linear equation

\[ E = C \left( \frac{\sigma}{\sigma'} \right)^P \]

where \( E \) is the elastic modulus, \( C \) and \( N \) are constants and \( \sigma' \) is a refer-
ence stress usually set to 14.5 psi. The degree of stress sensitivity of
modulus values can be determined from the constant \( P \) (Table II). Only the
Parks section showed a significant modulus sensitivity to load. This
indicates that the subgrade is relatively cohesive on the Parks and rela-
atively non-cohesive on the Geist, Old Steese and Parks 1 pavement sections.

Once the moduli for the four typical pavement structures were determined
at each loading configuration, the stresses and strains were calculated
using Boussinesq's equations with the method of equivalent thicknesses,
Ullidtz (1974, 1977). Based on the magnitude of stress or strain in the
pavement structure, an estimate of loads to failure was determined from
equations 1, 2 and 3$ was used to define the onset of damage. Two techniques
were used to calculate the effect of various wheel load configurations on
different pavement structures. First the remaining lifetime of each
pavement section in terms of the number of passes of a particular dual or
dual tandem load was calculated for a present serviceability rating change
of one (\( \Delta \) PSR = 1). The critical layer of the pavement using equations
1, 2 and 3 was determined, that is the layer in which damage first occurs.
### TABLE II

Elastic modulus values and stress sensitivity of the moduli at the prevailing FWD test temperature found with the ELMOD program:

<table>
<thead>
<tr>
<th>Roadway</th>
<th>Stress Sensitivity</th>
<th>Approx. FWD Load</th>
<th>Surface Asphalt E&lt;sub&gt;1&lt;/sub&gt;</th>
<th>Base E&lt;sub&gt;2&lt;/sub&gt;</th>
<th>Sub-base E&lt;sub&gt;3&lt;/sub&gt;</th>
<th>Subgrade E&lt;sub&gt;m&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Geist Road</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>On</td>
<td>-0.05</td>
<td>3,100 lbs</td>
<td>1,249</td>
<td>102</td>
<td>34</td>
<td>27</td>
</tr>
<tr>
<td>[Thicknesses used =]</td>
<td>5,600 lbs</td>
<td>&quot;</td>
<td>119</td>
<td>40</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>1 in BST</td>
<td></td>
<td>8,100 lbs</td>
<td>&quot;</td>
<td>129</td>
<td>43</td>
<td>31</td>
</tr>
<tr>
<td>6 in Base</td>
<td></td>
<td>11,900 lbs</td>
<td>&quot;</td>
<td>165</td>
<td>55</td>
<td>39</td>
</tr>
<tr>
<td>6 in Sub-</td>
<td></td>
<td>16,200 lbs</td>
<td>&quot;</td>
<td>182</td>
<td>61</td>
<td>42</td>
</tr>
<tr>
<td>base. Test</td>
<td></td>
<td>20,900 lbs</td>
<td>&quot;</td>
<td>206</td>
<td>69</td>
<td>43</td>
</tr>
<tr>
<td>Temp = 61°F]</td>
<td></td>
<td>23,400 lbs</td>
<td>&quot;</td>
<td>216</td>
<td>72</td>
<td>43</td>
</tr>
<tr>
<td><strong>Old Steese</strong></td>
<td>-0.05</td>
<td>3,100 lbs</td>
<td>900</td>
<td>34</td>
<td>11</td>
<td>8</td>
</tr>
<tr>
<td>[Thicknesses used =]</td>
<td>5,600 lbs</td>
<td>&quot;</td>
<td>38</td>
<td>13</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>1 in BST</td>
<td></td>
<td>8,100 lbs</td>
<td>&quot;</td>
<td>40</td>
<td>13</td>
<td>6</td>
</tr>
<tr>
<td>6 in Base</td>
<td></td>
<td>11,900 lbs</td>
<td>&quot;</td>
<td>42</td>
<td>14</td>
<td>6</td>
</tr>
<tr>
<td>6 in Sub-</td>
<td></td>
<td>16,200 lbs</td>
<td>&quot;</td>
<td>46</td>
<td>15</td>
<td>6</td>
</tr>
<tr>
<td>base. Test</td>
<td></td>
<td>20,900 lbs</td>
<td>&quot;</td>
<td>48</td>
<td>16</td>
<td>6</td>
</tr>
<tr>
<td>Temp = 77°F]</td>
<td></td>
<td>23,400 lbs</td>
<td>&quot;</td>
<td>50</td>
<td>17</td>
<td>6</td>
</tr>
<tr>
<td><strong>Parks 1</strong></td>
<td>0.00</td>
<td>3,100 lbs</td>
<td>833</td>
<td>27</td>
<td>9</td>
<td>21</td>
</tr>
<tr>
<td>[Thicknesses used =]</td>
<td>5,600 lbs</td>
<td>&quot;</td>
<td>39</td>
<td>13</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>2 in AC</td>
<td></td>
<td>8,100 lbs</td>
<td>&quot;</td>
<td>41</td>
<td>14</td>
<td>15</td>
</tr>
<tr>
<td>6 in Base</td>
<td></td>
<td>11,900 lbs</td>
<td>&quot;</td>
<td>45</td>
<td>15</td>
<td>13</td>
</tr>
<tr>
<td>8 in Sub-</td>
<td></td>
<td>16,200 lbs</td>
<td>&quot;</td>
<td>48</td>
<td>16</td>
<td>14</td>
</tr>
<tr>
<td>base. Test</td>
<td></td>
<td>20,900 lbs</td>
<td>&quot;</td>
<td>53</td>
<td>18</td>
<td>14</td>
</tr>
<tr>
<td>Temp = 61°F]</td>
<td></td>
<td>23,400 lbs</td>
<td>&quot;</td>
<td>57</td>
<td>19</td>
<td>14</td>
</tr>
<tr>
<td><strong>Parks</strong></td>
<td>-0.20</td>
<td>3,100 lbs</td>
<td>833</td>
<td>29</td>
<td>10</td>
<td>33</td>
</tr>
<tr>
<td>[Thicknesses used =]</td>
<td>5,600 lbs</td>
<td>&quot;</td>
<td>33</td>
<td>11</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>2.5 in AC</td>
<td></td>
<td>8,100 lbs</td>
<td>&quot;</td>
<td>36</td>
<td>12</td>
<td>20</td>
</tr>
<tr>
<td>6 in Base</td>
<td></td>
<td>11,900 lbs</td>
<td>&quot;</td>
<td>55</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>6 in Sub-</td>
<td></td>
<td>16,200 lbs</td>
<td>&quot;</td>
<td>66</td>
<td>22</td>
<td>19</td>
</tr>
<tr>
<td>base. Test</td>
<td></td>
<td>20,900 lbs</td>
<td>&quot;</td>
<td>65</td>
<td>21</td>
<td>19</td>
</tr>
<tr>
<td>Temp = 60°F]</td>
<td></td>
<td>23,400 lbs</td>
<td>&quot;</td>
<td>71</td>
<td>24</td>
<td>19</td>
</tr>
</tbody>
</table>
The number of loads to failure for the pavement structure after a one inch overlay of asphalt had been applied was also calculated (Table III). The calculated number of loads to failure using the one inch overlay technique is significantly larger than from the remaining lifetime method. This is natural since the overlay would add strength to the pavement structure. This should have little effect on the damage factor calculations since they are determined by the ratio given in equation 4. The effect of the overlay was to smooth the calculated values for loads to failure. This can be seen by examining the regression coefficients (Table IV) that describe the results of the number of loads to failure calculations for single axle loads in the form

