THE ECONOMIC, OPERATING, AND INFRASTRUCTURE IMPACTS OF
CONCENTRATED TRUCK TRANSPORT SERVICE AND
Designated COMMERCIAL HIGHWAY NETWORKS

Prepared by
The University of Iowa
Public Policy Center
in conjunction with the
Midwest Transportation Center

August, 1992
Commissioned by the
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This study was funded by the University Transportation Centers Program of the U.S. Department of Transportation. The results and views expressed are the independent products of university research and are not necessarily concurred in by the funding agency.
PREFACE

This report is the product of a third-year research project in the University Transportation Centers Program. The Program was created by Congress in 1987 to “contribute to the solution of important regional and national transportation problems.” A university-based center was established in each of ten federal regions following a national competition in 1988. Each center has a unique theme and research purpose, although all are interdisciplinary and also have educational missions.

The Midwest Transportation Center is one of the ten centers; it is a consortium that includes Iowa State University (lead institution) and The University of Iowa. The Center serves federal Region 7 which includes Iowa, Kansas, Missouri, and Nebraska. Its theme is “transportation actions and strategies in a region undergoing major social and economic transition.” Research projects conducted through the Center bring together the collective talents of faculty, staff, and students within the region to address issues related to this important theme.

This particular project was carried out by an interdisciplinary research team at The University of Iowa’s Public Policy Center. This center is a reflection of the University’s renewed commitment to applied research that seeks to advance the public interest. The Center’s projects generally involve close interaction with decision makers and resource people in both the public and private sectors.

The project is central to the Midwest Transportation Center’s theme in that it examines the relationship between road highway design and allowable truck axle weights and configurations. The principal investigator was Professor James W. Stoner, Civil and Environmental Engineering. Co-investigator was M. Asghar Bhatti, Civil and Environmental Engineering. They were assisted by Norman S.J. Foster, Research Associate at the Public Policy Center; Carlos Quintero Febres, Idelin Molinas-Vega, James D. Hingtgen, and Mahesh Ayyalasomaya, graduate students in Engineering; Jeffrey Barlow and Bryce Amhof, undergraduate students in Civil and Environmental Engineering, and Lin Baizhong, Visiting Scholar in Engineering. Support services for computing were provided by the Center for Simulation and Design Optimization and the Public Policy Center at the University of Iowa.
ACKNOWLEDGMENTS

The research and development performed during the second year of this project required the assistance of many people. The Project Advisory Committee satisfied many of our critical data needs, assisted in the preparation of our work program, and reviewed our drafts of this report. Members of the Advisory Committee included Bill McCull and Neil Volmer of the Iowa Department of Transportation; Bill Giles of the Ruan Companies; Don Cole of Trailer Train; and George Crouse of Crouse Cartage Company. In addition, technical data and design information was supplied by Brian McWaters of the Iowa Department of Transportation.

The University Transportation Centers Program of the U.S. Department of Transportation deserves our thanks for making it possible to carry out this research. Also, Professor Maze and his colleagues at Iowa State University provided direction and administrative guidance that helped us through the project.
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INTRODUCTION

This interim report summarizes the first year effort of researchers at the University of Iowa dealing with the design standards for a commercial highway network in the state of Iowa. This research builds on work performed as part of a previously funded project that developed a computer based procedure for evaluating the effects of vehicle axle loadings and trailer configuration on the deflections and stresses in rigid pavement. The results of this analysis consequently provide an estimate of the resulting effective service life of a statewide trucking network. This study looks specifically at the relative effect for vehicles currently authorized to operate on the interstate system, providing a basis for the future evaluation of the impact of alternative vehicles that could be implemented under the general construct of Longer Combination Vehicles, termed LCVs.

A collaborating research group at Iowa State University will be looking at the freight distribution pattern in the State of Iowa and developing a distribution model for potential LCV traffic in the state. The research groups will jointly define the resulting impact of LCV traffic on the state in terms of shipping costs, operator cost and investment structure, and recommend general changes in design and maintenance standards to the Iowa Department of Transportation.

LCV CHARACTERISTICS

Longer combination vehicles (LCVs) exceed length or weight minimums that the Federal Government currently requires states to accommodate on the designated interstate and federal aid primary highway segments. At the present time, 20 states allow various configurations of LCVs to operate on designated portions of their road systems, with widely varying restrictions. These designated portions include 110,000 miles of two lane roads and 28,000 miles on the National Truck Network. State regulations now allow the approved STAA vehicles (defined in the Surface Transportation Assistance Act of 1982, and hence known as STAA vehicles), such as doubles, to operate within a specified distance of the national network (five miles in Iowa) or a designated terminal (shortest practical distance in Maryland and Tennessee). The definition of a terminal or a description of what constitutes a terminal has been avoided in legislative or regulatory language. States have been legally held to have the right to exercise authority over access, but the courts have ruled that they must use appropriate criteria.

Local governments have adopted various controls, ranging from designations of local truck routes to outright bans on large trucks, or requirements for use permits or access approval.

Truck Load (TL) carriers have more problems than Less-than-Truck-Load (LTL) carriers because they do not typically operate over regularly scheduled routes and do not maintain their
own terminal facilities. Many carriers report that a very small percentage of their customer base is within the designated distances from the specified truck network.

Multiple Trailers

Double combinations

The Surface Transportation Assistance Act of 1982 (STAA) provided that states cannot prohibit operation of twin trailer trucks, with trailer lengths of up to 28 feet, on interstate highways and other designated principal roadways. This has become the configuration of choice for many LTL operators. States were specifically prohibited from enacting an overall length limit because of safety concerns with respect to tractor designs that could be implemented to meet that limit. The typical five axle configuration is not classified as an LCV vehicle. Twin trailer LCV designated vehicle configurations would include the Rocky Mountain Double, a tandem axle 45 to 48 foot semi-trailer, with a 28 foot single axle trailer on a dolly; and a Turnpike Double, two 45 to 48 foot tandem axle trailers with a tandem axle dolly.

The 48 foot van trailer with a 350 cubic foot volume capacity and a cube out density of 15 lb/ft$^3$ has become the standard single trailer. A twin 28 foot trailer has a volume capacity of 4,200 cubic feet and a cube out density of 12 lb/ft$^3$. Low density cargoes make up nearly $1/3$ of all truck shipments, and nearly all LTL shipments.

Triple combinations

The typical triple combination consists of three 28 foot trailers, two dollies, and a tractor, all on single axle sets. All triple combinations would be considered LCV combinations. Triples can operate in Nebraska, but the trailers must be empty.

Pup trailers

A large number of straight truck and trailer combinations exist, usually connected with a draw bar. These can be designated an LCV combination depending upon the overall length and number of attached trailers

Increased Overall Length

Overall length

Overall vehicle length limits range from 90 to 120 feet, depending on the number of trailers and the configuration. Some states have recently enacted an overall length limit of 60 feet on non
truck network roads or have regulations concerning wheel bases for vehicles traveling off the network. Other states, such as New Jersey, have designated a significant commercial network.

**Trailer lengths**

The maximum trailer length for single trailers varies from 48 to 59\(\frac{1}{2}\) feet. Triple combination trailers are each typically 28 feet long. Some states restrict the combined trailer lengths for Rocky Mountain Doubles. Maximum kingpin to rear axle limits for the longer STAA trailers were instituted in some states to reduce inward off tracking (Iowa has a 40 foot restriction), but this can be difficult to enforce.

