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Symposium on Geometric Design for Large Trucks

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Introduction

J. W. HALL, Chairman, Symposium Planning Committee

The Surface Transportation Assistance Act (STAA) of 1982 was a landmark piece of federal legislation intended to improve the quality of the nation's deteriorating highway system. One provision of this legislation, which is of great concern to highway engineers, introduced new truck size regulations. They required all states to allow tractors with 48-ft semitrailers, double-trailer combinations with 28-ft trailers, and 8.5-ft-wide vehicles. The 1982 STAA vehicle size provisions apply to a designated National Network that consists of the entire Interstate system and portions of the Federal-Aid Primary system designated by the Secretary of Transportation. The states were also required to provide access routes for the federally authorized vehicles between the National Network and terminals and fuel and rest stops. At the time this legislation was enacted, several states, primarily in the East, prohibited 48-ft semitrailers and double trailers, and most states prohibited trucks wider than 8 ft.

Some segments of the highway engineering community did not have in place the appropriate practices and procedures to design and operate roadways for these vehicles. Larger trucks can create problems because of their physical size and their potentially poorer operational characteristics. Several critical aspects of accommodating the 1982 STAA trucks are not addressed in AASHTO's "Policy on Geometric Design of Highways and Streets." The geometric design and operational factors of principal concern appear to be

- Sight distance and no-passing zones,
- Grades and climbing lanes,
- Intersection design and operation,
- Interchange and ramp design,
- Roadside design and traffic barriers,
- Traffic control device usage, and
- Safety.

Bureau of Engineering Research, Department of Civil Engineering, University of New Mexico, Albuquerque, N.M. 87131.

In response to the perceived gap in existing highway design standards, four TRB committees with interest in this subject planned and sponsored a Symposium on Geometric Design for Large Trucks in August 1985. The intent of this meeting was to document the state of the art in design and operational practices for these vehicles.

The symposium relied heavily on the expertise of designers in several states who had experience in accommodating large trucks on their highway systems and on researchers who had thoroughly studied selected aspects of the problem. The 24 papers presented at the symposium and included in this Record highlight many of the unusual demands that 1982 STAA vehicles place on the geometric design of highways. It is not possible to state with certainty which of the design and operational issues discussed in this Record are the most critical; however, highway engineers should be cognizant of the special problems posed by the 1982 STAA vehicles and attempt to ameliorate them in the design or redesign of roadways on the National Network.

Appreciation is expressed to the members of the Symposium Planning Committee whose work and dedication made the symposium possible:

J.A. Cirillo, Federal Highway Administration
R.W. Eck, West Virginia University
D.B. Fambro, University of Tennessee,
J.C. Glennon, John C. Glennon Chartered
T.E. Mulinazzi, University of Kansas
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C.M. Walton, University of Texas

National Network for Trucks: Development, Performance, and Outlook

JOHN P. EICHER, THOMAS E. KLIMEK, and SHELDON G. STRICKLAND

ABSTRACT

The Surface Transportation Assistance Act (STAA) of 1982, as amended, contains provisions that concern the length, weight, and width of commercial motor vehicles. By enacting STAA Congress preempted state authority completely with respect to width and partly with respect to length. Congress also extended length and width controls to those portions of the Federal-Aid Primary system designated by the Secretary of Transportation. The STAA also requires the states to provide access for commercial vehicles from the Interstate and other designated highways to terminals and facilities for food, fuel, repair, and rest, and for household goods carriers to points of loading and unloading. In this paper the development of the networks is explained, some observations on how well the system is working are presented, forthcoming changes are described, and some speculation about the near future is offered.

If Paul Revere had been alive in 1983 and had felt compelled to warn the people of the New England countryside of a threat to their way of life, he might well have borrowed a 1965 Plymouth with loud speakers on the roof from a fundamentalist preacher and with shrieks of hysteria sounded the alarm that "the doubles are coming, the doubles are coming."

There have been few issues in recent time that have tested federal-state relations, strained old friendships, and evoked such public outcry as the federal law that allows larger trucks to operate on certain highways. And, although the dust is beginning to settle after two years, it has been costly. The FHWA has been in a federal district court more than 10 times (twice as plantiff), and one case is still pending. Thousands of pieces of correspondence, which have required untold thousands of person-hours to answer, have been received at FHWA headquarters and field offices. Federal Register issuances pertaining to the large truck network totaled more than 20 as of July 1985, an unprecedented average of almost one every 1 1/2 months; and hundreds of unanswered questions, which will require the dedication of resources for the next decade to fully answer, have been raised. Perhaps a little history is in order.

On January 6, 1983, the Surface Transportation Assistance Act (STAA) of 1982 became law. Several provisions of the law concern the length and weight of commercial motor vehicles. On April 6, 1983, the STAA was amended to include truck width provisions.

Before the enactment of these laws, federal involvement in these areas was limited to matters involving permissible maximum vehicle weights and widths and was limited in applicability to the National System of Interstate and Defense Highways.

The changes created by the STAA have been dramatic because, as far as the Interstate system is concerned, Congress has preempted state authority completely with respect to width and partly with respect to length. Congress also extended length and width controls to those portions of the Federal-Aid Primary (FAP) system designated by the Secretary of Transportation. The secretary has been authorized to seek injunctive relief as the method of enforcing these provisions.

The dimensional limits established by the STAA include

1. Weight--All states must now allow on the Interstate system 20,000 lb on a single axle, 34,000 lb on a tandem axle, and a gross weight limit determined by the bridge formula with a cap of 80,000 lb. The bridge formula develops a maximum gross weight by taking the number of axles and their spacing into account.
2. Width--All states must establish a 102-in. width limit, excluding safety devices, applicable to what is now called the National Network, which will be fully explained later in this paper. All but three states (Connecticut, Hawaii, and Rhode Island) had to enact legislation on this issue to come into conformance.
3. Length--All states must allow on their portion of the National Network:

- A 48-ft semitrailer in a tractor-semitrailer combination; however, semitrailer lengths in normal, nonpermitted use on December 1, 1982, must continue to be allowed.
- A tractor-semitrailer-trailer or "doubles" combination vehicle. This has now been interpreted by the U.S. Department of Transportation (DOT) as including tractor-semitrailer-semitrailer vehicles in order to allow the use of new coupling methods for the units.
- Twenty-eight-foot trailer and semitrailer units as a part of "doubles." Twenty-eight-and-one-half-foot-units in legal operation within a 65-ft overall length limit on December 1, 1982, must also be allowed. However, more than 97 percent of these particular units belong to one company, and they are phasing them out in favor of the 28-ft units.
- Tractor-semitrailer and tractor-semitrailer-trailer (or second semitrailer) to operate without being subject to an overall length limit.

As an indication of the regulatory changes required by the length provisions of the STAA consider

Office of Motor Carrier Transportation, FHWA, U.S. Department of Transportation, 400 7th Street, S.W., HCT-1, Washington, D.C. 20590.

this list of conditions in effect just before passage of the STAA.

- Eleven states had semitrailer limits of less than 48 ft; 38 states had no semitrailer length limit but governed the combination by an overall length limit.
- Fifty states and the District of Columbia had overall length limits on tractor-semitrailer combinations applicable to what are now National Network highways.
- Twelve states and the District of Columbia did not allow doubles to operate at all; 11 states that allowed doubles restricted their movement to certain highways or required permits.
- Thirty-eight states had overall length limits on doubles applicable to what are now National Network highways.

Obviously, the length provisions of the STAA required at least some regulatory changes in almost every state.

Finally, the STAA also requires that the states provide access for commercial motor vehicles from the Interstate and other designated roads to terminals and facilities for food, fuel, repair, and rest, and for household goods carriers to points of loading and unloading.

In this paper the development of the networks is explained, some observations on how well the system is working are presented, forthcoming changes are described, and speculation about the near future is offered.

NATIONAL NETWORK

The STAA mandates that the full Interstate system be available for the operation of commercial vehicles of the dimensions authorized. In addition, the Secretary of Transportation was required to designate qualifying Federal-Aid Primary (FAP) system highways on which the larger vehicles must be allowed to operate. The term "National Network" was coined to designate the combination of the Interstate system and those portions of other FAP highways on which commercial vehicles of the dimensions authorized by the STAA would be permitted to operate.

The FHWA could have undertaken the designation process solely as a federal initiative without input from the states. This option was quickly dismissed. In the highway program that has existed since 1916, policy and practice have always been matters of state initiation and federal review and, if appropriate, approval. Thus the FHWA decided to designate a network in cooperation with the states. Cooperation with the states in this exercise was essential because the FHWA (headquarters, regions, or divisions) does not maintain files on the detailed geometrics of the highway system. Further, the FHWA is not staffed to undertake such a detailed task covering the 256,000 mi of the non-Interstate FAP system.

Two distinct approaches were available for drafting the message to be communicated to the states through the initial policy statement. One approach was to designate the entire FAP system in each state and let the states request removal of all mileage that they believed was unsafe for operation of the larger vehicles. The second approach was to designate only those FAP routes that met the highest standards, namely multilane, divided, full-control-of-access facilities, and let the states propose additions to this system that they believed were safe for the operation of the larger vehicles. The final decision was to adopt the second approach because it fit the traditional pattern of the federal-state relationship and it was anticipated that all states would cooperate in the development of a consistent interim network. The goal of the FHWA was to designate a consistent system that could safely accommodate these vehicles. Under either approach, FHWA viewed the FAP system as a generic class that could safely accommodate the larger vehicles.

The responses from the states varied greatly. For example, 13 states recommended 100 percent of their FAP systems, 6 states recommended more than 50 percent of their FAP systems, and 11 other states recommended from 10 to 50 percent of their FAP systems. The remaining 22 states recommended from 0 to 10 percent of their FAP systems. Furthermore, several of the lean submissions consisted of short and unconnected segments. In total, the states initially recommended about 38 percent of the non-Interstate FAP system, or approximately 96,000 mi.

Many states appeared unresponsive to FHWA policy statements of February 3 and March 10, 1983, and because of the extremely limited networks proposed by those states, it appeared that Interstate commerce would be impeded. The FHWA decided to supplement the recommendations of the states.

On April 5, 1983, the FHWA published the interim National Network for the larger vehicles. The 96,000 mi recommended by the states and accepted by FHWA were supplemented by an additional 40,000 mi selected by the FHWA. To emphasize the interim nature of the network and the continuous refining process that the FHWA had earlier announced, the April 5 publication also offered an opportunity to request exceptions to the interim network.

Thus was set in motion a process that was designed to refine the interim network, relying heavily on the judgment of and input from the state highway agencies.

Also immediately following the April 5 publication, the states of Alabama, Florida, Georgia, Pennsylvania, and Vermont requested U.S. District Courts to enjoin the designation of all highways on the interim network that had not been recommended by the individual states. In response the FHWA removed from the interim network all routes not recommended by the five states. These cancellations resulted in a reduction of 8,800 mi.

Between April 5 and July 8, 1983, the FHWA actively sought recommendations for revisions to and did revise the interim National Network. The result was an interim network in 32 states and the elimination of more than 7,200 FAP system miles. Furthermore, the total cancellation of FHWA-designated mileage in Alabama, Florida, Georgia, Pennsylvania, Vermont, and later Connecticut (due to litigation brought by FHWA against Connecticut) resulted in a reduction of more than 9,000 mi.

This refined and reduced network of approximately 162,000 mi was subsequently offered for public comment in the September 14, 1983, Notice of Proposed Rulemaking (NPRM). As a result of public comments and recommendations by state highway agencies, further additions and deletions were made that resulted in a net addition of about 19,000 mi for a total of approximately 181,000 mi.

As of June 5, 1984, 181,000 mi of FAP routes were open to vehicles authorized by the STAA.

12-FT LANES

The final National Network is undergoing an additional formal examination that has the potential for causing some adjustment involving the inclusion of segments with less than 12-ft lanes.

In part because of language in a Memorandum Opinion issued March 27, 1984, by the U.S. District Court

for the District of Columbia in a suit challenging interim designations of highways open to STAA vehicles, the preamble to the June 5, 1984, Final Rule proposed to establish a definition for the statutory term "highway with traffic lanes designed to be a width of twelve feet or more," and requested comments. In October 1984, Congress passed the Tandem Truck Safety Act (TTSA) of 1984. Section 105 of the TTSA amended the STAA to provide the FHWA the authority to designate FAP system highways for use by 102-in-wide vehicles, if such designation is consistent with highway safety.

This amendment clarified the authority of the FHWA to designate highways with less than 12-ft-wide lanes and disposed of the need to define further the phrase "highways with traffic lanes designed to be a width of twelve feet or more."

In accordance with the TTSA the FHWA is again reviewing those highways that have sections with less than 12-ft lanes that were designated in the June 5, 1984, rule to determine their suitability for STAA vehicles. Only 2,200 mi of the 181,000-mi network are involved in this review. Those that are inadequate will be removed or improved.

REASONABLE ACCESS

"Reasonable access" is another term from the STAA that has caused major consternation in some states. The STAA provides that states may not deny reasonable access to vehicles of the weights and linear dimensions authorized by the STAA between the National Network and terminals or service facilities. The September 14, 1983, NPRM stated the intent of the FHWA to allow the states to establish individual reasonable access provisions. The subsequent comments did not reveal evidence that the states would not provide reasonable access; thus the intent of the NPRM was retained in the Final Rule.

The FHWA continues, however, to monitor the access policies of the states. Should the FHWA determine a state's position to be unreasonable, it has the authority to seek injunctive relief.

The following list indicates the variety of policies that have been established to define reasonable access:

- Twenty-one states allow essentially unlimited access;
- Ten states allow from 2 to 20 mi;
- Four states allow 1 mi or less with no provisions to go farther;
- Two states have not yet established an access policy;
- One state allows access to all terminals via the shortest practical route;
- Nine states have a limited free access of from 1/2 to 2 mi for food, fuel, and lodging, but require permits for all terminal access; and
- Five states have a terminal access system that requires terminals to apply for access rights; the state evaluates the service road and either grants or rejects access; if access is granted, this route is publicized.

The FHWA is especially concerned with the provisions requiring permits for all access or that allow non-permitted access for only very restrictive distances such as 1/4 mi or less.

TANDEM TRUCK SAFETY ACT

In addition to the 12-ft lane clarification, the TTSA contains two other significant provisions. First, the act allows 28 1/2-ft "pup" trailers the same access as household goods carriers (i.e., to any point of loading or unloading). Second, a mechanism was established whereby certain Interstate segments may be withdrawn from the National Network.

Historically local motor carrier pickup and delivery operations have been conducted using substantially the same equipment used for over-the-road operations. In the past this meant an 18 wheeler that included a semitrailer that was nominally 45 ft long by 96 in. wide. Most companies now plan to use the individual 28- or 28 1/2-ft trailers allowed in a doubles combination for pickup and delivery after splitting the STAA-authorized combination at the terminal. This should improve local traffic flow because even though these vehicles will be an imperceptible 6 in. wider, they will be a quite perceptible 17 ft shorter.

The TTSA also gives the Secretary of Transportation the authority to exempt sections of the Interstate system from the National Network. Originally the STAA had mandated that the entire Interstate system be opened to STAA vehicles. This meant that several segments, primarily older, urban sections, built to less than current Interstate design standards, were to be made available to these vehicles at the same time as newly built wide-open rural segments. Many of the urban segments antedated the Interstate system and were subsequently included as logical connecting links but have not been updated to current Interstate design standards.

The decision to excluded a section of the Interstate can be based on the request of a governor or on the secretary's own initiative.

In requesting an exemption a governor must consult with the local government or governments involved and, if appropriate, the governor of any neighboring state concerned. Any request must show consideration of alternate routes and include specific evidence of safety problems. In acting on an exemption, the secretary must follow a notice and comment procedure through the Federal Register.

The FHWA is now in the process of developing specific regulatory instructions for both Interstate exemptions and pup-trailer access.

NETWORK PERFORMANCE

As a cook would say, the real test is in the tasting. In the case of the National Network, what's happening? Let us look at it from three perspectives: combination truck traffic, industry conversion, and safety experience.

- Truck traffic--Indications are that the trucking industry is switching to vehicles with the larger dimensions to take advantage of the increased payload, and this is resulting in a reduction in the overall vehicle miles of travel (VMT) by combination vehicles. VMT of combination trucks has increased by more than 32 percent since 1975, but because all other vehicle VMT has likewise increased, the combination truck share of total VMT has remained at a steady 3.5 to 3.8 percent since 1975. Although exact data are not available for STAA-dimensioned vehicles, it is estimated that by 1990 the total VMT for all trucks will be 1.2 percent less than it would be if the STAA had not been passed. Included in this estimate is the prediction that VMT of tractor-semitrailer combinations will decrease by 20 percent, but that VMT of 28-ft double combinations will increase by 25 percent. From the safety perspective this means less exposure of automobiles to large trucks and, it is hoped, fewer truck-involved accidents.

• Industry conversion--The Truck Trailer Manufacturing Association indicates that more than 75 percent of current van production is of 48-ft semitrailers, 102 in. wide. The remainder is of different lengths, but almost all are 102 in. wide. Equipment orders for STAA dimensions exist at an estimated value of more than $1 billion. Many carriers are aggressively changing fleet dimensions. Roadway, for example, has committed $200 million to upgrade its fleet to 15,000 twin trailers, 102 in. wide, by 1986. In 1983 United Parcel Service had approximately 1,000 trailers 102 in. wide. By the end of 1984 that number had increased to 3,000. Obviously, the industry has confidence in the network and intends to use it and take advantage of the productivity gains it offers.

• Safety experience--Much of the concern heard by the FHWA pertains to a perception that the larger dimensioned trucks, and especially the doubles, are less safe than are conventional sized trucks. Experience to date, though limited, shows the opposite. On the basis of 1984 data from six states that agreed to watch closely the twin trailer experience and to report accident data to the FHWA, both the fatality rate and the nonfatal injury rate per 100 million VMT for multitrailer trucks was about one-half that of single-trailer trucks. The FHWA has asked all state highway departments to revise their accident recording systems to include separate classifications of the STAA-authorized vehicles in order that accurate surveillance and experience can be analyzed and evaluated.

THE FUTURE

National uniformity in all aspects of trucking operations has long been a goal of the trucking industry. On the other side of the coin, the individual states have been necessarily provincial in their outlook, seeking to protect local industry and shippers. If at any time these two philosophies coincided, it was strictly coincidental.

By enacting the STAA, Congress has come down on the side of the trucking industry in the first battle over uniformity.

In the years to come, industry is likely to continue pressing for more uniformity, but that uniformity, no matter what the issue, is always to be at increasing levels, limits, or amounts. In commenting on these proposals, the traffic engineering community must be able to respond with factual information about the operation and effect of existing vehicles and sound estimates of what longer and larger vehicles are likely to do.

The FHWA has under way several research studies that are designed to provide some information about many unanswered questions, including

• "Impact of Specific Geometric Features on Truck Operations and Safety at Interchanges," which will help improve interchange designs through updated offtracking models and turning templates;

• "Operation of Larger Trucks on Roads and Streets with Restrictive Geometry," which will provide criteria for the safe operation of large trucks on local roads and streets and suggest under what conditions the larger trucks should be allowed or prohibited; and

• "Techniques for Improving the Dynamic Ability of Multi-Trailer Combination Vehicles," which involves the development of improved dollies or coupling devices.

These three studies are scheduled for completion within the next 12 months. Additional studies scheduled for later completion include

• "Effectiveness of Truck Roadway or Lane Restrictions," which examines current truck lane roadway restrictions, such as prohibiting trucks from using certain lanes of a multilane highway, to determine their impact on operations and safety;

• "Safety Implications of Various Truck Configurations," which will examine several possible near-term changes in size and weight limits that may influence future truck design; and

• "Safety Criteria for Multi-Trailer Highway Network," which will determine what controls are necessary to ensure the safe operation of even longer combination trucks on the Interstate system nationwide.

These projects should be completed in the next 2 years.

Currently only 60 percent of the eligible Federal-Aid mileage is available to STAA-authorized vehicles. As economic pressures mount from the trucking industry, and as research and experience expand the body of knowledge on operational and safety requirements, an expansion of the National Network can be expected.

The transportation engineer is being pulled in two directions. The large truck interests want access to their terminals and other points of loading and unloading now. The public wants to be protected. How are access and productivity gains to be balanced against safety? Perhaps research and experience will provide some tools for use in making these determinations. In the meantime, the FHWA would welcome any assistance or advice in any area pertaining to large truck operations.

This paper was presented at the Symposium on Geometric Design for Large Trucks but was originally prepared for the Annual Meeting of the Institution of Transportation Engineers. It is reprinted from the ITE Journal, September 1985, with permission of the Institute of Transportation Engineers, Washington, D.C. Copyright 1985.

Geometric Design for Large Trucks—An Overview of the Issues from the Perspective of the American Trucking Associations, Inc.

RICHARD A. LILL

ABSTRACT

Size and weight regulations represent compromises between conflicting needs. There is a need to accommodate diverse kinds of highway transportation demands and a need to fit trucking into the capabilities of the highway system. A consistent pattern of size and weight regulation, characterized as the "spread-the-load" strategy, has evolved and does a good job of meeting most of these needs. Under this strategy, increased productivity in trucking has come about largely through changes in vehicle type and gross weight, not through increases in the maximum allowable axle loads. For instance, there have been only two general axle weight limit increases in the United States since weight regulation became of central concern. One came during World War II and the second in 1974. The spread-the-load weight regulation strategy strongly influences vehicle design and thus also influences geometric highway design. The Federal-Aid Highway Amendments of 1974 formalized this weight regulation strategy, and the Surface Transportation Assistance Act (STAA) of 1982 accelerated the use of vehicles dictated by it. How the industry is adapting to the 1982 STAA criteria and why there is minimal overall impact on the highway network are described. Also presented are some specific examples of how size and weight regulations "design" the trucks used by the industry and a brief discussion of how future developments may be reasonably accommodated.

Issues that relate to geometric highway design and truck size in the broad context of the overall strategy that has governed the development of vehicle size and weight limits in the United States are addressed. This is necessary to obtain a balanced view of the subject.

Fundamentally, all size and weight regulations represent a series of compromises between conflicting needs. There is the need in the trucking industry to provide for the safe and efficient movement of commodities of widely different densities and characteristics. There is the need in the highway design community to provide for the safe and efficient movement of a motor vehicle population of significantly different sizes, weights, and operating characteristics.

If there is one thing that can be said about highway transportation, it is that it is dynamic. In a period of 75 years or so, highway transportation has moved from a nonexistent status to being the dominant form of transportation in the United States. The development of trucking has paralleled that of highway transportation in general.

An indication of the present significance of truck transportation for economic development and growth is contained in a series of surveys sponsored by Business Week magazine. The intent of these surveys was to find out what factors industry management considers most important when selecting a new plant site (1).

Table 1 gives selected excerpts of those factors company executives consider most important when they are siting new plant facilities. The data reflect responses in the survey years of 1976, 1980, and 1984.

TABLE 1 New Plant Location Survey—Selected Responses by Category and Ranking Within Survey

	1984		1980		1976	
	Percentage	Rank	Percentage	Rank	Percentage	Rank
Cost of property	84	1st	79	1st[a]	69	2nd
Trucking	75	2nd	79	1st[a]	76	1st
Reasonable taxes	73	3rd[a]	71	3rd	61	4th[a]
Construction cost	73	3rd[a]	69	2nd[a]	58	6th
Near airport	36	24th	31	28th	26	20th[a]
Rail freight access	25	29th	27	29th	31	17th

[a]Tied.

Overall, there were 49 categories of questions posed in the surveys. This table gives the categories of generally highest significance plus responses that related to other prominent transportation modes.

The survey indicates the significance of the costs of building and owning a new facility and the availability of truck transportation. The direct cost factors have become dominant, but truck transportation has remained a priority item. Nearness to an airport has gained, but availability of rail freight transportation has declined.

Highways and truck transportation are presently a major and integral part of the economic fabric of the United States. They have created a revolution in the way the nation does its business. It is a revo-

Department of Highway Policy, American Trucking Associations, Inc., 2200 Mill Road, Alexandria, Va. 22314.

lution that reflects the superior transportation service they provide for a wide range of activities. Moreover, the prediction is that the vehicle miles of combination truck travel will grow on the order of 2 to 4 percent annually through 1995, indicating that trucking will continue to be a dynamic industry.

From this point on, some of the factors that have shaped the trucking industry and that are expected to shape it in the future will be discussed. Simply put, the trucking industry exists in its present form because of the strategy that has governed developments in vehicle size and weight limits. In other words, trucks are "designed" to obtain the most effective use of what the size and weight laws permit.

Some observers may not be aware of this strong connection between the size and weight laws and vehicle design or recognize that a fairly consistent "strategy" of regulation has guided the changes in limits over the years. However, such is the case. The strategy can be termed the "spread-the-load" concept, and it has been in effect for the major part of the trucking industry from the earliest days of regulation.

For instance, there have been only two general axle weight limit increases in the United States since axle weight regulation became of central concern. One came during World War II, when maximum single axle limits were increased from 16,000 to 18,000 lb as a war emergency measure. Tandem axle limits were not increased at that time. The second general axle weight limit increase came in the Federal-Aid Highway Amendments of 1974, which permitted a 2,000-lb increase in both single and tandem axle limits up to the 20,000-lb single and 34,000-lb tandem level. It should be noted that these changes affected only some of the states of the nation because many states had always allowed limits that exceeded the values included in the 1974 action.

The 1974 amendments also added another important feature to the federal regulation of vehicle sizes and weights. This was the inclusion of a "bridge" or gross weight formula applicable to vehicles that took advantage of the new gross limit of 80,000 lb. This action formalized the spread-the-load strategy at the federal level for the first time, but the principle has been present in state regulations much longer.

For instance, the 80,000-lb maximum gross weight is permitted only if the overall axle spacing is 51 ft if a vehicle has five axles, or 57 ft if a vehicle has only four single axles. It is seen that options exist under the formula, but length limits, vehicle design considerations, and commodity type will determine how the industry will use it.

The present gross vehicle weight (GVW) formula has two major characteristics. One is that the maximum axle load limits cannot be legally obtained on all the load axles unless they have adequate distances between them. The other is that an increase in the number of axles within a given spacing may permit an increase in the maximum allowable GVW. In practice, however, the total permissible load will not usually be equal to the sum of the maximum allowable axle load limits.

The strategy has proven effective by increasing productivity in the trucking industry while reducing the demand on highway pavements from the transport of a given tonnage of freight. Figure 1 shows the equivalent single axle loads generated in the movement of a given amount of freight for a few different vehicle types. Because of the gross weight formula, the longer combination vehicles, on the average, exert less demand on pavements per ton carried than do shorter vehicles, even though they transport more freight per trip. These data for 1979 are based on the average loaded weights reported in the 1975-1979 National Truck Characteristic Report (2).

Truck transportation is not only flexible, it is also a "tailored" transportation mode. It is tailored to fit the commodity involved, it is tailored to fit the demands of shippers, and it is tailored to fit the size and weight regulations.

Thus the industry must always adjust its equipment to account for differences in density and characteristics of freight, operational requirements, and use of equipment. The spread-the-load strategy adversely affects the carriers of some commodities by causing the vehicle dead weight to be increased out of proportion to changes in total permissible weight. Nevertheless, the trucking industry generally accepts the viability of the concept.

Within the highway community there is the need to compromise among pavement costs, bridge costs, and geometric costs. To a limited extent, the bridge and

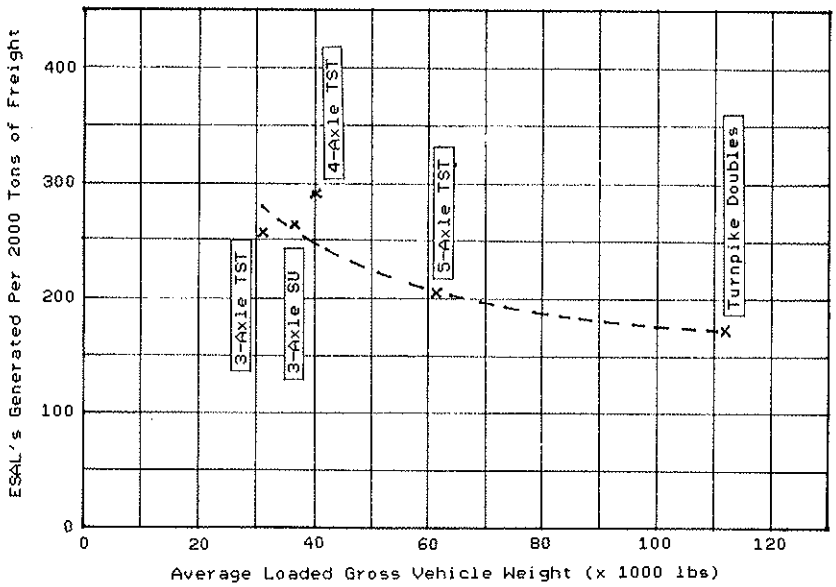

FIGURE 1 Generation of ESALs by vehicle type and average gross weight in moving 2,000 lb of freight—1979 national truck weight data computed for PCC pavement.

pavement costs have dominated while the geometric costs have received lesser concern. This may be because modern geometric features are desirable for all types of highway traffic.

Everyone, of course, has a strong interest in the safety of operations. There is no trucking company that will survive if it operates equipment that has inherent safety problems. The industry is sensitive to these issues and works hard, as a group, to have a good safety record. Because insurance rates are increasing rapidly, it is likely that safety of operations will become more and more significant in the future.

The remainder of this paper is a discussion of a few specific vehicles that demonstrate directly how trucking has been fitted into the regulatory environment. Presented here are only a few examples that show the broad relationship; hundreds of different adaptions could be shown.

Figure 2 is a picture of an old single-unit truck. The very first stages of axle weight regulation were applied to this vehicle. The regulation took the form of restricting the weight allowed per inch of hard rubber tire width. This type of regulation remains on the books of some states and is applied to pneumatic tires. The maximum axle loads permitted on these vehicles were on the same order as are allowed today.

FIGURE 2 Old single-unit truck.

Figure 3 is an early version of the three-axle tractor-semitrailer combination. Tandem axles were available in the early days of trucking, but most gross weight limits precluded their widespread use on combination vehicles.

The four-axle tractor-semitrailer vehicle shown in Figure 4 became the workhorse of trucking during the 1940s and 1950s, particularly on the East Coast where the heaviest axle loads are allowed. The passage of the 1956 Federal-Aid Highway Act led to increases in GVW limits up to 73,280 lb. This led in turn to the use of the five-axle tractor-semitrailer in most of the United States, although the four-axle semitrailer remained in substantial use in the East because of the heavier axle limits.

Figure 5 shows the vehicle that was designed to fit the 1946 AASHO bridge formula. Also, the present federal formula was slightly altered to permit a significant population of existing equipment to obtain the productivity gains envisioned by the 1974 Federal-Aid Highway Amendments. Even so, the restrictions of the formula have prevented some carriers, notably bulk commodity carriers on the East

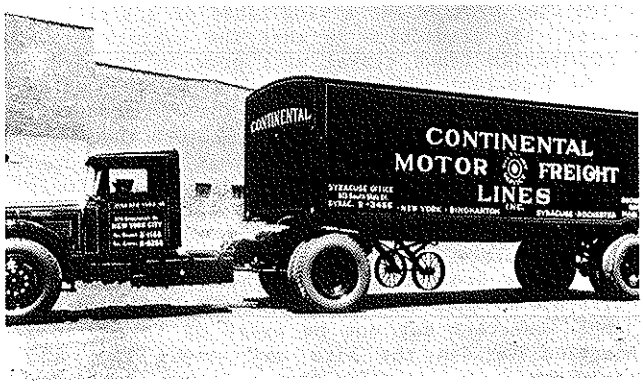

FIGURE 3 Early version of three-axle tractor-semitrailer combination.

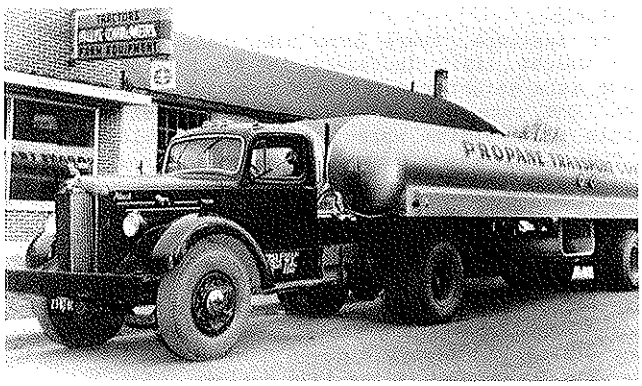

FIGURE 4 Workhorse of trucking in the 1940s and 1950s.