\[ N = (TL)^{1/B}/A^{1/B}. \]

The number of loads to failure is \( N \), TL is the total axle load; \( A \) and \( B \) are constants. The correlation coefficients for the remaining lifetime technique are all lower than for the one inch overlay method. In assessing both methods of arriving at the number of loads to failure (damage potential) the one inch overlay method was thought to provide the most reasonable results (Stubstad, 1983). The damage factor analyses presented in the following section therefore utilize the results of the one inch overlay method for determining damage potential.
TABLE III

Estimated Load Life of Pavement Structures

<table>
<thead>
<tr>
<th>Total group load</th>
<th>Geist Road On</th>
<th>Old Steese</th>
<th>Parks 1</th>
<th>Parks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Axle, Dual Wheel</td>
<td>(Number of Loads to Failure N)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6,300 lbs</td>
<td>1,438,000</td>
<td>40,070</td>
<td>267,000</td>
<td>(1,589,000)*</td>
</tr>
<tr>
<td>11,200 lbs</td>
<td>1,004,000</td>
<td>23,050</td>
<td>102,000</td>
<td>240,000</td>
</tr>
<tr>
<td>16,200 lbs</td>
<td>729,000</td>
<td>18,870</td>
<td>34,290</td>
<td>63,930</td>
</tr>
<tr>
<td>23,800 lbs</td>
<td>815,000</td>
<td>8,340</td>
<td>13,950</td>
<td>73,610</td>
</tr>
<tr>
<td>32,400 lbs</td>
<td>587,000</td>
<td>3,820</td>
<td>7,850</td>
<td>55,720</td>
</tr>
<tr>
<td>41,800 lbs</td>
<td>425,000</td>
<td>1,740</td>
<td>6,990</td>
<td>25,390</td>
</tr>
<tr>
<td>46,800 lbs</td>
<td>370,000</td>
<td>1,320</td>
<td>(8,730)*</td>
<td>32,280</td>
</tr>
<tr>
<td>Tandem Axle, Dual Wheel</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12,600 lbs</td>
<td>719,000</td>
<td>20,040</td>
<td>133,000</td>
<td>(794,000)*</td>
</tr>
<tr>
<td>22,400 lbs</td>
<td>502,000</td>
<td>11,530</td>
<td>51,190</td>
<td>230,000</td>
</tr>
<tr>
<td>32,400 lbs</td>
<td>365,000</td>
<td>9,430</td>
<td>17,140</td>
<td>31,970</td>
</tr>
<tr>
<td>47,600 lbs</td>
<td>407,000</td>
<td>4,170</td>
<td>6,980</td>
<td>36,800</td>
</tr>
<tr>
<td>64,800 lbs</td>
<td>293,000</td>
<td>1,910</td>
<td>3,930</td>
<td>27,860</td>
</tr>
<tr>
<td>83,600 lbs</td>
<td>212,000</td>
<td>870</td>
<td>3,490</td>
<td>12,690</td>
</tr>
<tr>
<td>93,600 lbs</td>
<td>185,000</td>
<td>662</td>
<td>(4,360)*</td>
<td>16,140</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Single Axle, Dual Wheel</th>
<th>(Number of loads requiring a one inch overlay of asphalt)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6,300</td>
<td>29,600,000 1,427,000 2,100,000 3,740,000</td>
</tr>
<tr>
<td>11,200</td>
<td>12,900,000 413,000 1,560,000 1,880,000</td>
</tr>
<tr>
<td>16,200</td>
<td>3,770,000 124,000 417,000 400,000</td>
</tr>
<tr>
<td>23,800</td>
<td>2,020,000 45,000 142,000 402,000</td>
</tr>
<tr>
<td>32,400</td>
<td>976,000 27,100 72,200 231,000</td>
</tr>
<tr>
<td>41,800</td>
<td>691,000 9,590 47,300 92,700</td>
</tr>
<tr>
<td>46,800</td>
<td>623,000 7,250 42,800 88,100</td>
</tr>
</tbody>
</table>