**Increased Axle Loads**

Federally mandated axle weight limitations are presently 20,000 lbs for single axles and 34,000 lbs for tandem axles. Numerous grandfather provisions exist in some states primarily impacting the LCVs used in the western states and the so-called Specialized Hauling Vehicles (SHVs), such as concrete delivery trucks and garbage trucks. Alternative gross weight and axle limits have been proposed to eliminate the 80,000 lb gross weight limit and the associated state specific limits on vehicle length.

At the present time approximately 25 percent of all operating combination trucks exceed either gross vehicle weight or axle weight limits (National Research Council 1990). Even allowing for overweight permits, the percentage would still remain between 10 and 20 percent for all states.

An American Association of Railroads study estimates that eliminating the weight limit would shift 2.2 percent of rail traffic to trucks, and a combined elimination of length limits (allowing 48 foot doubles for example), would divert nearly 9 percent.

**Increased Gross Weight**

**Legislated vehicle weights**

The maximum vehicle gross weight was initially set at 73,280 pounds for the interstate system. The weight limit was increased to 80,000 pounds in 1974, but many higher weight limits established by individual states were grandfathered in. Florida allows a maximum gross vehicle weight of 147,000 pounds.

**Bridge formulae**

The current federal bridge formula is based on stresses in the structural members of the bridge. HS-20 is used for interstate highways and H-15 is often used for non-interstate highways that are
exposed to lighter loads. The first formula has been criticized for being overly cautious, and several alternatives have been proposed. The Texas Transportation Institute proposed a modification that would not exceed the stress limits. This approach generally would result in slightly higher acceptable loads on short wheel base, five axle trucks, but would be more restrictive on some of the longer trucks currently operating under grandfather provisions. A Transportation Research Board Committee developed a hybrid formula that would encourage the use of more axles, resulting in more evenly distributed weight. This proposal recommended a limit of up to 30,000 lb for trailer tandem axle sets.

Canada has higher weight limits and no specific bridge formula. Minimum axle spacings are used to spread the load and avoid over stressing the bridges. Setback steering axles and other arrangements have been proposed in the U.S. to allow heavier loading on the steering axle, perhaps even approaching the 20,000 lb limit on single axles.

The current federal bridge formula is expressed as:

\[ W = 500 \left( L \times N(N-1) + 12N + 36 \right) \]

Where:

- \( W \) = Maximum weight in pounds carried on any group of axles
- \( L \) = Distance in feet between the extremes of any group of consecutive axles
- \( N \) = The number of axles under consideration

The HS–20 formula is based on a hypothetical vehicle with a single 8,000 lb steering axle and two 32,000 lb tandems. The H–15 bridge formula is based on a straight truck with a 6,000 lb steering axle and a single 24,000 lb tandem axle set. The gross vehicle weight is capped at 80,000 lbs, regardless of the bridge formula value. An important point, however, is that the Federal Highway Administration (FHWA) has not been able to exercise control over special permit practices by the states, since states do not have to acquire FHWA approval for allowing heavier loaded vehicles to operate under a special permit status. There are a large number of special cases that apply to SHVs, including solid waste vehicles that do not have to comply with any axle weight limits except on interstate highway segments. Standard vehicle descriptions are available (National Research Council 1990, pp. 96–97).

SHV operators often use lift axles that are lowered only when being weighed or dummy axles that increase the allowable weight, but actually accommodate very little of the weight of the vehicle, serving no useful purpose except compliance.
Effects on Economic Efficiency

Cubic capacities

The use of larger vehicles will increase productivity for shipping low density commodities, that are rarely shipped by rail anyway. Transportation costs usually represent 2-3 percent of the total cost of a manufactured product. A 25 percent larger trailer should provide a slightly smaller decrease in shipping costs.

Vehicle mileage

Increased LCV operation could result in decreased traffic, or it could divert additional freight from the railroads (estimated to be as much as a 52 percent diversion), which would increase truck traffic.
INTERSTATE SYSTEM

Early Route Location and Specification

Planning for the initial segments of the interstate system were based on the specification of a federal system connecting the major urban areas of the United States. It is important to understand that the original purpose of the system was to serve as a defense network that would allow for the rapid transfer of troops and material by truck during war time. The state highway departments and the Bureau of Public Roads identified a 41,000 mile network, which was later expanded to its present size. Portions of the previously existing turnpike systems were integrated into the defined system. This interstate network serves as the present day backbone of the commercial trucking network for the Nation.

Minimum distance

A prime criterion during the early planning was to minimize distance, because the constraints were not so much fiscal as they were based on total mileage. The new interstate segments tended to follow straight line paths, especially in midwest states like Iowa, and were not concerned with following existing ground lines or the natural terrain. Issues like energy conservation, facility life, preservation of existing drainage patterns, and minimization of construction costs were not totally ignored, but they were not given high priority either. The system was built to connect major cities, and to enhance the ability to control vast regions politically, economically, and militarily.

Geometric simplicity

The resulting network design standard is based on a certain geometric simplicity with an assumed specific design speed, somewhat depending upon the natural terrain. In Iowa, the typical design speed was 75 mph. Natural physical laws, combined with assumptions regarding friction coefficients, sight distances, physical comfort, and vehicle configuration, determined the minimum degree of horizontal curvature, lengths of vertical curves, and steepness of grade. Considerable excavation and embankment was undertaken to prevent these standards from being violated, without requiring extensive indirect routing. Pavement depth calculations were based on research conducted on airfield slabs during world war II, some experiential studies by state highway departments, and some very early empirical evaluations performed at universities and federal and state laboratories. The early interstate was based on the best engineering decisions given the data available and the assumptions that were required, but they were not necessarily efficient from a vehicle operating or goods transportation viewpoint, nor were they intended to provide a long functional life.
Maximize road building capability

The intent of these early route location and facility design decisions was to maximize the amount of roadway that could be completed within a specific political jurisdiction. This was somewhat different from the philosophy that guided other road building efforts, partially because of the procedure for allocating monies to states and determination of required match, the lack of understanding of the response of pavements to heavy axle loads, and the performance of thick pavement slabs. Sub-grade stabilization and reinforcement was not considered to be of great importance, and safety programs tended to take a back seat to the creation of additional capital facilities. It must be remembered that at that time the threat of foreign invasion and even nuclear attack was considered quite creditable, and the need for rapid implementation was assumed to be critical.

Pavement Materials and Thickness Design

Nearly all pavement thickness designs utilize the concept of an Equivalent Single Axle Load (ESAL), a term resulting from the AASHO road tests conducted in the 1950s. The tests were based on subjective evaluations of pavement serviceability after replications of different axle loads. An ESAL value is roughly a fourth power relationship directly with axle weight, with an 18,000 lb single axle being the standard. It is fair to say that many of the values were extrapolated from very little data. The pavement slabs evaluated were typically very thin, being 6 to 9 inches thick. The serviceability values, while appearing to be based on quantitative values, were in fact quite subjective. The correlation's between pavement load, slab thickness and subgrade quality, and load replications were often quite low if not non-existent. A number of researchers have returned to the original road test data for recalculation of ESAL values, and consequently provided quite different results (Small, Winston and Evans 1989). Use of the ESAL values for design, and the estimates of allowable ESALs for a facility life based on a terminal serviceability of 2.5, requires a large number of assumptions. These design procedures are based on very creative work with a very difficult experimental design, but they do not provide a good description of the deterioration process associated with pavements, nor are they even necessarily correct. They simply assume that the larger number of heavy loads applied, the faster the pavement will be consumed and then an estimate of that rate of consumption is applied that has a very significant error term.