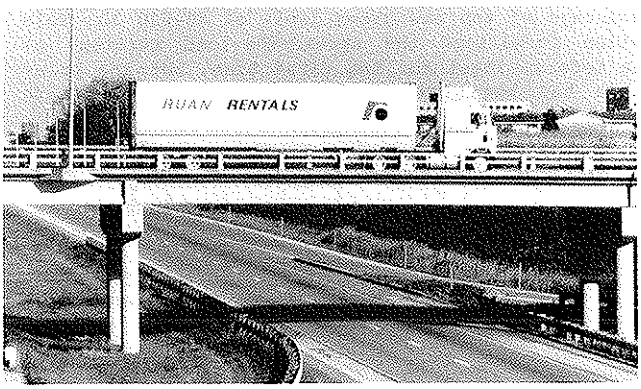

FIGURE 5 Vehicle designed to fit the 1946 AASHO bridge formula.

Coast, from converting to the five-axle unit because their existing trailers do not fit the formula.

This figure also shows the way in which many carriers with five-axle semis are using the 48-ft trailers. Because their existing equipment already meets the spacing requirements of the formula, most are adding 3 ft in back of the rear tandem. Doing this does not affect vehicle offtracking.

Figure 6 is an example of a vehicle that has been designed from the ground up to fit the current GVW formula. The multiple load axles are common but the "twin-steering" axles are not. This vehicle fits the formula well and keeps dead load to a minimum. It is desirable for certain types of dense commodities.

FIGURE 6 Vehicle designed to fit the current GVW formula.

The same twin-steer concept is applied to the truck-full trailer combination shown in Figure 7. Conventional five-axle semitrailer combinations have difficulty in reaching the 80,000-lb limit because of problems in shifting enough weight to the steering axle. This vehicle is designed for about 22,000 lb on the twin-steer assembly. It thus can be fully loaded and also give some flexibility in loading the regular load axles. Because of advanced design concepts, this vehicle comes in at about 5,000 lb less dead weight than many five-axle tractor-semitrailers.

Figure 8 shows another vehicle that fits the GVW formula very well. It is a five-axle twin-trailer combination that was built to haul a specific bulk commodity. Its dead weight is also about 21,000 lb, and it is easily adaptable to different regulations.

FIGURE 7 Truck-full trailer combination.

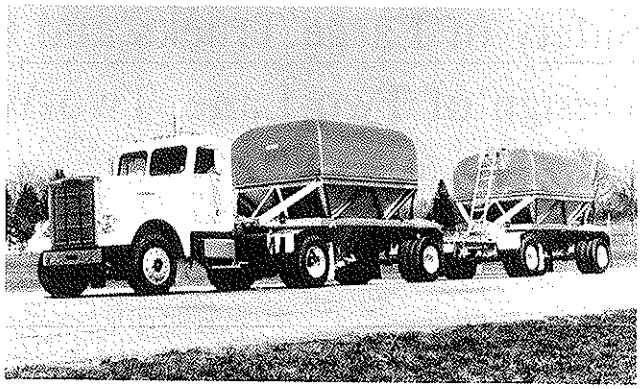

FIGURE 8 A second vehicle designed to fit the GVW formula.

In summary, the trucking industry adapts its vehicles to the regulations that govern their use. Productivity is the key word, and it is the primary factor in determining how the industry will make use of changes in regulations.

The spread-the-load size and weight regulation strategy is sound and there is little likelihood that it will be significantly altered. The strategy thus potentially affects geometric design in the areas of turning and offtracking of longer combination vehicles because these are the vehicles the strategy produces. It should be recognized, however, that modest adjustments in the implementation of the strategy may be in order and that there will be a continuing need for special-case exceptions.

The 1982 STAA introduced the concept of "designated highway systems" for the operation of more productive vehicles on the national level. Such designations provide a viable means of recognizing the greater capabilities of modern highways for meeting the crucial transportation demands of the nation. The need for balancing bridge, pavement, geometric, and safety concerns will remain, but problems will be worked out as they have been in the past. Goods transportation is too important for this not to be the case.

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Safety of Large Trucks and the Geometric Design of Two-Lane, Two-Way Roads

GERALD A. DONALDSON

ABSTRACT

Recent federal and state designations of primary highways for use by longer, wider, and heavier tractor-trailer trucks, pursuant to the Surface Transportation Assistance Act of 1982, contain a high percentage of arterials with deficient geometric and cross-sectional design features. Recent studies indicate the severe accident overinvolvement potential of larger, especially tandem-trailer, trucks on both rural undivided and urban divided highways. Research in the past few years demonstrates both the proclivity of larger trucks for certain kinds of accidents because of their design characteristics and their incompatibility with the substandard operating conditions found especially on older two-lane, two-way rural arterials. The safety-deficient design characteristics of larger trucks are reviewed and the incompatibility of their operation with horizontal curvature, superelevation, skid resistance, and, in particular, passing sight distance deficiencies is surveyed. Recent investigations of passing sight distance and marked passing zone deficiencies on roads designated for use by longer, wider trucks are explored. The results of these investigations are buttressed by a number of papers and research studies produced during the past 5 years. Preliminary comments are made on the new AASHTO geometric design guide, which appears to condone substandard design features on highways open to use by larger trucks. Last, the current paucity of accident data collection on large trucks, the need for better on-site investigation of large-truck accident causation, and the necessity of more sustained research on the behavior of large trucks on each functional class of roadway are indicated.

It is rapidly becoming clear that certain geometric design elements play a pivotal role in the safe operation of large commercial vehicles on our nation's roads. When these elements, some of which are considered in the body of this paper, are deficient, they provide the context for vehicle and driver responses that lead to truck accidents. Moreover, the operation of long, wide trucks, especially on two-lane, two-way (TLTW) roads with substantial geometric deficiencies, markedly compromises the safety of automobile motorists who must share the roadway with big trucks.

A discontinuity has emerged during the last few years between the results of research and public policy on the compatibility of large trucks with older arterial and collector roadways. The aims of increased productivity for American trucking interests and the perceived need for uniformity of truck size and configuration and of access privileges to these older roads have produced mandates and arguments in statute and regulation that attempt to establish the safety parity of larger trucks with automobiles.

This author thinks that the investigations of the past several years into such topics as passing, stopping, and decision sight distance on TLTW roads; superelevation; the behavior of large trucks at high speeds on roads with moderate to severe curvature; and other related subject areas should convince us all that there is a considerable divergence between the appearance and the reality of safety of big trucks on roads with impoverished design.

Center for Auto Safety, 2001 S Street, N.W., Washington, D.C. 20009.

It is unfortunate that the issue of highway design adequacy and big truck safety should have been propelled into the arenas of high politics and of court advocacy. Neither of these forums is the place to establish the acceptability of older, non-Interstate roads for use by longer and wider trucks. However, as has been the case innumerable times in the past, the actions of the legislatures and the courts in response to the pressures of important public interests have precipitated the intense scrutiny of a critical area of public health and safety. The results of impartial research nevertheless have the unenviable task of bringing up the rear and responsible practitioners have the Herculean job of attempting the reform of policies already in place.

As indicated at the outset, this author is strongly persuaded that a small number of geometric design features are of pivotal importance to the operating safety of large trucks on TLTW roads. The investigative efforts of a number of researchers during the past few years have shown that sight distance for passing and horizontal curvature are central design parameters for determining the operating safety margins of large trucks on TLTW roads. This is certainly not to say that other design and cross-sectional features do not also have a role to play in the safety of large trucks on these roads. The interdependence of these features (e.g., lane width, superelevation, vertical curvature, and stopping sight distance) along with pavement surface characteristics (coefficient of friction) is heavily determinative of the margins of safe vehicle operation. Furthermore, the traffic engineering applied to these roads, especially with regard to the marking of passing and no-passing zones, is of decisive im-

portance in fostering safe operations. Also, passing zone warrants, standards, and practices are woefully inadequate and will be discussed later in this paper.

The recent research of Lieberman (1) demonstrates the thorough inadequacy of the American Association of State Highway and Transportation Officials (AASHTO) sight distance formulas for the successful execution of the passing maneuver at higher speeds on TLTW roads. Lieberman has shown that significantly longer sight distances are needed when the impeding vehicle is a truck not an automobile. He also points out correctly that the inadequacy of the AASHTO passing sight distance formulas results from the postulation of automobiles passing only automobiles, an approach still used in the latest AASHTO geometric design guide (the "green book") (2). It should be noted that Lieberman's assumptions of acceleration capabilities for the passing maneuver and the speed differential between the impeding truck and the passing automobile are AASHTO's and these assumptions are unrealistically sanguine for many actual vehicles and on-the-road conditions. His analysis does stress that the issue of inadequate safety is particularly acute where vehicles with low height-of-eye, such as many subcompact automobiles, attempt to pass large trucks at 85th percentile traveling speeds in excess of 44 mph. However, it should be pointed out here that the acceleration capabilities of many legal vehicles in the United States in the 45 to 65 mph range are substantially below the figures assumed in the AASHTO formulas and calculations. Just to mention one or two examples, the Mercedes-Benz 240D (3) and the Peugeot 504 (4) diesels having passing-speed abilities far below the rates premised in the AASHTO criteria. There are many other examples of recent automobiles, light vans, and multipurpose vehicles with very poor passing abilities.

The research of Gericke and Walton (5) demonstrates that the AASHTO sight distance formulas for geometric design are inadequate for any vehicle attempting to pass any other vehicle on TLTW highways, and especially inadequate for automobiles passing trucks. Gericke and Walton stress that prospective increases in the length of trucks will correspondingly increase aborted passing maneuvers of automobiles and will thereby increase safety hazards. They emphasize that additional passing sight distance will be needed if safety is not to be compromised. Unfortunately, they do not fully address the issue of successful versus unsuccessful aborts. No existing analysis treats completely the nature, variety, and frequency of the maneuvers and consequences of the inability of vehicles to safely conclude an abort, although some inquiries do show conclusively the high percentage of aborts that are necessary in automobile-passing-truck attempts on TLTW roads. Moreover, at the present time there are no data on the consequences of unsuccessful aborts; that is, whether and to what extent the vehicle that cannot successfully abort runs off onto the roadside of the opposing lane, has a head-on collision with an opposing vehicle, has an accident with the impeding vehicle during the attempted drop-back, or has an accident with a trailing vehicle when the aborting vehicle attempts to reenter the queue. These data and research are badly needed. In contrast to the complexity and subtlety of the aborted passing maneuver problem and its impact on highway safety, the AASHTO Policy on Geometric Design of Highways and Streets (2,p.148) states:

> When required, a driver can return to the right lane without passing if he sees opposing traffic is too close when the maneuver is only partially completed.

And (2,p.156)

> Even on low-volume roadways a driver desiring to pass may, on reaching the passing section, find vehicles in the opposing lane and thus be unable to use the section. . . .

The research of Saito (6) shows the correlative inadequacy of the passing zone sight distance and pavement marking criteria and practices of the national Manual on Uniform Traffic Control Devices (MUTCD). Although a previous article in the ITE Journal by Weber (7) showed quite conclusively that the use of AASHTO sight distance and MUTCD passing zone standards results in the marking of thoroughly inadequate passing zones, particularly on vertical and horizontal curves, the only change to these formulas and resulting markings has been the marginal one brought about through the lowering of the height-of-eye criterion from 3.75 to 3.5 ft [48 Fed. Reg. 54336 (1983)]. As Weber (7,p.16) points out:

> Passing zones of lesser standards . . . are lethal to the inexperienced driver who has undue trust in the markings. Such marginal zones neither fulfill the expectations of safety experts nor do they increase the economic benefit of the road. . . .

Yet, in the last year and a half, many thousands of miles of roads marked consistently with excessively short, inadequate passing zones have been opened to use by longer trucks.

Saito (6) confirms Weber's analysis and adds to the small number of research considerations of aborts. He shows that successful aborts are impossible under most high-speed conditions on the basis of current MUTCD passing zone sight distance and striping standards. He argues that there is a high probability of collision potential due to the inability of the aborting vehicle to reenter the lane behind the impeding vehicle when the 85th percentile speed is greater than 40 mph. Through the use of kinematic modeling, Saito demonstrates algebraically that the impossibility of successful passing on the basis of MUTCD passing criteria can also be used to demonstrate that successful aborts of the passing maneuver cannot be performed after a certain point has been reached by the overtaking vehicle.

The importance of Saito's demonstration cannot be overestimated. However, one shortcoming of this approach is the postulation of only automobiles attempting to pass other automobiles. If the kinematic model is extrapolated, it shows substantial increases in the lengths of times and distances for successful aborts of automobiles attempting to pass long trucks and, moreover, shows that an increase in the percentage of unsuccessful aborts occurs as the impeding vehicle's length is increased. At one point Saito (6,p.21) does briefly consider an automobile passing a truck 55 ft long. His computations and graphic representation clearly imply that a significant increase in what he terms the "collision zone" is effected by the attempt of automobiles to abort the attempted passing of trucks, but his own consideration of this derivative conclusion is too brief. It appears that, on the basis of his model, a consistent arithmetic increment of additional length in the impeding vehicle, ceteris paribus, causes a corresponding logarithmic increase in the percentage of aborts.

In a paper offered last year, Garber and Saito (8) applied Saito's analysis to real-world sight distance and passing zone conditions on TLTW highways

in mountainous areas. Passing-attempt data from Virginia roads were used in the analysis to demonstrate that MUTCD passing zone values are inadequate for passing zone marking of TLTW highways with significant vertical and horizontal curvature. They show the functional relationship of AASHTO passing sight distance values and MUTCD passing zone length values, and the inadequacy of both sets to accommodate safe passing maneuvers. The minimum values of the MUTCD for passing zone length are inadequate at lower speeds and increasingly inadequate at higher speeds. This inadequacy begins as low as the 85th percentile speed of 30 mph. A 90 percent increase in the minimum length of passing zones over the MUTCD minimum values is needed in order to ensure the safe completion of the passing maneuver at the 50 mph 85th percentile speed. Even at the 85th percentile speed of 30 mph, a 35 percent increase in length is necessary. These recommended additional lengths will also accommodate successful aborts.

These research results make it abundantly clear that passing sight distance and passing zone standards are critically important engineering features on TLTW roads with significant curvature. The automobile-truck relationship in the passing maneuver is highly dangerous on many thousands of miles of rural arterial and collector routes that are designed with inadequate sight distance and marked for permitted passing maneuvers, which cannot be accomplished, in some cases, even by a majority of the vehicles making the attempts. It might be indicated here that the South Carolina Department of Highways and Public Transportation (SCHPT) reviewed their TLTW highways for passing capabilities in light of the federal designation of many of their primary routes for use by longer, wider, and tandem-trailer trucks. SCHPT found many instances in which only 30 percent, and sometimes as low as 25 percent, of the marked passing zones on a given route would allow an automobile to pass another automobile, and these percentages were further reduced when the pass was of a standard tandem-trailer or equivalently long truck [i.e., in the case of a tandem-trailer rig, two 27-ft-long trailing units plus the length of the cab (FHWA Docket 83-4)].

A few more geometric design elements and their bearing on the safety of large trucks need to be considered. Recent research on stopping sight distance (SSD) by Olson et al. (9) has shown the inadequacy of the current green book formulas for SSD by revealing the improper assumptions lying behind AASHTO calculations. These include a locked-wheel premise for braking that, on close examination, proves to be unrealistic and hazardous because the result of locked-wheel braking is the inability of the driver to maintain proper control of the vehicle, particularly to avoid encroachment into an adjacent lane. On TLTW roads, avoidance of opposing lane encroachment is, of course, crucial and when inadequate SSD combines with locked-wheel braking on a horizontal curve, the consequences can be catastrophic: the locked-wheel vehicle will proceed off the road tangentially to the curve and the crown or superelevation of the road will cause the locked-wheel vehicle to slide toward the downhill side of the road in a manner that the driver cannot correct by steering (9,p.55).

Olson et al. argue that locked-wheel stopping is not desirable and that it should not be portrayed in design standards as an appropriate course of action (9,p.55). An additional consideration about the hazardousness of a locked-wheel standard is the relatively low value of pavement skid resistance available at high speeds, particularly on wet surfaces.

As important as the research of Olson et al. is for the general issue of reforming SSD requirements, many important insights were gained from this major investigative effort into other issues that affect the safety of trucks. In the course of the study, the capabilities, design, and efficiency of truck braking systems were called seriously into question in their relation to the design features of typical highways. With regard to SSD, the authors concluded that, given the substantially inferior frictional capabilities of truck tires (approximately 0.7 the frictional capability of automoble tires), current SSD available on many highways is thoroughly inadequate. For controlled (i.e., unlocked-wheel) stops in which the truck driver modulates his brakes to prevent spinning or jackknifing and maintain steering control, it was found that trucks require stopping distances that are approximately 1.4 times those required for automobiles. In tests conducted by Olson et al. involving repeated stops by heavy trucks from only 40 mph on a 12-ft-wide lane on a curve with a 1,000-ft radius, of 60 runs performed by professional drivers, 11 resulted in loss of directional control and departure of the vehicles from their lanes (9,p.90). Because Olson et al. argue for allowable stopping distances for automobiles of 85 ft for 50 mph, 190 ft for 60 mph, and 350 ft for 70 mph (9,pp.2-3), it is obvious that trucks are not accommodated in their stopping distance requirements in current design standards despite the supposed compensation for inferior braking lent by the operators' superior height of eye.

Olson et al. also address the behavior of vehicles on horizontal curves, a topic that is particularly important for trucks. Neuman (10) and Glennon et al. (11) have, along with Olson et al., demonstrated the critical importance of spiral transitions in the design of horizontal curves. Olson et al. showed that one of the salient effects of spiral transitions is to reduce the need for object clearance on the inside of the path of travel when the driver is in the tangent section (9,p.45). Neuman and Glennon et al. showed conclusively that, even if spiral transitions are not provided, drivers always will tend to guide their vehicles through a path that essentially duplicates the behavior of a vehicle in a spiraled transition. When spirals of sufficient length are provided, dramatic effects are achieved in reducing the most critical aspects of vehicle path behavior. Properly spiraled curves radically decrease the hazards of path overshoot. This in turn substantially lowers lateral tire acceleration, thereby ameliorating undue reliance on tire side friction demands.

However, there are many thousands of continuous curves on highways and in some states the policy is explicitly not to design and build spiraled curves of appropriate length. Although the new AASHTO design policy does imply throughout its treatment of spiral transitions the superior accommodation of actual driver and vehicle behavior achieved by designing spiraled curves (2,pp.195,198), in the end the endorsement by the green book is really only tepid and the overall analysis of the functional importance of spiraled transitions is insufficient and simplistic, particularly in the failure to correlate the differing natures of spiraled versus nonspiraled curves with regard to run-off-the-road encroachments and the functional interdependence of different curve geometries and roadside environment.

The question of spiral curve transitions is of special value in viewing the needs of large trucks and, in the interrelationship of spiral curves with lane width and superelevation, constitutes a pivotal design matrix that should be investigated carefully for trucks. In the data base used by Glennon et al. (11) in their study of safety on rural highway curves, it was found that the average accident rate for curves is three times the rate for tangents, that the average single-vehicle run-off-the-road accident

rate for curves is four times that for tangents, and that accident severity on curves is greater than on tangents. In addition, Glennon et al. pointed out the high sensitivity of accident rates on curves to the nature of the roadside environment, including the severity of the slope and clear-zone width. They also show that drivers traveling at or near a curve's design speed will tend to exceed the tire side friction demands implied by AASHTO friction factors and that the actual margin of safety on wet pavement is only 50 percent of that currently set forth in the green book. Moreover, they stress that the current design manual should explicitly consider the trade-off necessary between curvature and superelevation, which it does not.

In concluding with specific recommendations, Glennon et al. (11,p.9) suggest that

> [T]he avoidance of large central angles between successive tangents is recommended. AASHTO policy should state that central angles of no more than 45° are preferred. Larger central angles require either sharp curvature or long curves, both of which adversely affect safety.

It is interesting to note here another rebuttable presumption of the AASHTO green book in regard to curves. The text would have it that, "[f]or most curves the average driver can effect a suitable transition path within the limits of normal lane width" (2,p.195). The expression "normal lane width" is left undefined in this passage, but this author submits that, indeed, many curves, especially those that are unspiraled with lanes less than 12 ft wide on TLTW roads, cannot be properly and safely negotiated by a large truck even when traveling at the posted speed. Opposing lane encroachment on a TLTW road by the truck is guaranteed and this due partly to the excessive demands made on the vehicle and driver by the inadequate design of the curve and partly, with some rig configurations, to substantial inboard offtracking. [For a discussion of offtracking behavior see Millar and Walton (12) and Ervin et al. (13,p.156).] Any attempt to correct for this by the driver will result in encroachment onto the shoulder if, indeed, there is any shoulder.

The manifest hazardousness of both opposing lane and roadside encroachment should be apparent to all. Yet, although the green book nowhere has a unified, coherent treatment of the unique needs of design for very large vehicles, it nevertheless sees through a glass darkly that big rigs will encroach beyond the delineated travel lanes. At one point, in the consideration of turning roadways, the guide recommends that large vehicles pass each other on inadequately wide pavements by intentionally employing the shoulder or stabilized roadside area (2,p.234). But the green book recognizes neither that such encroachments at high speeds on many curves are compelled by the nature of curve geometry and the behavior of many big rigs nor the consequent harazardousness of this maneuver. The green book's exhortation (2,p.233) that

> [i]n negotiating pavements designed for smaller vehicles, larger vehicles will have less clearance and will require lower speed and more caution and skill by drivers. . . .

is naive and an admission that design cannot control traffic safety. [The issues of truck wheel lateral placement and typical ranges of lateral variation are relevant here. For some recent data, see Shankar and Lee (14,p.9).]

The work by Olson et al. (9) and Glennon et al. (11) appropriately complements a recent paper by Zador et al. (15) in furthering understanding of the relationship of horizontal curves and superelevation. These three studies make it apparent that present green book guidance is insufficient to provide adequate superelevation rates that guarantee a margin of safety for lateral acceleration of truck tires given the usual ranges of pavement surface friction coefficients found on older roads. Glennon et al. (11) concluded that, given the predictable curve overshoot behavior of the typical driver, more superelevation is required than is called for by AASHTO policy to produce AASHTO-specified lateral tire accelerations at design speed for nominally critical driver behavior. Olson et al. (9,pp.21,55) intimate the critical contribution of superelevation to safe, controlled braking in curves that will allow the driver to maintain his lane throughout his maneuver. Zador et al. (15) showed that after adjustments were made for both curvature and grade, fatal rollover crash sections were nevertheless still found to have less superelevation than comparison sections. The results of Zador et al. were based on comparisons of the linear regression estimates of superelevation rates as functions of curvature. Therefore the deficiencies in superelevation found in the study cannot be due to curvature differences between flat road sections and those with grades. Furthermore, if downhill grades were designed for realistic (i.e., higher) travel speeds, the rate of superelevation would be higher for curves with downhill grades than for comparable flat curves because of the higher average speeds of vehicles traveling downhill. The new manual only asserts that (2,p.194):

> On long or fairly steep grades, drivers tend to travel somewhat faster in the downgrade than in the upgrade direction. In a refined design this tendency should be recognized, and some adjustment in superelevation would follow.

How much adjustment to make is not mentioned; no sets of recommended values are provided in the green book to link the earlier discussion on curve design with the later consideration of the effects of grades (2, pp.252-265). The green book (2,p.264) does acknowledge that "[s]teep downhill grades can . . . have a detrimental effect on capacity and safety on facilities with high traffic volumes and numerous heavy trucks"; however, ". . . criteria are not established for these conditions. . . ."

It is evident that no such compensatory design has been provided on tens of thousands of miles of rural TLTW roads for downhill curve superelevation, a condition particularly serious for large truck safety given the usual modus operandi of big trucks of highballing downgrades to offset the gradual deceleration that accompanies the traversal of moderate to severe upgrades. When combined with the poor brakes and braking efficiency found in many big rigs (16), inadequately superelevated curves on downgrades are especially dangerous both for negotiation of the curve and for any necessary braking maneuvers. This is a point at which the SSD insights of Olson et al. (9) integrate with the superelevation discoveries of Zador et al. (15) and the curvature insights of Glennon et al. (11).

It is clear by now to any reader that older TLTW roads are riddled with substandard, hazardous design and operating features. Moreover, the latest design guide, the green book, gives inadequate direction for the substantial improvement of those geometric features that, in their interactive influence on driver and vehicle behavior, provide a context for predictably higher accident rates. And whatever the

serious shortcomings of these badly designed, and oftentimes inadequately rehabilitated, roads may be for automobiles, their adverse effects on the safety of big trucks are magnified. The compatibility of big trucks with the operating environment produced by the interaction of narrow lanes, deficient superelevation, unspiraled and severe horizontal curves, and severe grades is largely fortuitous; and the attempt by some to rationalize away these systemic incompatibilities by appealing to the experienced compensating driving behavior of many truckers is sheer folly. An approach to geometric design on older roads that rationally accommodates the actual ranges of legally licensable drivers and vehicles is totally lacking. On TLTW roads with significant curvature and grades, there is no vehicle more disenfranchised from the protection that should be afforded any highway user than the big truck. Current efforts at a national level to argue the adequacy of these geometrically deficient facilities is nothing more than lame ex post facto constructions.

As Glennon and Harwood point out in their deservedly famous article of 1978 (17,p.80),

> The apex of the objective design process is the requirement that desired goals be defined and completely quantified. In addition, of course, these goals must be defined within the framework of a functional classification of highways. This points to a primary weakness of the AASHTO policies. Although they name the goals of safety, efficiency, economy, and comfort, they do not operationally define these goals.

And (17,pp.80,82)

> In the design process, a lack of understanding of basic design constraints and how they affect the solution contributes to piecemeal optimization. The current approach tends to ignore the consistency of various combinations of design elements and thus oversimplify the process and limit the reliability of relations for most design purposes.
>
> [D]esign consistency means that combination of design elements (and their dimensional specification) . . . does not violate the abilities of the driver to guide and control the vehicle. Therefore, the concept of driver expectancy is wholly embodied in the general definition of design consistency.

And, finally (17,p.82),

> Although the concept of design consistency has been given substantial attention in the design policies, there is a general lack of explicit criteria for the contiguous combination of basic design elements or for the longitudinal variations of such features as horizontal alignment, vertical alignment, and cross section. Without these explicit criteria, highway designers will continue to build inconsistent geometric details into highways.

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Big Trucks in New Jersey: From Crisis Management to Strategy

MARK L. STOUT

ABSTRACT

This paper is a discussion of problems created or brought to the fore by passage of the Surface Transportation Assistance Act of 1982 and of the efforts of the New Jersey Department of Transportation to address them.

Historically, the New Jersey Department of Transportation (NJDOT) had little need to be interested in trucks and trucking. Intrastate trucking was and is "unregulated." Such regulation as has existed--registration, licensing, and inspection--has been done by the Division of Motor Vehicles, which in New Jersey reports to the Attorney General not to the Commissioner of Transportation. The state police enforce truck size and weight laws.

Passage of the Surface Transportation Assistance Act (STAA) of 1982 and subsequent regulations issued by the FHWA made longer and wider trucks a major issue in New Jersey and a high-priority concern of the NJDOT. The watershed event was notification on April 5, 1983, of FHWA's proposed "designated routes" for longer and wider trucks in the state. That network was much bigger than had been anticipated and contained many routes that were totally unacceptable both on political and public opinion grounds and on technical grounds. What had been an issue for transportation professionals became, overnight, front-page news. There ensued a period of crisis management, involving legislation, emergency regulations, negotiations with FHWA, considerable press attention, and the personal involvement of Governor Thomas Kean. The issue of designated routes was not resolved until September 1984 when FHWA published its final rule, which acquiesced to New Jersey's designated network.

Although the department "won" on the issue of designated routes, the main effect on transportation professionals was an increased awareness of the complexity and apparent intractability of many truck problems and the high level of controversy attached to them. This effect was compounded by a variety of other truck issues that arose in the same period. Perhaps most notable for its complexity has been the implementation of the federal bridge formula. The bridge formula was incorporated into state law only in September 1983 and only after years of discussion among lawyers for the New Jersey Attorney General and for FHWA, an ultimatum from FHWA, substantial political pressure from the motor carrier industry over the issue of permanent and temporary exemptions from the bridge formula, and agreement by the department to become actively involved in the exemptions issue. NJDOT is still involved in ironing out residual legal and regulatory issues concerning these exemptions.

During these periods of crisis management, the department found itself in a reactive, defensive posture. As one complication and policy issue succeeded another, it became apparent that the department needed a comprehensive strategy for incorporating trucks into the overall transportation system on satisfactory terms. Although this strategy is far from complete, progress has been made in defining objectives on several issues that form key pieces of the puzzle. The main issues that are being worked on are

1. Development of settled truck size and weight standards in conformity with the needs of the highway system. It appears that a more or less stable situation has been achieved with regard to rules governing two of the three STAA combinations, but there still is no solution for the third. Combinations with 48-ft semitrailers are accepted universally under New Jersey law. This outcome was a result of the reasoning that because 48-ft semitrailers had been in actual and lawful use in New Jersey before STAA (within a 55-ft overall limit and no semitrailer limit), it would be impossible to "roll back" the limit to something less than 48 ft on the non-STAA system. Indeed, the department had agreed in principle with the trucking industry before passage of the STAA to accept an increase in the overall limit to 60 ft, provided that a 48-ft semitrailer limit was enacted.

A settlement appears to have been reached--temporarily at least--on the issue of "double bottoms." Doubles are now permitted to travel on an integrated network of Interstates, freeways, and toll roads. Reasonable access is granted freely for services within 1 mi of the system. Access to terminals is much more restricted, requiring a written permit, but has caused relatively few complaints. Although there are problems with the regulations for doubles--especially due to gaps in the Interstate system--in general the introduction of these vehicles into New Jersey has gone remarkably smoothly. This is especially interesting because these vehicles have accounted for the lion's share of controversy over big trucks.

The third STAA vehicle--102-in.-wide trucks--provides an interesting contrast to doubles. Wide trucks attracted almost no attention in the press but have proved to be the most difficult problem for the department. At the moment, 102s are limited to the STAA network in New Jersey. Although this dual system--102-in.-wide trucks on the STAA system and 96-in.-wide trucks off it--is far from satisfactory, the department's engineers remain skeptical about the advisability of permitting wide trucks free travel throughout the state, especially on roadways with substandard lane widths.

Truck size and weight issues are not confined to STAA vehicles of course. Two examples will suffice

Office of Policy Analysis, New Jersey Department of Transportation, 1035 Parkway Avenue, Trenton, N.J. 08625.

to demonstrate the continuing problems. First is the question of applying the federal bridge formula off the Interstate system. NJDOT is still assessing the costs and benefits associated with what would be, at best, a difficult change to accomplish. The department is under pressure from segments of the industry to support their legal and political efforts to carve out "grandfather" exemptions from the bridge formula even on the Interstate system.

A second example comes from the solid waste industry. In New Jersey the transportation of solid waste has become a subject of considerable controversy and complexity as landfills close and environmental rules tighten. The solid waste hauling industry, which is now relieved from axle weight limits for 60,000-lb collector vehicles, has recently approached the NJDOT with the proposal that they be relieved from axle weight limits for 70,000-lb collectors and 80,000-lb transfer trailer combinations. It is easy to say "no" to such a request, but to what extent should the department become involved in the problems of a troubled industry that is already notorious for running overweight? Should the department be attempting to find and promote transportation solutions?

These examples illustrate the size and weight policy problems that confront decision makers in New Jersey and other states. In New Jersey, the process of achieving stability in this area has been slow and painstaking and is, of course, always subject to upset from sources beyond the control of the NJDOT.

2. Integration of new truck requirements into design and operations standards. The department's current view on this subject is that the main requirement is to maintain an open process for receiving, reviewing, developing, and applying technical information that may affect design and operations standards. In 1985 the department undertook a study of the need to correct geometric deficiencies on the designated system and also made plans to participate in a study sponsored by the FHWA entitled "Operation of Larger Trucks on Restricted Geometry."