*Questionable values
### TABLE IV
Regression Coefficients for Equation 6 Describing Single Axle Dual Wheel Loads

<table>
<thead>
<tr>
<th>Roadway</th>
<th>Remaining Lifetime Technique</th>
<th></th>
<th>One Inch Overlay Technique</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>r²</td>
<td>A</td>
</tr>
<tr>
<td>Geist Road</td>
<td>$9.64 \times 10^{12}$</td>
<td>-1.48</td>
<td>0.93</td>
<td>$28.9 \times 10^6$</td>
</tr>
<tr>
<td>Old Steese</td>
<td>$2.47 \times 10^6$</td>
<td>-0.54</td>
<td>0.93</td>
<td>$1.36 \times 10^6$</td>
</tr>
<tr>
<td>Parks 1</td>
<td>$2.41 \times 10^6$</td>
<td>-0.47</td>
<td>0.98</td>
<td>$5.47 \times 10^6$</td>
</tr>
<tr>
<td>Parks</td>
<td>$31.8 \times 10^6$</td>
<td>-0.65</td>
<td>0.83</td>
<td>$14.4 \times 10^6$</td>
</tr>
</tbody>
</table>
RESULTS AND DISCUSSION

Damage Factors for Alaskan Pavement Structures

The relative damage potential of various load configurations on different pavement structures is described by equation 4. The results of the remaining lifetime to failure analysis shown in Table III can be used to estimate the influence of different axle configurations. These indicate that the calculated number of loads to failure for a tandem axle load configuration was virtually half the corresponding number for the single axle when the per axle loading remains constant. This is most likely due to the very thin asphalt concrete pavements on Alaskan roads which assume individual small radius surface deflection basins under each wheel set of multiple axle groups, even with interaxle spacings as little as 3 feet. The tandem axle consequently has the same effect as two single axle passes with each individual axle carrying half the total tandem axle load. The number of loads to failure for multiple axle loads can then be described by

\[ N_n = (T_{L_n})^{1/B} / (n^n A_1^{1/B_1})^{1/B}. \]

This equation assumes equal distribution of weight between axles within a given axle group. The number of loads to failure for a multiple axle load is \( N_n \), \( T_{L_n} \) is the total axle group load, \( n \) is the number of axles, \( A \) and \( B \) are the regression coefficients given in Table IV. For a one axle load configuration \( n = 1 \) and equation 7 becomes equivalent to equation 6.

An average strength pavement, Parks 1, was chosen as the reference pavement structure used in this study and equations 4 and 7 were used to develop the comparative damage factor for the pavements

\[ CDF = (18,000)^{1/B} A^{-1/B} A_1^{1/B_1} n^{(1+1/B_1)} (T_{L_n})^{-1/B_1}. \]
The standard axle load of 18,000 lb has been inserted into the formula. A and B are the regression coefficients for the standard pavement structure using the one inch overlay method. $A_1$ and $B_1$ are the regression coefficients for the pavement structure of interest and the total group load acting on the pavement structure of interest is given by $TL_n$. Equation 8 was used to calculate damage factors for each of the four pavement structures, that is for $A = A_1$ and $B = B_1$, with the number of axles varying from 1 to 6 (Figures 1, 2, 3 and 4). It is apparent that damage factors do depend slightly on the pavement structure. It is also apparent that multiple axle load configurations in which the axles are more than three feet apart do not reduce the damage caused by a given per axle loading. For example an 18,000 lb single axle, dual wheel load has a damage factor of one, while a tandem dual wheel load of 36,000 lbs and any normal axle spacing has a damage factor of two. Previous damage factor analyses have concluded that multiple axle loads should reduce the per axle damage for given axle loadings. That is, a single axle dual wheel 18,000 lb load would have approximately the same damage factor as a 36,000 lb tandem axle dual wheel load (Havens et al., 1979). These conclusions are primarily the result of assuming that the superposition of stresses and strains from the multiple axles increases the critical stress or strain values only slightly above the value for single axle loads (Figure 5A). The deflection basin, and consequently the critical stresses or strains would be only slightly greater than for the individual axles. However, the length of time for which the elevated stress and strain acts on the pavement would also be greater for the tandem load as compared to individual axle loads. This results because the total deflection basin is broader between the axles for thick pavements with respect to thin pavements.
FIGURE 1