PCC pavement

Pavement sections made with Portland cement, sand, and coarse aggregate are usually referred to as Portland Cement pavements. A wide variety of designs, with slab thicknesses varying from 6 to 15 inches. When the slabs are connected at transverse joints with dowels, usually with a 12
inch spacing either for the full width of the pavement or just in the wheel path, then it is called jointed concrete pavement.

**Continuous reinforced concrete**

Concrete pavement that uses reinforcing steel in the form of welded wire fabric or bars is termed Continuous Reinforced Concrete, or CRC. The amount of steel used varies widely between jurisdictions, but generally the steel must be strong enough to drag both ends of each individual slab toward its center, over the subgrade.

**Composite designs**

A design that has come under increasing use either because of initial design or resurfacing practice is a composite pavement that may utilize a concrete base course with an asphalt surface or wearing course. These designs make use of the bearing and compressive strengths of concrete and the smoothness and wearing qualities of the asphalt, or flexible, pavements. A major factor in the selection of the materials to be used in road construction is the relative cost and availability. Consequently a roadway design may be preferred in the Midwest because of the relative economies provided by certain materials and the availability of contractors with experience in using them.

**Iowa’s Interstate Highways**

The state of Iowa has 779 miles of interstate highway within its borders. This section describes this system of highways in terms of age, characteristics, and level of service. The data presented are drawn from the Pavement Management System (PMS) maintained by the Iowa Department of Transportation, and are current as of 1991. Since each roadbed is rated separately, these data relate to a total of 1,558 miles for Iowa, twice the actual interstate highway length.

Iowa has eight segments of interstate, varying in length from 307 miles for I80 to less than 10 miles for I280. Three interstates, I80, I35 (204 miles) and I29 (149 miles), account for the bulk of the interstate miles in Iowa.

**Age**

Iowa’s interstates were largely built in the 1960s and early 1970s. Consequently, most of the pavement is over 20 years old (Figure 2). Indeed, almost 1,000 miles, two-thirds of the total, is over 22 years old. The newer sections of roadway are either rebuilt sections of I80 and other older interstates, or sections of recently completed interstates, most notably I380.
Depth of pavement

Most of Iowa’s interstate pavements have pavement depths of less than 12 inches (Figure 2). The thicker sections tend to be rebuilt portions of the system, or repeated overlays of flexible pavement.
Type of pavement

Three types of pavement predominate on Iowa’s interstates (Figure 3), jointed concrete, continuous reinforced concrete (CRC), and composite designs of PCC overlayed with asphalt paving. Only about 100 miles of highway are completely asphalt.

![Bar chart showing distribution of Interstate lane miles in Iowa by type of pavement, 1991.](chart.png)

**FIGURE 3. DISTRIBUTION OF INTERSTATE LANE MILES IN IOWA BY TYPE OF PAVEMENT, 1991**

Present Serviceability Index (PSI)

Iowa’s interstate pavement is in generally good condition as measured by PSI rating. Figure 4 shows that only a small fraction of the highway miles has a PSI below 2.5, the traditional terminal value for interstates. Moreover, a significant majority of highway miles are above 3.5, a fairly high value.

Previous Year’s 18k Axle Loads

Traffic on Iowa’s interstates is concentrated on small portions of the system. Figure 5 shows how 18k ESAL values vary across the system. On over half the system, the previous year’s total 18k ESALs were less than 1 million. On only 12 percent of the system did 18k exceed 2 million in the previous year. This variation in pavement loading will contribute to differential rates of wear on different parts of the system.
Most of Iowa’s interstates have soil support values above 100 (Figure 6). About half of all miles have k values in the range of 100 to 150 and the other half exceed 150. Figure 7 shows how PSI and Lifetime 18k vary by each of the three k value groupings (below 100, between 100 and 150, and above 150).
Comparative characteristics

Iowa’s interstate highways vary in age, condition and traffic, both with respect to each other and with respect to different segments. Figures 8 to 13 illustrate these differences for the three
major interstates in Iowa, by length: I80, traversing the state on an East-West alignment; I35, a major North-South route; and I29, also a North-South route.

These three interstates were all originally built in the late 50s to early 70s, as Figure 8 shows. The pavement on I29 is still the original laid down 20 to 35 years ago. In contrast, both I80 and I35 are presently undergoing significant reconstruction. Several segments of I80 have been rebuilt in the last five years, as has a segment of I35.

Pavement depth is much more uniform on I29 than on the other two interstates (Figure 9). Most of the pavement on I29 is less than 12 inches deep, whereas the southern sections of I35 tend to be about 15 inches or more in depth. Pavement depth on I80 varies most, because of changed construction methods and varying geography.

Age and Pavement Evaluations

Present Serviceability Index

The Present Serviceability Index (PSI) is a qualitative measure of user based road quality developed during the original 50s' road tests. The PSI is a five point scale varying from poor to good. The question of what constitutes acceptable pavement varies from one jurisdiction to another, but many pavement design procedures assume a terminal serviceability index of 2.5. This means that the pavement requires replacement or reconstruction at that point. Most pavements were designed based on loading assumptions that would produce a PSI of 2.5 after approximately 20 years of forecast traffic loads. Depending on how the index is calculated, the definition of what level corresponds to unacceptable ride quality and high operator costs can vary considerably. Pavement with a value less than 3.0 is certainly not desirable.

The basic Iowa D.O.T. formula for calculating PSI is:

\[ PSI = LPV - 0.9\sqrt{C+P} \]

Where:

PSI = Present Serviceability Index

LPV = Longitudinal Profile Value (as measured by a Profilometer)

C = percentage of cracked area (square feet per 1,000 square feet)

P = percentage of patched area (square feet per 1,000 square feet)
Figure 8. Pavement Age for Iowa Interstates, 1991

Figure 9. Pavement Depth for Iowa Interstates, 1991

Interstate 29 Northbound

Interstate 35 Northbound

Interstate 80 Eastbound

Figure 10. Pavement Type for Iowa Interstates, 1991

Figure 11. Pavement PSI for Iowa Interstates, 1991

Figure 12. Last Year’s 18k for Iowa Interstates, 1991

Interstate 29 Northbound

Interstate 35 Northbound

Interstate 80 Eastbound

Figure 13. Average k value for Iowa Interstates, 1991

The primary term for governing the PSI values is the longitudinal profile value, which is calculated based on the relative roughness of the pavement as measured by a variety of instruments. The roughness may be expressed in a variety of different ways, but each instrument’s output values are correlated with equivalent values of the LPV. These roughness measures accumulate and are averaged over long segments of roadway. Deviations are measured relative to the previous value, and are quite sensitive to faulting or cracking conditions, but not low frequency variations in the vertical geometry of the roadway. Bridge approaches, deck joints, and other significant obstructions or variations in the road surface are ignored. The relatively low weight assigned to the cracking and patching term make it only a correction procedure that is typically insignificant. Road rater values are often used to quantify the ride quality and smoothness of new construction to determine if the contractor meets quality performance specifications for the roadway.

A correlation between the Present Serviceability Index and the design loading parameters of the pavement was generated as part of this study from the 1990 Pavement Management System Database supplied by the Iowa Department of Transportation. The data analysis was conducted using the JMP package on an Apple Macintosh personal computer (SAS Institute 1989). This is an interactive package that allows on-line data display, reduction, data editing, and statistical evaluation.