3. Recognition of the needs of truck movements and truck access in the planning and project selection processes. To date this has happened only on a case-by-case basis. It is expected that improved knowledge of actual commodity flows; stronger comprehensive planning at state, county, and municipal levels; better liaison with industry; and more sophisticated programming techniques will lead to a more comprehensive approach.

4. Requiring trucks to pay their "fair share" of highway costs. Highway finance in New Jersey is not supported by dedicated user fees. However, the New Jersey Transportation Trust Fund Authority, established in 1984, provides stable funding for transportation capital projects through short-term bonds backed by anticipated earmarked appropriations. An essential element in enacting and implementing this legislation was a $30 million increase in truck user fees--including increases in registration fees, motor fuel decal fees, and diesel taxes--that were earmarked for the trust fund program. The $30 million figure is a major step toward a fair share contribution, but it was not based on any systematic highway cost allocation study.

5. Development of an adequate data base on trucks. Progress has been made in a number of areas, notably in a statistical understanding of pavement damage caused by trucks and in mapping commodity flows. However, in general, the department's knowledge of the population of trucks on the state's highways is poor and is inadequate for supporting informed decisions on many of the issues discussed here. This frustration is no doubt widely shared in state and federal agencies where decisions about trucks must be made. Time and time again in New Jersey all the obtainable, relevant data on a policy question have been gathered and found to be pitifully inadequate. Unfortunately, it is all too clear that the efforts being made in this area will still leave NJDOT in an unsatisfactory situation in the foreseeable future.

6. Better enforcement of truck size and weight laws. The department is now in the process of designing modern, new truck weigh stations for enforcement by the state police. In addition, a comprehensive review of the current penalties for size and weight violations has been started. This is another area in which the department traditionally had little interest.

7. Better liaison with motor carriers and shippers. The controversies over truck size and weight and increased truck fees caused serious strains between the industry and the department. Fortunately, the atmosphere has cooled considerably on these issues and the department and the industry have established a joint advisory committee that has led to vastly improved communications.

8. Rationalization of truck policy and regulatory responsibilities within the department and with other state agencies. The unsystematic and uncoordinated growth of truck responsibilities within the department and in other agencies has created the need for a fresh look at the best way to assign these responsibilities.

In sum, the New Jersey Department of Transportation has been forced by circumstances--especially enactment of the STAA--to move from a posture in which truck issues received minimal attention to a position in which a steady stream of truck problems and issues has made apparent the need for a comprehensive truck strategy. Although that comprehensive strategy has not yet been achieved, progress has been made in identifying important component issues, in defining goals for a number of those issues, and in moving toward several of these goals.

Keeping Up with Big Trucks: Experiences in Washington State

STAN A. MOON

ABSTRACT

Changes in Washington State trucking regulations necessitated by the Surface Transportation Assistance Act of 1982 and attempts to standardize regulations among states are discussed.

Whether the public is ready for it or not, the big truck is here to stay. A concern that must be faced both now and in the future is that there may be more of them and they may get still bigger.

It can be agreed that the truck is a viable means of transporting goods or materials from one point to another. This means transporting raw materials from various areas such as forests, mines, ports, and farming communities to manufacturers and intermodal distribution points. Also of major importance are intrastate and interstate distribution of the manufactured products and final distribution to the user at the local level.

Why is the trucking industry important to Washington State? The Cascade Mountains divide the state into two quite distinct areas. West of the mountains the climate is mild with moderate precipitation. The region supports logging, lumber, pulp mills, wood products, commercial fishing industries, and poultry and dairy farming, the latter being the second largest agricultural business in the state. Puget Sound with its deep-water harbors is a major shipping area and supports such major industries as shipbuilding and the manufacture of aircraft, clothing, furniture, construction materials and equipment, aluminum, and glassware.

Eastern Washington has drier continental weather and greater temperature fluctuations. Vast expanses of previously open grasslands have been developed for agriculture through expanded use of irrigation. Fruits and grains are predominant and wheat is the state's number one agricultural product. Cattle and sheep ranches flourish as well.

These diverse activities of Washington State require an extensive transportation system to market their raw or finished products. The 7,000-mi state highway system as shown in Figure 1 serves this need.

During the past 15 to 20 years trucks have moved more tons of freight than railroads at the national level. In Washington State, however, railroads carried nearly twice as much tonnage as did trucks in 1967. By 1977 that figure had been almost reversed. Rail movement had dropped from 45 percent to 27 percent while truck movement had increased from 24 percent to 43 percent. These comparisons are shown in Figure 2.

This trend reversal is in part the result of railroad abandonments that have decreased the availability of railroad spur lines for the moving of crops from farms to the usual distribution centers.

Washington State Department of Transportation, Transportation Administration Building, Olympia, Wash. 98504.

The increase in the size of trucks has also been a factor. The number of big trucks (tractor-semitrailers or double combinations) increased more than 10 percent from 1972 to 1977 alone.

This increase in the number and size of trucks has presented a number of concerns to the Washington State Department of Transportation (WSDOT) and the traveling public. The primary concerns are the effects trucks have on the safety of the highway system and the life of the pavement structure.

Although existing transportation facilities serve all regions of the state, major movements of people and goods are concentrated within a limited number of travel corridors. These corridors, which are shown in Figure 3, connect principal activity and population centers. These corridors provide for movement of people and goods by various forms and are where most big truck movements are occurring.

The major corridors shown by hachure lines serve two major functions. They serve the interstate trucker making long hauls to and from the Puget Sound region and the intrastate trucker for getting the raw materials to markets or transfer points to rail or water facilities. Corridors west of Interstate 5 are used primarily by the logging industry to get timber to market whereas those east of the Cascades are generally used for transporting agricultural products to market or transfer facilities.

It is recognized that Washington, along with several other western states, may be considered liberal in its treatment of trucks on the highway system. Table 1 gives a summary of regulations for truck size both before and subsequent to the Surface Transportation Assistance Act (STAA) of 1982.

Before 1982 the typical legal vehicle was the semi with a width of 8 ft, which was 0.5 ft less than the AASHTO design vehicle at the time. Doubles could legally operate on the system if the total vehicle length was less than 65 ft. Because the tractor-semitrailer was the typical legal vehicle and also the predominant big truck in the traffic stream, it was considered the design vehicle and conformed to the characteristics of a WB-50.

Through a permit process larger trucks could also operate over the highway system. For general freight handling, annual permits for vehicles up to 75 ft in length, 8.5 ft in width, and with gross vehicle weight of 105,000 lb could be secured. These permits imposed no restrictions on when or where the vehicle could be operated.

For vehicles in excess of the noted limits, special oversize permits could be secured. This type of permit restricted the user's hours of travel, speed limit, and special route to his destination. No absolute maximum length was established by permit.

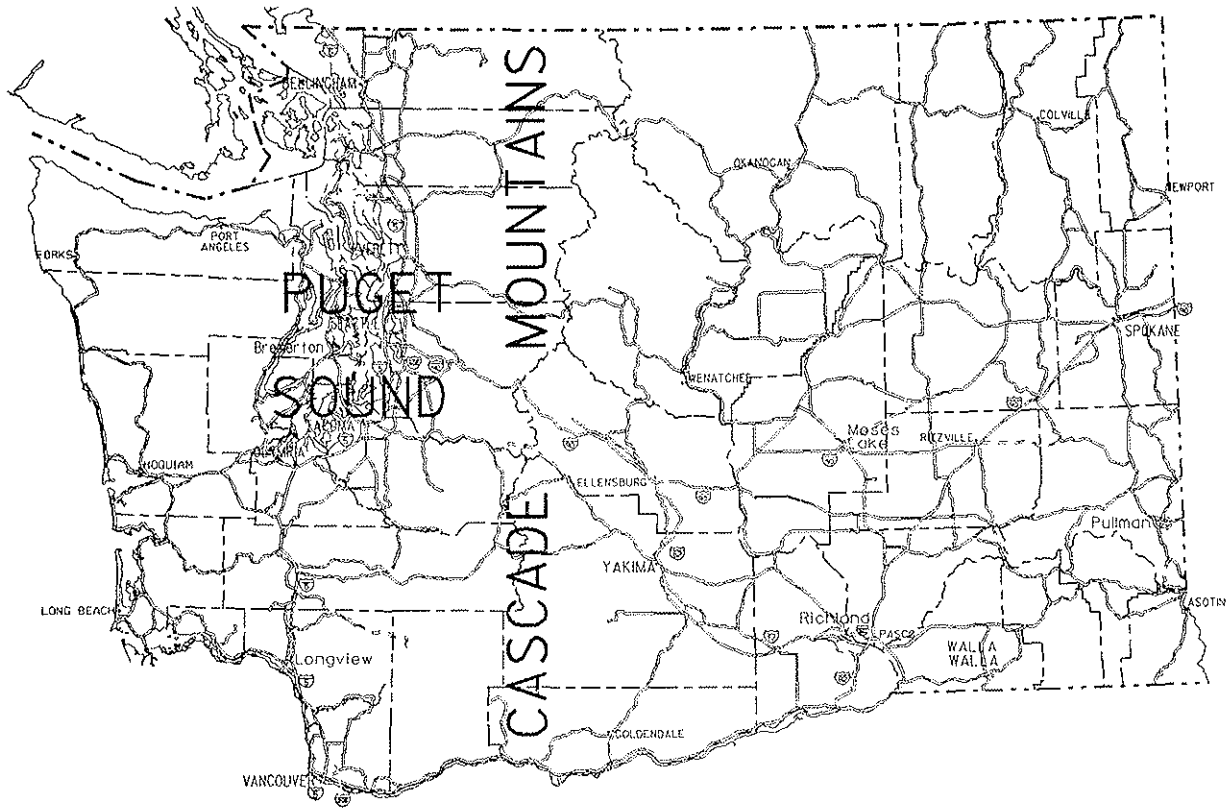


FIGURE 1 Washington State's highway system.

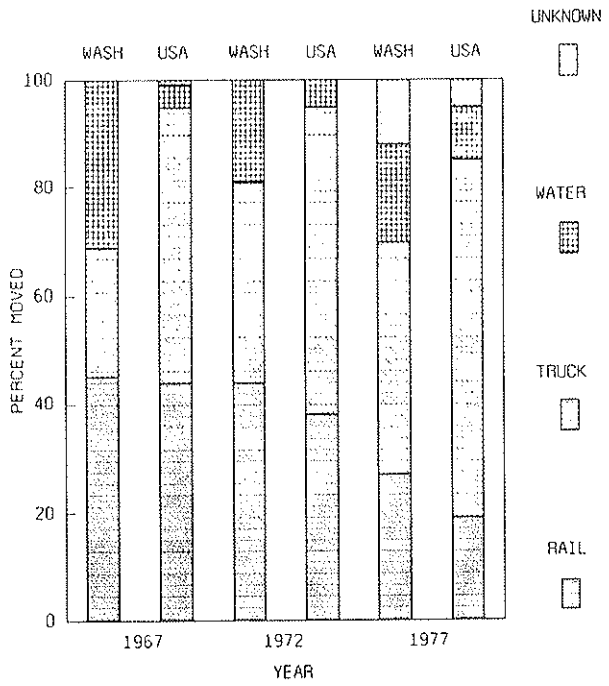


FIGURE 2 Percentage of freight (tons) movement by mode, Washington State versus national (exclusive of pipelines).

Route geometrics established the maximum length and the permissible route.

Classification by truck size on the highway system is not an exact science. The data for vehicle mix and accident rates given in Table 2 will provide at least some insight into their use of the highways.

Trucks heavier than 10,000 lb account for about 6 percent of travel on the urban system, and the rural system has about 9 percent trucks. Overall the mix is about 7 percent. The data in Table 2 indicate that these 7 percent trucks are traveling 12 to 15 percent of the mileage on the state highways.

It is estimated that 1 to 1 1/2 percent of the vehicles on the highway are big trucks (i.e., tractor-semitrailers or double combinations). The percentage of accidents involving big trucks during the 4-year period 1980-1983 ranges from 5 to 6 percent (Table 2). Big truck accident rates are 0.1 per million vehicle miles (MVM) of all vehicle miles traveled and 0.8 per MVM of all big truck miles traveled. The statewide accident rate for all vehicles ranges from 1.5 to 2.1 per MVM. The accident rate for big trucks therefore appears to be about one-half of that for all vehicles.

The surface condition of the roadway appears to play a big part in the big truck accident picture. Figure 4 shows the surface condition at the time of the accident for big trucks. Snow and ice were a factor in 16 percent of all big truck accidents. Wet pavements were evident in another 23 percent of these accidents. An interesting note is that a large number of these snow- and ice-related accidents occur on the best highways, the Interstate system. This is especially true on I-90 as it crosses the Cascade Mountains. Overall, 60 to 70 percent of the big truck accidents during the 1980-1984 period occurred within the previously discussed interstate or intrastate corridors.

There are two other potential safety problems that are not well documented. The first is those cases in which a large vehicle turning right strikes an object outside the roadway because of the off-tracking characteristics of the vehicle. The second is when a right-turning vehicle swings wide onto a crossroad to correct for offtracking and strikes a

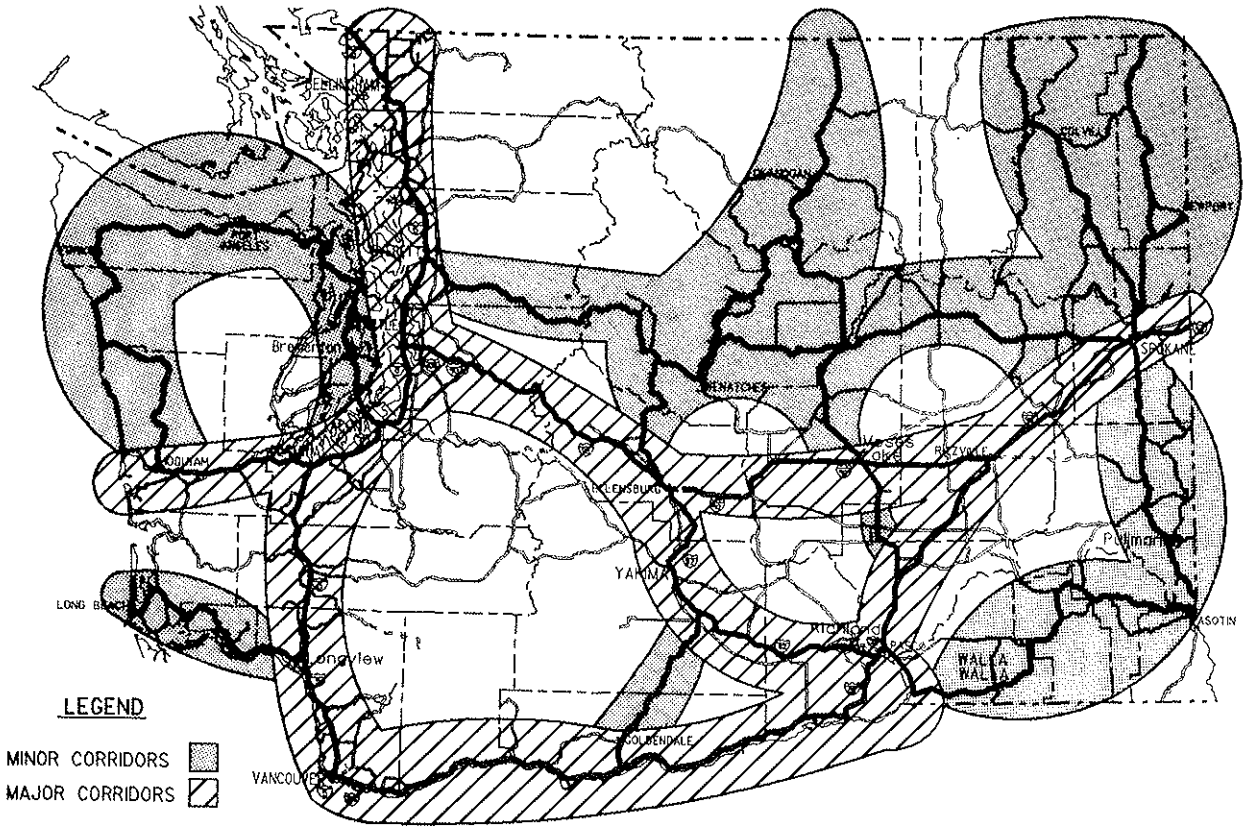


FIGURE 3 Statewide transportation corridors.

TABLE 1 Summary of Washington State's Truck Size Regulations

	Legal Width (ft)	Legal Length of Trailer (ft)	Total Legal Vehicle Length (ft)	Trailer Length by Permit (ft)	Total Vehicle Length Allowed by Permit for General Freight Hauling (ft)
Before 1967	8.0	40	60		
1967-1971	8.0	40	65		
1971-1975	8.0	45	65		73
1975-1982	8.0	45	65		75
1982-1985					
Semitrailer	8.5	48	65	56	75
Double	8.5	59[a]		68[a]	
1985					
Semitrailer	8.5	48	75	67	
Double	8.5	60[a]		68[a]	

[a]Combination vehicle.

TABLE 2 Big Truck Miles Traveled Versus Total Miles Traveled by All Vehicles

Year	Vehicle Miles Traveled by All Vehicles (billions)	Vehicle Miles Traveled by Trucks Heavier than 10,000 lb (billions)	Percentage of Total Miles Traveled by Big Trucks	Percentage of Accidents Involving Big Trucks
1977	15.34	1.87	12.2	
1978	16.50	2.20	13.3	
1979	16.34	2.32	14.2	
1980	16.33	2.39	14.6	6.1
1981	17.48	2.47	14.1	5.1
1982	17.70	2.41	13.6	5.6
1983	18.67	2.33	12.5	6.0

vehicle on the crossroad. It is anticipated that, with more big trucks in the traffic stream, those types of accidents will be more noticeable and therefore better documented in the future.

From the point of view of WSDOT, the 1982 STAA as it relates to big trucks can be summarized as follows:

- 102-in-wide trucks must be allowed;
- A state may grant special use permits for vehicles wider than 102 in.;
- 48-ft-long trailers must be allowed in tractor-semitrailer combinations and 28-ft-long trailers must be allowed in double combinations;
- Truck tractor lengths are not restricted;
- A grandfather clause requires that any trailer length previously legal must not be prohibited; and
- Reasonable access to the Interstate system; Federal-Aid system; terminals; facilities for food, fuel, repairs, and rest; and points of loading and unloading must be maintained.

Because the 1982 STAA required that certain vehicles be allowed to use both the Interstate and the designated primary Federal-Aid system, it became necessary for the states to review their systems to aid in the final determination of what roads they would designate. The approach in Washington State was to look at the system as a whole and review the historical use of the system by these types of vehicles. It was believed that the routes used by larger trucks before the 1982 STAA would be generally the same after the 1982 STAA.

Because the large trucks had been allowed by permit on the system before passage of the 1982 STAA, it was determined that legalizing of them as a result of the 1982 STAA would be little cause for hardship on the system. Except for the truly over-

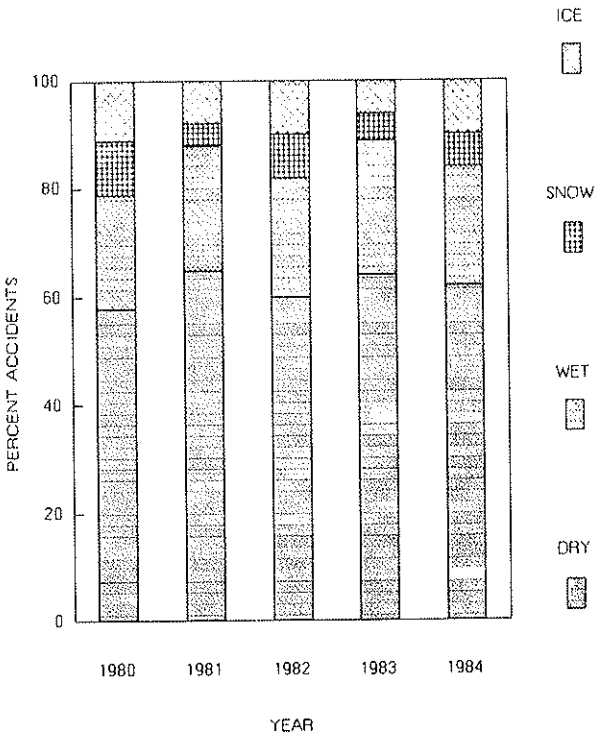


FIGURE 4 Percentage of accidents by surface condition.

sized loads, these vehicles were not restricted from any part of the system before and therefore should not be restricted as a result of the 1982 STAA. For this reason the entire 7,000-mi state highway system was designated and included both primary and secondary Federal-Aid highways.

As a result of the 1982 STAA, width increased from 8.0 to 8.5 ft. Single semitrailer length increased from 45 to 48 ft with no change in total length (Figure 5). Combined length of double trailers was established at 59 ft and total vehicle length was deleted (Figure 6). The legal vehicles in Washington State are the same ones mandated by the 1982 STAA. Through the permit process the trailer on a semi can be increased from 48 to 56 ft and the combined length of trailers on doubles can be increased from 60 to 68 ft. Once again there is no restriction on where these longer permit vehicles can operate. Special permits are still required for oversized loads and include restrictions on time of operation and the route they must take.

Figure 7 shows both total traffic and big truck traffic throughout the state since passage of the 1982 STAA and illustrates that there are some changes. Throughout the state the overall traffic volumes have generally increased about 2 percent per year with some areas seeing as much as a 4 percent increase. Big truck traffic, however, has remained relatively constant during the same period of time on the minor corridors. On the long-haul major corridors the percentage of big trucks has increased by as much as 40 percent along the I-90 corridor {on

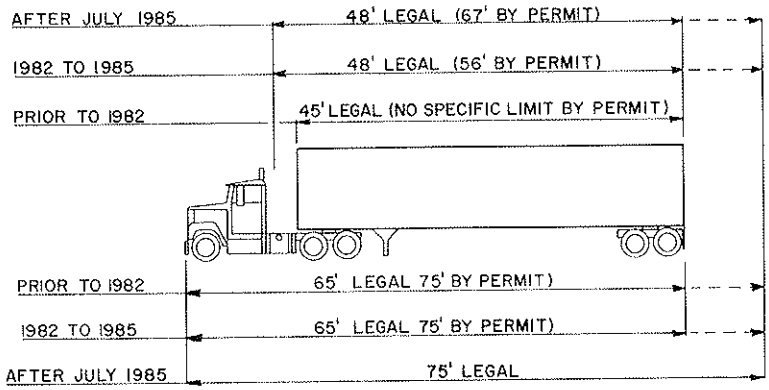


FIGURE 5 Legal and permit semi.

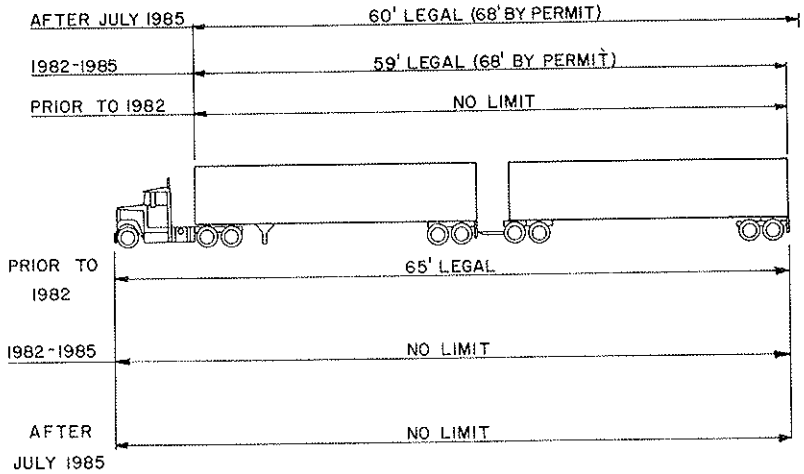


FIGURE 6 Legal and permit double.

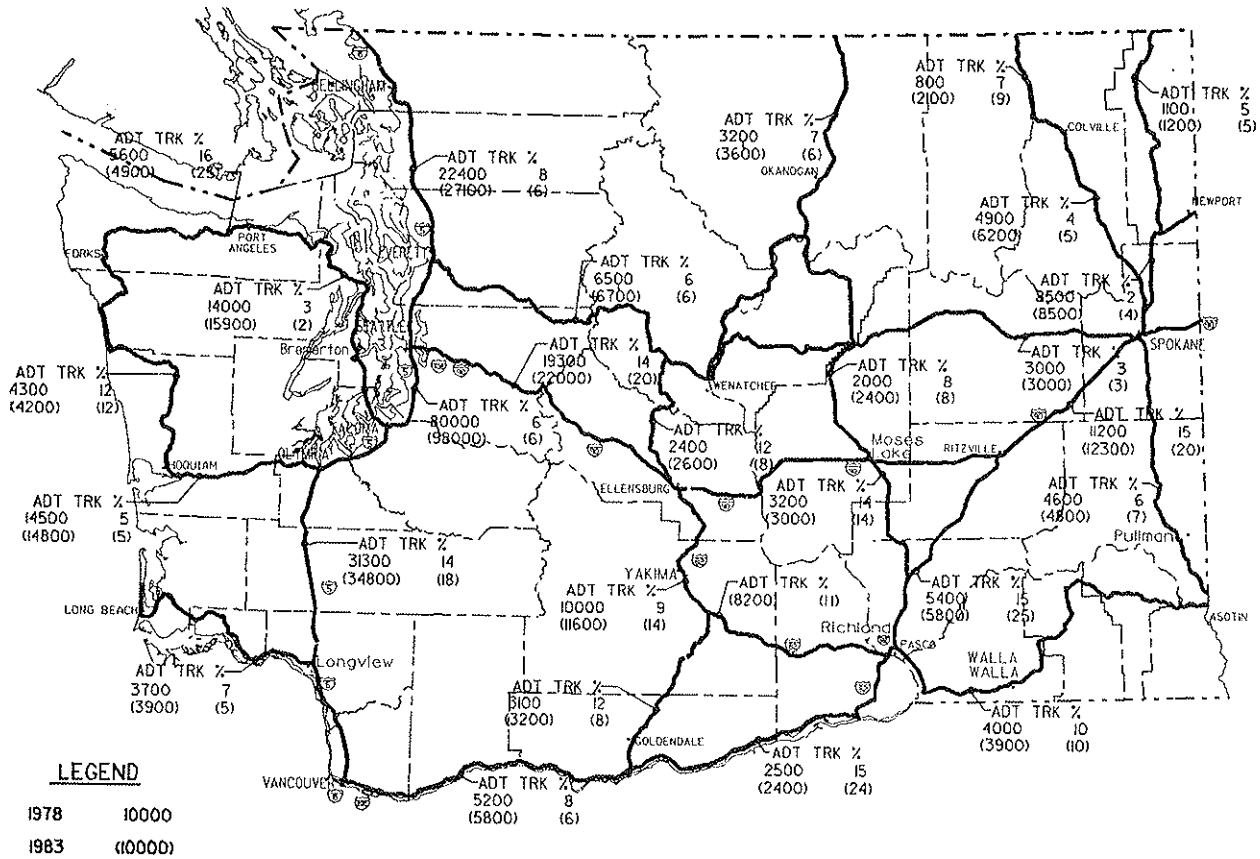


FIGURE 7 Weighted traffic volumes and big truck percentages in statewide corridors.

I-90 at Snoqualmie Pass average daily traffic (ADT) = 22,000 and trucks increased from 14 to 20 percent], 30 percent along the southern I-5 corridor (on I-5 at Centralia ADT = 35,000 and trucks increased from 14 to 18 percent), and 67 percent along the WA-395 corridor (on WA-395 near Mesa ADT = 6,000 and trucks increased from 15 to 25 percent).

As previously noted the larger trucks had been allowed on the system before the 1982 STAA. In general, it was believed that they were using mostly the major corridors that were designed to reasonably high geometric standards. However, because the number of these vehicles is increasing, it has caused WSDOT to take a closer look at even these high geometric standard areas. In the major corridors horizontal alignment on the Interstate roadways meets 80 mph design standards with 12-ft lanes. For the remaining major corridors, with the exception of the Washington side of the Columbia River corridor, 12-ft lanes and good alignment exist.

The minor corridors have at least 11-ft lanes but their alignments vary considerably. Most of these corridors will handle the larger vehicles within the available lane width but there are some isolated areas where tight curves would require encroachment on the adjacent lane to keep the vehicle on the roadway surface.

Strictly on the basis of geometrics, the current Interstate system cannot handle the legal vehicle within the designated lanes at every access point because of the offtracking characteristics of the vehicles. In rural areas the primary problem is the ramp terminal area at the crossroad. In suburban and urban areas ramp curvature comes into play especially on the loop ramps of parclo and cloverleaf interchanges. The cost to upgrade these ramp terminals and interchange ramps just for geometrics to accommodate the 1982 STAA vehicle is estimated at $32 million in 1983 dollars.

In a sampling of the rest of the non-Interstate system in both the major and minor corridors, it was noted that intersection geometrics are the biggest problem area. The cost for upgrading all non-Interstate routes is estimated to be $85 million. It should also be noted that these costs do not reflect any cost for providing access to nearby services. As can be seen, even though the entire system was designated, it was not designated in ignorance of its deficiencies.

Bigger trucks have also created challenges for cities and towns in providing local access to terminal or distribution centers. Generally speaking, truck routes within these localities have been established over a period of time. These routes have provided reasonable access to distribution points. There have always been some tight spots on city streets in the industrial areas. Bigger trucks now require more roadway for making turns because of their offtracking characteristics. Those municipalities that have large concentrations of big trucks are involved in modifying areas along existing truck routes to accommodate the bigger sizes. Intersection widening and increased curb return radii are the majority of modifications. In some cases new truck routes may be established as well.

Since passage of the 1982 STAA there has been little information published that relates specifically to the design characteristics of the larger vehicles mandated by the law. California prepared a study in 1983 that provided guidance for that state. This study determined that the tractor-trailer with the 48-ft box presented the worst case for offtracking and, as a result, offtracking curves for various turning radii were developed. For interim design

purposes WSDOT has adopted California's offtracking curves as appropriate design guidance for Washington.

WSDOT has not established rigid criteria for when it will design for large trucks; it is left up to the designer to make a decision based on existing and anticipated land use, traffic volumes, and localized condition. WSDOT has, however, revised its intersection geometric requirements to generally reflect the anticipated use of these larger vehicle types under various conditions. For freeway ramp terminals, the 75-ft right-turn radius has been increased to 95 ft.

There are other areas in which WSDOT is continuing to work to update both standards and the highway system to handle these larger vehicles. Of major concern are the sight distance requirements for negotiating intersections and for stopping as well as passing. Turning roadway widths and approaches to commercial establishments are being reviewed to determine what changes must be made. Storage areas at weigh stations and rest areas are also being studied to determine appropriate turning areas and stall length and angle. Doubles affect length whereas the 48-ft box combination affects angle and turning areas.

Although it would appear that the 1982 STAA would work a hardship on state highway systems, inconsistent laws in adjacent states also work a hardship on the trucking industry. What is legal in one state may not be legal next door. This was true of Washington, Oregon, and Idaho.

Typical areas of concern that have been a detriment to the trucking industry are the legalizing of doubles or triples, the legal weight per tire inch allowed, the combined legal trailer length of doubles, and the total legal length of a tractor-semitrailer. Table 3 gives differences that exist in Washington and the adjacent states of Oregon and Idaho.

To resolve these differences joint meetings have been held between the DOT staffs of all three states to discuss the standardization of these items in particular. As of late 1984 general agreement was reached on the figures shown in the "Proposed" column. The changes affecting Washington State were introduced in the legislative process and became effective July 28, 1985. Some of these values are in excess of those mandated by the STAA and yet because they were not standardized they continued to create problems. It is expected that this attempt at standardization will smooth the road for trucking firms operating in the Pacific Northwest.

TABLE 3 Regulatory Differences for Big Trucks in Washington, Oregon, and Idaho

	Washington	Oregon	Idaho	Proposed
Triples legal	No	Yes	Yes	NC
Legal weight carried per tire inch (lb)	550	550-600	300-800	600
Combined legal trailer length for doubles (ft)	59	60	60	60
Total legal length for semis (ft)	65	75	75	75
Mobile home transporting (ft)	14	14	17	14 and grandfather for Idaho

Note: NC = no change.