DAMAGE FACTOR CURVES FOR GEIST ROAD TEST SECTION
(STRONG TYPE)

\[ n = \text{number of axles in group} \]

TOTAL GROUP LOAD KIPS
FIGURE 2

DAMAGE FACTOR CURVES FOR
PARKS HIGHWAY TEST SECTION
(Moderately Strong Type)

n = number of axles in group

TOTAL GROUP LOAD KIPS
FIGURE 3

DAMAGE FACTOR CURVES FOR
PARKS (1) HIGHWAY TEST SECTION
(AVERAGE STRENGTH TYPE)

\[ n = \text{number of axles in group} \]

TOTAL GROUP LOAD KIPS
FIGURE 4

DAMAGE FACTOR CURVES FOR
OLD STEESE HIGHWAY TEST SECTION
(Weak Type)

\[ n = \text{number of axles in group} \]

TOTAL GROUP LOAD KIPS
FIGURE 5

HOW PAVEMENT THICKNESS EFFECTS VEHICLE LOADING SUPERPOSITION

[Diagram showing how pavement thickness affects vehicle loading through superposition]

Illustration of Superposition Effect on a Thick Pavement

[Second diagram showing a thin pavement]

Illustration of Superposition Effect on a Thin Pavement
(Figure 5). This implies that any fatigue damage criterion should account for the number of loads (cycles) to failure at different loading periods. The fatigue criterion used in this study and previous studies does not describe the influence of load duration on fatigue damage. Until such a fatigue relationship is used, the true damage potential of individual as compared to multiple axle loads will remain in question.

The damage factors for single axle dual wheel loads \((n = 1)\) shown in Figures 1-4 do, however, agree quite well with the results of the ASSTHO tests, Havens et al., (1979) and Deacon (1969). This implies that relative damage factor analyses for single axle loads are not very sensitive to differences in pavement structure type.

**Comparative Damage Factors for Different Pavement Structures**

The relative damage potential of a given wheel load configuration for any particular pavement structure is reasonably well described by the damage factor concept. Figures 1-4 show that damage factors for different strength pavements do not vary significantly over a wide range of pavement structure types. This is advantageous when determining the relative damage potential for wheel loads on a given roadway without regard to pavement strength. Such damage factors do not provide any information about how the fatigue damage changes when a given load configuration acts on pavements with different strengths. A wheel load configuration that exceeds the ultimate strength of a pavement will fail the pavement irrespective of its relative damage factor, for example, through simple shear failure. This means that a wheel load configuration that can traverse a strong pavement causing little damage may immediately fail a weaker pavement structure even though the calculated damage factors for the two roads are almost identical.
A comparative damage factor that describes the relative damage potential of given wheel load configurations for different pavement types can be defined using Equation 8. This is accomplished by setting A and B in equation 8 to the values of a standard pavement (Parks 1) and setting $A_1$ and $B_1$ to the values of the pavement of interest:

$$(9) \quad \text{CDF} = 3.29 \times 10^5 A_1^{1/B_1} n^{(1+1/B_1)} (T_{Lm})^{-1/B_1}.$$ 

Equation 9 describes the relative damage potential of a given wheel load configuration for a given pavement structure as compared to the damage potential of an 18,000 lb single axle dual wheel load acting on a standard pavement structure (Parks 1). Figures 3, 6, 7 and 8 show the comparative damage factors (CDF) for the four pavement structures used in this study (Parks 1, Geist, Parks and Old Steese). The CDF for the Parks 1 pavement is just the normal pavement damage factor described by equation 8 with $A = A_1$ and $B = B_1$. The remaining comparisons are between the Geist, Parks and Old Steese pavements and the Parks 1 pavement section. The relative strengths of the pavement structures are readily seen in Figures 6, 7 and 8. The CDF for the Geist/Parks 1 is significantly lower than the standard DF for Parks 1 as would be expected since the Geist pavement is much stronger than Parks 1. The Parks/Parks 1 CDF is slightly less than the standard DF and the Old Steese/Parks 1 CDF is significantly greater than the standard DF for Parks 1. The CDF approach is valuable in that the relative damage potential of any wheel load configuration on a given pavement can be compared to the standard dual wheel load configuration on a standard pavement.

An example will best illustrate the use of the CDF. Table V shows the CDF's for the four pavement structures with both single and tandem axles dual
FIGURE 6

COMPARATIVE DAMAGE FACTOR CURVES, GEIST ROAD (STRONG PAVEMENT) REFERENCED TO PARKS (1) (AVERAGE PAVEMENT)

\[ \text{DAMAGE FACTOR} \]

\[ \text{TOTAL GROUP LOAD KIPS} \]

\[ n = \text{number of axles in group} \]
COMPARATIVE DAMAGE FACTOR CURVES,
PARKS HIGHWAY (MODERATELY STRONG PAVEMENT)
REFERENCED TO PARKS (1) (AVERAGE PAVEMENT)

\[ \text{Damage Factor} \]

\[ 10^{-2} \rightarrow 10^{-1} \rightarrow 10^0 \rightarrow 10^1 \rightarrow 10^2 \]

\[ n = \text{number of axles in group} \]