It must be remembered that the PSI is a calculated variable with the primary term being pavement smoothness, which may or may not directly relate to the structural integrity of the pavement slab. Many descriptors of the slab were included in an attempt to define the relationship. The final estimating equation is linear additive and is relatively simple, containing a dummy variable representing pavement type, total ESALs over the life of the pavement, average soil support value for the pavement segment, and the age since last lift or age since initial construction. The parameter estimates and goodness of fit statistics, shown in Figure 14 and Tables 1 and 2, result in a reasonably descriptive equation that carries a good deal of statistical and intuitive validity. The variable describing slab thickness never made a reasonable contribution to the statistical fit because all of the pavements were designed with forecast loadings as a design factor, and the slabs, within pavement type, are all approximately equal in depth. The data do not reflect a fully factored experimental design.

The plot of predicted and observed PSI values, also shown in Figure 14, demonstrates the strength of the relationship. The pavement type variable effectively modifies the point of intercept. Type 1 produces a value of 3.48 for PCC, 3.85 for Continuously Reinforced Concrete (CRC), 2.9 for composite, and 3.34 for asphalt. A normal value for subgrade support value would be approximately 150, which would increase the new construction average PSI for each
type by approximately 0.5, or values of 4.0, 4.35, 3.4, and 3.84 respectively. This value is affected to some degree by the fact that most composite pavements are older, weakened pavements that have had an asphalt overlay added.

**TABLE 1. SUMMARY OF FIT FOR PSI ESTIMATION EQUATION**

<table>
<thead>
<tr>
<th>Summary of Fit</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$R^2$</td>
<td>0.640189</td>
</tr>
<tr>
<td>Root Mean Square Error</td>
<td>0.293676</td>
</tr>
<tr>
<td>Mean of Response</td>
<td>3.455786</td>
</tr>
<tr>
<td>Observations (or Sum Weights)</td>
<td>280</td>
</tr>
</tbody>
</table>

**FIGURE 14. ACTUAL PSI V. PREDICTED PSI FOR IOWA INTERSTATE SEGMENTS, 1991**

**TABLE 2. PARAMETER ESTIMATES FOR PSI ESTIMATION EQUATION**

<table>
<thead>
<tr>
<th>Term</th>
<th>Estimate</th>
<th>Std Error</th>
<th>t Ratio</th>
<th>Prob&gt;</th>
<th>t</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>3.449474</td>
<td>0.19286</td>
<td>17.89</td>
<td>0.0000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type[2]</td>
<td>0.4270171</td>
<td>0.05174</td>
<td>8.25</td>
<td>0.0000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type[3]</td>
<td>-0.607898</td>
<td>0.06891</td>
<td>-8.82</td>
<td>0.0000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type[4]</td>
<td>-0.142711</td>
<td>0.08988</td>
<td>-1.59</td>
<td>0.1135</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total 18k ESALs</td>
<td>-1.5e-8</td>
<td>0</td>
<td>-4.04</td>
<td>0.0001</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average k value</td>
<td>0.0035597</td>
<td>0.00108</td>
<td>3.29</td>
<td>0.0011</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Surface layer age</td>
<td>-0.031572</td>
<td>0.00358</td>
<td>-8.81</td>
<td>0.0000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

It should also be pointed out that some segments of pavement were removed from analysis because of bad data. This included one segment of I80 near Des Moines because the PSI was not updated after new reconstruction, a short segment of I29 near Monona, and several short segments of I80 that had been grounded or resurfaced just prior to the PSI being estimated.
LCV CHARACTERISTICS AND PAVEMENT LIFE

As part of a previously funded project, an integrated software system was developed for evaluating the effects of vehicle characteristics on the pavement life. The details of the software system are presented in two Midwest Transportation Center reports (Stoner et al. 1990; Stoner et al. 1991).

In this section, the software system is used to study the effects of LCVs on the rigid concrete pavement response. In the first part of the section, the focus is on the computation of dynamic loads on the pavement from different LCVs. In the later part of the section some of these dynamic loads are applied to typical rigid pavements to predict the pavement life.

Dynamic Loads on Rigid Pavements from Heavy Commercial Vehicles

The following two different truck types are used for a parametric study on the dynamic loads from the heavy commercial vehicles.

1. Five-Axle Tractor-Semi-trailer (48 foot trailer)
2. Five-Axle Double (27 foot trailers)

The three Road Profiles used in this study include warping, random and sine-random. The warping road profile is used to model pavement warp due to joint spacing, as shown in Figure 15. The profile consists of a half sine wave between the pavement joints. Both the joint spacing and the amplitude of the sine wave are input parameters.

![Figure 15: Warping Road Profile with 0.5 Inch Amplitude and 30 Foot Wavelength](image-url)
The random road profile is actual profilometer data for both a rough and smooth road as shown in Figure 16 and 17 respectively. The sine-random profile is a combination of the random profile with a long full wavelength sine curve. The wavelength, amplitude and random profile type are all data input parameters. A rough sine-random road profile is shown in Figure 18.

**FIGURE 16. RANDOM ROUGH ROAD PROFILE**

**FIGURE 17. RANDOM SMOOTH ROAD PROFILE**
The truck parameters used in this study include the payload weight, trailer wheel base, and speed. The nominal tractor-trailer configurations are as follows:

<table>
<thead>
<tr>
<th>Type</th>
<th>Payload</th>
<th>Wheel Base</th>
<th>Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single (G.V.W. 80,000 lbs)</td>
<td>42,200 lbs</td>
<td>41.3 feet (1st trailer)</td>
<td>45 mph</td>
</tr>
<tr>
<td>Double (G.V.W. 80,000 lbs)</td>
<td>24,600 lbs (1st trailer)</td>
<td>21.3 feet (1st trailer)</td>
<td>45 mph</td>
</tr>
<tr>
<td></td>
<td>16,600 lbs (2nd trailer)</td>
<td>21.3 feet (2nd trailer)</td>
<td></td>
</tr>
</tbody>
</table>

The payload was varied from 50 to 150% of nominal payload weight. The pitch moment of inertia of the payload was varied simultaneously. The trailer wheel base was varied from 75 to 125% of the nominal trailer wheel base. The distance to the trailer and payload center of gravity was also proportionately varied. The truck speed was varied from 40 to 70 mph.

The road profile and truck parameters were varied to determine their effects on the Dynamic Load Coefficient (DLC). The DLC values represent an increase in the static axle loads due to dynamic effects. The reported DLC values are an average over the entire simulation time history. The static axle loads are multiplied by the average DLC values to get the dynamic loads on the pavement for later study on the pavement life prediction.

Complete DLC graphs for both the single and the double trailer configurations are shown in the appendix in Figures A-1 though A-18. The axles 4 and 5 (the rear tandem for the single trailer
configuration, and the rear trailer for the double trailer configuration) usually exhibit greater
dynamic load coefficients than the first three axles and are graphed in the following figures:

<table>
<thead>
<tr>
<th></th>
<th>Axle 4</th>
<th>Axle 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Truck Speed</td>
<td>Figure 19</td>
<td>Figure 20</td>
</tr>
<tr>
<td>Warping Wavelength</td>
<td>Figure 21</td>
<td>Figure 22</td>
</tr>
<tr>
<td>Sine-Random Wavelength</td>
<td>Figure 23</td>
<td>Figure 24</td>
</tr>
<tr>
<td>Payload Weight</td>
<td>Figure 25</td>
<td>Figure 26</td>
</tr>
<tr>
<td>Trailer Wheel Base</td>
<td>Figure 27</td>
<td>Figure 28</td>
</tr>
</tbody>
</table>

The effect of truck speed on the DLC is shown in Figures 19 and 20. For the smooth road
profile there is almost no change in the DLC from 50 to 65 mph but there is a slight increase
above 65 mph to a maximum smooth DLC of 0.30 for the double configuration. The rough
profile also shows a slight peak at 63 mph but exhibits a dramatic increase below 45 mph with
the double trailer configuration increasing to 0.43 and the single trailer to 0.70 at 40 mph.