WSDOT intends to continue collecting and monitoring traffic and accident data for the entire system. This should provide appropriate information necessary to identify areas needing attention and what might be done to alleviate any problems that surface. Also on the horizon is high-speed weigh-in-motion (WIM). Its primary purpose is weight enforcement but it appears to have excellent capabilities of providing data on truck type and size. This information will then provide a better data base for determining type, number, and route usage of these vehicles. This information can also be used to help develop accident statistics and lead to possible corrective measures.

Currently WSDOT is participating in the "Crescent Demonstration Project" along with the states of Arizona, California, New Mexico, Oregon, and Texas and the province of British Columbia, Canada. The objective of this project is to develop and implement methods to improve the monitoring of truck usage, including measurements of mileage, size, weight, and speed. Automation of such data gathering will result in more efficient highway planning, design, and management.

Although the future is uncertain, WSDOT expects to keep abreast of the changing conditions and implement updated standards as necessary to keep the big trucks rolling. WSDOT would also expect to take an active role in areas of standardization among the states by suggesting that the steps taken by Washington, Oregon, and Idaho be expanded to include a larger area.

Existing Design Standards

BOB L. SMITH

ABSTRACT

Truck operations have a pronounced effect on the design of highways. Various characteristics of trucks are reflected in the standards used today for planning, designing, and operating highways. In determining geometry, the type, size, weight, gradability, acceleration, deceleration, and turning features of trucks all play an important part. These are accounted for by a classification of design vehicles that are represented by the largest trucks and their most imposing characteristics of operation. In the design process, one class of vehicles is selected for a particular type of highway or set of conditions. The application of standards, which reflect design vehicle performance, generally produces appropriate results. There are a few areas in which operational aspects of trucks may be further considered. Also, the more recent introduction of "extra" large trucks not yet included in national geometric highway standards for certain conditions should be addressed. The features and adequacy of present standards are reviewed and areas in which reinforcement or inclusion of additional standards or concerns is needed are highlighted.

Truck operations have a pronounced effect on the design of highways. Various characteristics of trucks are reflected in the standards used today for planning, designing, and operating highways. In determining geometry, the type, size, weight, gradability, acceleration, deceleration, and turning features of trucks all play an important part.

The primary guide or policy on highway geometric design is the American Association of State Highway and Transportation Officials (AASHTO) "A Policy on Geometric Design of Highways and Streets, 1984" (1), better known as the green book. The green book, for geometric design purposes, replaces the 1965 blue book on rural highways (2), the 1973 red book on urban highways and arterial streets (3), and other AASHTO publications. The technical data for the policy in the green book were essentially completed before the enactment of the Surface Transportation Assistance Act (STAA) of 1982, which increased the allowable maximum dimensions for truck tractor-trailer combinations ("extra large" trucks). The AASHTO subcommittee on design is currently updating these criteria and addenda to the green book will be published reflecting the effects of the extra large trucks.

In 1981 Gericke and Walton (4) published the results of a study of the effects that an increase in legal truck limits would have on geometric design elements and the implications that it would have for segments of the Texas highway system.

The physical characteristics of vehicles and the proportions of variously sized vehicles using the highways are positive controls in geometric design. Design vehicles are selected motor vehicles with the weight, dimensions, and operating characteristics used to establish highway design controls for accommodating vehicles of designated classes. The green book describes two general classes of vehicles: automobiles and trucks. The truck class includes single-unit vehicles, recreational vehicles, buses, truck tractor-semitrailer combinations, and trucks or truck tractors with semitrailers in combination with full trailers (1,pp.19-36).

Department of Civil Engineering, Kansas State University, Manhattan, Kans. 66506.

TURNING RADIUS

Scale drawings showing the minimum turning paths of the 10 design vehicles are included in the green book. These turning paths are often reproduced at various scales on transparent material in sets of "turning radius templates." They are excellent design aids in determining the design of such critical features as radii at intersections, radii of turning roadways, channelization details, and pavement edges at curved sections.

Of the three truck tractor-semitrailer combinations, WB-40, WB-50, and WB-60, the WB-50 is critical for design purposes. Figure 1 (1, Figure II-6)

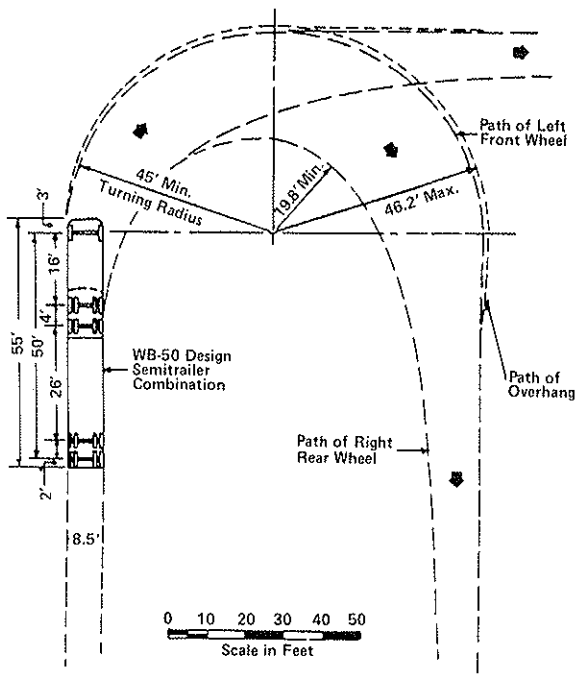


FIGURE 1 Minimum turning path for WB-50 design vehicle (reprinted with permission of AASHTO).

shows that an inside radius of 19.8 ft. and an outside radius of 46.2 ft. should be considered in design.

PAVEMENT WIDENING ON CURVES

Pavements on curves are sometimes widened to make operating conditions on curves comparable to those on tangents. Pavement widening is needed on certain curves because (a) trucks occupy a greater width because rear wheels generally track inside front wheels in rounding curves or (b) the drivers experience difficulty in steering their vehicles in the center of the lane. The need for widening is greater for curves that are unspiralized or unsuperelevated, or both. Two-lane highways with radii larger than 400 ft generally do not require widening as is shown in Table 1 (1, Table III-22). Minimum pavement inner edge curves for at-grade intersections and the effect of curb radii on turning paths are described in the green book (1,pp.727-751).

Figure 2 (1, Figure II-11) shows the swept path of vehicles similar to those of the STAA of 1982. It is noted (1,p.28) that "continuing research is being conducted into off-tracking of these vehicle configurations and the designer should verify the type and characteristics of the vehicle being used for design purposes."

SIGHT DISTANCES

The derived minimum stopping sight distances in the green book (1,p.138) are for automobile operation. Trucks generally require longer braking distances, but, because truck drivers are generally able to see the vertical features of obstructions substantially farther ahead because of the higher position of the seat in the vehicle, separate stopping sight distances for automobiles and trucks are not used in highway design standards. It is cautioned, however, that when horizontal sight restrictions occur on downgrades, particularly at the ends of long downgrades, the greater height of eye of the driver is of little value to him. It is recommended that designers use stopping sight distances that meet or exceed the values in Table 2 (1, Table III-1). The issue of lack of front wheel brakes and poor brake adjustment is discussed in a following section.

Necessary sight distances at intersections for stopped vehicles (automobiles or trucks) crossing a major highway, turning left onto a two-lane major highway, and turning right onto a two-lane major highway are presented in the green book (1,p.785).

Of particular concern is the required sight distance along the crossroad at terminals of ramps at interchanges. The data given in Table 3 (1, Table IX-9) indicate that the required sight distances for trucks are substantially greater than are those for automobiles (P vehicle). Figure 3 (1, Figure IX-29) shows how sight distances are measured at ramp terminals.

Passing sight distances are discussed in considerable detail in the green book (1,pp.148-162) but with almost no mention of trucks.

HORIZONTAL CURVES

Tables are presented in the green book for various values of rate of superelevation, design speed, degree or radius of curve, and recommended length of spiral or transition curve (1,pp.188-191). Spiral (transition) curves provide the only practical way in which superelevation can be attained in a theoretically correct manner. When the superelevation runoff is effected without a spiral curve, usually partly on curve and partly on tangent, the driver may have to steer opposite to the direction of the curve ahead when on the superelevated tangent portion in order to keep his vehicle on tangent (1, p.195). In most agencies that do not use spirals, the current design practice is to place approximately two-thirds of the runoff on the tangent approach and one-third on the curve. Without the use

TABLE 1 Calculated and Design Values for Pavement Widening on Open Highway Curves—Two-Lane Pavements, One or Two Way (reprinted with permission of AASHTO)

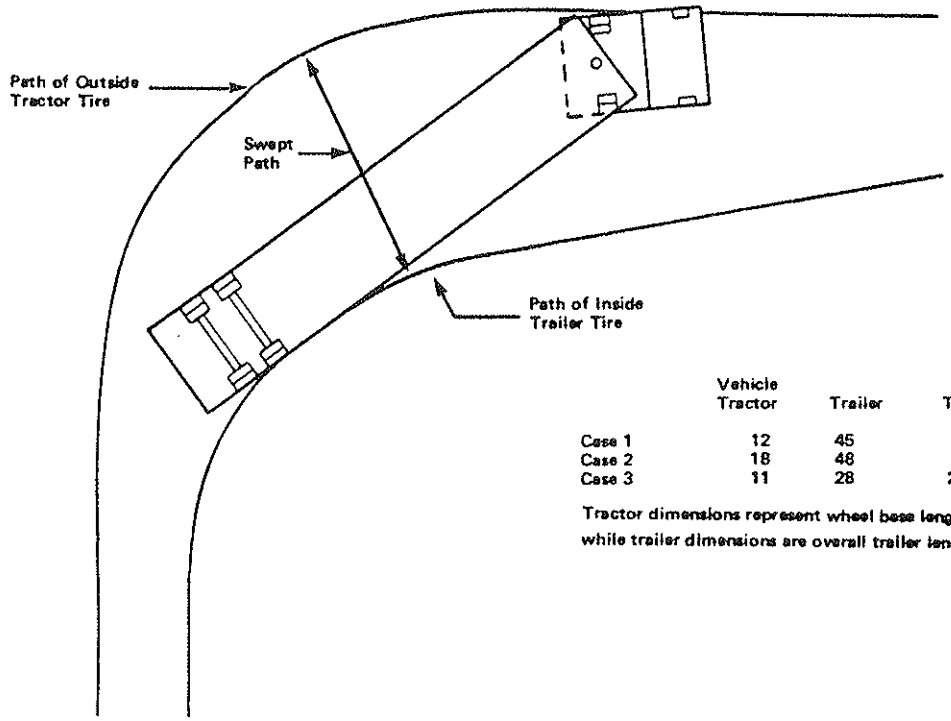
	Widening (ft) for Two-Lane Pavements on Curves for Width of Pavement on Tangent of:													
	24 ft					22 ft					20 ft			
Degree of Curve	Design Speed (mph)					Design Speed (mph)					Design Speed (mph)			
	30	40	50	60	70	30	40	50	60	70	30	40	50	60
1	0.0	0.0	0.0	0.0	0.0	0.5	0.5	0.5	1.0	1.0	1.5	1.5	1.5	2.0
2	0.0	0.0	0.0	0.5	0.5	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5
3	0.0	0.0	0.5	0.5	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5
4	0.0	0.5	0.5	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5	3.0
5	0.5	0.5	1.0	1.0		1.5	1.5	2.0	2.0		2.5	2.5	3.0	3.0
6	0.5	1.0	1.0	1.5	–	1.5	2.0	2.0	2.5		2.5	3.0	3.0	3.5
7	0.6	1.0	1.5	–		1.5	2.0	2.5			2.5	3.0	3.5	
8	1.0	1.0	1.5			2.0	2.0	2.5			3.0	3.0	3.5	
9	1.0	1.5	2.0			2.0	2.5	3.0			3.0	3.5	4.0	
10-11	1.0	1.5				2.0	2.5				3.0	3.5		
12-14.5	1.5	2.0				2.5	3.0				3.5	4.0		
15-18	2.0					3.0					4.0			
19-21	2.5					3.5					4.5			
22-25	3.0					4.0					5.0			
26-26.5	3.5					4.5					5.5			

NOTES: Values less than 2.0 may be disregarded.

3-lane pavements: multiply above values by 1.5.

4-lane pavements: multiply above values by 2.

Where semitrailers are significant, increase tabular values of widening by 0.5 for curves of 10° to 16°, and by 1.0 for curves 17° and sharper.



	Vehicle Tractor	Trailer	Trailer	Swept Path
Case 1	12	45		28.1'
Case 2	18	48		28.3'
Case 3	11	28	28	22.8'

Tractor dimensions represent wheel base length while trailer dimensions are overall trailer length.

FIGURE 2 Swept path width for various truck vehicles low-speed offtracking in a 90-degree turn (reprinted with permission of AASHTO).

TABLE 2 Stopping Sight Distance on Wet Pavement (reprinted with permission of AASHTO)

Design Speed (mph)	Assumed Speed for Condition (mph)	Brake Reaction		Coefficient of Friction (f)	Braking Distance on Level[a] (ft)	Stopping Sight Distance	
		Time (sec)	Distance (ft)			Computed[a] (ft)	Rounded for Design (ft)
20	20-20	2.5	73.3- 73.3	0.40	33.3- 33.3	106.7-106.7	125-125
25	24-25	2.5	88.0- 91.7	0.38	50.5- 54.8	138.5-146.5	150-150
30	28-30	2.5	102.7-110.0	0.35	74.7- 85.7	177.3-195.7	200-200
35	32-35	2.5	117.3-128.3	0.34	100.4-120.1	217.7-248.4	225-250
40	36-40	2.5	132.0-146.7	0.32	135.0-166.7	267.0-313.3	275-325
45	40-45	2.5	146.7-165.0	0.31	172.0-217.7	318.7-382.7	325-400
50	44-50	2.5	161.3-183.3	0.30	215.1-277.8	376.4-461.1	400-475
55	48-55	2.5	176.0-201.7	0.30	256.0-336.1	432.0-537.8	450-550
60	52-60	2.5	190.7-220.0	0.29	310.8-413.8	501.5-633.8	525-650
65	55-65	2.5	201.7-238.3	0.29	347.7-485.6	549.4-724.0	550-725
70	58-70	2.5	212.7-256.7	0.28	400.5-583.3	613.1-840.0	625-850

[a]Different values for the same speed result from using unequal coefficients of friction.

TABLE 3 Required Sight Distance Along the Crossroad at Terminals of Ramps at Interchanges (reprinted with permission of AASHTO)

Assumed Design Speed on Crossroad Through the Interchange	Sight Distance Required to Permit Design Vehicle to Turn Left from Ramp to Crossroad (ft)[a]			Sight Distance Available to Entering Vehicle When Vertical Curve on Crossroad is Designed for Stopping Sight Distance[b]	
	Design Vehicle Assumed at Ramp Terminal				
	P	SU	WB-50	P	SU or WB-50
70	740	1,060	1,430	920	1,040
60	630	910	1,230	730	820
50	530	760	1,030	540	600
40	420	610	820	420	480
30	320	460	620	310	350

[a]Sight distance measured from height of eye of 3.50 ft for P, SU, and WB-50 design vehicles to an object 4.25 ft high.

[b]Minimum available stopping sight distance based on the assumption that there is no horizontal sight obstruction and that $S < L$.

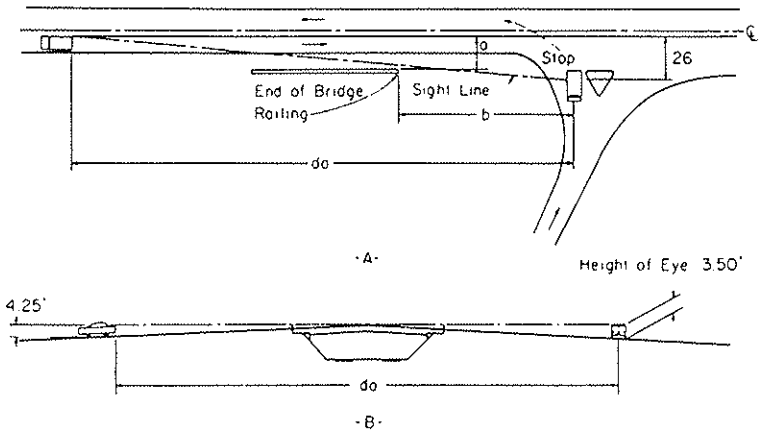


FIGURE 3 Measurement of sight distance at ramp terminals (reprinted with permission of AASHTO).

of spirals there is superelevation on the tangent where none is needed, and there is not enough superelevation on a substantial part of the circular curve (end sections). Vehicles traveling at the design speed thus develop side friction factors in excess of the allowable minimum on the end sections of the curve. Although the side friction factor developed on the tangent is undesirable, the development on curves of friction factors greatly in excess of the design basis results in hazardous conditions (1,p.203).

Compound circular curves are advantageous in effecting desirable shapes of turning roadways at at-grade intersections and at ramps at interchanges. On compound curves for open highways it is generally accepted that the ratio of the flatter radius to the sharper radius should not exceed 1.5 to 1. For compound curves at intersections where drivers accept more rapid changes in direction and speed, the radius of the curves can be as high as 100 percent greater than the radius of the sharper arc, a ratio of 2 to 1 (1,p.223). It is pointed out that spiral curves have an advantage in providing for natural travel paths and a correct transition from one superelevation rate to another (1,pp.222-223,249-250).

A reverse curve should have spiral transitions between the curves in order to properly handle the superelevation (1,p.250). As is shown later in this paper, circular curves, compound curves, and reverse curves, without proper spiral transitions, can present particularly dangerous situations for truck operations.

VERTICAL ALIGNMENT OF CURVES AND GRADES

The "critical length of grade" is used to indicate the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction of speed. To establish design values for critical lengths of grade for which gradability of trucks is the determining factor, the following data or assumptions are needed:

1. Size and power of a representative truck to be used as a design vehicle along with gradability data for this vehicle. A loaded truck, powered so that the weight-to-horsepower ratio is about 300 is representative of the size and type of vehicle normally used for design control on main highways.
2. Speed at entrance to critical length of grade.
3. Minimum speed on the grade below which interference to following vehicles is considered unreasonable. The common basis for determining critical length of grade is a reduction in speed of trucks below the average running speed. It is recommended that a 10-mph reduction criterion be used as the general design guide for determining critical lengths of grade. A design technique is suggested in the green book (1,pp.259-264).

For increased safety, climbing lanes are considered where the length of grade causes a reduction of 10 mph or more in the speed of loaded vehicles provided the volume of traffic and percentage of heavy trucks justify the added costs (1,pp.265-278).

Leisch et al. (5,6) and Rowan and Johnson (7) have suggested the use of a speed profile as a technique to achieve a consistent design speed, critical length of grade, and the design of creeper lanes for both existing highways and new designs.

The speed profile provides a continuous plot of the average speed of vehicles along the roadway in each direction of travel at a time when traffic is sufficiently light to represent a condition that may be termed "free flowing." Both automobile and representative loaded truck speeds are plotted along with the vertical and horizontal alignment. This allows a complete analysis, in an easy fashion, for a "new" highway and allows the designer to change design speeds, grades, and curves to achieve a consistent design.

On existing highways the technique can be used to determine the location of creeper lane beginnings and ends based on the 10-mph speed differential rule. Note in Figure 4 (5) that a climbing lane should begin at about Station 230 and extend to about Station 315. Note that the speed differential decreases to about 10 mph, an acceptable figure, at 305, but this would place the end of the creeper lane in a sharp horizontal curve.

The green book also describes a procedure for the design of emergency escape ramps for runaway trucks on steep downgrades (1,pp.293-303).

CROSSOVER CROWN (algebraic difference of cross slopes)

It is suggested that the use of cross slopes steeper than 2 percent on high-type, high-speed pavements with a central crown line is not desirable. In a passing maneuver, drivers must cross and recross the crown line and negotiate a total rollover (crossover crown) or cross-slope change of more than 4 percent. The reverse curve path of travel of the passing vehicle causes a reversal in the direction of the cen-

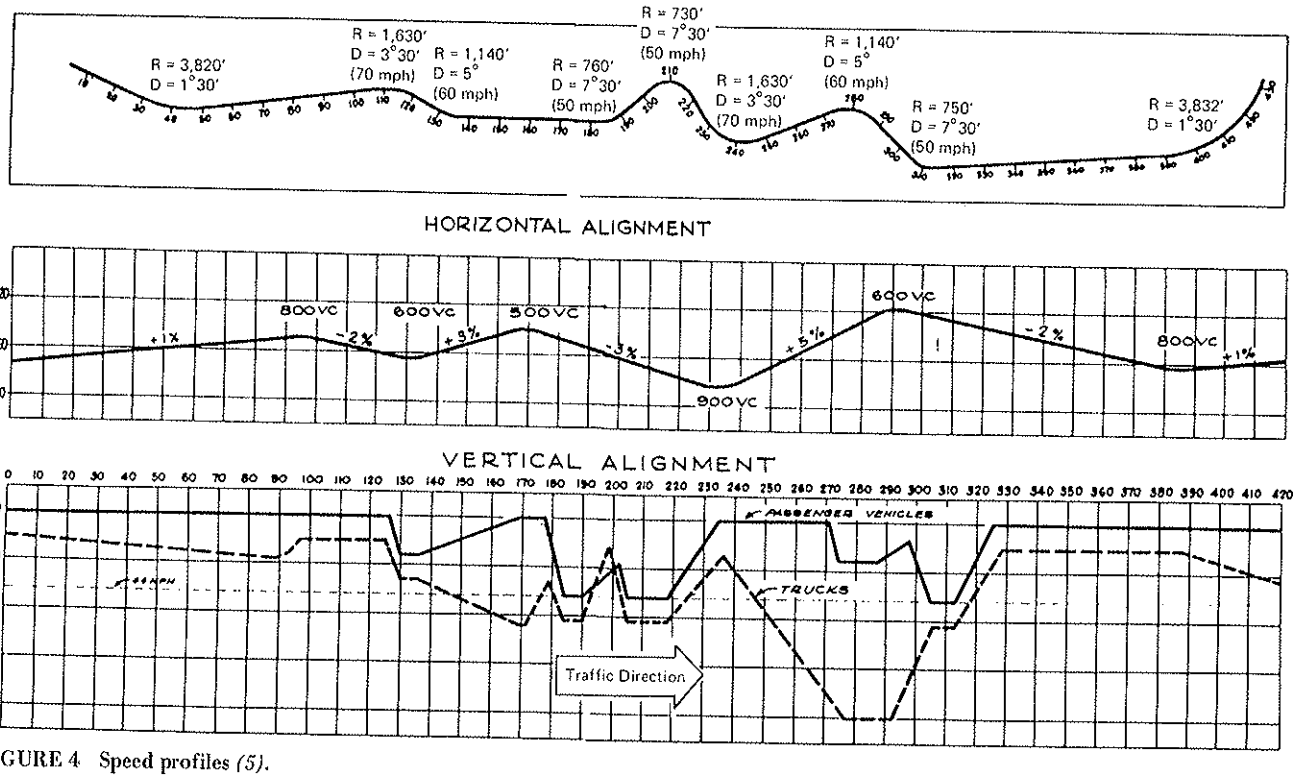


FIGURE 4 Speed profiles *(5)*.

trifugal force, which is further exaggerated by the effect of the reversing cross slopes. Trucks with high body loads crossing the crown line are caused to sway from side to side when traveling at high speed, at which time control is difficult (1,p.357).

For turning roadway and ramp terminals a desirable maximum algebraic difference at a crossover crown line is 4 or 5 percent but it may be as high as 6 percent at low speeds and where there are few trucks (1,pp.814,1018). The maximum crossover crown values have severe safety implications for trucks. This is, of course, a problem similar to that of designing proper transitions for superelevated sections on compound, reverse, and simple curves.

MEDIAN OPENINGS

An important factor in designing median openings is the path of each design vehicle making a minimum left turn at 10 to 15 mph (1,p.847). The paths of design vehicles making right turns were discussed earlier (Figure 1). Any differences between the minimum turning radii for left turns and those for right turns are small and are insignificant in highway design. In using turning radius templates, simply "turn the template over" to go from right turn to left turn. Note that the objective is to have the turning vehicles stay entirely in their own lanes (no encroachment on adjacent lanes) as is shown in Figure 5 (1, Figure IX-55).

SUBTLETIES OF DESIGNING FOR TRUCKS

Rollover

Hutchinson and Shapley (8) present some sobering implications regarding the potential for truck rollover.

In assessing the rollover potential of tractor-trailers, the conclusion arrived at will depend on the extent to which the various flexibilities and other properties of the trucks are considered.

For example, a perfectly rigid simple vehicle with a height of center of gravity above the ground (h) and an overall width of assembly (t) would roll over at a steady lateral acceleration of

$$A_{max} = tg/2h$$

where g is the acceleration due to gravity (ft/sec^2).

Note that t/2h is often called the "tripping coefficient" of friction. However, none of the components can really be considered rigid, especially the tires. Flexible tires further reduce effective truck width. Note also that the forces attempting to overturn the vehicle will also tend to deflect the tires and wheels.

Roll and lateral movement can also be generated by such things as looseness in the spring mounts and clearance in the fifth wheel. Both of these effects serve to reduce the lateral acceleration required for overturning.

Road surface irregularities, entering a curve, superelevation templet warp, and roughness as well as transient roll inputs induced in response to steering can directly disturb a vehicle in roll. When certain dynamic effects are present these vehicles may be caused to overturn at levels of lateral acceleration approaching half of their steady-state limit even without any special outside tripping force inputs.

Hutchinson and Shapley (8) give an example in which it is shown that the cornering ability of a loaded 18-wheeler does not compare at all favorably with that of the average well-designed automobile. In the example curve, the automobile would slide out at about 84 mph whereas the flexible truck would overturn at 46 mph using a tripping coefficient of friction of only about 0.17.

Surely then, simple curves without spirals, reverse curves, compound curves, and areas of high

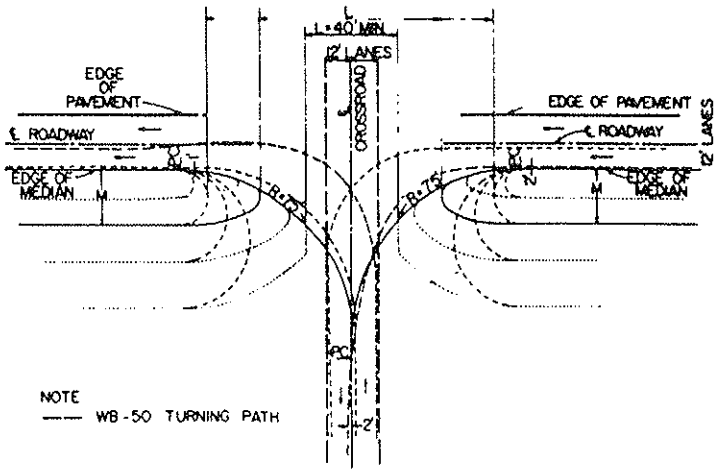


FIGURE 5 Minimum design of median opening for WB-40 design vehicle, control radius 75 ft (reprinted with permission of AASHTO).

crossover crown values are potential locations for truck rollovers at modest speeds.

The rollover of trucks can also be increased dramatically by a nudge (tripping force), in the direction of centripetal acceleration, by an automobile in the truck's fifth wheel or jackstand area. For example, suppose a truck on the inside lane of a curve and an automobile on the outside lane (side by side at the truck's fifth wheel area and traveling in the same direction) bump or nudge each other because one or both leave their respective lanes. This can easily increase the tripping action so the truck will quickly overturn onto the automobile.

Guardrails

One of the reasons a truck may strike a guardrail is a flat front tire. Some trucks are uncontrollable in the event of a flat front tire. This uncontrollability may reflect either the original design of the vehicle or poor maintenance. With a centerpoint front axle and a well-maintained rig, the alert driver of an 18-wheeler can often be expected to correct for flat front tire vehicle yaw within the lateral clear zone on modern highways. Unfortunately there is frequently a truck wreck anyhow. A flexible automobile-type single-beam W-section steel guardrail is often encountered parallel to the pavement about where the truck is brought under control. Portions of the guardrail often damage the brake and steering systems and have enough rail strength remaining to guide the truck into the obstructions the guardrail was "protecting" (8). Some guardrails, such as rigid concrete "New Jersey" barriers, are effective in guiding trucks yet minimizing vehicle damage and penetration (9).

Dished Wheel Tracks

The abrasion of bare pavements by studded tires and the compression or lateral displacement of unstable flexible pavement often result in depressed wheel tracks. This causes a properly loaded set of dual tires to have one of the tires overloaded when it runs along the hump while the mate overhangs the dish or depression.

If the brakes are applied this can cause a yaw to the right on dry pavements and to the left under certain other circumstances (8). Hydroplaning is also a distinct possibility.

Washboard Pavement

Washboard pavements can cause the tires of a lightly loaded 18-wheeler to bounce up and down and skitter off the crown of a dry road into the ditch without braking at speeds as low as 30 mph (8).

Pavement Warp

For reasons already given in the foregoing discussion of truck rollover, compound curvature, excessive crown templet warp, and superelevation templet warp that "feels tricky" but "not too bad" in an automobile can be enough to cause load shift or rollover, or both, in large trucks traveling at the posted speed limit or advisory speed (8).

The causes of such templet warps may lie in original faulty design, but originally satisfactory design and construction (especially superelevation) may have been so altered during routine maintenance and overlays that no superelevation or even reversed superelevation may now exist! The use of a ball-bank indicator mounted on the dashboard of an automobile is recommended for quickly checking safe speeds versus superelevation. Is anybody checking superelevations after overlay projects?

Truck Brakes

Many trucks are running with no front tractor brakes. They have been disconnected to prevent "lockup" and lack of steering. No front brakes and lockup of driving wheel brakes are virtually certain to force the tractor to try to "reverse ends" resulting in a jackknife situation. Tractor and trailer brakes are often in poor adjustment. The resulting lack of brakes or adjustment increases the truck braking distance even more and can more than negate the positive effect of higher driver eye height in all braking situations.

Pavement Edge Dropoffs and Surface Discontinuities

In "The Influence of Roadway Discontinuities--A State-of-the-Art Report" (10,pp.42,37) the authors caution: "Large commercial vehicles, because of their size and design, may be more sensitive than passenger cars to some surface discontinuities. . . . From the knowledge of truck dynamic properties, it

may be expected that certain of these road features can create a greater vibration disturbance to trucks than to cars."

SUMMARY

The preceding overview of existing design standards, coupled with the stated concerns about the subtleties in designing for trucks, points to the need for a definitive highway design and maintenance guide to satisfy the unique safety-critical operational requirements of trucks.

It is hoped that this symposium will be of assistance to AASHTO's Subcommittee on Design in its efforts to update the green book to reflect the "large trucks" allowed under the STAA of 1982 (1,p.iv).

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Sight Distance Problems Related to Large Trucks

P. S. FANCHER

ABSTRACT

In this paper are discussed the influences of the properties of large trucks on (a) sight distances for accelerating across intersections, (b) passing sight distances on two-lane highways, and (c) stopping sight distances for crest vertical curves. The vehicle properties considered include power-to-weight ratios (acceleration capabilities), overall lengths, driver eye heights, and braking capabilities. The findings presented here indicate that (a) current policy of AASHTO may be used to obtain conservative estimates of the time required to accelerate across intersections, (b) longer periods of time in the left lane are needed for passing longer trucks, and (c) if controlled stops without jackknifing, trailer swinging, or vehicle spins are to be performed by truck drivers, the required stopping sight distances at high speeds are much longer than those recommended in the AASHTO policy.

The intent of this paper is to provide an understanding of how sight distance requirements are influenced by the properties of large trucks. Whether large trucks are involved in crossing intersections, passing situations on two-lane roads, or stopping to avoid objects on the highway, pertinent truck characteristics are enough different from those of automobiles that design policies based on automobile characteristics cannot be assumed to be appropriate. With regard to crossing intersections, there is a recommended AASHTO policy for heavy trucks (the WB-50 design vehicle) (1). However, AASHTO policy for passing sight distance is based on acceleration capabilities of automobiles. And, although trucks are mentioned, the policy for stopping sight distance on crest vertical curves is based on the

Transportation Research Institute, University of Michigan, Ann Arbor, Mich. 48109.

locked-wheel performance of automobile tires. In the following sections of this paper relationships between truck performance and sight distance policies based on AASHTO recommendations are examined. [The AASHTO policy recognizes that it is less than adequate for large trucks traveling at high speed (1, p.iv).]