TOTAL GROUP LOAD KIPS
FIGURE 8

COMPARATIVE DAMAGE FACTOR CURVES,
OLD STEESE HIGHWAY (Weak Pavement)
REFERENCED TO PARKS (1) (Average Pavement)

\[ n = \text{number of axles in group} \]

TOTAL GROUP LOAD KIPS
### TABLE V
Comparative Damage Factors

<table>
<thead>
<tr>
<th>Total Group Load lbs</th>
<th>Number of Axles</th>
<th>Comparative Damage Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Geist/Parks 1</td>
</tr>
<tr>
<td>10,000</td>
<td>1</td>
<td>0.028</td>
</tr>
<tr>
<td>15,000</td>
<td>1</td>
<td>0.065</td>
</tr>
<tr>
<td>18,000</td>
<td>1</td>
<td>0.094</td>
</tr>
<tr>
<td>25,000</td>
<td>1</td>
<td>0.18</td>
</tr>
<tr>
<td>20,000</td>
<td>2</td>
<td>0.05</td>
</tr>
<tr>
<td>30,000</td>
<td>2</td>
<td>0.13</td>
</tr>
<tr>
<td>36,000</td>
<td>2</td>
<td>0.188</td>
</tr>
<tr>
<td>50,000</td>
<td>2</td>
<td>0.36</td>
</tr>
</tbody>
</table>
wheel loads. The Parks 1/Parks 1 comparison is simply the standard damage factor describing the relative damage potential of wheel load configurations on an average strength pavement. The remaining CDF comparisons show the influence of different wheel load configurations. Table V illustrates that identical wheel load configurations will have different CDFs on different pavement structures. Pavements that are stronger than the Parks 1 will have lower CDFs, while pavements that are weaker than Parks 1 will have greater CDFs, than the standard DF calculation implies.

Current methods of evaluating the effects of heavy loads on flexible pavements do not allow for comparisons between different pavement structures. The DF can be used by transportaiton personnel to evaluate wheel loading in the standard fashion by using the standard pavement damage factor. Additionally, information about the influence of variations in pavement structure strength as compared to the standard pavement can be obtained by using the CDF. Comparative damage factors for the four pavement structures used in this study have been tabulated in Appendix II.
IMPLEMENTATION

Results of this search suggest implementation of a procedure to supplement present regulations regarding overweight permits for vehicles operating on paved roadways. The permit cost estimate is based on the amount of pavement damage done by the overweight vehicle on a selected route of travel relative to a legally loaded vehicle on a standard type pavement. None of the findings obtained from this study provided evidence which would reduce the level of control exerted by present overweight vehicle regulations. Implementation of this report's findings requires adoptive action, by the Department, of the following:

1) Based on deflection basin shapes determined from Falling Weight Deflectometer data, all paved highways intended as truck routes should be placed into one of the four structural (strength) categories indicated in this report. This may be effectively done either by visual inspection of plotted deflection basins or through use of a computerized curve matching process. The structural classification of roads provides information required in the following item.

2) Overweight permit fees should be obtained using the presently prescribed method ("Alaska Oversize and Overweight Permit Movements," Chapter 4, Paragraph 4.02) and also according to the method described in Appendix IV of this report ("Vehicle Overweight Policy Using Comparative Damage Factor Analysis"). Actual permit fees charged to the vehicle owner will be the larger of the two determined values.

Single axle loads of up to 55 Kips have been allowed on strong roadway sections, having 3 inches of asphalt pavement, without obvious damage. However, requests for axle loadings in excess of 50 Kips should be treated with a high degree of caution. Pavements subjected to such loads should be monitored during the entire vehicle movement for signs of immediate failure such as cracking, faulting, or rutting. Axle loads of more than 50 Kips should operate only on those roads which are verified as being moderately strong or better as defined in this report.
REFERENCES


APPENDIX I

Deflection Basins, Bulk Dynamic Moduli and Peak Deformation-Load Plots Used to Categorize the Pavement Types Used in this Study
LOAD-DEFLECTION BASINS, GEIST ROAD SECTION (Strong)

Figure A1a

INCHES

THOUSANDS OF AN INCH

DISTANCE METERS

DEFORMATION MICRONS
DEFLECTION AT LOAD CENTER, GEIST ROAD SECTION (STRONG)
FIGURE A1c

BULK DYNAMIC MODULUS, GEIST ROAD SECTION (STRONG)
FIGURE A2A

LOAD-DEFLECTION BASINS, PARKS HIGHWAY SECTION (Moderately Strong)
DEFLECTION AT LOAD CENTER, PARKS HIGHWAY SECTION (MODERATELY STRONG)
FIGURE A2c

BULK DYNAMIC MODULUS, PARKS HIGHWAY SECTION (MOLLERATELY STRONG)
FIGURE A3A

LOAD-DEFLECTION BASINS, PARKS (1) SECTION (AVERAGE)
FIGURE A3B

DEFLECTION AT LOAD CENTER, PARKS (1) HIGHWAY SECTION (AVERAGE)
FIGURE A3c

BULK DYNAMIC MODULUS, PARKS (1) HIGHWAY SECTION (AVERAGE)
LOAD-DEFLECTION BASINS, OLD STEESE HIGHWAY SECTION (WEAK)
FIGURE A4b