The warping road profile is used to simulate the effect of pavement warp between joints. This
effect is most noticeable for the single trailer configuration as shown in Figures 21 and 22. For
the single trailer configuration there is a dramatic peak at a wavelength of 23 feet. The double
trailer reaches a maximum at 24 feet for the 5th axle and at 28 feet for the 4th axle.

Varying the wavelength of a sine-random profile within the range of 100 to 300 feet has no
effect on the DLC, as shown in Figures 23 and 24. However Figure A-13 in the appendix
suggests an slight increasing trend for wavelengths below 100 feet for the smooth road profile.

Figures 25 and 26 show the DLC is maximized at 70% of the nominal payload for the single
trailer on a rough road. For the double trailer on a rough road the largest DLC values are for the
lighter payloads. As the payload increases up to nominal there is a rapid decrease in the dynamic
load coefficient.

Increasing the payload above nominal seems to only gradually reduce the DLC. For the double
trailer on a smooth road the maximum DLC is reached at 80% of the nominal weight.

The DLC as a function of nominal trailer wheel base is shown in Figures 27 and 28.
FIGURE 19. COMPARISON OF AXLE 4 DLC VS. TRUCK SPEED FOR RANDOM ROUGH ROAD PROFILE

FIGURE 20. COMPARISON OF AXLE 5 DLC VS. TRUCK SPEED FOR RANDOM ROUGH ROAD PROFILE
FIGURE 21. COMPARISON OF AXLE 4 DLC VS. WAVELENGTH FOR WARPing ROAD PROFILE WITH 0.5 INCH AMPLITUDE

FIGURE 22. COMPARISON OF AXLE 5 DLC VS. WAVELENGTH FOR WARPing ROAD PROFILE WITH 0.5 INCH AMPLITUDE
FIGURE 23. COMPARISON OF AXLE 4 DLC VS. WAVELENGTH FOR SINE-RANDOM ROAD PROFILE WITH 0.5 INCH AMPLITUDE

FIGURE 24. COMPARISON OF AXLE 5 DLC VS. WAVELENGTH FOR SINE-RANDOM ROAD PROFILE WITH 0.5 INCH AMPLITUDE
Figure 25. Comparison of Axle 4 DLC vs. Percentage of Nominal Payload for Random Road Profile

Figure 26. Comparison of Axle 5 DLC vs. Percentage of Nominal Payload for Random Road Profile
**FIGURE 27.** COMPARISON OF AXLE 4 DLC VS. PERCENTAGE OF NOMINAL TRAILER WHEEL BASE FOR RANDOM ROAD PROFILE

**FIGURE 28.** COMPARISON OF AXLE 5 DLC VS. PERCENTAGE OF NOMINAL TRAILER WHEEL BASE FOR RANDOM ROAD PROFILE
For the rough road profile there is a slight increase at 80% of nominal trailer wheel base and dramatic increase at the trailer wheel base increases toward 125% of nominal. The minimum appears at 95% of nominal. For the single trailer configuration on the smooth road there is a slight increase below 100% of nominal up to a DLC of 0.25 at 75% of nominal. The DLC of the double trailer on a smooth road is constant for all wheel base lengths above 90% of nominal with slight fluctuations below 80%.

Rigid Pavement Response to Heavy Commercial Vehicles

The finite element program for analysis of concrete pavements developed previously (Stoner et al. 1991) has been modified to incorporate the effect of vehicle configurations in the pumping model originally developed by Larralde (1984). Further improvements in the program were aimed at reporting suitable measures to indicate cumulative pavement damage as the numbers of axle load applications is increased. The modified pumping model as implemented in the program and other damage indices are described in the following sections.

Description of Larralde’s Pumping Model

Pumping in rigid pavements is defined as the decay in the subgrade support due to the ejection of fine material and water with repeated applications of the traffic loadings. Water can accumulate in initial voids and be ejected through cracks and joints. The initial voids can be a consequence of temperature effects, or plastic deformation of the subgrade. The pumping of fine material produces zones of low bearing capacity underneath the concrete slab thereby modifying the support conditions. After severe pumping, voids are created with the consequent loss of subgrade support.

The amount of material pumped depends on a number of factors, such as structural properties of the pavement, magnitude and number of load applications, climatic conditions, and type of material used in the subgrade. Field observations have indicated that the initial pumping is more severe at the edges of the pavement and along the joints (Gulden 1983; Yoder and Witczak 1975). Once the support condition has deteriorated beyond a critical limit, cracks begin to form providing additional locations for pumping to develop.

A pumping model developed at Purdue University (Larralde, 1984), and based on the results of the AASHO Road Tests (AASHO 1962b) is used as the basis for pumping model in this program. This model accounts for the effects of structural properties of the concrete pavement and the amount of traffic imposed. Climatic conditions and granular composition of the subgrade, as well as quality of drainage, are taken into account through modifying factors (Van
Wijk et al. 1989). The model is defined in terms of two basic parameters; the total energy of deformation imposed on the pavement and the normalized Pumping Index.

The energy of deformation is defined as:

$$ E = \sum_{i=1}^{n} k_i A_i \omega_i^2 $$

where

- $E$ = energy of deformation due to one application of loading (in.-lb)
- $\omega_i$ = vertical deflection in excess of $\omega_{\text{min}}$ (in.)
- $\omega_{\text{min}}$ = minimum vertical deflection necessary to induce pumping. In this model it is taken as 0.020 inches (20 mils)
- $A_i$ = area of the slab associated with $\omega_i$
- $k_i$ = subgrade modulus associated with location of $\omega_i$
- $n$ = number of location at which $\omega_i > \omega_{\text{min}}$

The Pumping Index is defined as the volume of material pumped per unit length of pavement. To account for different slab lengths used during the AASHO Road Tests, a normalized pumping index was defined by dividing the pumping index value by the number of joints per 100 lineal feet.

The relationship between the total deformation energy, after a given number of load repetitions, and the normalized pumping index is given by:

$$ NPI = e^{1.652 \log_{10}(\frac{E_t}{10,000}) - 2.884} $$

where

- $NPI$ = normalized Pumping Index (in$^2$)
- $E_t$ = total deformation energy (inch-pounds) = energy of deformation due to one application of load times the number of load applications.

Using the normalized pumping index, the total volume of pumped material is computed as follows.

$$ V = NPI \times \text{Slab\_length} \times \text{N\_joints} $$

where

- $V$ = total volume of material pumped from underneath the slab.
- Slab\_length = length of the slab on which damage is being computed.
- N\_joints = number of joints for each 100 ft of pavement.
Once the total volume of material pumped has been estimated, it is used to define an equivalent void under the pavement slab. Larralde assumed a uniform void under the entire slab and adjusted the void depth to get at the total volume of pumped material.

This model was incorporated into the pavement analysis program IowaRigidPav (Stoner et al. 1991). The implementation of the model was verified by comparing results obtained from IowaRigidPav with those presented by Larralde (1984). The problem used for comparison purposes is the one presented by Larralde (1984) and depicted in Figure 29. It consists of two concrete slabs 180 inches long and 140 inches wide. The slabs are connected every 12 inches by dowels of 1.25 in. diameter. Two loads equivalent to a 40 Kips axle were placed at the joint as shown in the figure.