SIGHT DISTANCE FOR ACCELERATING ACROSS INTERSECTIONS

The weight-to-power ratio of heavy trucks (up to 80,000 lb) has experienced a decreasing trend since 1949 (Figure 1). This means that modern trucks can cross an intersection from a stop in less time than was required previously. A recent study (2) has shown that the accelerating time for the assumed WB-50 design vehicles given in the AASHTO policy (1) for geometric design is conservative compared with (a) measured results for a 273-lb/hp truck and (b) calculated results for a 300-lb/hp truck. Given current trends toward vehicles with higher power-to-weight ratios and less rolling resistance, the AASHTO curve of accelerating time versus distance traveled (Figure 2) will provide reasonable estimates of vehicle performance until some unforeseen factor causes a major change in the power-to-weight ratios of heavy vehicles.

Although accelerating time versus distance traveled during acceleration is not changing rapidly, there is a trend toward longer vehicles (e.g., the use of more doubles and longer semitrailers). The distance traveled for these longer vehicles to clear an intersection may increase by approximately 10 or 15 ft in some cases. At first approximation, the AASHTO recommendation (Figure 2) yields an additional accelerating time of 1 sec per each 15 ft of travel for distances of from 60 to 160 ft. If the cross traffic were traveling at 55 mph, this additional second of accelerating time would mean an additional intersection sight distance of approximately 80 ft. As long as future longer vehicles do not have lower power-to-weight ratios than current vehicles, the AASHTO design recommendations will apply with the added length being accounted for by using the appropriate distance traveled during acceleration when reading accelerating time from the design curves.

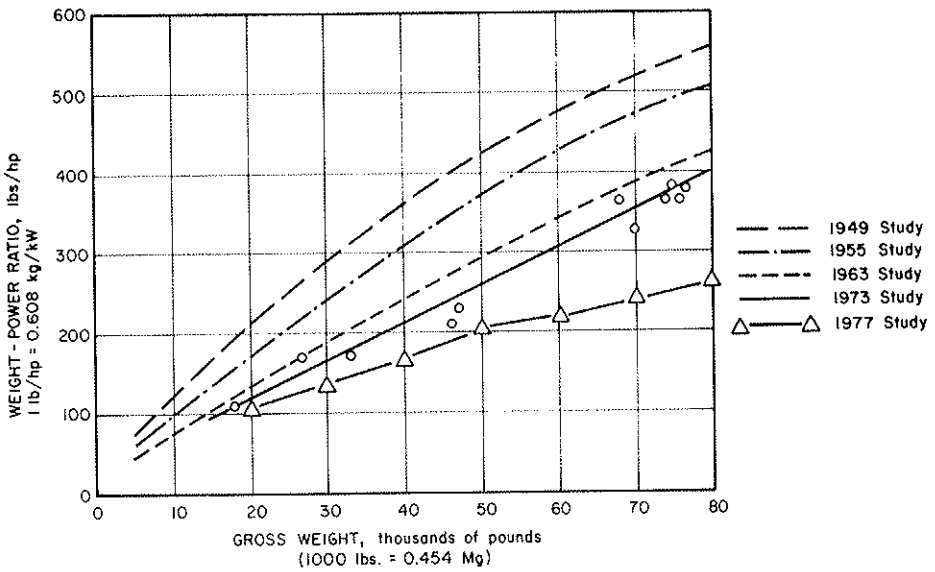


FIGURE 1 Trend in weight-to-power ratios from 1949 to 1977 (2).

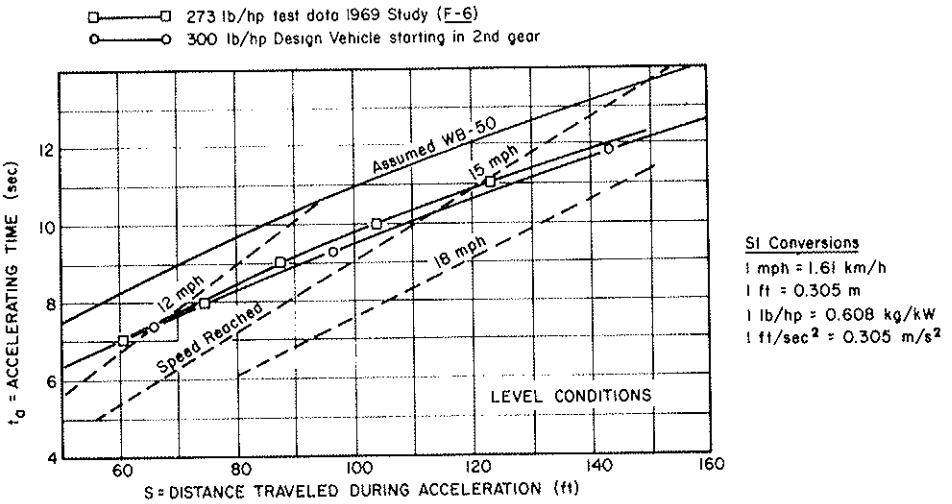


FIGURE 2 Accelerating time versus distance traveled during acceleration.

PASSING SIGHT DISTANCE ON TWO-LANE HIGHWAYS

For passing on two-lane highways, the AASHTO design policy specifies the sight distances needed for one vehicle to pass another before encountering oncoming traffic. The total passing sight distance is divided into four parts: (a) initial acceleration distance including perception and reaction time, (b) distance traveled in the left lane, (c) clearance safety margin with respect to the opposing vehicle, and (d) distance traveled by the opposing vehicle during two-thirds of the time the passing vehicle occupies the left lane.

Vehicle acceleration performance is involved in this maneuver. For automobiles the contribution of the initial acceleration part of the maneuver is approximately 15 percent of the total passing sight distance. However, some heavy trucks have sustained speeds on level ground of no more than 60 mph when fully laden, and, at speeds near 40 mph, distances on the order of 2,500 to 3,000 ft may be needed to accelerate to 50 mph.

On the basis of these observations, the AASHTO passing sight distance model used for automobiles does not appear to be appropriate for heavy trucks. It might be better hypothesized that trucks pass when they have already attained passing speed before encountering a vehicle to be passed. Because of the height of their eyes, truck drivers can see over cars in front of them and decide, without slowing down or pulling out into the left lane, whether to pass. If this hypothesized scenario is accepted, the passing sight distance used for automobiles would be adequate for trucks that have not had to slow down. However, if trucks must slow down for slowly moving vehicles, they will require long distances to accelerate to speeds high enough to pass vehicles traveling at velocities above 40 mph.

Furthermore, researchers (3) have defined a critical point at which the passing vehicle comes abreast of the vehicle to be passed. At this point, the driver decides whether to complete the pass or to abort the maneuver. Under this model of the passing situation, the initial acceleration distance is not included in the minimum passing sight distance. Hence, the acceleration characteristics of trucks do not influence the passing sight distance required for heavy trucks if this model is used.

In addition to difficulties encountered in "seeing around" trucks, the distance traveled in the left lane by an automobile passing a long truck is longer than that needed to pass another automobile. This increase in passing distance and, consequently, passing time will increase the time during which approaching traffic will travel. An additional 30 ft of vehicle to be passed means that an additional 2 sec are needed for passing at a speed differential of 10 mph. An oncoming vehicle would travel approximately 160 ft during these 2 sec if it were traveling at 55 mph. The presence of long trucks could add more than 300 ft to the passing sight distance recommended for passing shorter vehicles, when allowance is made for (a) 2 more seconds of travel in the left lane and (b) 2 more seconds of travel for the oncoming vehicle. (Following this line of reasoning, a truck-passing-truck situation might require 4 additional seconds in the left lane and also 4 additional seconds for travel of the oncoming vehicle--perhaps 600 ft more than the distance recommended for automobiles passing automobiles.)

STOPPING SIGHT DISTANCE

Sight distance for passing and for crossing intersections depends on the acceleration capabilities of the vehicles involved. Clearly, the acceleration capabilities of heavy vehicles have little to do with stopping sight distance. Nevertheless, acceleration capabilities do influence a number of situations in which heavy trucks are able to travel at high speeds. For example, when climbing a long upgrade section before a crest vertical curve, a heavy truck may proceed slowly. Calculations, made for studying the stopping sight distance of trucks at those particular locations, might well consider the speed of approach of the vehicles involved.

Given an initial speed, the primary parameters that affect stopping sight distance include (a) perception and reaction time, (b) driver eye height, (c) height of the object in the roadway, and (d) braking distance. Values of these parameters are used to calculate the lengths of vertical curves that will not hide significant hazards from the driver until the driver is too close to be able to deal with them effectively.

In the United States, AASHTO recommends a perception and reaction time of 2.5 sec and an object height of 6 in. In this paper, it is assumed that these values apply to all vehicles, including heavy trucks. Matters related to eye height and braking distance will be examined in detail in the following discussions.

Influence of Eye Height of Truck Drivers

The AASHTO policy for crest vertical curves is based on automobile characteristics (1). When trucks are compared with automobiles, the additional eye height of the truck driver is believed to compensate for the reduced braking capabilities of trucks.

Geometric relationships are available for calculating the length of crest vertical curves for given values of eye height (h_e), object height (h_o), and available (specified) sight distance (S_A). These relationships are derivable from the basic properties of parabolas and tangents to these parabolas (2, Appendix E). In this context, the vertical distance between a parabola and its tangent (as shown in Figure 3) is given by the following simple equation:

$$h = CS^2 \tag{1}$$

where

h = vertical height,
S = horizontal distance from a selected point of tangency, and
C = coefficient of x^2 in the parabolic expression

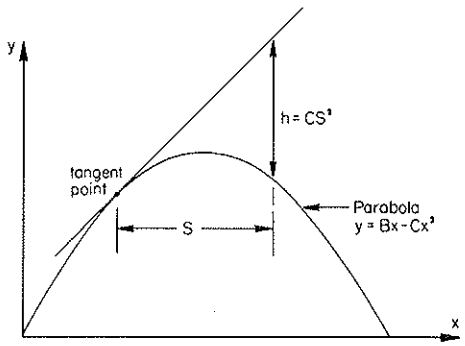


FIGURE 3 Sight distance with respect to a parabola.

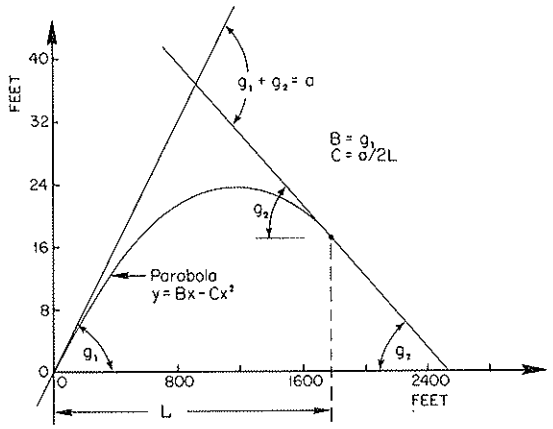


FIGURE 4 Geometric properties of crest vertical curves.

$$y = Bx - Cx^2 \tag{2}$$

The coefficients B and C in Equation 2 are related to the geometric properties of a crest vertical curve by the following equations that use the symbols shown in Figure 4.

$$B = g_1 \tag{3}$$

$$C = (g_1 + g_2)/2L = a/2L \tag{4}$$

Now consider the sight distance between a driver's eyes and an object when both the driver and the object are on the vertical curve; that is,

$$S_e = (h_e/C)^{0.5}$$

$$S_o = (h_o/C)^{0.5}$$

and

$$S_A = S_e + S_o = (h_e/C)^{0.5} + (h_o/C)^{0.5}$$

$$= (2L/a)^{0.5}(h_e^{0.5} + h_o^{0.5}) \tag{5}$$

However, the highway design problem is to find the length of the vertical curve (L) given the needed available sight distance (S_A). Solving Equation 5 for L yields the following design equation that applies when $L > S_A$:

$$L = S_A^2/[(2/a)(h_e^{0.5} + h_o^{0.5})^2] \tag{6}$$

For $L < S_A$, maximum L, corresponding to minimum sight distance, is obtained when both the object and the eye are on either side of the vertical curve. For this case, the following equation is used in design (2, Appendix E):

$$L = 2S_A - [(2/a)(h_e^{0.5} + h_o^{0.5})] \tag{7}$$

Either Equation 6 or Equation 7 can be used to examine the situation in which $L = S_A$. (Clearly, Equation 6 or Equation 7 will give the same result because they are equivalent for $S = L$.) Let L* be the value of L if L were equal to S_A; specifically,

$$L^* = (2/a)(h_e^{0.5} + h_o^{0.5})^2 \tag{8}$$

The quantity L* has at least three interesting properties: (a) it does not depend on sight distance, (b) it can be used to simplify Equations 6 and 7, and (c) it can be used conveniently to gain an understanding of the influence of differences in eye heights.

Using L*, the design equations can be expressed as follows:

For $S_A < L^*$, $L = 2S_A - L^*$

(i.e., for $L < S_A < L^*$) (9)

For $S_A > L^*$, $L = S_A^2/L^*$

(i.e., for $L > S_A > L^*$) (10)

For either $S_A > L^*$ or $S_A < L^*$, the length of vertical curve (L) depends on two separable quantities: (a) S_A, the needed available sight distance, and (b) L*, which is a function of eye height. The influence of eye height can be illustrated by comparing L_t^*, evaluated for eye heights typical of truck drivers, with L_c^*, evaluated for drivers of automobiles. For example, let the algebraic difference in grades a = 0.06 (6 percent) and h_{et} = 100 in. for trucks and h_{ec} = 40 in. and h_o = 6 in. for automobiles. Then, for the truck,

L_t^* = 431 ft,

and, for the car,

L_c^* = 214 ft.

In general, regardless of the algebraic difference in grades,

$$L_t^*/L_c^* = [(h_{et}^{0.5} + h_o^{0.5})/(h_{ec}^{0.5} + h_o^{0.5})]^2 \tag{11}$$

For h_{et} = 100 in., h_o = 6 in., and h_{ec} = 40 in.:

$L_t^*/L_c^* = 2.01$

For some heavy trucks and drivers, h_{et} might be as low as 90 in. In this case, $L_t^*/L_c^* = 1.85$. Clearly, the significant sight distance advantages of truck drivers (compared with automobile drivers) would greatly reduce the lengths of vertical curves needed for trucks if it were not for the longer stopping distances of trucks.

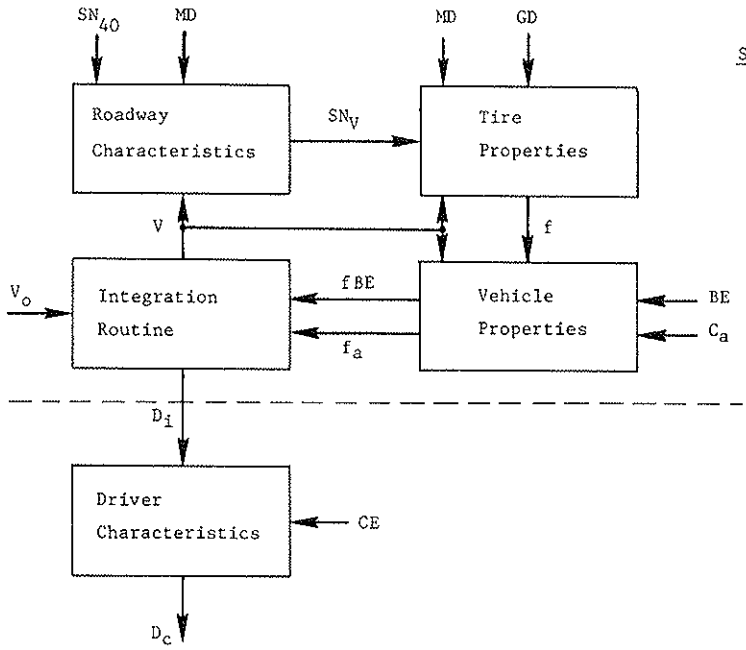
Stopping Distances for Trucks

In this section the significance of providing enough sight distance to allow trucks to make a controlled stop on a "poor, wet road" is addressed.

Stopping sight distance consists of (a) the distance traveled during the time required to perceive the object and to react by applying the brakes plus (b) the braking distance of the vehicle involved. Both the perception and reaction distance and the braking distance depend on the initial velocity of the vehicle. Perception and reaction distance is simply equal to the initial velocity multiplied by the perception and reaction time (i.e., 2.5 sec).

In addition to initial velocity, braking distance depends on the properties of the tire-road interface. Furthermore, for safe, controlled stops, braking distance depends on the braking efficiency of the vehicle and the control efficiency of the driver in modulating the brakes (2).

The following discussion outlines the elements of a procedure for predicting the braking distances of trucks operating on poor, wet roads (2,4). The items considered in this procedure are (a) roadway charac-



SYMBOLS

SN_{40}	-- pavement skid number at 40 mph
MD	-- mean texture depth
GD	-- tire groove depth
SN_V	-- skid number at velocity V
V	-- instantaneous velocity
V_o	-- initial velocity
f	-- tire road friction capability
BE	-- braking efficiency
CE	-- driver control efficiency
C_a	-- aerodynamic coefficients
f_a	-- aerodynamic drag divided by vehicle weight
D_i	-- ideal braking distance (perfect controller)
D_c	-- braking distance for a controlled stop

FIGURE 5 Diagram illustrating the calculation of braking distance.

teristics, (b) tire properties, (c) vehicle properties, and (d) driver control factors. The flow diagram shown in Figure 5 illustrates the sequence of calculations that are to be performed as speed decreases. Because the forces acting on the vehicle are functions of velocity, the equations of motion are solved using an integrative procedure [i.e., a numerical integration routine such as that given in Appendix B of Olson et al. (2)].

The roadway characteristics employed in the basic model are skid number and skid number gradient. The skid number at 40 mph and the skid number gradient are used in an exponential function to predict the skid number at the velocity of current interest in the iterative procedure; that is,

$$SN_V = SN_{40} \exp[P(V - 40)] \quad (12)$$

where

$$P = -0.0016(MD)^{-0.47} \quad (13)$$

V = velocity (mph), and
MD = mean texture depth in inches as determined by the sand-patch method (5).

For wet roads in the United States, the 15th percentile values (representing the poor, wet road) are given by the following equation (2,6):

$$SN_V = 28 \exp[-0.0115(V - 40)] \quad (14)$$

where $SN_{40} = 28$ and MD = 0.015 in. [Note that the poor, wet road used in the AASHTO design policy is indeed a slippery surface. A reasonable alternative to extreme changes in geometric design may be an improvement in pavement skid resistance (2).]

The equations given in Table 1 have been used for estimating the braking performance of a prototypical truck stopping on a poor, wet (15th percentile) road. The coefficients in these equations have been selected to represent (a) worn truck tires with 2/32 in. of groove depth, (b) the braking efficiency of an empty heavy vehicle with typical brake proportioning, and (c) the aerodynamic drag of a typical heavy truck. These selections correspond to a set of unfavorable conditions that reflects a conservative, safety-biased approach to design.

TABLE 1 Equations for Estimating Braking Distances

Equation	Explanation	Equation No.
$f_s = 0.0084\ SN_V$	f_s is the locked-wheel friction capability of a new truck tire	15
$f_s(2/32\text{ in.}) = f_s - (0.5918)\,\Delta f$	$\Delta f = -0.0762 + 0.008045V$ and V is the instantaneous forward velocity in mph	16 17[a]
$f_p = 1.45\ f_s$	f_p is the maximum friction capability for a braked but unlocked tire	18[b]
Braking efficiency = BE = $(0.47)/(0.75 + 0.23f_p)$	BE ≈ 0.55 to 0.59 for an empty truck. For locked-wheel calculations, BE is set equal to 1.0.	19
Control efficiency = CE = 0.62		
Aerodynamic drag = f_a = $0.00238\ A\ C_D\ V^2/W$	A = frontal area (100 ft²) CD = drag coefficient (0.8) W = weight (14,600 lb for an empty truck)	20

[a]Equations 16 and 17 are for estimating the influence of tread wear on friction (2,7).
[b]Equation 18 is applied to either new or worn tires.

Figure 6 shows the influences of velocity, tire wear, and sliding and rolling friction on the estimated frictional capabilities of truck tires. When these frictional capabilities are combined with braking efficiencies and aerodynamic drag factors, the deceleration capabilities at various velocities may be predicted. Deceleration capabilities for new and worn tires and for the vehicle making locked-wheel and perfectly modulated stops (CE = 1.0) are shown in Figure 7.

Locked-wheel values can be used (as they are in the AASHTO procedure) to calculate locked-wheel stopping distances, but these values are not deemed appropriate for predicting stopping distances that allow drivers to control trucks during stops from highway speeds on poor, wet roads. Truck drivers will modulate their brakes to eliminate wheel lock in order to maintain directional control (2, Appendix B). However, professional truck drivers are not

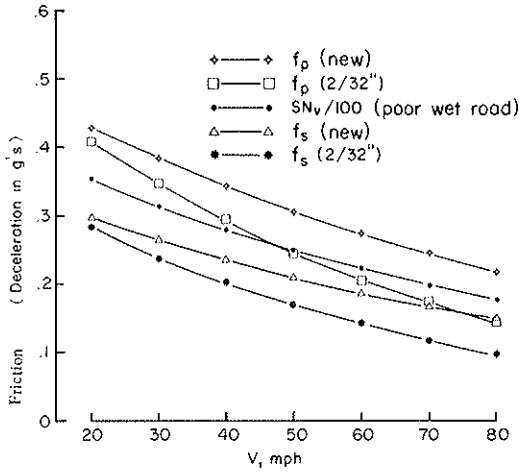


FIGURE 6 Friction capabilities of truck tires on poor, wet roads.

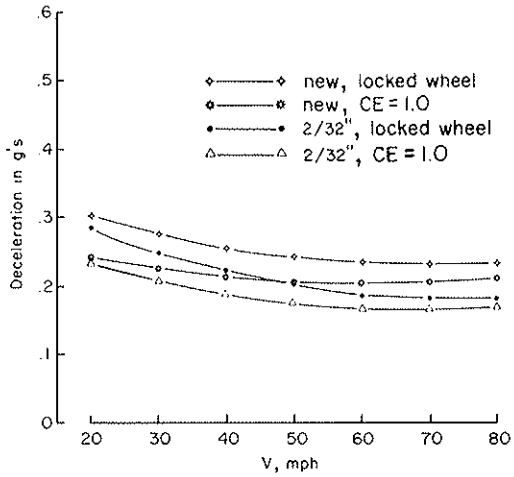


FIGURE 7 Truck deceleration on poor, wet roads.

able to perfectly modulate their brakes to obtain performance corresponding to the maximum capability of the road-tire-vehicle system. Experimental results have been used to estimate that truck drivers attain approximately 62 percent (CE = 0.62) of the performance capabilities of the road-tire-vehicle system (2).

The results of these considerations of truck performance show that trucks with worn tires will require stopping distances that are substantially longer than those recommended in the AASHTO policy. Furthermore, if spins, trailer swings, and jackknifing are to be avoided, controlled stops will require exceedingly long stopping distances at highway speeds (Figure 8).

The notion of attempting to design for trucks passing over crest vertical curves at 60 mph or faster may not be economically reasonable. At 60 mph the braking distances for controlled braking exceed the AASHTO policy for 80 mph (Figure 8). At 55 mph, controlled stops of trucks require braking distances that are approximately equal to the AASHTO policy for 80 mph (i.e., approximately 800 ft).

Consider the cost implications of restructuring a crest vertical curve to allow a braking distance of 800 ft for trucks instead of 340 ft for automobiles at 55 mph. Let the total difference in grade be 0.06 (6 percent) and the initial velocity be 55 mph. Un-

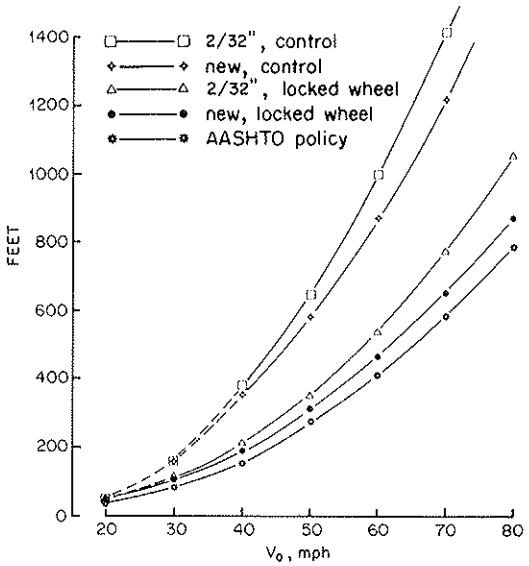


FIGURE 8 Truck braking distances on a poor, wet road.

der these circumstances, the controlled stopping sight distance (CSSD) for trucks is 1,002 ft and the AASHTO stopping sight distance for automobiles is 542 ft. From an earlier example, $L_t^* = 431$ ft and $L_c^* = 214$ ft and, applying Equation 10 with S_A = CSSD for the truck, it is found that $L_t = 2{,}329$ ft; for the automobile, $L_c = 1{,}373$ ft.

Another way to consider this situation is to evaluate the acceptable speed of trucks operating on crest vertical curves built for automobiles traveling at 80 mph. In this case (with a = 0.06 still), $L = 5{,}654$ ft $= S_t^2/431$; or $S_t = 1{,}561$ ft. Using the braking distance for a 2/32-in. controlled stop, as shown in Figure 8, it is found that, at 67 mph, braking distance equals 1,315 ft and the perception and reaction distance equals 246 ft. Hence, trucks traveling at 67 mph will be able to make controlled stops on the vertical curve designed for automobiles traveling at 80 mph. Carrying out similar calculations for curves designed for 70 mph and 60 mph yields the results given in Table 2. From this point of view, crest vertical curves designed according to AASHTO recommendations for 70 or 80 mph will be more than adequate for trucks traveling at speeds of less than 59 mph.

SUMMARY AND CONCLUSIONS

This short review of sight distance issues related to the characteristics of heavy trucks has presented technical arguments supporting the following positions:

- The AASHTO curve (1) displaying accelerating time as a function of distance traveled for the WB-50 design vehicle is applicable to current longer trucks as long as the additional length of the truck is included in the distance traveled.
- The initial acceleration distance employed in estimating passing sight distance does not apply to heavy trucks. This portion of the conceptual framework used for determining passing sight distance needs to be revised. Nevertheless, automobiles passing long trucks will spend more time in the left lane than is required for passing another automobile. If the average relative passing speed is 15 ft

TABLE 2 Truck Control Speeds

Car Speed (mph)	AASHTO SSD (ft)	L_c (ft)	S_t (ft)	Controlled Truck Speed (mph)
For a = 0.06 (6%), L_c^* = 214 ft, L_t^* = 431 ft				
60	650	1,974	922	52
70	850	3,376	1,206	59
80	1,100	5,654	1,561	67
For a = 0.12 (12%), L_c^* = 107 ft, L_t^* = 215 ft				
60	650	3,949	922	52
70	850	6,752	1,206	59
80	1,100	11,308	1,561	67

Note: L_c = length of vertical curve based on the AASHTO SSD (h_e = 40 in.) and S_t = available sight distance for a truck driver (h_e = 100 in.) operating on a vertical curve of length L_c.

per second, the additional time in the left lane can be readily estimated using the additional length of the larger vehicle.

• The stopping sight distances given in the AASHTO policy for crest vertical curves are much shorter than those needed for stopping trucks while maintaining directional control. The primary factors that contribute to the longer stopping distances estimated for heavy trucks are (a) truck tire properties on poor, wet roads; (b) braking efficiencies of heavy trucks; and (c) driver control efficiencies in modulating the brakes to avoid wheel lock. It is concluded that vertical curves designed for design speeds of more than 60 mph in accord with the AASHTO policy are adequate for trucks traveling at speeds of less than 52 mph. A vertical curve designed in accord with the AASHTO policy for a design speed of 70 mph is adequate for trucks traveling 59 mph.

Although stopping sight distances for horizontal curves were not considered in the body of this paper, the braking distance material presented here is applicable to that situation. For many horizontal curves, the additional eye height of the truck driver will not be an advantage. In those cases, the longer braking distances of trucks will greatly increase the width of the zone to be kept free of sight obstructions, if the heavy truck is used as the design vehicle. [See Appendix E of Olson et al. (2) for a calculation procedure for sight distances on horizontal curves.]

ACKNOWLEDGMENTS

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Operational and Safety Problems of Trucks in No-Passing Zones on Two-Lane Rural Highways

SNEHAMAY KHASNABIS

ABSTRACT

Two-lane rural roads in mountainous terrain with large truck volumes pose a real problem for the motorist. Extended roadway sections with severe sight restrictions and inadequate passing opportunities can make overtaking and passing a slow-moving vehicle extremely difficult. Further, the ability of large trucks to maintain speed decreases drastically over lengthy gradient sections. The inability of motorists to pass slower moving vehicles on these highways causes reductions in throughput and increases in delay, conflict, and hazard. Passing maneuvers on two-lane highways require a series of complex information-processing and decision skills, which makes these maneuvers one of the most demanding and risky operations performed by the motorist. Trucks require considerably longer distances than do automobiles to pass on two-lane rural roads. Even with their greater eye heights, truckers are placed at a disadvantage under the current system of marking no-passing barriers. The purpose of this paper is to discuss the interactive effects of geometric design elements and traffic composition (with particular emphasis on truck traffic) on traffic accidents and operational aspects on two-lane highways in mountains. Included in this analysis are passing-related accidents, human factors elements, and the impact of passing lanes and four-lane sections. Conclusions and recommendations, which draw on the findings of various research studies on the topics of truck traffic and no-passing zones, are also presented.

There are more than 3 million miles of two-lane rural highways in the United States that comprise about 97 percent of the total rural system and 80 percent of all U.S. roadways (1). These highways have about 3.4 million intersections and more than 100,000 railroad crossings. Further, more than two-thirds of the two-lane mileage is in mountainous or rolling terrain characterized by steep grades and sharp curves. Geometric design standards vary considerably within this rural system, and the use of traffic control devices is sparse. An estimated 68 percent of rural travel and 30 percent of all travel occur on the rural two-lane system. Many of these roadways experience significant increases in traffic on weekends and during peak vacation periods.

It has become increasingly evident that there are some serious safety and operational problems on rural two-lane highways resulting from their age, geometric standards, and traffic composition. About 80 to 90 percent of two-lane accidents occur in the rural environment and certain accident categories are prevalent among them, including passing maneuver, run-off-the-road, and railroad crossing accidents.

The recent growth in recreational vehicle and truck traffic on these roads has led to serious operational problems. The limited ability of these large vehicles to maintain speed on long grades causes following motorists to initiate passing maneuvers, often in the most hazardous situations. Trucks and recreational vehicles are also likely to encroach on the opposing lane because of their widths and dynamic characteristics. The terrain, pavement widths, and traffic characteristics of the rural two-lane system frequently limit passing opportunities and make this maneuver difficult and hazardous. These operational problems manifest themselves in reduced levels of service, delay, and increases in passing attempts, as well as in aborted passes and greater driver frustration.

Possible solutions to this problem include the addition of lanes, vehicle turnouts, and truck climbing lanes and the installation of signal controls; however, factors such as limited funds, the nature of the terrain, and potential environmental impacts often restrict the use of these solutions. Alternate passing zones on three-lane roads, or short four-lane sections with appropriate traffic control devices, may also represent feasible solutions. Unfortunately, little is known about the relative characteristics and effectiveness of these solutions.

SURFACE TRANSPORTATION ASSISTANCE ACT

The passage of the Surface Transportation Assistance Act (STAA) of 1982 makes it possible for wider, longer, and heavier trucks to operate on selected Interstate and other federally aided highway systems. The increased limits specified by this new act are as follows:

- Length: Truck unit 65 ft long, twin-trailer combinations with 28-ft-long trailers, semitrailer combinations with 48-ft-long trailers.
- Width: Maximum width of 102 in.
- Weight: Maximum single axle weight of 20,000 lb, maximum tandem axle weight of 34,000 lb, maximum gross weight of 80,000 lb.