DEFLECTION AT LOAD CENTER, OLD STEESE HIGHWAY SECTION (WEAK)
FIGURE A4c

BULK DYNAMIC MODULUS, OLD STEESE HIGHWAY SECTION (WEAK)

LOAD Kips

5 10 15 20

BULK DYNAMIC MODULUS MN/m

90 80 70 60 50 40 30

LOAD KN

470 420 370 320 270 220
APPENDIX II

Comparative Damage Factors for Single Axle Dual Wheel Loads on Alaskan Pavements
APPENDIX II

Comparative Damage Factors for Single Axle Dual Wheel Loads on Alaskan Pavements

The following tables contain the comparative damage factors for the Geist, Parks, Parks 1 and Old Steese pavement sections. Only the CDF for single axle dual wheel loads have been tabulated. CDF for multiple axle dual wheel configurations can be obtained by simple division of the tabulated damage factor values by the number of axles

\[ \text{CDF}_n(\text{TL}) = \text{CDF}_1(\text{TL})/n. \]

The comparative damage factor for an n axle wheel configuration with a total group load of TL is CDF\(_n\)(TL). Table AI shows the CDF for single axle dual wheel loads and Table AII shows the present fatigue damage in excess of that caused by a 20 kip single axle load. Table AII illustrates the dependence of excess fatigue damage above legal load limits on pavement strength characteristics.

The comparisons are all to the Parks 1 pavement section which has been chosen as the standard pavement structure in this study. The Parks 1/Parks 1 damage factor results are conceptually the same as have been calculated in the past (Havens et al., 1979; Deacon, 1966).
TABLE AI

Comparative Damage Factors for Single Axle Dual Wheel Loads
Acting on the Geist, Parks, Parks 1 and Old Steese
Pavement Sections

<table>
<thead>
<tr>
<th>Total Axle</th>
<th>Comparative Damage Factor</th>
<th>CDF1(TL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load KIPS</td>
<td>Strong Geist/Parks 1</td>
<td>Strong Parks/Parks 1</td>
</tr>
<tr>
<td>5</td>
<td>0.007</td>
<td>0.04</td>
</tr>
<tr>
<td>10</td>
<td>0.03</td>
<td>0.16</td>
</tr>
<tr>
<td>15</td>
<td>0.07</td>
<td>0.36</td>
</tr>
<tr>
<td>18</td>
<td>0.09</td>
<td>0.51</td>
</tr>
<tr>
<td>20</td>
<td>0.12</td>
<td>0.64</td>
</tr>
<tr>
<td>25</td>
<td>0.18</td>
<td>0.99</td>
</tr>
<tr>
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<td>0.27</td>
<td>1.43</td>
</tr>
<tr>
<td>35</td>
<td>0.37</td>
<td>1.95</td>
</tr>
<tr>
<td>40</td>
<td>0.48</td>
<td>2.54</td>
</tr>
<tr>
<td>45</td>
<td>0.61</td>
<td>3.22</td>
</tr>
<tr>
<td>50</td>
<td>0.76</td>
<td>3.98</td>
</tr>
<tr>
<td>55</td>
<td>0.92</td>
<td>4.81</td>
</tr>
<tr>
<td>60</td>
<td>1.10</td>
<td>5.72</td>
</tr>
<tr>
<td>65</td>
<td>1.30</td>
<td>6.72</td>
</tr>
<tr>
<td>70</td>
<td>1.51</td>
<td>7.79</td>
</tr>
</tbody>
</table>
TABLE AII

Percent Fatigue Damage in Excess of That Caused by a 20 kip Single Axle Dual Wheel Load

<table>
<thead>
<tr>
<th>Axle Load (kips)</th>
<th>Strong</th>
<th>Moderately Strong</th>
<th>Average Strength</th>
<th>Weak</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>25</td>
<td>6.7%</td>
<td>35.0%</td>
<td>81.7%</td>
<td>395.7%</td>
</tr>
<tr>
<td>30</td>
<td>15.0%</td>
<td>78.3%</td>
<td>185.0%</td>
<td>948.3%</td>
</tr>
<tr>
<td>35</td>
<td>25.0%</td>
<td>131.7%</td>
<td>311.7%</td>
<td>1,671.7%</td>
</tr>
<tr>
<td>40</td>
<td>36.7%</td>
<td>190.0%</td>
<td>463.3%</td>
<td>2,586.7%</td>
</tr>
<tr>
<td>45</td>
<td>48.3%</td>
<td>258.3%</td>
<td>638.3%</td>
<td>3,708.3%</td>
</tr>
<tr>
<td>50</td>
<td>63.3%</td>
<td>333.3%</td>
<td>840.0%</td>
<td>5,051.7%</td>
</tr>
<tr>
<td>55</td>
<td>80.0%</td>
<td>416.7%</td>
<td>1,068.3%</td>
<td>6,633.3%</td>
</tr>
<tr>
<td>60</td>
<td>98.3%</td>
<td>508.3%</td>
<td>1,323.3%</td>
<td>8,468.3%</td>
</tr>
<tr>
<td>65</td>
<td>118.3%</td>
<td>608.3%</td>
<td>1,605.0%</td>
<td>10,501.7%</td>
</tr>
<tr>
<td>70</td>
<td>138.3%</td>
<td>715.0%</td>
<td>1,916.7%</td>
<td>12,901.7%</td>
</tr>
</tbody>
</table>
APPENDIX III