![Figure 29. Pavement System Used to Compare Results](image)

The displacement values predicted by IowaRigidPav and those reported by Larralde (1984) were compared for two different pavement thicknesses. In the first case a thickness 7 inches was used. The maximum deflection predicted by IowaRigidPav is 12% larger than that reported by Larralde (the actual values were 49 mils from IowaRigidPav vs. 43 mils from Larralde). In the second case the pavement thickness was taken as 10 in. The maximum deflection predicted by IowaRigidPav is 15% higher than the one reported by Larralde (1984); i.e. 31 mils vs. 27 mils. The difference between these results, for the same finite element discretization, is consistent with
the different element types used in the two programs. IowaRigidPav uses a nine-node isoparametric plate element whereas the Larralde’s results were based on a rectangular four-node element, which is a comparatively stiffer element.

The pumping index was computed for an 8 inch thick pavement with 1,000,000 load applications. The NPI was found to be 30% higher than that reported by Larralde (1984). It is believed that this difference is due to the larger displacements computed by IowaRigidPav.

**Modifications to the Pumping Model**

The above described Larralde’s pumping model was developed for prediction of ultimate pumping effect under the action of a single axle load located at a joint. Two fundamental modifications were made to the model to make it a suitable tool for predicting pumping damage after a specified number of truck passes. The first modification was related to the way the deformation energy was computed and the second was in the way in which the amount of material pumped was distributed under the pavement. These modifications are discussed in this section.

*Deformation energy associated with one complete pass of a truck over a pavement joint*

Larralde’s model does not account for the axle configuration of different types of heavy trucks. The axle spacing will influence deformations at a joint when a truck passes over it. If the axles are closely spaced there will be a very strong influence of the adjacent axles. When a truck passes over a pavement joint, portions of the slabs adjacent to the joint will deform with different magnitudes under the action of different axles. The energy of deformation imposed by each of these axles on a given segment of the pavement will be a function of the axle weight and the spacing between axles. Even if all axle weights are the same for a given truck, the energy of deformation imposed by any one of the axles will be influenced by the others and will likely be different for each axle. To account for this situation, the basic definition of energy of deformation used in computing NPI is generalized from the one based on a single axle to the one based on one pass of the entire truck over a given joint in the pavement. It is assumed that the damage computed at a given joint will be representative of the damage observed by other nearby joints in the pavement. For this reason concept of a “reference slab” and “reference joint” is introduced. All pumping related calculations are performed across the reference joint. The deformation energy is computed in the reference slabs. After determining the pumping damage, the same damage is applied to all the joints in the model. The reference slab and joint configuration is represented by the shadowed area in Figure 30.

To compute energy of deformation the first truck axle is placed at the reference joint. From the geometry of the truck the location of the other axles is calculated. An elastic analysis of the
pavement is then performed. Based on the deformations in the reference slab, the energy of deformation associated with the current axle is then calculated. This computation is repeated for each of the other axles in the truck and the energy of deformation for all the axles is added together. It is this total energy of deformation which is used to compute the normalized pumping index as a function of the number of truck passes. The tandem axles are treated as single axles with the heavier of the two axles being placed at the joint itself. Thus three separate analyses are required for computing deformation energy associated with one pass of a typical 18 wheel tractor-trailer.

![Diagram of reference segment used for calculation of pumping damage.](image)

**Figure 30. Reference segment used for calculation of pumping damage.**

**Distribution of pumped material**

The normalized pumping index is calculated using the modified definition of the deformation energy. This value of NPI is used in calculating the total volume of material pumped out from under the reference slab. Larralde's assumption of uniform void depth under the entire slab does not seem reasonable. Instead it appears more realistic to define larger voids near the joints where the deformations are the greatest and reduce the void depths gradually as one moves away from the joint. As a second modification to the Larralde's model, the void depth is made a function of the deformation energy instead of being constant. To implement this concept the energy of deformation associated with the current material and subgrade support conditions is calculated. The volume of material pumped associated with each finite element is assumed to be proportional to the contribution of that element towards the current deformation energy of the slab as follows.

\[
V_{pump. \, elem.}^i = \frac{E_{elem}^i}{E_{slab}} \cdot V_{pump. \, slab} \tag{3}
\]

where

\[
V_{pump. \, elem.}^i = \text{Volume of material pumped associated with } i^{th} \text{ element.}
\]

\[
E_{elem}^i = \text{Energy of deformation associated with } i^{th} \text{ element.}
\]
\[ E_{\text{slab}} = \text{Total energy of deformation of the reference slab.} \]

\[ V_{\text{pump, slab}} = \text{Total volume of material pumped.} \]

The actual void depth under an element is then calculated by assuming it to be uniform under the element. This operation is repeated for all the elements that were included in the deformation energy calculations. This process leads to the total area of the pavement that is covered by pumping near the reference joint. The subgrade support for the area covered by pumping is removed from around all joints in the model. Because the distribution of the material pumped is based on the current configuration, modification of the subbase support condition will alter the corresponding energy of deformation and the contribution of each element to pumping. Therefore iterations must be performed until convergence of the area covered by pumping is achieved.

This procedure for distribution of the material pumped is repeated after each increment in the number of load applications and the growth in the area covered by pumping damage is monitored.

It is important to note that the model developed by Larralde (1984) was based on the results presented in the AASHO Road Test, Special Report 61E, Tables 54 and 55 (AASHO 1962a). These tables present the number of load repetitions necessary to bring the pavement to failure. Thus Larralde’s model was designed to predict the total number of load applications necessary to bring a pavement to failure based on the imposed energy of deformation. With the modifications introduced in the present work, the basic model is extended to predict pumping damage after any number of truck passes up to failure of the pavement. Because of lack of intermediate data from experimental investigation and because of time constraints these extensions have not been validated so far. The only justification for their use is that the results predicted qualitatively agree with the field observations. Work is underway in validating these modifications and perhaps coming up with a better pumping model using the original AASHO data. The main difficulty so far has been in actually going through the complete AASHO data and isolating parts related to the pumping study because of lack of readily available key to the contents of the data tape.

**Rigid Pavement Damage Indices**

A series of damage indices has been incorporated into the IowaRigidPav program. These indices are reported after a specified number of truck passes. These indices correspond to the reference slab and are assumed to be the same for any slab in the pavement system used in the analysis.
Surface Area affected by Cracking

This index represents the percentage of the top surface of the reference slab that has been cracked.

Cracked Volume.

In the IowaRigidPav program each element is divided into several layers. In each layer the stress calculations are performed at the Gaussian points used for numerical integration of the stiffness matrix. The cracking is therefore monitored at the Gaussian integration points in each concrete layer. Thus it is possible to monitor crack propagation through the thickness of the pavement. To reflect the severity of cracking in the pavement slabs a “cracked volume index” is defined as follows.

\[
C.I. = \frac{1}{V_t} \sum_{i=1}^{N} A_i d_i
\]  

(4)

where

C.I. = cracking index,
N = total number of cracked integration points,
V_t = total volume of reference slab,
A_i = area of slab associated with i\textsuperscript{th} integration point
d_i = thickness of layer associated with i\textsuperscript{th} integration point.

Volume of Subgrade Material Pumped from Underneath the Reference Slab.