Before 1982 the federal government, as well as individual states, placed size and weight restric-

Department of Civil Engineering, Wayne State University, Detroit, Mich. 48202.

tions on trucks operating on Interstate and state-maintained highways. A complete description of the pre-STAA limit ranges is presented in NCHRP Report 198 (2). The 1982 act has raised questions about possible hazards associated with the operation of larger, heavier trucks on roadways with inadequate geometrics. Concurrently, there is an increased need to identify roadways that can support these larger trucks. The 1982 act not only increased the allowable weights and size of vehicles but also designated a National Highway Network consisting of Interstate and other primary highways on which these longer and wider vehicles could operate. A set of criteria has been defined for designating such a national network. Among the criteria are included two factors that have raised some questions in the minds of traffic safety experts:

1. The route has adequate geometrics to support safe operations, considering sight distance, severity and length of grades, pavement width, horizontal curvature, shoulder width, bridge clearances and load limits, traffic volumes and vehicle mix, and intersection geometry.
2. The route consists of lanes designed to be a width of 12 ft or more.

The controversial part of the act is the lack of exact definition of geometric adequacy, and the question of inclusion of highways with 12-ft or wider lanes over much but not all of the length. The outcome of these unresolved issues and other clarification in the near future may dictate whether large trucks will operate on roadways with restrictive geometrics. Specifically, for a given stretch of highway, "What should be the maximum allowable percentage of no-passing barriers that can be considered safe for such heavy truck operation?" Similarly, "How should a two-lane section be regarded, if, over a given stretch, a majority (but not all) of the facility has 12-ft pavements?" The passage of the act has raised new questions about truck safety and operational problems, an area that has been of great concern to many highway engineers for a long time.

PASSING-RELATED ACCIDENTS

Overtaking and passing maneuvers require motorists to make complex decisions regarding roadway, environmental, and vehicular characteristics. The average motorist is considered a poor judge of speed and distance and has difficulty performing the four basic tasks required in a typical passing situation (3). It has also been estimated that passing-related accidents constitute about 3 to 4 percent of the total number of accidents reported in the United States (4). Furthermore, nationally, approximately 1,500 fatalities may be related annually to passing maneuvers. Moreover, the incidence of passing-related accidents is much greater on two-lane roads. For example a 1972 study conducted by Kemper et al. (5) found that approximately 20 to 23 percent of total reported accidents in Virginia were passing related. Also, between 40 and 50 percent of all passing-related accidents generally occurred at intersections and driveways (4,5). Further, many of these accidents at intersections and driveways were the result of a motorist attempting to pass another vehicle making a left turn at an intersection. It should be noted here that none of these studies mentioned any specific analysis with regard to truck traffic.

A recent FHWA-sponsored study conducted by Parker et al. (6) attempted to assess the nature and magnitude of passing-accident problems at rural intersections on two-lane highways. As a part of this study, the authors collected accident, traffic, and geometric data from 1,028 rural intersections in Michigan in an effort to identify and analyze specific passing-accident problems. On the basis of analysis of these data and a detailed review of the accident reports, roadway deficiencies and other causal factors were identified. The feasibility of using geometric design treatments to reduce the number and severity of intersection-related passing accidents was examined.

The major finding of the study is that a passing accident is a rare event at a rural intersection. Only 20 percent of the 1,028 intersections sampled experienced any passing accident during a 3-year period. Fewer than 8 percent of the intersection accidents were found to involve passing maneuvers. However, in the context of passing accidents only, those that occurred at intersections and driveways comprised a major proportion. Approximately 58 percent of the passing accidents involved intersections and driveways. Although this high percentage of passing-related intersection accidents might suggest a major safety problem, an analysis of the distribution of passing accidents by intersection revealed that fewer than 1 percent of the 1,028 intersections had an average of one or more passing accidents per year. A rural intersection with two or more passing accidents during the 3-year period was a rarity. Thus, to summarize this finding, passing-related accidents comprise a small fraction of all intersection accidents. However, of all passing-related accidents, those that occurred at or near an intersection comprise a major proportion. The findings of Parker et al. generally agree with those of an earlier study that also concluded that a high percentage of passing accidents occurred at intersections and driveways (1).

Another important finding of the study by Parker et al. is that the severity of injuries in passing-related intersection accidents is significantly less than that in other types of rural intersection accidents. This result is because a majority of passing-related accidents at intersections are the result of collisions of vehicles traveling in the same direction caused by a motorist attempting to pass another motorist making a left turn. The authors, however, note that "the results of this study should not be construed to imply that there are no safety problems at intersections." The authors conclude that some specific accident problems occur in sufficient numbers at specific sites to economically justify the implementation of geometric design and traffic engineering treatments.

An earlier FHWA study, conducted by the Texas Transportation Institute (7), used passing-accident data from three states, California, Kentucky, and Texas, for the purpose of developing improved criteria and guidelines for establishing no-passing zones. The findings of this study generally correspond with those of the study by Parker et al. in that a high percentage of passing accidents was found to occur at intersections and driveways; again, the severity of these accidents was much less than that of those at nonintersections. A second FHWA study conducted by the Texas Transport Institute used accident data from North Carolina, Texas, and Utah to identify passing-related problems (4). The study concluded that passing accidents are rare events for any special highway condition, including rural intersections. However, the study also reconfirmed the earlier finding that a high percentage of passing accidents occurs at intersections and driveways. In none of these studies was the phenomenon of truck accidents studied in any depth, nor was any

conclusion reached regarding any possible relationship among passing accidents and geometric and other roadway or traffic factors.

TRUCK ACCIDENTS

Numerous studies of truck accidents have been conducted during the last 10 years. Unfortunately, there does not appear to be a consensus among researchers as to whether large trucks have a higher or a lower accident rate compared with other vehicles. One of the earlier studies conducted by the author of this paper in 1977 used the 7-year (1970-1977) Michigan accident data base to assess the relative magnitude of truck accidents compared with those involving all other vehicles (8). The primary finding of the study was that the relative involvement of large trucks in fatal accidents was much greater than that of all other vehicles. The author, in a later study, used the concept of "opportunity for interaction" in estimating exposure and used the same Michigan accident data base to demonstrate the approach (9). This second study reconfirmed the earlier finding that large trucks are involved in a high percentage of fatal accidents.

A comprehensive study of large truck safety was sponsored by NHTSA and conducted by Wagner-McGee & Associates with the objective of synthesizing all significant information relative to large trucks and large-truck accident countermeasures (10). Nearly 200 references identified from previous studies were reviewed in this project. This study found that single-vehicle truck accidents account for 32 to 50 percent of fatal truck accidents. Run-off-the-road and overturning were the two most frequent dynamics for single-vehicle truck accidents. In multiple-vehicle accidents, trucks are more likely to be the striking vehicle, and angle accidents produce the most fatalities. Accidents involving trucks hauling hazardous cargo are infrequent.

Table 1 gives the results of three studies that have developed accident rates for trucks and for all other vehicles, including those developed by the author of this paper in 1977. The obvious disparity among results of these studies can be attributed to differences in data base and criteria for measuring exposure. In none of the studies was any attempt made to categorize accident data by type of facility. A later study by Vallette et al. (11) developed accident rates for large trucks and nontrucks for four types of roadways in California and Michigan. The data in Table 2, reproduced from the Vallette study, indicate that large-truck accident rates were lower than those for nontrucks in three of the four roadway types. It is clear from Tables 1 and 2 that the basic issue of whether trucks experience more or fewer accidents per unit exposure compared with all other vehicles has not been satisfactorily resolved. The Vallette study, however, had a few other important findings:

TABLE 1 Accident Rates for Trucks and Other Vehicles

Source	Accidents per Million Vehicle-Miles			
	Straight Trucks	Tractor-trailer Combinations	All Trucks	Other Vehicles
Six states, 1976-1977[a]			2.35	3.12
Michigan, 1977[b]	37.2	4.98	7.47[c]	5.42
21 toll expressways 1976-1978	1.95[d]	1.79		1.13

[a] From Vallette et al. (11).
[b] From Khasnabis and Atabak (8).
[c] Includes pickups, panel trucks, and vans.
[d] Identified as light trucks.

TABLE 2 Accident Rates of Total Traffic, Nontrucks, and Large Trucks by Roadway Type (11)

Vehicle Type	Accidents per Million Vehicle-Miles			
	Roadway Type			
	Rural Freeway	Rural Nonfreeway	Urban Freeway	Urban Nonfreeway
Total traffic	0.90	2.61	3.59	4.92
Nontrucks	0.87	2.69	3.65	5.07
Total large trucks	1.12	2.34	2.73	3.02

1. Empty trucks have substantially higher accident rates than do loaded vehicles.
2. Otherwise, within the range of vehicle sizes observed, there were no major differences in accident rates or severity in the heavier truck weights, lengths, and widths.

Since 1975 the National Highway Traffic Safety Administration has initiated a program of collecting comprehensive data on all accidents nationwide. This data base, commonly referred to as the Fatal Accident Reporting System (FARS), was used by O'Day et al. (12) in 1980 to analyze the first 5 years of accident experience of combination trucks (tractor-trailers). The O'Day study generally shows that a majority of truck accidents occur on U.S. or state routes and that freeways are safer than nonfreeways.

GEOMETRIC FEATURES

Perhaps more important than the type of road is the specific location on the various road types where truck accidents are prevalent. An effort to identify specific geometric features by the Wagner-McGee study led to the general conclusion that particularly hazardous locations for trucks are interchanges and intersections, with off-ramps being more hazardous than on-ramps. For example, a data base containing approximately 34,000 reports on truck accidents prepared in 1978 for the Bureau of Motor Carrier Safety (BMCS) by various motor carriers shows a 53:47 split between off-ramp and on-ramp accidents (13). When the BMCS accidents are divided between collision and noncollision, there are, however, more collision accidents at on-ramps (likely due to merging) and more noncollision accidents at off-ramps (likely due to overturning on sharp curves). The FARS data file provides even further evidence of the off-ramp hazard reported by O'Day et al. (12).

The study by O'Day et al., using the 5-year FARS data, also indicated that approximately 25 percent of fatal truck accidents are intersection related and 4.7 percent occur near a driveway. A similar finding is reported by Lohman et al. (14) from an analysis of truck accidents in North Carolina in 1973, which showed that nearly 33 percent of large-truck accidents occur at intersections and another 13.5 percent occur at driveways and alley intersections.

The question of truck operation and safety on steep grades and sharp curves has been a topic of research for many years. The scope of this paper does not allow any elaborate discussion of this topic, other than to mention that large trucks have special safety problems on vertical grades. On an

upgrade, they are likely to be struck by overtaking vehicles, and on a downgrade they may strike slower moving vehicles. O'Day et al. report that 30 percent of fatal truck accidents occur on grades, and that gradient sections generally experience higher levels of fatal truck accidents than do nongradient sections. Among recent studies that have addressed the question of truck operation on grades are the works of Glennon (15), Walton and Lee (16), Humphreys (17), and Polus et al. (18).

Sharp curves are considered hazardous for vehicles, particularly for trucks. Past studies by Vallette et al. and others, using the FARS data, generally attest to this hypothesis, although the analysis of O'Day et al. indicates that, compared with straight sections, curved sections showed a slightly lower accident rate. Obviously, more information is needed before generalized conclusions can be drawn.

Two recent research studies conducted by Chira-Chavala et al. (19,20) at the University of Michigan deal with the topic of truck accidents. In their 1984 study, the authors investigated the severity of accidents involving large trucks and combination vehicles using the 1980 BMCS data. This study reports that on undivided rural roads collisions involving trucks can be severe under all conditions, particularly at night. Truck-car collisions on divided rural roads were found to be less serious than those on undivided rural roads. The second study attempted to investigate the degree of association between truck accidents and other influencing factors. The analysis indicates that most doubles and singles showed higher accident involvement rates than did straight trucks. In both of these studies, the authors reiterated their concern about the safety of undivided rural highways.

PASSING BEHAVIOR

A recent study by Sequin et al. (21) attempted to assess the effects of truck width on passing behavior on a two-lane rural road. This study is considered particularly pertinent in view of the 1982 STAA. The authors used as an experimental vehicle a tractor-trailer combination that was systematically varied in width from 96 to 114 in. by 6-in. increments. A summary of the data compiled in this study, with the truck as the "passed vehicle," is given in Table 3. The authors inferred from the results that there are no significant differences in passing time, distance, and speed with varying truck width. The authors also analyzed speed data of oncoming vehicles. These results showed greater variation among "oncomers" than among "overtakers" as a function of truck width.

As a further test to assess any possible "intimidation effect" due to truck width, the authors analyzed the "accepted gap size"--the sum of decision time, passing time, and time margin (Table 4). Contrary to common expectation, total gap size and time margin measures were found to be significantly lower for trucks of greater widths. However, the extent of driver uncertainty, as reflected by the amount of decision time, was found to be independent of truck width.

An analysis of the prepass and minimum headways demonstrated that following vehicles maintained greater separation when encountering wider trucks. This was due to the need for greater sight distance or to the intimidation effect. Regardless of the cause, the authors found that truck followers are definitely sensitive to truck width but found no evidence of increased hazard resulting from wider trucks. The authors also concluded that truck width is an intimidating factor in lateral placement of vehicles during passing.

Sequin et al. also analyzed the effects of increasing truck size on the speed and lateral placement of oncoming vehicles at or near a narrow bridge site in Nevada. No significant differences were noted in the speed behavior of nontrucks when interacting with oncoming trucks of increasing length. Similarly, no significant differences in lateral placement were found to occur during approach, bridge, or exit interactions involving longer trucks. In spite of the general reduction in lateral distance from the road edge, increased truck width was not shown to be a source of increased hazard in this regard.

Sequin et al. also studied the impact of increased truck length on driver behavior. Unfortunately, none of these analyses were related to passing maneuvers; more specifically, these analyses included driver behavior in freeway entrance merges

TABLE 3 Summary of Passing Time, Distance, and Speed Statistics by Truck Width

	Truck Width (in.)											
	96			102			108			114		
	$\bar{X}$	σ	N	$\bar{X}$	σ	N	$\bar{X}$	σ	N	$\bar{X}$	σ	N
Passing time (sec)	10.3	2.4	81	10.3	2.5	85	11.0	2.8	84	10.7	2.7	98
Passing distance (ft)	786.1	184.5	81	786.7	185.9	86	843.1	200.0	84	814.0	164.7	97
Passing speed (ft/sec)	76.7	8.1	81	76.6	6.3	85	76.8	5.6	84	77.1	7.8	97

TABLE 4 Summary in Seconds of Decision Time, Time Margin, and Accepted Gap Size Statistics by Truck Width *(21)*

	Truck Width (in.)							
	96		102		108		114	
	$\bar{X}$	σ	$\bar{X}$	σ	$\bar{X}$	σ	$\bar{X}$	σ
Decision time	7.3	8.1	5.6	7.6	6.3	6.5	8.1	9.5
Passing time	10.3	2.4	10.3	2.5	11.0	2.8	10.7	2.6
Time margin	29.9	18.1	24.6[a]	16.7	24.9[a]	14.5	24.8[a]	15.0
Accepted gap size	47.4	20.5	40.4[a]	18.6	38.3[a]	17.9	43.6	20.2

[a]Significant at or beyond $p = 0.05$ compared with 96-in. value.

and interactions of oncoming vehicles at narrow bridge sites. Although the authors noted increased traffic turbulence associated with longer trucks (such as forced lane changes, gore encroachments, and sudden braking), there was no basis for the argument that increased track lengths are associated with increased safety hazards.

There is some controversy about the adequacy of the procedure for determining passing zones, particularly when trucks are involved. A recent study by the FHWA (22), using information compiled by the Swedish Road Research Institute, concluded that truck-automobile passing zones should be at least 1.5 times as long as those for one automobile passing another. This observation is based on the assumption that passing distance is proportional to passing time. The Swedish study also concluded that passing zone markings based on automobiles passing trucks should be 1.25 to 2.0 times longer than those needed for one automobile to pass another. In the event of a truck passing a truck, the passing zone should be even longer. The increased distance can partly be attributed to the fact that an automobile driver passing a truck starts further back than he does when passing another automobile and thus requires longer decision distances. It thus appears that passing zones designed for automobiles are not adequate for trucks. Although trucks have a 17 to 27 percent sight distance advantage over automobiles on crest vertical curves, this does not fully compensate for the 50 percent greater truck passing distances. A more recent study by Gericke and Walton (23) essentially confirms the Swedish study results. The authors contend that if current pavement marking practice [as described in the Manual on Uniform Traffic Control Devices (MUTCD)] is maintained, an adverse safety impact may be expected.

The use of passing lanes and short four-lane sections has been suggested as a means of alleviating safety and operational problems on two-lane highways. A passing lane is defined as an added third lane that is placed to provide passing opportunities on a two-lane highway. A four-lane section on a two-lane highway is generally less than 3 mi long and is provided for the specific purpose of providing passing opportunities in both directions at the same location. A recent study by Harwood et al. (24) attempted an operational and safety evaluation of passing lanes and short four-lane sections to improve traffic services on two-lane highways. The authors used 5-year accident data collected at selected sites in 12 states for 66 passing lanes and 10 short four-lane sections. Some of the important conclusions of this study are discussed next.

Passing lanes and short four-lane sections are likely to provide significant operational benefits on two-lane highways. Both types of treatments significantly increase the passing rate in the direction of travel compared with a conventional two-lane highway. The percentage of vehicles platooned is reduced by nearly one-half in a passing lane. The percentage of vehicles platooned immediately downstream of a passing lane is even less than the upstream value. Further, the operational benefit of passing lanes can persist for several miles downstream from the treated section. On the question of highway safety, the study found that the installation of a passing lane does not increase accident rates and, indeed, probably reduces them. No unusual safety problems were found to be associated with either lane-addition or lane-drop transition areas. The rate of accident involvement for vehicles traveling in opposite directions was found to be the same or lower on passing-lane sections than on untreated two-lane highways, even for passing lanes where passing by vehicles moving in opposite directions is permitted. There was also no indication of any major safety problem in the lane-addition or lane-drop transition areas of passing lanes. No safety problems associated with vehicles making left turns from the treated direction of a passing lane were found.

A substantially lower accident rate was found for short four-lane sections than for comparable untreated two-lane highways. The authors, however, were not able to conduct any statistical significance tests on the addition of four-lane sections.

SIMULATION MODELS

Since the mid-1960s, computer simulation has been used extensively as an analytical tool in the field of traffic and transportation engineering. During the period 1969-1972 a computer simulation model was developed at the Civil Engineering Department of North Carolina State University (referred to as the NCSU model) to determine the effect of systematic alteration of no-passing zones (NPZs) on throughput traffic. The model was calibrated with traffic flow data from rural highways in North Carolina and then applied on a specific field site to evaluate traffic flow consequences of systematic reductions of no-passing barriers. The model was developed as an outgrowth of its predecessor developed at the Franklin Institute of Research Laboratory.

The major findings of this study have been reported in the literature, but for the most part these are somewhat irrelevant to the topic of this paper (25-27). However, during the initial model development process a series of sensitivity analyses, using the original computer simulation model (FIRL model), was conducted with the specific objective of evaluating the traffic operational impact of percentage NPZ, truck percentage, and input volume on speed, delay, and passing-related output. The model used for the sensitivity analysis was not calibrated with field data; however, the trends in the output data, as a result of changing the three input variables, are worth noting.

The input to the model consisted of a hypothetical 30,000-ft-long roadway on which five levels of no-passing barriers (imposed by horizontal or vertical sight restrictions singly or in combination), along with vertical grades, had already been established. Two types of trucks were specified: Type 1, a single-unit vehicle and Type 2, a heavy tractor-trailer combination. The distribution assigned to these two types was 43:57 and was taken from the AASHO policy manual, which reported the results of a nationwide survey of truck travel on rural roads in 1963.

The results of the sensitivity analysis for an input volume level of 800 vehicles per hour (vph) are presented in figures (25). The important features of these figures are as follows:

- Figure 1: An increase in the percentage of trucks shows a consistent decrease in mean speed for the 50 and 70 percent no-passing zone configuration.
- Figure 2: Increases in truck percentages generally produce an increase in the number of attempted passes per hour per mile.
- Figure 3: An increase in the no-passing zone percentage from 20 to 50 percent, or from 25 to 70 percent, reduces the number of completed passes per hour per mile approximately two- to sixfold. An increase in the percentage of trucks is accompanied by a substantial increase in the number of completed passes per hour per mile.
- Figure 4: The number of vehicles passed per hour per mile increases beyond the 50 percent no-

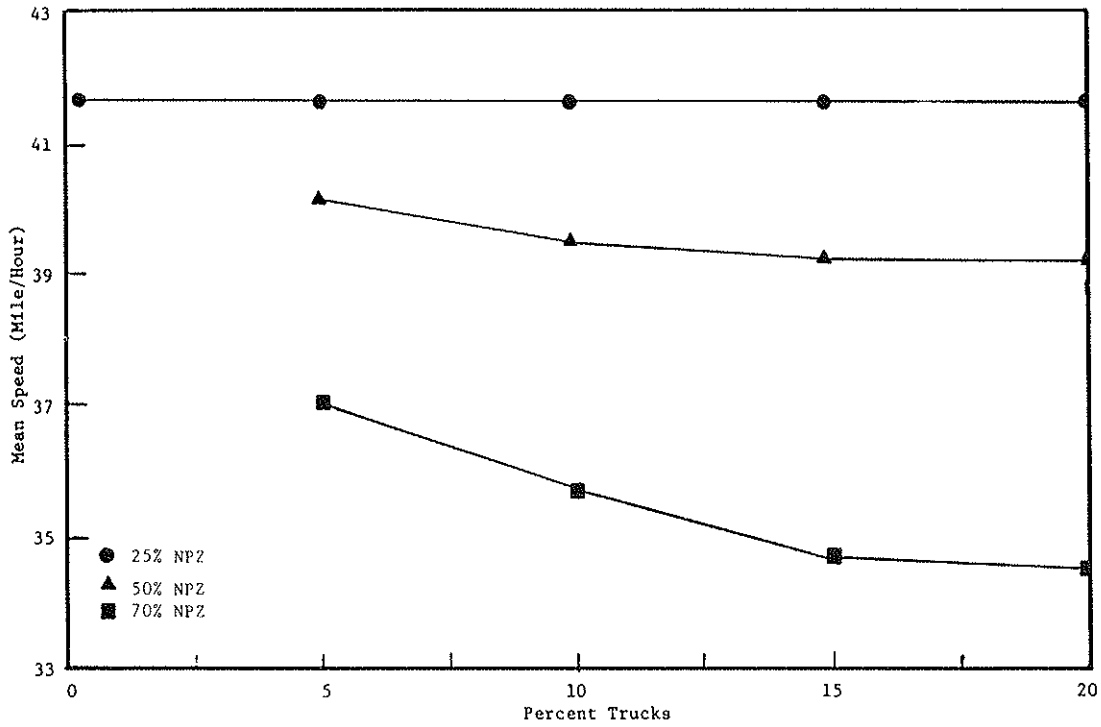

FIGURE 1 Mean speed for an input volume of 800 vph (two ways) versus percentage trucks *(25)*.

passing zone configuration when the percentage of trucks is 15 percent or greater. An increase in no-passing zones from 25 to 50 percent results in a decrease in the number of vehicles passed for all input volumes and for all truck percentages. For truck percentages greater than 10 percent, the 70 percent no-passing zone shows a greater number of vehicles passed than do the 25 or 50 percent zones.

• Figure 5: A reduction in the no-passing zone percentage causes a clear decrease in the delay per hour per mile for all truck percentages from 0 to 20 percent. The change in delay for increasing percentages of trucks is negligible for the 25 and 50 percent no-passing zone classifications. For 70 percent no-passing zones, there is a clear increase with increasing truck percentages.

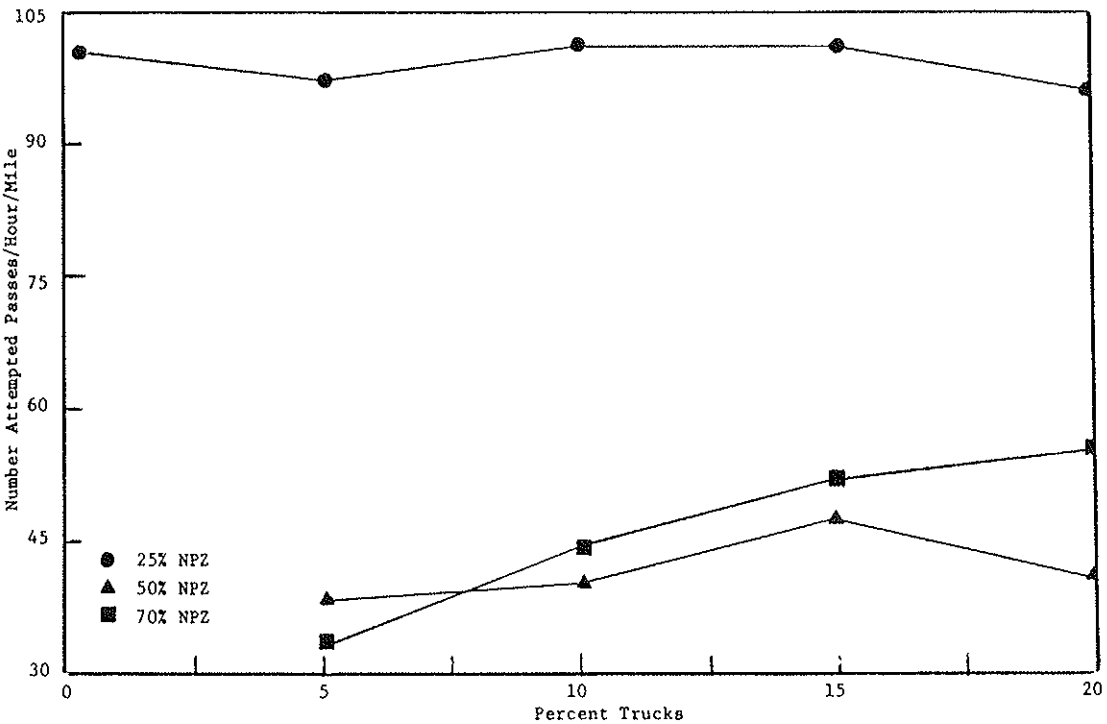

FIGURE 2 Number of attempted passes per hour per mile for an input volume of 800 vph (two ways) versus percentage trucks *(25)*.

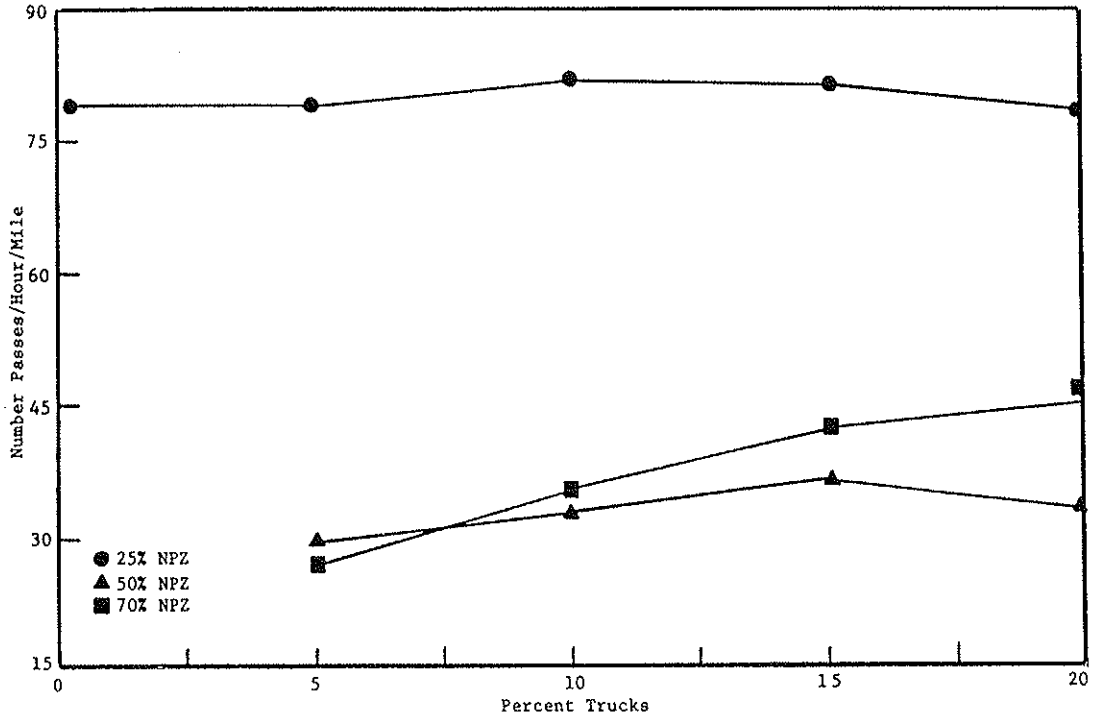

FIGURE 3 Number of passes per hour per mile for an input volume of 800 vph (two ways) versus percentage trucks *(25)*.

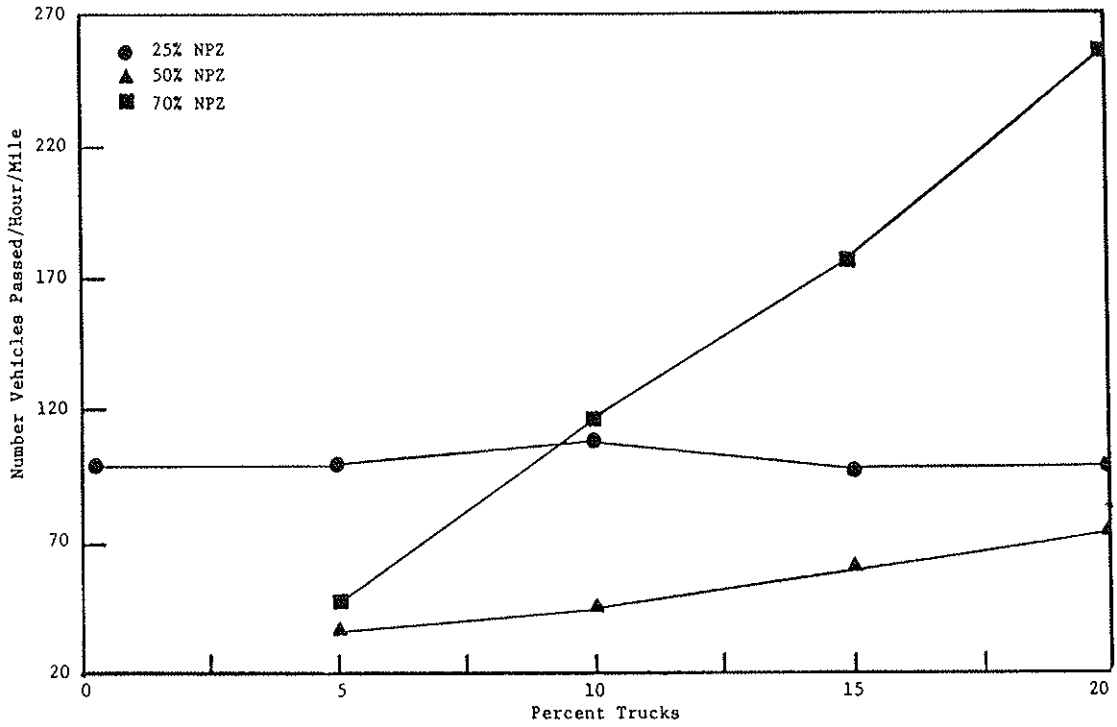

FIGURE 4 Number of vehicles passed per hour per mile for an input volume of 800 vph (two ways) versus percentage trucks *(25)*.

CONCLUSIONS AND RECOMMENDATIONS

The purpose of this paper is to review the basic issues pertaining to truck safety and operational problems in no-passing zones on two-lane rural highways. As a result of this review and limited discussions with a number of researchers, the following conclusions are drawn:

1. Passing-related accidents are generally prevalent among the accident categories within the two-lane rural system of U.S. highways; however, these accidents comprise a small fraction of all accidents in the country.