Table of Pavement Load Deflection Measurements Used in This Study
## TABLE AIII
Load-Deflection Data for the Geist, Parks 1, Old Steese and Parks Pavement Sections

<table>
<thead>
<tr>
<th>Date of Measurement</th>
<th>Air Temp.</th>
<th>Pavement Section</th>
<th>Applied Pressure in kPa*</th>
<th>Sensor Location in meters (0 0.2 0.3 0.45 0.65 0.9 1.2)</th>
<th>Strength Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Geist Rd. on</td>
<td>225</td>
<td>128 77 48 29 20 12 8</td>
<td>strong</td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Geist Rd. on</td>
<td>379</td>
<td>187 120 79 50 44 23 16</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Geist Rd. on</td>
<td>511</td>
<td>259 166 117 75 51 33 22</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Geist Rd. on</td>
<td>771</td>
<td>329 216 161 111 77 53 37</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Geist Rd. on</td>
<td>1113</td>
<td>447 304 225 159 112 78 56</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Geist Rd. on</td>
<td>1348</td>
<td>509 342 264 191 136 95 68</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Geist Rd. on</td>
<td>1528</td>
<td>562 377 294 214 156 109 80</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Parks 1</td>
<td>176</td>
<td>211 136 92 57 39 27 21</td>
<td>Average strength</td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Parks 1</td>
<td>352</td>
<td>380 259 189 126 91 66 48</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Parks 1</td>
<td>515</td>
<td>560 392 296 204 145 104 77</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Parks 1</td>
<td>741</td>
<td>793 577 445 315 227 161 119</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Parks 1</td>
<td>1012</td>
<td>1036 743 579 415 300 216 158</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Parks 1</td>
<td>1313</td>
<td>1305 970 760 551 401 287 211</td>
<td></td>
</tr>
<tr>
<td>82/08/27</td>
<td>61</td>
<td>Parks 1</td>
<td>1465</td>
<td>1390 1027 814 599 443 322 240</td>
<td>Weak</td>
</tr>
<tr>
<td>82/09/01</td>
<td>77</td>
<td>Old Steese</td>
<td>194</td>
<td>384 283 219 150 112 80 55</td>
<td></td>
</tr>
<tr>
<td>82/09/01</td>
<td>77</td>
<td>Old Steese</td>
<td>340</td>
<td>684 509 406 299 218 143 90</td>
<td></td>
</tr>
<tr>
<td>82/09/01</td>
<td>77</td>
<td>Old Steese</td>
<td>503</td>
<td>1011 766 623 464 336 215 129</td>
<td></td>
</tr>
<tr>
<td>82/09/01</td>
<td>77</td>
<td>Old Steese</td>
<td>708</td>
<td>1435 1112 922 701 513 328 197</td>
<td></td>
</tr>
<tr>
<td>82/09/01</td>
<td>77</td>
<td>Old Steese</td>
<td>961</td>
<td>1840 1424 1178 893 651 406 236</td>
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</tr>
<tr>
<td>82/09/01</td>
<td>77</td>
<td>Old Steese</td>
<td>1244</td>
<td>2357 1824 1534 1189 872 549 316</td>
<td></td>
</tr>
<tr>
<td>82/09/01</td>
<td>77</td>
<td>Old Steese</td>
<td>1393</td>
<td>2597 1989 1692 1305 965 606 354</td>
<td></td>
</tr>
<tr>
<td>82/09/07</td>
<td>60</td>
<td>Parks</td>
<td>197</td>
<td>150 76 52 27 15 10 7</td>
<td>Moderately strong</td>
</tr>
<tr>
<td>82/09/07</td>
<td>60</td>
<td>Parks</td>
<td>349</td>
<td>283 197 137 76 40 24 17</td>
<td></td>
</tr>
<tr>
<td>82/09/07</td>
<td>60</td>
<td>Parks</td>
<td>529</td>
<td>427 286 209 123 68 38 28</td>
<td></td>
</tr>
<tr>
<td>82/09/07</td>
<td>60</td>
<td>Parks</td>
<td>779</td>
<td>544 375 283 179 107 63 43</td>
<td></td>
</tr>
<tr>
<td>82/09/07</td>
<td>60</td>
<td>Parks</td>
<td>1006</td>
<td>653 473 357 231 141 86 58</td>
<td></td>
</tr>
<tr>
<td>82/09/07</td>
<td>60</td>
<td>Parks</td>
<td>1345</td>
<td>893 626 479 318 199 120 82</td>
<td></td>
</tr>
<tr>
<td>82/09/07</td>
<td>60</td>
<td>Parks</td>
<td>1520</td>
<td>980 694 535 361 230 139 95</td>
<td></td>
</tr>
</tbody>
</table>

1 psi = 6.895 kPa  
1 ft = 0.3048 m  
One thousandths of an inch = 25.4 microns  
*Loading plate area = 113 sq. in.
APPENDIX IV