This index quantifies the severity of pumping in a given pavement system. Its calculation is outlined in the previous section.

Area Over which Pumping Damage has Occurred.

The procedure for calculating the area covered by pumping is described in the previous section. It is reported by the IowaRigidPav program as a percentage of the area of the reference slab. It represents the extent of the pavement over which there is no subgrade support.

Decay in Concrete Slab Stiffness.

This index is associated with the fatigue behavior of concrete. It is defined as follows.

\[
F.I. = \frac{1}{E_o V_t} \sum_{i=1}^{N} (E_o - E_i) A_i d_i e
\]  

(5)
where

\[
\begin{align*}
F.I. &= \text{fatigue index}, \\
N &= \text{total number of cracked integration points}, \\
V_t &= \text{volume of the reference slab}, \\
A_i &= \text{area of slab associated with } i^{th} \text{ integration point}, \\
d_i &= \text{thickness of layer associated with } i^{th} \text{ integration point}, \\
E_{0i} &= \text{initial modulus of elasticity of concrete corresponding to } i^{th} \text{ integration point}, \\
E_r &= \text{modulus of elasticity at } i^{th} \text{ integration point after modifications due to fatigue damage}.
\end{align*}
\]

**Damage Indices for Different Truck and Pavement Parameters**

The IowaRigidPav program is used along with the Truck simulation program to study the effect of different truck and pavement parameters on the pavement damage in terms of the damage indices defined in the previous section.

**Pavement Parameters**

- Pavement thicknesses: 11 inches & 12 inches
- Subgrade modulus: 150 lbs/ft³

**Concrete**

- Modulus of Elasticity: 4.20 \(10^6\) psi
- Compressive Strength: 5570 psi
- Tensile Strength: 550 psi
- Density: 150 lbs/ft³
- Dowel Modulus: 1.50 \(10^6\) psi

**Dowel**

- Modulus of Elasticity: 2.70 \(10^7\) psi
- Diameter: 1.25 in
- Spacing: 12.0 in

**Truck Parameters: Five-Axle Tractor-Semi-trailer with full payload**

- Speed: 45 mph
- Road profiles: Rough (r) and smooth (s)

The following figures compare damage indices as function of number of truck passes when the pavement thickness and road profiles are changed. Figures 31 through 35 show, respectively, the cracked area, cracked volume, volume of material pumped, area covered by pumping and the fatigue index.
FIGURE 31. ROAD PROFILE AND PAVEMENT THICKNESS EFFECTS ON SURFACE CRACKING

FIGURE 32. ROAD PROFILE AND PAVEMENT THICKNESS EFFECTS ON CRACKED VOLUME INDEX
**Figure 33.** Road profile and pavement thickness effects on volume of material pumped.

**Figure 34.** Road profile and pavement thickness effects on area covered by pumping.
Figure 35. Road Profile and Pavement Thickness Effects on the Fatigue Index
HIGHWAY DESIGN, VEHICLE CONFIGURATION, AND ACCIDENT RISK

Accident Experience by Vehicle Type and Driver

Iowa experience for trucks

Trucks tend to have safety records that are quite good when measured against passenger cars, but this is partially because persons under 25 are not employed as truck drivers. The accident rate for large trucks in Iowa, when adjusted for the fact that a large number of the miles are on interstate highways that generally have lower accident rates, is not significantly different than for passenger cars. The distribution for age of truck drivers involved in vehicle accidents does not appear to be significantly different than the age distribution for all truck drivers. The only factor that appears to stand out is the larger numbers of accidents where fatigue or falling asleep was listed as a factor. Data on size and weight of vehicles involved in accidents is not readily available, but over sized vehicles were listed as a factor in only 1.2 percent of the truck tractor with trailer accidents. The accident rate by location appears to be directly related to the level of congestion, resulting in extraordinary descriptive statistics such as thirty percent of all interstate system accidents occur in Polk County and nearly two-thirds occur in Polk County and on interstate 80 East of Des Moines, the segments of the system that coincidentally have the highest traffic volumes.

Involvement versus fatality

Large trucks tend to have a lower overall accident rate than other types of vehicles, but nearly double the probability of a fatality when there is an incident. Combination vehicles are involved in a relatively small number of accidents compared to other vehicles, but the accidents they are in tend to be more severe and involve more fatalities. In 1986, passenger cars were involved in 633 accidents per 100 million Vehicle Miles of Travel (VMT), over twice the accident rate of 287 for all combination trucks. In contrast, the fatal accident rate for passenger car only accidents was 2.87 per 100 million VMT, versus 4.95 for those involving combination trucks.

This comparison highlights one major factor in analyzing accident rates: VMT. Researchers relate accidents to exposure by using an assumed value for VMT. When relative accident rates of twin trailers versus tractor-semi-trailers are the focus of attention, the relative VMTs become very important. Accidents, and especially fatal accidents are nearly always recorded. The generation of trips, the basis for VMT, is not centrally recorded. Relative measures of VMT by various vehicle configurations, for rural versus urban roads, by time of day, are thus often estimates rather than actual measures. Given the low rate of fatal accident occurrence,
approximately 3 per 100 million VMT, the method for estimating VMT is the determining factor in how much confidence can be ascribed to the estimate of accident rates by vehicle type.

Accident research also suggests that driver attitude and experience, as well as the driving environment are the major factors that affect truck accident rates.

Vehicle mass

Data on accident involvement for combination vehicles on both high and low design standard highways would appear to be very inconclusive, being very dependent on how the data are aggregated into class. There would seem to be some support for an assumption that higher gross vehicle loads can lead to higher fatality involvement rates, especially on non-interstate level highways. (National Research Council 1990).

Vehicle type

Multiple trailer vehicles are less likely to be involved in accidents on urban roads, and more likely to be involved in accidents on rural roads (Graf and Archuleta 1985). An Insurance Institute for Highway Safety study used a case control method and found that multiple trailer vehicles were two to three times more likely to be involved in an accident than single trailer vehicles. It should be noted that the highest accident rates are for tractors operating without a trailer. Large carriers typically report lower accident rates for the more exotic combinations, but that may be due to a generally good internal training program and the assignment of drivers with the greatest amount of experience.

Accident Experience by Geometric Design Standard and Functional Class

Heavy truck safety is highly dependent on the type of roadway that they are operated over. interstate design standard divided roadways with grade separated interchanges are inherently safer. Specific investigations have found deficiencies in accident involved drivers with respect to training and experience. Drivers can become more easily fatigued because of the increased vigilance required for multiple trailer vehicles, especially in bad weather.

Additional lane and shoulder widths would increase lateral clearance with respect to obstructions or other vehicles in the traffic stream, but the reduction in accident rate becomes quickly non-linear depending upon the initial conditions. Additional widths for bridges are also critical. Larger vehicles will have greater difficulty negotiating sharp roadway curves and intersections. Roughly half of all urban accidents and nearly a third of all rural accidents occur at intersections.
Interchange exit and entrance ramp design standards are not based on the higher center of gravity and other stability related factors that are peculiar to any truck, much less those imposed by larger and heavier LCV vehicles.

**Effects of Vehicle Configuration on Vehicle Stability and Response**

**Sight Distances**

Braking distances for safe stopping of multi-trailer trucks can be extended (up to 800 feet from 55 mph), which is more like the distance to stop a passenger car safely from 80 mph. This can be ameliorated somewhat by the higher height of eye of the driver. The resulting correction provides a resulting effective speed differential of around 10 mph. Research is conflicting with respect to the influence of vehicle length on required passing sight distances.