2. Passing-related accidents comprise a small fraction of all rural intersection accidents. However, of all passing-related accidents, those that

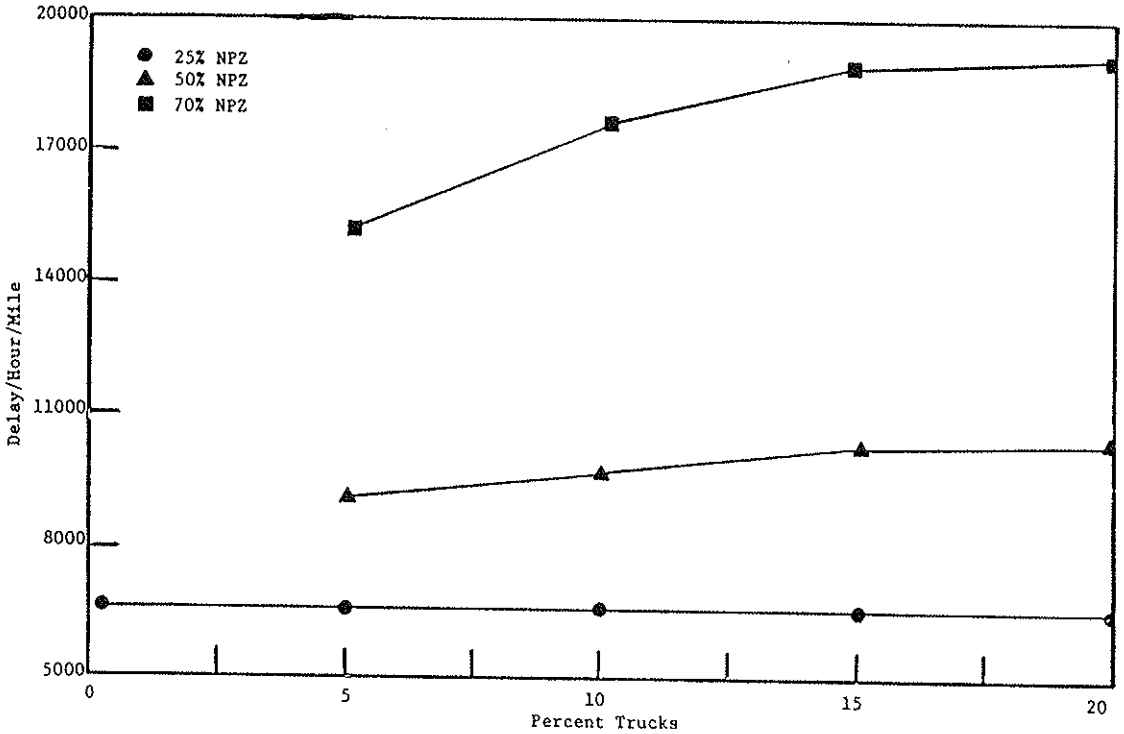

FIGURE 5 Delay per hour per mile for 800 vph (two ways) versus percentage trucks *(25)*.

occur at or near intersections comprise a major proportion.

3. There does not appear to be a consensus among researchers as to whether the accident involvement rate of large trucks is significantly higher or lower than that of all other vehicles. Most truck accident studies, however, appear to indicate that the involvement rate of large trucks in fatal accidents is much higher.

4. Among fatal truck accidents, single-vehicle accidents comprise a major category. These single-vehicle fatal accidents may be indicative of roadway and geometric deficiencies. There is, however, no conclusive evidence in the literature of such geometric deficiencies, other than some limited indication of hazards on off-ramps.

5. There is no information available in the literature on the incidence of truck accidents in no-passing zones. However, a number of recent studies have indicated that on undivided rural roads collisions involving trucks are severe under all conditions.

6. Truck size (length and width) appears to be an intimidating factor in lateral placement of vehicles during passing, as well as longitudinal separation (gap) from the following vehicle. Also, increased traffic turbulences are associated with longer trucks (as evidenced by forced lane changes, gore encroachments, etc.). However, there is no evidence of increased hazard resulting from wider trucks.

7. The current MUTCD practice of marking passing zones designed for automobiles may not be adequate for trucks. The increased eye height of truckers does not compensate for increased truck passing distance.

8. Limited evidence from the literature suggests that both passing lanes and short four-lane sections are likely to provide significant operational benefits on two-lane highways. These operational benefits appear to extend several miles downstream from the treated area. Further, there does not appear to be any indication of a safety problem in the lane-addition or lane-drop transition areas of passing lanes.

9. The use of simulation techniques appears to provide a means of assessing operational impact (on delay, speed, and passing maneuvers) of increased truck traffic as well as altered roadway geometrics (as reflected by various measures of no-passing zones). With the proper calibration of such simulation models it may be possible to quantify some of these operational effects.

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Use of the WHI Offtracking Formula

KENNETH L. HEALD

ABSTRACT

Offtracking is the phenomenon that occurs when the trailing axles of a turning vehicle increasingly migrate toward the curve center until they finally reach a maximum steady-state offset from the steering alignment path. Steady-state offtracking is achieved when the projected extensions of all fixed axles pass through the curve center. For turns of 120 degrees or less, maximum offtracking observed will seldom fully achieve that of the steady state; however, the clean geometric relationships that exist at the steady-state condition make it possible to readily quantify and use this worst-case performance as a basis of comparison for various vehicle configurations. The Western Highway Institute (WHI) offtracking formula provides a relatively straightforward method of closely approximating the steady-state expectations for any given vehicle or combination. However, the vehicle dimensions required and the implications of their use in the formula need to be fully understood to ensure that calculations are performed and interpreted correctly. The purpose of this paper is to establish the basis for a correct understanding of the data requirements and the use of the WHI offtracking formula.

The Surface Transportation Assistance Act (STAA) of 1982 provided badly needed new funding for U.S. highway facilities but is also a "mixed blessing" from several different points of view. Surely, no thinking person can deny that the 48-ft semitrailer, now mandated nationwide, has brought about a major upheaval in the arena of geometric design standards. Further, the double-trailer phenomenon, new to some sectors of the country, has given rise to a renewed interest in a reexamination of truck turning requirements.

Properly used, the WHI offtracking formula can provide considerable insight into the highly variable turning requirements associated with different vehicle configurations. The purpose of this paper is to establish the basis for a correct understanding of the data requirements and use of this relatively simple offtracking formula.

OFFTRACKING DEFINITIONS

Offtracking is most frequently recognized by its consequences, but the subject has a history of documented study going back at least 25 years. During the period of recorded study, several different definitions have been advanced, each typically reflecting the concerns of the research approach. A brief explanation of the basic research methodologies and perspectives will be presented to help develop a basis for understanding the concepts currently used to define and quantify offtracking. These methodologies include full-scale tests; scale-model tests; mathematical and graphic procedures; and, most recently, computer-model simulations.

Full-Scale Tests

The earliest offtracking research involved measurements using actual vehicles on test-track curves of known radius. One offtrack definition that evolved from this type of work dates back to 1966 and comes from Stevens, Tignor, and LoJacono (1), who indicated that

> Offtracking is the path of the outside of the outer tire on a rear or trailing axle that deviates inward toward the center of a turn from the circular path of the outside of the outer front tire, while the vehicle or trailer combination is making a turn.

The definition obviously comes from a practical highway engineering perspective and accounts for the entire minimum pavement width required. This perspective establishes the overall objective for the final offtrack measurement of interest, and the methodology ultimately provides the basis for the validation of the alternative estimating procedures.

Scale-Model Tests

Scale-model work proved much more expeditious than did dealing with actual vehicles, and these tools provided most of the source drawings from which existing turning templates were originally developed. The definitions of offtracking that evolved were much less explicit and are typified by this 1962 statement from a Society of Automotive Engineers (SAE) report (2):

> In general, offtracking is defined as the difference in the path of the first inside front wheel and of the last inside rear wheel as a vehicle negotiates a curve.

The tractrix integrator, Figure 1, is perhaps the most widely known and used of the template-drawing scale-model devices. It is distinctive in that the line-width relationships developed capture the relative distances desired, but the physical aspect of tire width must be considered an additive factor.

Western Highway Institute, 1200 Bayhill Drive, San Bruno, California 94066

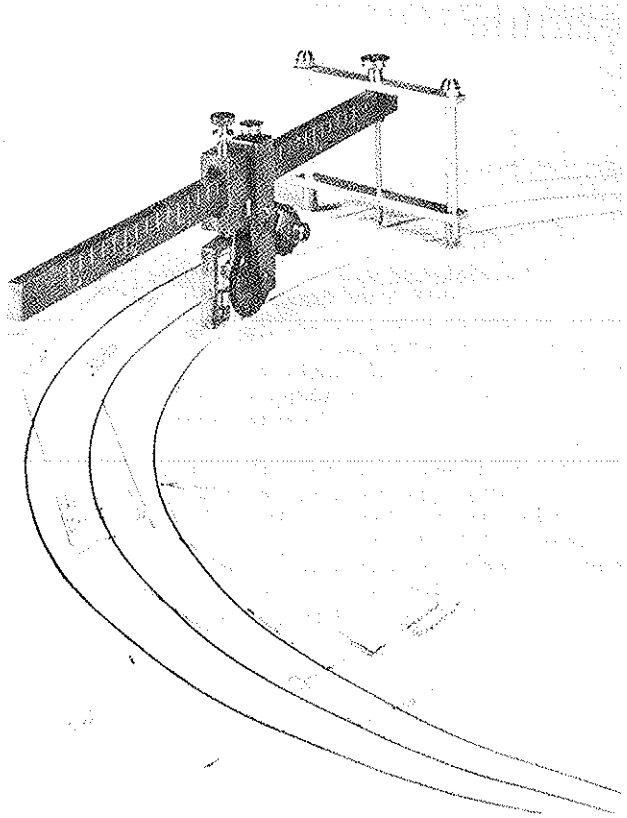


FIGURE 1 Tractrix integrator.

Mathematical and Graphic Techniques

The mathematical and graphic techniques depend quite explicitly on the geometric relationships demonstrated in offtracking. These properties are shown in Figure 2 and are pictured verbally in this definition, which appeared in the 1964 SAE Handbook (3,p.877).

> Offtracking is the difference in radii from the turning center to the vehicle centerline at the foremost and rearmost axles of a vehicle or combination and represents the increase beyond the tangent track occasioned by a turn.

Notice that, as illustrated, the vehicular centerline taken in combination with the adjacent radius lines from the turning center form a series of right triangles.

The oft-referenced SAE offtracking formula is based on the well-known Pythagorean theorem from geometry that the square of the hypotenuse of a right-angle triangle is equal to the sum of the squares of the other two sides. Although simple in theory, the vehicle-specific formulas proceed from a basis that is less than obvious for a two-axle vehicle and become increasingly more complex as additional axles are considered.

The SAE formula for offtracking of a single two-axle vehicle is

$$OT = \{WB^2 + \{[(TR^2 - WB^2)^{1/2}] - HT\}^2\}^{1/2} - [(TR^2 - WB^2)^{1/2}] + HT \quad (1)$$

where

OT = offtracking,
WB = wheelbase,
HT = front wheel track ÷ 2, and
TR = turning radius of outside front wheel.

The complexity of the SAE formula stems from the need to deal with turning centers located at the vehicular centerline rather than on the path of the turning radius. The annually published SAE handbooks carefully defined and explained this formula up to and through the 1972 issue. However, beginning with the 1973 volume (4,p.1209), SAE dropped much of the prior detail and revised the text to include this statement:

> In recent years, there have been developed data which are accurate enough to use for

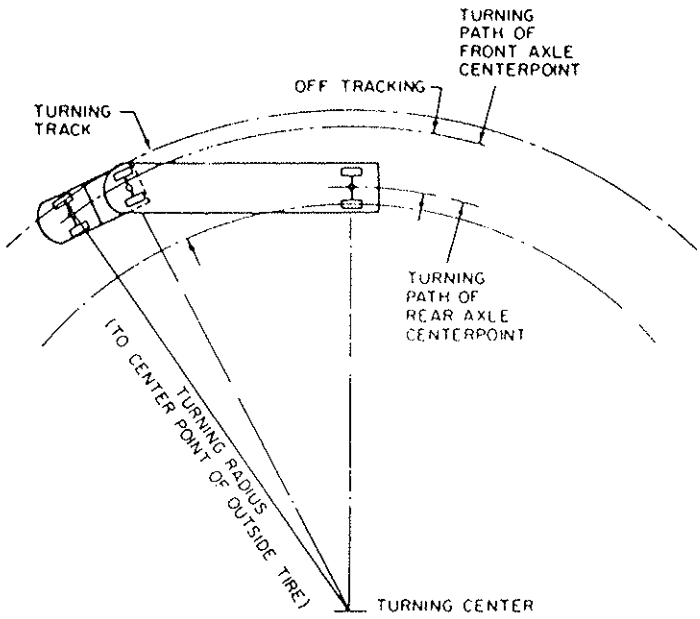


FIGURE 2 Typical offtracking geometrics.

> all practical purposes. The method was developed by the Western Highway Institute. . . . It is this method, easy to calculate and simple to apply, which is recommended as a general practice.

A detailed discussion of the WHI formula, its derivation, and its accuracy in comparison with results obtained via other methods is presented in WHI's Research Committee Report 3 (5).

The WHI equation for the calculation of maximum offtracking uses as one basis the summation of the squares of the components of the overall wheelbase. Thus the WHI concept is frequently referred to as the "sum of the squares" and, all too often, mistakenly as the "sum of the squares of wheelbases." This latter misconception can be and has been the source of some grave misjudgments concerning relative turning capability. Before further discussion of the WHI formula, one remaining offtrack methodology demands consideration.

Computer Modeling

Manual methods of computation previously constrained mathematical offtracking analyses to a comparison of the maximum values occurring at some variable but unspecified degree of turn. However, the mathematical theory and the computational muscle now exist to fully define the transient offtrack values for any turning condition. As might be expected, an expanded definition of offtracking results from this new capability and one such is as follows (6,p.4):

> Offtracking is the phenomenon which occurs when the trailing axles of a turning vehicle increasingly migrate toward the curve center until finally reaching a maximum steady state offset from the steering alignment path. The measured quantity of offtracking is defined as the radial distance separating the rear axle center path from the front axle center alignment path.

This definition recognizes that offtrack values increase along with the degree of turn--up to the point at which a steady-state condition finally develops. Offtrack at the steady-state condition is, obviously, that which the manual methods have typically termed "maximum."

Figure 3, taken from the report of Woodrooffe et al. (6), quoted previously, graphically portrays the offtrack cycle for one specific vehicle combination as it enters into and completes a 360-degree turn.

Notice that the resulting offtrack is apparent well before and ends well beyond the curvature points defined by the steering input. As indicated for this combination, offtrack increases rapidly through the first 60 degrees of turn and very quickly approaches the steady-state value even though the theoretical point of occurrence may be referenced to a significantly larger angle. Overall, this Canadian transient offtrack model appears to replicate observed movements reasonably well. It should be recognized that field tests and their associated measurements do tend to be somewhat erratic and may also display some sensitivity to deviations from the planned steering input.

The positional relationships, graphically shown in Figure 4, illustrate the points of the following technical definition of steady-state offtracking (6,p.4):

> [A] condition where a projected extension of all fixed axles of a vehicle pass through the curve center forming a right angle triangle with the vehicle where the hypotenuse is the alignment curve radius and the right angle is formed by the vehicle wheelbase and the radius of curvature at the trailing axle center.

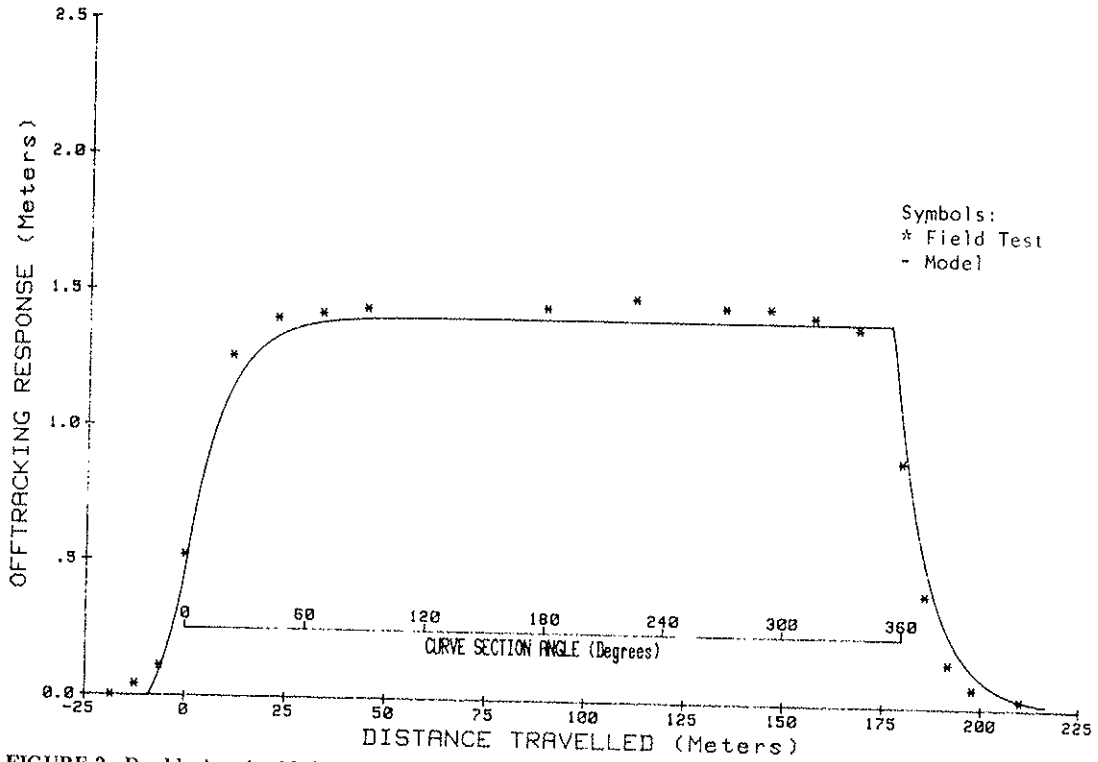


FIGURE 3 Double A-train, 28.6-m radius, 360-degree turn *(6)*.

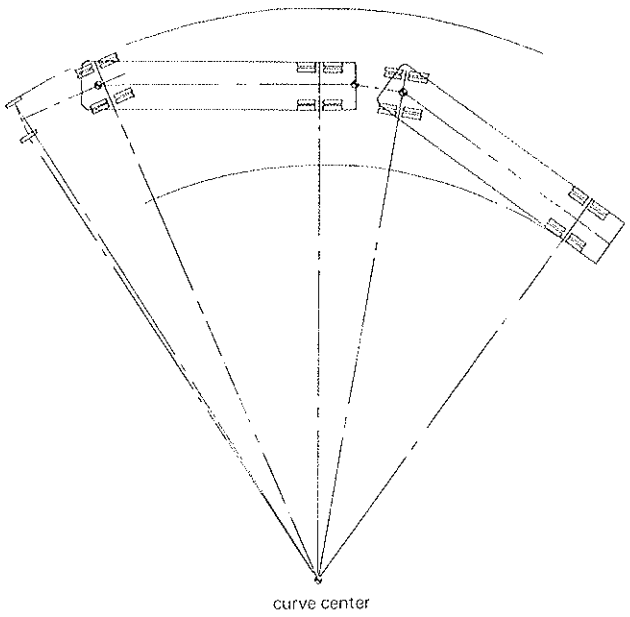


FIGURE 4 Graphic representation of steady-state offtracking.

It can be seen that when the steady-state condition has been reached the SAE and WHI formulas are both directly applicable. However, if the degree of turn is less than that required to reach steady state, the maximum-point formulas will always overstate the expected response. It should also be noted that the maximum-point formulas may break down for long vehicles on short-radius curves. This will occur when the curve center falls between the path of the rearmost axle and the curve itself.

CONSIDERATIONS FOR USE OF FORMULAS

The general form of the WHI offtracking equation is

$$OT_{max/ss} = R - (R^2 - \Sigma L^2)^{1/2} \quad (2)$$

where

R = radius of the curve followed by the front axle center and
L = individual component distances between points effecting or directly affecting turnability.

The offtrack value to be calculated is in actuality an estimate of the maximum or steady-state value. As computed, the value is centerline related but can readily be correlated with any other comparison points given the correct add-on constants. Figure 5 shows several of the significant offtracking components and terms.

Turning Track

Although comparative offtrack values for different vehicle configurations are the primary products visualized from WHI formula use, turning track comparisons are of interest as well. This dimension may also be referred to as swept width or track width; however, the latter term should be avoided if at all possible because it has another quite different connotation as a vehicle dimension. Turning track, or swept width, can be computed as the sum of offtrack and effective width where effective width includes an overhang component. AASHTO policy suggests the use of 8 ft 6 in. as the appropriate add-on factor for effective width.

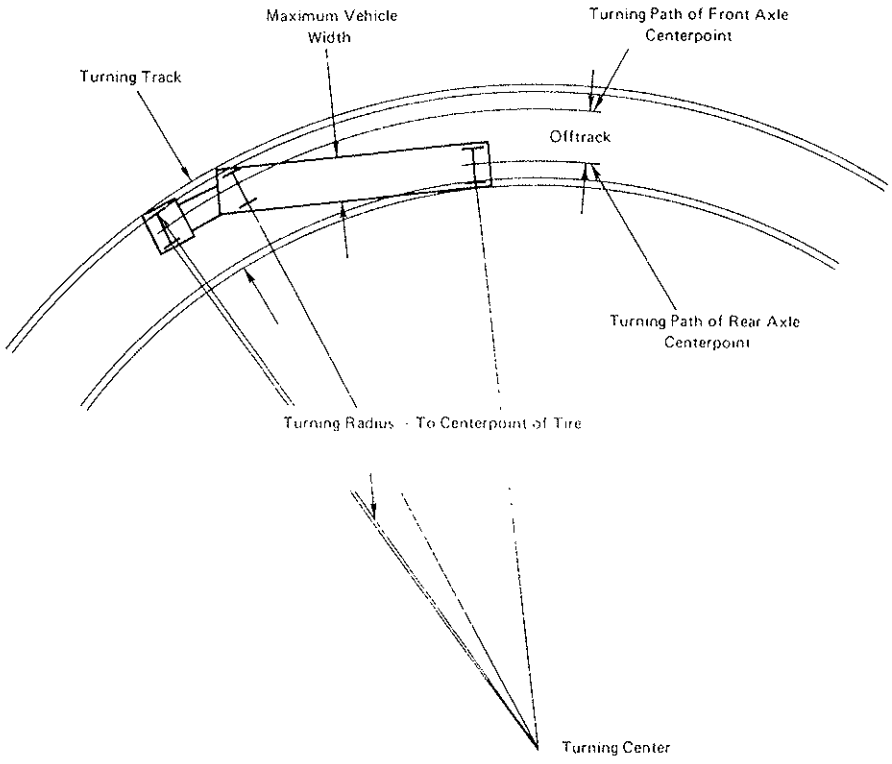


FIGURE 5 Schematic of turning track components and terms.

Turning Radius

In offtrack computation, the first radius of concern is the turning radius. As used, turning radius (TR) is taken to be the alignment path of the outer front tire at its centerpoint. For computational purposes, however, the formulas relate to the geometry found at the centers of axles. The computational centerline radius (R) is therefore that of the centerline of the outer front tire (TR) less one-half of the front axle track (i.e., half-track or HT). Restated in algebraic form:

$$R = TR - HT \quad (3)$$

Front Axle Track

A common mistake is to assume that the half-track value is one-half the maximum vehicle width. A simple function of vehicle width obviously results in overstating the effective turning radius. However, the point is that front axles on heavy-duty trucks are typically narrower than are all other axles in the unit.

The information shown in Figure 6 comes directly from a major manufacturer's data book and clearly indicates that the front axle can reasonably be assumed to be approximately 80 in. This being the case, half-track as used for computational purposes should be taken as 40 in. or 3.33 ft.

Elements of Vehicle Length

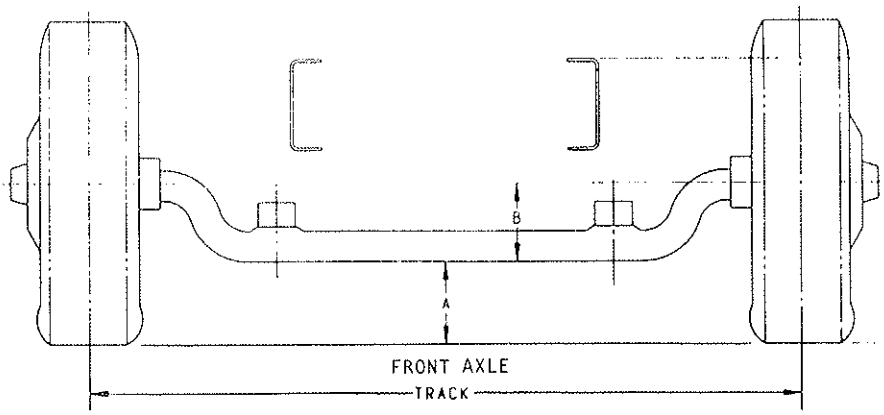
It is worthy of note that, from an offtracking standpoint, multiple axles of an axle group operating together within a single suspension system are treated as though they were a single axle located at the geometric center of the group. As shown in Figure 7, this is consistent with the general definition of wheelbase. Other typical dimensions in Figure 7 are suggested to serve as a test of reasonableness for the designation of representative length dimensions.

Overall length of truck combinations is still an active controlling factor even though trailer-only limits are now the rule on the Interstates and other designated highways. These length limits, coupled with various bridge table requirements, have acted to largely predetermine many truck length characteristics. Although this subject is admittedly a separate, unrelated issue, it is mentioned briefly here to encourage some consistency in the selection of dimensions for representative "analytical" models of various vehicle combinations.

Understanding the sum of L^2s (ΣL^2) portion of the WHI formula is undoubtedly the key to ensuring proper use. The Ls are defined as the distances between points involved in or directly affecting turnability. Figure 8, an axles-only schematic of a tractor-semitrailer-full trailer (double-trailer unit), will be used as the basis for the explanation of Ls.

Wheelbase dimensions are obviously significant

TRACK CHART				TRACK
FT6	20-7.5	DISC	5.40	81.29
FT6	20-7.5	CAST	4.75	80.57
FS6	20-7.5	DISC	5.40	81.29
FS6	20-7.5	CAST	4.75	80.57
FXG	20-7.0	DISC	6.06	82.13
FXG	20-7.0	CAST	4.75	79.40
F54	20-7.5	DISC	5.40	80.46
F54	20-7.5	CAST	4.25	80.12
FT7	20-7.5	DISC	5.40	80.46
FT7	20-7.5	CAST	4.25	80.12
FS7	20-7.5	DISC	5.40	80.46
FS7	20-7.5	CAST	4.25	80.12
FT4	20-7.5	DISC	5.40	80.46
FT4	20-7.5	CAST	4.25	80.12
FS4	20-7.5	DISC	5.40	80.46
FS4	20-7.5	CAST	4.25	80.12
F43	20-7.5	DISC	6.06	77.58
F43	20-7.5	CAST	4.75	76.98
FT2	20-7.5	DISC	6.00	78.18
FT2	20-7.5	CAST	4.00	77.86
FS2	20-7.5	DISC	6.00	78.18
FS2	20-7.5	CAST	4.00	77.86
F48	20-7.5	DISC	6.00	78.18
F48	20-7.5	CAST	4.00	77.86
FRW	20-7.5	DISC	6.00	78.18
FRW	20-7.5	CAST	4.00	77.86
FRD	20-7.5	DISC	6.00	78.18
FRD	20-7.5	CAST	4.00	77.86
FVM	20-7.5	DISC	6.00	69.00
FVM	20-7.5	CAST	4.00	76.42
AXLE RPO	WHEEL SIZE	TYPE	WHEEL OFFSET	TRACK



AXLE RPO

FVM	F050	FRONT AXLE	5000 POUND CAPACITY	HYDRAULIC
FS2	F070	FRONT AXLE	7000 POUND CAPACITY	AIR TRUCK
FT2	F070	FRONT AXLE	7000 POUND CAPACITY	AIR TRACTOR
F48	F070	FRONT AXLE	7000 POUND CAPACITY	HYDRAULIC
FRD	F070	FRONT AXLE	7500 POUND CAPACITY	AIR TRUCK
FRW	F070	FRONT AXLE	7500 POUND CAPACITY	HYDRAULIC
FS4	F120	FRONT AXLE	9000 POUND CAPACITY	AIR TRUCK
FT4	F120	FRONT AXLE	9000 POUND CAPACITY	AIR TRACTOR
F43	F120	FRONT AXLE	9000 POUND CAPACITY	HYDRAULIC
FS7	F120	FRONT AXLE	12000 POUND CAPACITY	AIR TRUCK HD
FT7	F120	FRONT AXLE	12000 POUND CAPACITY	AIR TRACTOR HD
FS4	F120	FRONT AXLE	12000 POUND CAPACITY	HYDRAULIC
FS6	FL931	FRONT AXLE	18000 POUND CAPACITY	AIR TRUCK
FT6	FL931	FRONT AXLE	18000 POUND CAPACITY	AIR TRACTOR

Front Axle Track ~ 80"
Other Comparables (℄ Outside Duals)
Tractor ~ 84½"
Trailer (Std.) ~ 84½"
Trailer (Wide) ~ 90½"

FIGURE 6 Front axle dimensions.

General:

Wheelbase - The distance between the ℄ (centerline) of the lead axle and the ℄ of the trailing axle or axle group for any given vehicle unit: i.e., to ℄ of tandems

Tractors:

Bumper to ℄ front axle - 28"-30" typical for long haul

Wheelbase - highly variable

- cab-over-engine (COE) without sleeper (4x2) - 116" typical
- conv. with sleeper (6x4) - 212" typical

5th wheel offset - adjustable typical

- forward for load transfer to front axle
- back - nonsensical
- zero offset yields greatest offtrack

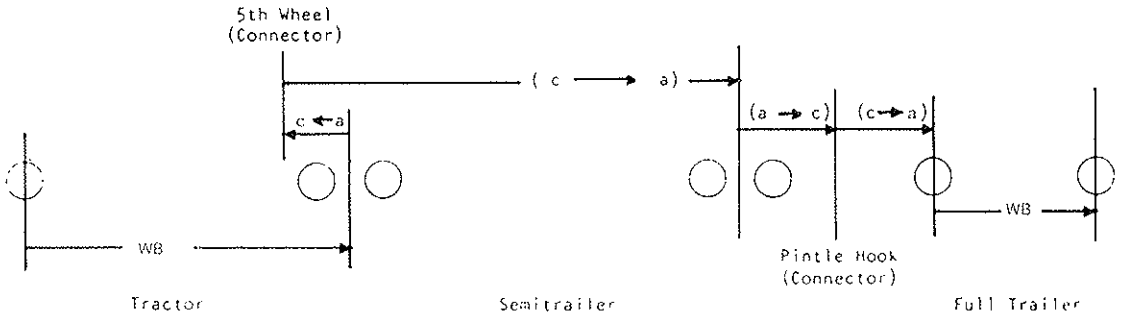
Trailers

Kingpin setting fixed - 28"-48", 36" typical

Rear overhang - from ℄ of rear axle/axle group

- short doubles units: 3-4 ft. (fixed)
- longer conv. units: 4-12 ft. (fixed/slider)

FIGURE 7 Elements of vehicle length.



General Form Formula:

$$OT_{max} = R - \sqrt{R^2 - \Sigma L^2}$$

where

L's may be = WBs (wheelbases)
= ac's (axle→ connectors)
= ca's (connector→ axle)

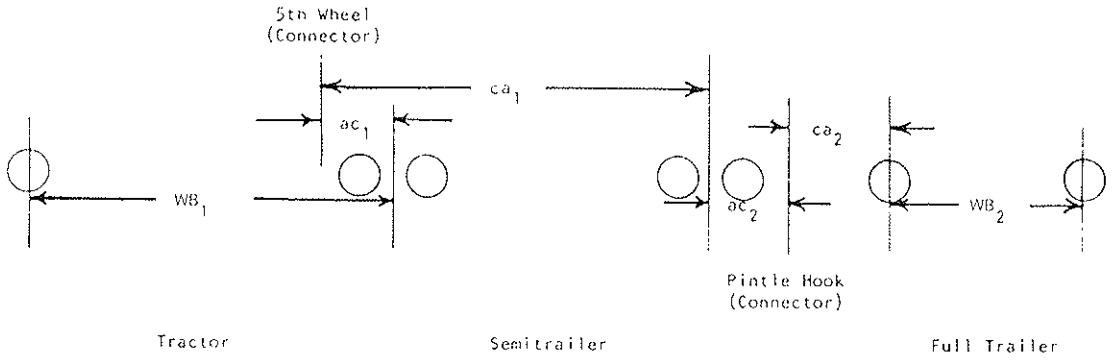
FIGURE 8 Axles and connectors concept.

but are definitely not all used in the calculations. The Ls actually represents the trace or single path from front to rear that connects adjacent axles or axle-group centerlines with each other or with intervening points of articulation (connectors). As indicated, the relevant axle-related dimensions are wheelbases (WB) and the unit connectors are typically fifth wheels or pintle hooks, or both.