Vehicle Overweight Policy Using Comparative Damage Factor Analysis
APPENDIX IV

Vehicle Overweight Policy Using Comparative Damage Factor Analysis

Road use permitting policies are, in general, implemented to limit or charge a fee for any excess damage caused by overweight vehicles. Before reasonable permit policies can be developed, some estimate of the average excess damage cost needs to be determined. The comparative damage factor analysis developed in this study can be used to estimate the average excess fatigue damage cost based on the known costs of pavement replacement. Connor (1980) estimated a base cost of $0.06 per equivalent axle load mile (EAL-mile) for loads in excess of the legal limit. The average cost per axle, per mile for a given axle load can be estimated from CDF analysis using

\[ [CDF(\text{TL}, \text{PL}) - CDF(20 \text{ kips}, \text{PL})] \times ($0.06) \]

where \( CDF(\text{TL}, \text{PL}) = N_r(18 \text{ kip, PA})/N_L(\text{TL, PL}) \) in (EAL) is the comparative damage factor for a given total group load on a given pavement with respect to the base load on an average strength pavement, and

\( CDF(20 \text{ kips, PL}) = N_r(18 \text{ kip, PA})/N_L(20 \text{ kip, PL}) \)

is the comparative damage factor in EAL for a legal 20 kip load on a given pavement with respect to the base load on a given pavement. The effects of heavy loads on front axles cannot be evaluated since the FWD data used to develop the above equations simulated dual wheel loadings. Consequently, calculations using the CDF developed in this study requires that the front axle be within legal load limits.
Table AIV shows the average fatigue damage costs for late summer conditions when average pavement deflection values are minimum for the thawed pavement structure. These were calculated using the results of Table AI. The damage costs based on CDF analysis show that costs decrease as the pavement strength increases. The following example illustrates the use of Table AIII to estimate excess damage costs for different strength pavements.

**Example**

Use Table AIII to find the average excess fatigue damage cost for a 3-52 (a vehicle with a single axle single wheel front axle, tandem axle dual wheel drive axles and tandem axle dual wheel trailer axles) commercial vehicle with the following axle loadings:

- Front Axle: 18 kips
- Drive Axles: 50 kips (25 kips per axle)
- Trailer Axles: 50 kips (25 kips per axle)

The average excess damage costs are calculated from

\[ F_C = \sum_{i=1}^{n} w_i K_i \]

were \( F_C \) is the excess fatigue damage cost, \( w_i \) is the number of axles in each axle group, \( K_i \) is the cost per axle per mile for the axle group load and \( n \) is the number of axle groups. For the above example on an average strength pavement

<table>
<thead>
<tr>
<th>Axle Group</th>
<th>Cost per Mile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front Axle</td>
<td>($0.00)</td>
</tr>
<tr>
<td>Drive Axles</td>
<td>2 x ($0.049)</td>
</tr>
<tr>
<td>Trailer Axles</td>
<td>2 x ($0.049)</td>
</tr>
</tbody>
</table>

\[ F_C = 0.196 \text{ per mile} \]
If the load is moved from Anchorage to Fairbanks (360 miles) the total cost would be

\[ F_C = 360 \times 0.196 = 68.60 \]

The CDF analysis that was developed in this study to provide estimates of damage cost per axle is only valid for late summer road conditions. Any load policy based on the above estimates should be limited to the dates when the average pavement surface deflection values are a minimum for thawed soil conditions.
TABLE A IV  
Excess Damage Cost Per Axle, Per Mile

<table>
<thead>
<tr>
<th>Axle Load (kips)</th>
<th>Strong</th>
<th>Moderately Strong</th>
<th>Average Strength</th>
<th>Weak</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>25</td>
<td>$0.004</td>
<td>$0.021</td>
<td>$0.049</td>
<td>$0.238</td>
</tr>
<tr>
<td>30</td>
<td>$0.009</td>
<td>$0.007</td>
<td>$0.111</td>
<td>$0.569</td>
</tr>
<tr>
<td>35</td>
<td>$0.015</td>
<td>$0.079</td>
<td>$0.187</td>
<td>$1.003</td>
</tr>
<tr>
<td>40</td>
<td>$0.022</td>
<td>$0.114</td>
<td>$0.278</td>
<td>$1.552</td>
</tr>
<tr>
<td>45</td>
<td>$0.029</td>
<td>$0.155</td>
<td>$0.383</td>
<td>$2.225</td>
</tr>
<tr>
<td>50</td>
<td>$0.038</td>
<td>$0.200</td>
<td>$0.504</td>
<td>$3.031</td>
</tr>
<tr>
<td>55</td>
<td>$0.048</td>
<td>$0.250</td>
<td>$0.641</td>
<td>$3.980</td>
</tr>
<tr>
<td>60</td>
<td>$0.059</td>
<td>$0.305</td>
<td>$0.794</td>
<td>$5.081</td>
</tr>
<tr>
<td>65</td>
<td>$0.071</td>
<td>$0.365</td>
<td>$0.963</td>
<td>$6.301</td>
</tr>
<tr>
<td>70</td>
<td>$0.083</td>
<td>$0.429</td>
<td>$1.150</td>
<td>$7.741</td>
</tr>
</tbody>
</table>