Stopping distances are dependent upon the dynamic loads and brake torque at each wheel, and the prevailing road surface friction coefficient. When the torque level exceeds the level dictated by the wheel load, the wheel will lock causing a loss of control. If the wheel load is too high then the brake shoes will be saturated and not contribute to slowing the vehicle. There are wide variations in the capabilities of truck braking systems. Increased payloads and attendant axle loads do not contribute to greater stopping distances, especially if the weight is concentrated toward the rear of the vehicle or in the rear most trailers.

**Speed Variation**

Greater speed variation caused by higher weight to horsepower ratios for LCVs and rolling or hilly terrain, can result in higher accident rates. This situation can be made worse by poor aerodynamics resulting from gaps between trailers.

**Low Speed Off-tracking**

Low speed off-tracking is the tendency for the trailers to track to the inside of the curve, usually below 35 to 40 mph. Multiple trailer combinations typically display less low speed off tracking than single long wheel base trailers.

**High Speed Off-tracking**

High speed off tracking is usually characterized by the rearward trailers tracking towards the outside of the curve. The swept area for multi-trailer vehicles can extend into other lanes. Conditions that would result in lower level of low speed off tracking, such as shorter trailers and more articulation points, typically result in greater high speed off-tracking.
Rear Amplification

Rear amplification describes the exaggeration of sudden steering movements as the lateral motion extends to the rearmost trailers. This effect is usually a direct function of the number of trailers, but can be affected by the trailer wheel bases, load distribution within individual trailers, relative load between contiguous trailers, and the type of converter dolly.

Jack Knifing and Trailer Swing

Additional trailers can serve as a stabilizing influence if the vehicle has well-maintained traditional braking systems or if outfitted with anti-lock breaking systems. If the last trailer is light, then the braking system could cause the breaks to lock causing significant trailer swings. Brakes locking up on the rear axle of the tractor can cause a jack knife effect. Braking systems that are biased toward the last trailer or rear most axles seem to positively affect straight line braking.

Multiple trailers display a tendency to oscillate continuously: a potential hazard only if following drivers make unnecessary actions to avoid the potential path of the rear trailer.

Load Distribution

Trucks are more susceptible to rollover than passenger cars because of their higher center of gravity. This can be evaluated on individual vehicles by examining the center of gravity height relative to track width, and calculating or measuring the rollover threshold. A typical CG height for baseline vehicles would be 80 inches (203 cm). Suspension type and coupling equipment can also affect the rollover threshold for combination vehicles. Increasing the allowable payload and holding commodity densities and trailer design constant, will obviously decrease the rollover threshold.

The decrease in rollover threshold is most pronounced for short, single axle trailers, because the rise in CG height with an increase in axle load is more rapid than for the longer trailers (Ervin 1986). For any combination vehicle, the probability of rollover seems to increase directly with the relative loading, most likely because of the direct relationship between payload and CG height.

Vehicle stability issues, including stopping distances, handling and steering response, rollover threshold, rear amplification, and yaw stability are perhaps more a function of payload placement than gross vehicle weight or vehicle configuration. Lateral offset of the load, placement of heavier loads in forward trailers, or having an empty rear trailer, and non uniform load distributions in or on a trailer can have very deleterious effects on the ability of the driver to control the vehicle, yet they may be virtually impossible to regulate or enforce because of the day
to day requirements imposed on the freight vehicle. It is also possible that more liberalized payload and axle weight distributions may result in a perfectly safe vehicle in a fully loaded condition, but a very sub-optimal one when common sense rules of vehicle loading are violated. For a more thorough discussion and the results of a wide variety of vehicle simulations see Ervin (1986).

**INTERIM CONCLUSIONS**

The finite element model for rigid pavement performs quite well and has been validated against both known solutions and experimental data. The portion of the model most open to question concerns the validity of the modified Larralde pumping model when extended to the determination of intermediate pavement damage resulting from pumping rather that just predicting the number of replications to slab failure.

The vehicle modeling packages used to estimate dynamic load coefficients for alternative vehicle configurations and road roughness conditions appear to be quite accurate.

The initial tests of alternative vehicle configurations indicate that the traditional doubles combination vehicles cause considerably less damage than a single 48 foot trailer with the same payload. The 28 foot double trailer configuration with the maximum gross weight permitted by bridge formula, 107,000 lbs, does approximately the same level of damage to an 11 inch slab as an 80,000 lb. single trailer unit. This is primarily due to the more even load distribution and lower overall dynamic load coefficients. The single 48 foot trailer may also be among the worst configurations for damage to a plain jointed concrete pavement with a 20 foot slab length. It is important to note that the findings could be quite different for flexible or composite pavement designs.

The addition of multi-trailer vehicles to the traffic stream is not likely to increase the overall level of safety, and may not result in any significant contribution to the accident and fatality rates. Certain LCV combinations may be quite dangerous, however, depending upon the number of articulation points and the load distribution within and between trailers.

The final project report will examine the pavement damage, geometric design, and safety issues in greater detail.
REFERENCES


APPENDIX

Dynamic Load Coefficient (DLC) for Different Vehicle Configuration

This appendix contains a complete set of DLC graphs for different vehicle configurations. Each graph has a separate curve for each axle of the vehicle. The graphs for the 4th and 5th axles, usually those with the largest DLC, are also presented in the main body of the report.

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Figure A-1. DLC vs. Percentage of Nominal Payload with Random Smooth Road Profile Single 48 Foot Trailer Configuration

Figure A-2. DLC vs. Percentage of Nominal Payload for Random Smooth Road Profile Double 27 Foot Trailer Configuration
FIGURE A-3. DLC VS. PERCENTAGE OF NOMINAL PAYLOAD FOR RANDOM ROUGH ROAD PROFILE 48 FOOT SINGLE TRAILER CONFIGURATION

FIGURE A-4. DLC VS. PERCENTAGE OF NOMINAL PAYLOAD FOR RANDOM ROUGH ROAD PROFILE 27 FOOT DOUBLE TRAILER CONFIGURATION
FIGURE A-5. DLC VS. TRUCK SPEED FOR RANDOM SMOOTH ROAD PROFILE SINGLE 48 FOOT TRAILER CONFIGURATION

FIGURE A-6. DLC VS. TRUCK SPEED FOR RANDOM SMOOTH ROAD PROFILE DOUBLE 27 FOOT TRAILER CONFIGURATION
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**Figure A-8.** DLC vs. Truck Speed for Random Rough Road Profile 27 Foot Double Trailer Configuration
Figure A-9. DLC vs. Percentage of Nominal Trailer Wheel Base for Random Smooth Road Profile Single 48 Foot Trailer Configuration

Figure A-10. DLC vs. Percentage of Nominal Trailer Wheel Base for Random Smooth Road Profile Double 27 Foot Trailer Configuration
FIGURE A-11. DLC VS. PERCENTAGE OF NOMINAL TRAILER WHEEL BASE FOR RANDOM ROUGH ROAD PROFILE 48 FOOT SINGLE TRAILER CONFIGURATION

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FIGURE A-13. DLC VS. WAVELENGTH FOR A SMOOTH SINE-RANDOM ROAD PROFILE WITH A 0.5 INCH AMPLITUDE SINGLE 48 FOOT TRAILER CONFIGURATION

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FIGURE A-18. DLC VS. WAVELENGTH FOR WARPING ROAD PROFILE WITH 0.5 INCH AMPLITUDE DOUBLE 27 FOOT TRAILER CONFIGURATION