Traces from axles (a) to connectors (c) are referenced as ac's and will always be found to produce contra-offtrack behavior (negative offtracking). The ac's when squared are always treated as negative values in the L^2s summation. (The negative offtrack phenomenon has long been recognized and is used to advantage in some types of combinations; however, personal verification is left to intuition or other sources.)

Traces from connectors (c) to axles (a) complete the normal list of options for L measurements. Referred to as ca's, these traces react in the same manner as a wheelbase in both the calculations and the turning process. It perhaps goes without saying that in the Ls summation process the number of ac's must equal the number of ca's.

Summarizing then, the Ls may be wheelbases (WBs), axle-to-connectors (ac's) or connector-to-axles



$$OT_{max} = R - \sqrt{R^2 - \Sigma L^2}$$

$$\Sigma L^2 = WB_1^2 - ac_1^2 + ca_1^2 - ac_2^2 + ca_2^2 + WB_2^2$$

FIGURE 9 General case illustration.

(ca's). Further, ac's will always generate negative L^2s. Referring to Figure 9, the ΣL^2 is shown written in algebraic form for the illustrative, general case doubles unit. Following the L trace from front to rear with all ac's negative after squaring, the ΣL^2 statement is as follows:

$$\Sigma L^2 = WB_1^2 - ac_1^2 + ca_1^2 - ac_2^2 + ca_2^2 + WB_2^2 \quad (4)$$

where

WB_1 = tractor wheelbase (as properly defined),
ac_1 = fifth wheel offset (irrespective of forward or back),
ca_1 = kingpin to rear tandem centerline,
ac_2 = centerline of rear tandem to pintle hook,
ca_2 = pintle hook to centerline of dolly axle (trailer 2), and
WB_2 = trailer 2 wheelbase (dolly fifth wheel offset = 0).

Note again that the number of ac's equals the number of ca's, even though an ac value, for example, may actually be zero. It will also be observed that the second trailer could also have a small fifth wheel offset, in which case an ac_3 and a ca_3 would replace the WB_2 shown.

For the fifth wheel offset, small values, say 1 ft or less, have virtually no impact as an ac because values less than 1, when squared, become even smaller. However, the ca difference compared with the corresponding wheelbase can and does become significant. The rule is: when a fifth wheel offset is the desired assumption, always use the ac and ca terms rather than the corresponding wheelbase.

SAMPLE CALCULATIONS

The first step in any offtrack calculation is to sort out and harmonize the various length components such that (a) all of the Ls are accounted for and realistic and (b) the sum of the parts equals the overall. This is not always as easy as might be imagined; and some visual aid, such as an axles-only schematic, will prove invaluable. It is recommended that this step precede any calculation effort--even if done on a computer.

Selected Combination

The Ls concepts were worked through with a somewhat unusual doubles configuration, a Rocky-Mountain double; this unit will also be used to illustrate the calculation process. Figure 10 is a fully dimensioned drawing of one such combination with the Ls components identified hereafter.

These particular units were assembled and used in the 1984 Multistate Highway Transportation Agreement (MHTA) over-the-road demonstration tests. The dimensions are exaggerated somewhat in comparison with normal practice. The purpose of the test was to gather real-world data for input to the FHWA Longer Combination Network Study, and FHWA insisted that the STAA dimensional maximums be used as the basis for the study.

L^2 Calculation

When the L components have been identified, the sum of L^2s computation is straightforward. Referring to Figure 10, the math, including the conversion of all measurements to the common units base, is as follows:

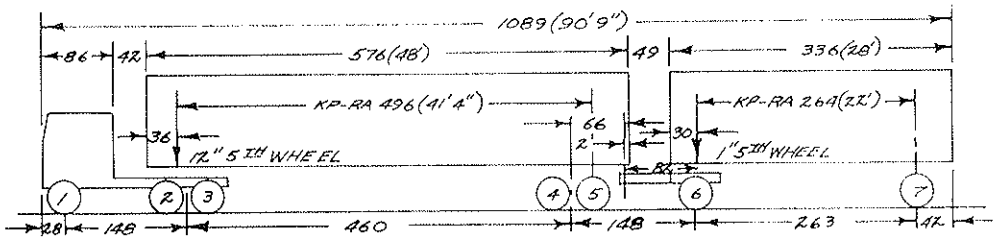
Component (in.)	L(ft)	L^2
WB_1 = 148	12.333	152.10
(-) ac_1 = 12	1.000	(1.00)
ca_1 = 472	39.333	1,547.08
(-) ac_2 = 66	5.500	(30.25)
ca_2 = 82	6.833	46.69
(-) ac_3 = 01	.083	Ignore
ca_3 = 264	22.000	484.00

$\Sigma L^2 = 2{,}198.62$

The selection of feet as the basis for computation is primarily a matter of number size convenience; however, because offtrack comparisons are often a matter of inches it is recommended that calculation accuracy be maintained to at least tenths of a foot and preferably to hundredths, as shown. Notice that the ΣL^2 is a characteristic of the vehicle itself and is totally independent of any turning radius considerations.

Selection of Intersection Radii

The turn radius of interest may be a special case (a given) or alternatively may be selected to represent



"Ls" Components

$$\Sigma L^2 = WB_1^2 - ac_1^2 + ca_1^2 - ac_2^2 + ca_2^2 - ac_3^2 + ca_3^2$$

where:

WB_1 = 148" ac_2 = 66"

ac_1 = 12" ca_2 = 82"

ca_1 = 496" - 24" ac_3 = 1"

= 472" ca_3 = 264"

FIGURE 10 Sample calculations, Rocky-Mountain double.

some particular class of facility. WHI analysis indicates that the following turning radii assumptions are reasonable and generally representative:

- Principal city streets = 60 ft,
- Rural state highways = 100 ft, and
- Freeways (cloverleaf) = 165 ft.

To complete the sample calculation process, two of these radii will be used to estimate the offtrack performance expected for the selected combination.

60-Ft Turning Radius Calculation

Given

TR = 60 ft (principal city street),
HT = 3.33 ft (1/2 front axle),
R = TR - HT
= 56.67 ft,
R^2 = 3,211.49, and
ΣL^2 = 2,198.62 (constant calculated earlier),

then

$$MOT_{60} = R - [(R^2 - \Sigma L^2)^{1/2}]$$
$$= 56.67 - [(3,211.49 - 2,198.62)^{1/2}]$$
$$= 56.67 - 31.83$$
$$= 24.84 \text{ ft.}$$

100-Ft Turning Radius Calculation

Given

TR = 100 ft (rural state highway),
HT = 3.33 ft (1/2 front axle),
R = TR - HT
= 96.67 ft,
R^2 = 9,345.09, and
ΣL^2 = 2,198.62 (same constant),

then

$$MOT_{100} = R - [(R^2 - \Sigma L^2)^{1/2}]$$
$$= 96.67 - [(9,345.09 - 2,198.62)^{1/2}]$$
$$= 96.67 - 84.54$$
$$= 12.13 \text{ ft.}$$

The simplicity of the final calculation points out once again the significance of the ΣL^2 as the key to the proper understanding and use of the WHI formula.

Maximum Turning Track Calculation

Consideration of the practical highway engineer's definition of offtracking leads again to the determination of minimum pavement requirements (i.e., turning track or swept width). AASHTO policy guidance makes this a simple task and suggests that a constant of 8.5 ft be added to the indicated offtrack value. If

U = turning track and
$$U_{tr} = MOT_{tr} + 8.50 \quad (5)$$

then for the selected combination,

U_{60} = 24.84 + 8.50 = 33.34 ft
U_{100} = 12.13 + 8.50 = 20.63 ft

The AASHTO constant, on closer examination, appears to have been somewhat overgenerous as originally specified for use with 96-in.-wide units. As a result, there appears now to be no strong argument for using a larger value in conjunction with 102-in.-width turning performance.

OVERVIEW OF WHI FORMULA

In introducing the subject, various definitions of offtracking were given to illustrate the unique characteristics of the WHI formula. It was pointed out that the formula yields a maximum or steady-state value related to the centerline of the vehicle. An understanding of front axle track width and its relationship to turning radius was emphasized, and the real world of truck length components was discussed with respect to the identification and handling of the Ls in the ΣL^2 determination.

The sample calculations used an uncommonly long vehicle configuration and short-radius curves to illustrate

- The concepts of axle-connector (negative offtrack term) and connector-axle distances as they interrelate with overall wheelbase to determine the vehicle-dependent value ΣL^2,
- The correct half-track adjustment of the turning radius to determine the vehicle centerline radius (R), and
- The relative simplicity of the calculations when the formula components have been properly defined.

As a general overview of the formula application, it can be said that the WHI offtrack formula is an accurate and expeditious tool for comparing worst-case vehicle turning performance. Worst case is emphasized because the steady-state values as computed virtually always exceed those for a 90-degree turn. Be aware, however, that the formula may break down for long units on short-radius curves.

Not even mentioned was that the mathematical formulation of the Canadian transient offtrack model now offers the capability to compute maximum offtrack for any given degree of turn. The formulas available in the report by Woodrooffe et al. (6) can be used to adjust the steady-state value when it has been determined. That discussion, however, is a follow-on subject and will not be attempted as part of this presentation.

Offtracking calculations and their interpretations are indeed skills that are "honed" only with frequent use. Further, when improperly used any special-purpose tool will fail to do the job for which it was designed. The purpose of this presentation has been to outline the concepts and procedures required to correctly use the WHI offtracking formula.

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Vehicle Offtracking Models

MICHAEL W. SAYERS

ABSTRACT

When a vehicle turns, the rear wheels track inside the path traced by the front wheels. This behavior is called offtracking and can lead to problems when large trucks operate in confined areas. The methods that have been used by designers to estimate the offtracking of heavy trucks are reviewed, and then a computer method for graphing the complete swept path of an arbitrary vehicle making any type of turn at low speed is described. The method is valid for nearly all truck configurations in use on the highways, including double and triple combinations. The paper includes several example plots, and a computer program that uses this method, developed for the Apple II computer, is described. The program is available free from the Federal Highway Administration.

Motor vehicles typically employ a single steered front axle followed by one or more unsteered rear axles. In low-speed turns, the rear wheels track inside the paths taken by the front wheels, such that the path swept by the vehicle is wider than the vehicle itself. Figure 1 shows how this behavior results in an additional swept width, called offtracking, for the vehicle. Offtracking can pose problems whenever there is not enough space to accommodate both the width of the vehicle and the additional offtracking displacement. Thus engineers laying out geometric designs for intersections, parking areas, and other locations with restricted geometry need to address the potential offtracking requirements of the largest vehicles that will be using the area.

University of Michigan Transportation Research Institute, University of Michigan, Ann Arbor, Mich., 48109.

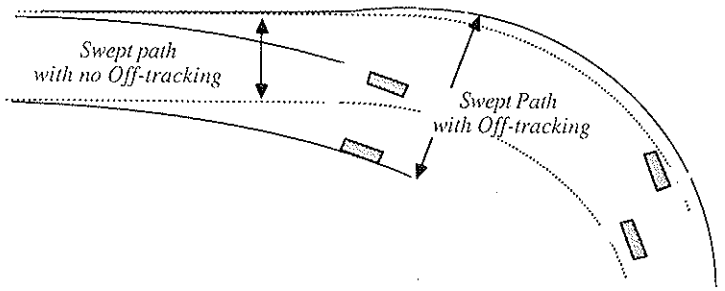

FIGURE 1 Example of the offtracking effect for a single-unit vehicle.

The methods that have been used by designers to estimate the offtracking of heavy trucks are reviewed and their limitations are mentioned. A computer-based method for graphing the complete swept path of an arbitrary vehicle making any type of turn at low speed is then described and demonstrated. The computer method presented is valid for nearly all truck configurations in use on the highways, including double and triple combinations. A computer program that uses this method has been developed for the Apple II computer and is available to the public from FHWA. When equipped with the appropriate plotting hardware, the program produces high-quality scaled drawings of vehicle offtracking. By using this program or an equivalent, the designer can see just how much space will be required by various vehicles to navigate a turn.

Most vehicle models used for offtracking predictions are one dimensional and neglect effects of vehicle width during low-speed turns. The assumptions underlying a one-dimensional "bicycle model" are relevant to the range of applications for which the models are valid and are described first.

BICYCLE MODEL

Description

In this paper are discussed models that assume that all nonsteered wheels that are rigidly connected can be represented by a single "equivalent wheel" located near the centroid of the actual wheel positions. Because highway vehicles are symmetric from right to left, with each wheel on the right side of the vehicle having a corresponding wheel on the left side, the model is based on a single wheel located at the center of the axle. Thus the vehicle is modeled geometrically as a bicycle. Multiple-axled suspensions are similarly modeled as a single effective axle, usually located at the geometric center of the nonsteered axles.

Figure 2 shows how an 18-wheeled tractor-semitrailer combination vehicle would be represented by two linked bicycle models. The bicycle model for the tractor has the front point coinciding with the center of the front axle and the rear point coinciding with a point midway between the two rear axles. The wheelbase, designated L_1, is the distance between these points. Note that the wheelbase parameter is less than the longest wheelbase dimension of the tractor because it does not extend to the second rear axle. Naturally, it is also less than the overall length of the tractor. The wheelbase for the semitrailer, designated L_2, is the distance between the hitch and the center point of the two axles. The front point of the semitrailer does not necessary coincide with the rear point of the tractor unit, and therefore the offset distance, designated λ_1, is also needed. The offset is shown as a positive quantity in the figure because it is in front of the

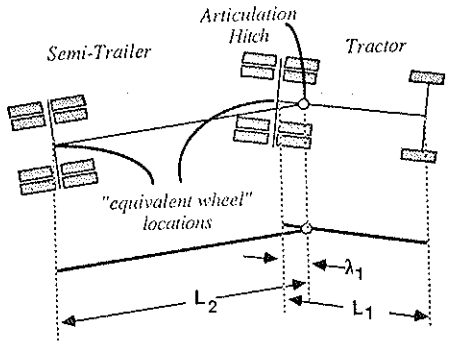

FIGURE 2 Two linked "bicycle" vehicle models.

equivalent wheel position. When the hitch point is located behind the rear wheel, a negative value is used.

Terms

In this paper, each component (tractor, semitrailer, dolly, and so forth) of a combination vehicle is called a "vehicle unit." The rear point in the bicycle model will be referred to as the "rear axle" for convenience, although it is recognized that it is actually the center of the two or more rear wheels in the actual vehicle being modeled. For a single-unit vehicle, such as a truck, bus, or automobile, the front point always corresponds to the center of the steered front axle. The same is true for the tractor in a combination vehicle. For towed units, the front point always corresponds to the hitch location. Because the front point may represent either the center of a steered axle or a hitch location, it will be referred to simply as "the front point."

Limitations

When turning at low speeds, the unsteered rear wheels of a vehicle follow a path that is determined mainly by two factors: (a) the paths taken by the front wheels and (b) the fixed geometric relationship between the front and rear axle or axles. At higher speeds, the masses of the vehicle components cause forces that resist change in direction. These forces interact with tire forces to determine where the vehicle goes. Usually, the effect of the masses is to force the rear wheels to the outside of the turn, reducing the offtracking. Although it is possible in some cases for the rear axle or axles to actually track outboard at high speeds, the swept width is generally largest at low speeds. Thus models based only on kinematic relations can be used to predict "worst-case" offtracking.

There is a great deal of additional mathematical

complexity that is encountered when going from a simple bicycle model to one that includes details of tire mechanics. Fortunately, these effects have only a slight influence in most of the situations that concern pavement layout design. Therefore this paper deals exclusively with methods based on the bicycle model. Before continuing, however, a few exceptional cases will be mentioned for which the bicycle model is not appropriate. The bicycle model is inaccurate in these cases because the tire mechanics act in such a way that identically steered wheels do not perform the same as a single equivalent wheel, as is assumed in the bicycle model.

For an axle with dual tires to follow a curved path, it is necessary for the individual tires to assume nonzero slip angles, generating forces that cancel but nonetheless create a steering moment. This moment acts about the center of the axle and resists any turning. The same effect is created when there are two or more nonsteered axles rigidly connected in a tandem or triple suspension. A result of these tire forces is that significant steering effort is required to navigate a turn in contrast to the zero steering effort that would be adequate if all tires were to roll with no slip.

The steering effort is not of concern as long as the required forces are available. One limitation is the friction of the pavement surface. If the required steering forces at the front axle exceed the friction available, then the front tires will slide and the vehicle will not make the turn as predicted with a no-slip bicycle model. This might happen for a special vehicle with many heavily loaded rear axles that are unsteered and a lightly loaded front axle, but such behavior would be most uncommon for a highway vehicle.

Another case in which a single wheel with zero slip is not a good representation of a group of unsteered wheels is that in which the tires on the wheels are not identical or all the tires in a group are not loaded equally.

When the tires are more or less the same, and equally loaded as intended, the steering moments generated by the nonsteered tires have only a minor influence on the vehicle tracking performance on high-friction surfaces (that is, dry pavement) and can be included in a bicycle model by modifying the wheelbase parameter (1). This effect is typically so slight that a simple geometric averaging is generally acceptable for obtaining the wheelbase parameters of the bicycle model.

On slippery surfaces some of the tires can reach the frictional limits while others do not, resulting in different offtracking performance than would be obtained on a high-friction surface (2). The bicycle model applies only for the case of a high-friction surface.

Although there is nothing to prevent the bicycle model from being used for vehicles with steerable rear axles, steerable rear axles are not treated in this paper. All of the analyses that follow apply only to vehicles with unsteered rear axles, for which an equivalent vehicle unit wheelbase can be assumed.

ANALYSES IN USE

Maximum Offtracking

For a given radius of turn, the maximum offtracking occurs when the vehicle has reached a steady-state condition. The case of steady turning is relatively simple to analyze for the bicycle model. Because the vehicle is a rigid body, there is a center of rotation about which every point in the body rotates. The no-slip condition at the rear means that the circle traced by the rear wheel must be perfectly tangent to the vehicle body, as shown in Figure 3. This condition of tangency means that the Pythagorean theorem can be used to calculate the radius of the circular path at the rear wheel:

$$R_1^2 = R_{in}^2 - L_1^2 \quad (1)$$

Thus the no-slip condition requires that the rear axle must follow a smaller radius than does the front, such that steady-state offtracking is always inboard. Note that the offtracking is maximum at the equivalent rear axle position because it is only at the position of the rear axle that the vehicle frame is exactly tangent to the curve. Either forward or aft of this position, the vehicle must be further from the circular curve (and thus it must lie on a longer radius curve) as can be seen from the figure.

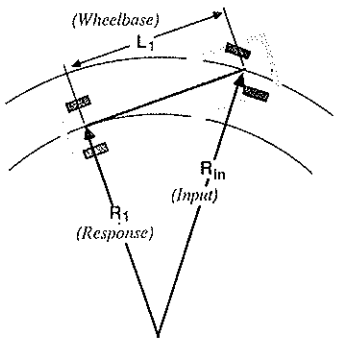

FIGURE 3 Use of the Pythagorean theorem to compute maximum (steady-state) offtracking in a constant-radius turn.

Just as the steady turning of a motor vehicle is determined by the radius of the path followed by the front axle, the turning of a towed trailer is determined by the radius of the articulation point (hitch location). Figure 4 shows the geometry for the case of a tractor-semitrailer vehicle. As before, the Pythagorean theorem applies to the tractor, such that the radius of the effective rear axle (R_1) can be calculated using Equation 1. The Pythagorean theorem

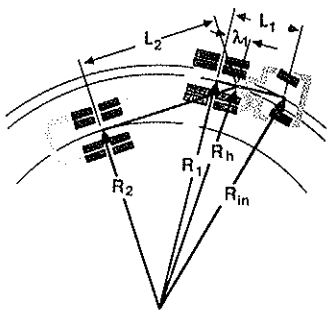

FIGURE 4 Use of the Pythagorean theorem to analyze maximum (steady-state) offtracking of a tractor-semitrailer in a constant-radius turn.

is used consecutively to next calculate the radius of the path traced by the hitch

$$R_h^2 = R_1^2 + \lambda_1^2$$
$$= R_{in}^2 - L_1^2 + \lambda_1^2 \quad (2)$$

and it is used once more to compute the radius of the effective trailer axle:

$$R_2^2 = R_h^2 - L_2^2$$
$$= R_{in}^2 - L_1^2 + \lambda_1^2 - L_2^2 \quad (3)$$

The path taken by the hitch is the input for the trailer, just as the path taken by the front axle is the input for the tractor. This procedure can be extended for more trailing units. Thus complex combination vehicles can be analyzed for steady turning (maximum offtracking) simply by repeated application of the Pythagorean theorem. When this is done, the cumulative offtracking of the rearmost effective axle can always be calculated directly. A simplified version of the general formula was originally recommended by the Western Highway Institute (WHI) (3) and has been adopted as a recommended practice by the Society of Automotive Engineers (SAE) (4). (The hitch offset parameters, designated λ_i in this paper, are neglected because they are usually much smaller than the wheelbase parameters and thus have a negligible effect when squared in Equation 3.)

The steady-turning scenario, represented by the SAE formula, gives only the maximum offtracking that will eventually occur for a given vehicle configuration and input radius. However, for many large trucks the steady turn condition is not reached until the vehicle has turned more than 360 degrees. For tight (small-radius) turns, Equation 3 may not have a solution. For example, typical length parameters for a 60-ft tractor-semitrailer combination vehicle are

L_1 = 16.5 ft (5.0 m),
L_2 = 37.0 ft (11.3 m), and
λ_1 = 0.

Equation 3 will give a zero radius for the rearmost axle when R_{in} = 40.5 ft (12.4 m); for any shorter radius the trailer is forced backward and Equation 3 cannot be used.

In practice, nearly all situations for which offtracking performance is desired are transient. The steady-turning relations were presented here mainly as an introduction to the more generalized analyses of transient turning that follow.

Tractrix Integrator

The transient path followed by the rear axle in a bicycle model is called the general tractrix of the path followed by the front point. The tractrix is defined by the two mathematical constraints that have been illustrated in Figures 3 and 4, namely,

1. The rear axle is always a constant distance from the front axle (wheelbase) and
2. The path traced by the rear axle is at all times tangent to the line connecting the rear axle to the front axle (no-slip condition for the unsteered wheel).

The tractrix integrator is a drafting instrument that can be used to trace the tractrix of a curve (3,5). The instrument consists of a bar supported at one end by a stylus (the front point) and at the other by a single knife-edge wheel (the rear axle). The tractrix integrator is essentially a physical bicycle model.

The distance between the wheel and the stylus or the integrator can be adjusted to model different wheelbases. To use the integrator, a scaled drawing is prepared for the input curve, which would be followed by the front axle of the vehicle of interest. The distance between the stylus and the wheel of the tractrix integrator is adjusted to match the wheelbase of the vehicle according to the scale chosen for the drawing. The wheel of the integrator is coated with wet ink, and the input curve is carefully traced with the stylus. The inked wheel, rolling in line with the bar, draws the tractrix. For a combination vehicle, the instrument would next be adjusted to match the wheelbase of the trailer, and the process would be repeated using the tractrix of the lead unit vehicle as the input for the second unit. Thus the path followed by the rear axle of the trailer is the tractrix of a tractrix.

The tractrix integrator can be used for any tractor-trailer combination and any type of input path. The procedure described for tractor-trailer combinations can be extended to include double and triple combinations by using the tractrix of the previous unit as the input for the following unit. The tractrix integrator gives only the paths that would be taken by the center of the vehicle--the wheels in a bicycle model. To obtain the swept path, the draftsman must manually add the width of the vehicle.

The procedure used for multiple vehicle combinations does not allow for hitch locations that are offset from the equivalent axle locations. Often these offsets are fairly small relative to the wheelbase measurements so this error is negligible.

Exact Solution

General mathematical solutions for the tractrix of both straight-line and circular steering curves have been derived (5) and can be used to show quantitatively just how the radius of the rear axle varies during the turn. Considering the simplicity of the vehicle geometry and path inputs, the relations are striking in their complexity. Because the path of the rear axle of the tractor is not circular, the recursive approach used with the Pythagorean theorem and tractrix integrator cannot be used with the exact solutions. The exact solution is therefore limited to a single-component vehicle, unless great liberties are taken when formulating engineering approximations.

Design Templates

The most popular method for estimating offtracking requirements involves overlaying a template with a scale drawing of the design area. The template shows the swepth path of a specific vehicle in a specific turn--typically a 45-ft radius for the outside wheel, which corresponds to a 41-ft radius for the center of the front axle. For these templates, the vehicle approaches the turn along a straight line, follows the constant radius arc for a specified arc angle, and then departs in a straight line. The arc angles are typically 90 and 180 degrees, although other angles are sometimes also shown. The AASHTO green book includes figures for several design vehicles (6), and similar templates are available from other sources (3). Most of the design templates were prepared graphically using a traxtrix integrator.

NUMERICAL METHOD

None of the methods discussed thus far are completely satisfactory as an everyday design tool. The templates can offer only an approximate indication of the offtracking that a design vehicle would exhibit in a reference turn. The radii used as inputs for the templates may have little in common with the design area. A more immediate problem is that templates are not necessarily available for the vehicle of interest, particularly if it was not previously allowed on public roads. Only the tractrix integrator is capable of providing a representative simulation of an arbitrary vehicle following an arbitrary path. As a drafting instrument, however, it requires scale drawings and a certain amount of skill in its use and interpretation, and it has not proven practical for everyday use. The alternative that follows is basically a computer simulation of the tractrix integrator.

General Approach

A numerical offtracking solution must duplicate the operation of the tractrix integrator. Thus the constraints that define a tractrix need to be translated into mathematical equivalents. Although a generalized mathematical solution to the tractrix problem is not known, it is relatively simple to solve the tractrix equations for very short distances. The general solution method is therefore one of stepping through the trajectories.

Because the computations are intended to be programmed into a computer, it is convenient at this point to consider a flowchart of the simulation, which is shown as Figure 5. The flowchart shows three "loops" through which the flow of the program might be redirected to repeat computational sequences. The main loop, which goes from the bottom diamond box to the box labeled b, indicates that, when all of the calculations have been performed for a specific point along the input path, the vehicle is moved forward slightly and the process is repeated for the new position. The increment (Δs) is usually set to a value of 1 ft (0.3 m). When the vehicle has reached the end of the path, the program finishes as indicated by QUIT at the bottom of the chart.

There are also two inner loops in which calculations are repeated for each unit in the combination vehicle. The letter n in the decision diamonds indicates the number of vehicle units and would be set to n = 1 for an automobile, n = 2 for a tractor-semitrailer, n = 4 for a doubles combination (tractor, semitrailer, dolly, pup semitrailer), and so forth. Note that for a single-unit vehicle, none of the calculations would be repeated and there would only be the single loop involving the calculation of position as the vehicle stepped through the maneuver.

Coordinates of a Point in a Vehicle Unit

When the position of the rear wheel of a vehicle unit and the heading angle are both known, the position of any point associated with that unit can be calculated on the basis of the position of the point within the vehicle unit. Figure 6 shows the x-y coordinate systems used in this paper to describe points lying on a vehicle unit. There is an absolute coordinate system needed to describe the positions of the vehicle units as they trace a path, designed with capital letters, X,Y. The origin of the system is arbitrary and can be set to any convenient location. (The beginning of the input path is one such convenient location.) In addition, each unit has its

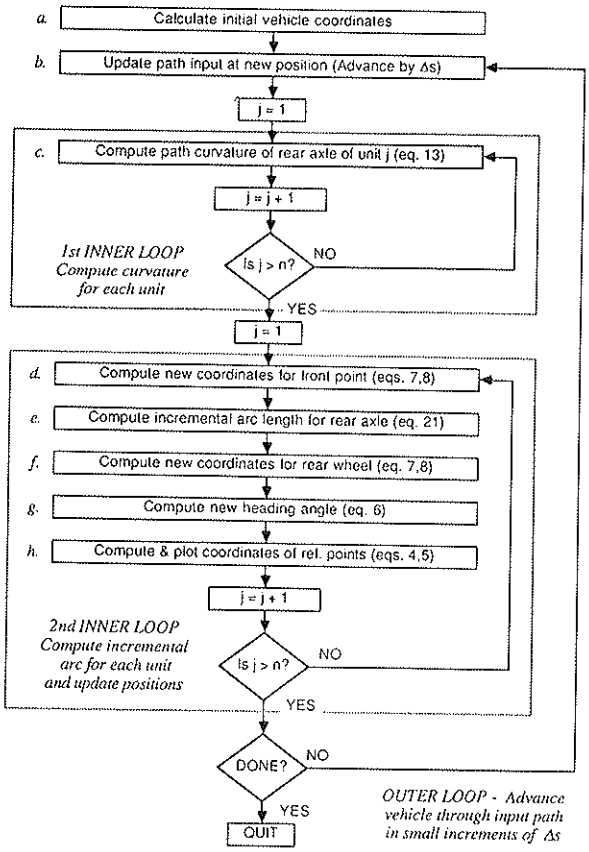

FIGURE 5 Flow chart for offtracking computation method.

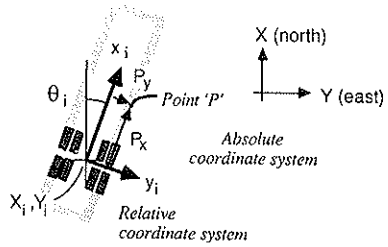

FIGURE 6 Coordinates of arbitrary point P in vehicle unit.

own relative coordinate system, designated by subscript lower-case letters (x_i,y_i). As shown in the figure, the origin of the relative coordinate system (x_i, y_i) has absolute coordinates (X_i, Y_i) and is located at the position of the rear axle of that unit. Note that positive x values lie in front of the axle and that positive y values lie on the right side of the vehicle centerline. The absolute coordinates of the point indicated in the figure are

$$X_p = X_i + P_x \cos \phi - P_y \sin \phi \quad (4)$$

$$Y_p = Y_i + P_x \sin \phi + P_y \cos \phi \quad (5)$$

where X_p and Y_p are the absolute coordinates of the point P, P_x and P_y are the relative coordinates of the point P within the vehicle unit, and X_i and Y_i are the absolute coordinates of the rear wheel of the unit.

Points of interest that would be located using Equations 4 and 5 are the front point (with relative

coordinates $P_x = L_i$, $P_y = 0$) and the hitch location (coordinates $P_x = \lambda_i = P_y = 0$). Two other points of interest are (a) the outer-front corner of the leading vehicle unit, which usually defines the outer edge of the swepth path, and (b) the inner-rear wheels on the rearmost unit, which usually define the inner edge of the swept path.

Equations 4 and 5 are used directly in plotting reference points (Step h in Figure 5) and are also used to determine the initial vehicle position (Step a in Figure 5). When used to begin the simulation, X_i and Y_i represent the front point of the unit, and X_p and Y_p are the calculated initial position of the rear axle, using $P_x = -L_i$, and $P_y = 0$.

Characterization of Input Path

Most of the time, designers are interested in the case of the vehicle making a circular turn for some angle of interest (typically 90 degrees) and then exiting the turn in a straight line. Thus the path input is represented by a circular arc and a tangent line. A more general representation would be helpful, however, so that offtracking simulations could deal more realistically with the types of maneuvers that truck drivers actually make. For example, when turning to the right in an intersection, the driver might first turn to the left to make better use of available space.

A generalized input path could be specified as a series of x-y coordinates at closely spaced intervals, but this would reduce flexibility in selecting an appropriate distance increment, and requires an assumption of how the points are connected (straight lines, arcs, polynomial function) in order to derive a solution. In this paper the input path will be characterized as a sequence of arcs. The end point of one arc is also the beginning point of the next, and the arcs are constrained to be tangent where they meet. Using this method, most input paths of interest can be represented with just two arcs--the first a constant-radius turn, and the second a straight line out of the turn. When more complex paths are desired, they can be built up easily from congruent arcs, as shown for three examples in Figure 7. Figure 7a shows the simple case for two arcs, the second of which has zero curvature. Figure 7b shows a more complex type of turn that could be used to model a maneuver in which the driver first turns to the left in order to obtain more room for a right turn. It is composed of four arcs, the fourth of which has zero curvature. Figure 7c shows a lane-change type of path, which could be used to model the maneuver made by a bus pulling into a bus-stop lane and then leaving.

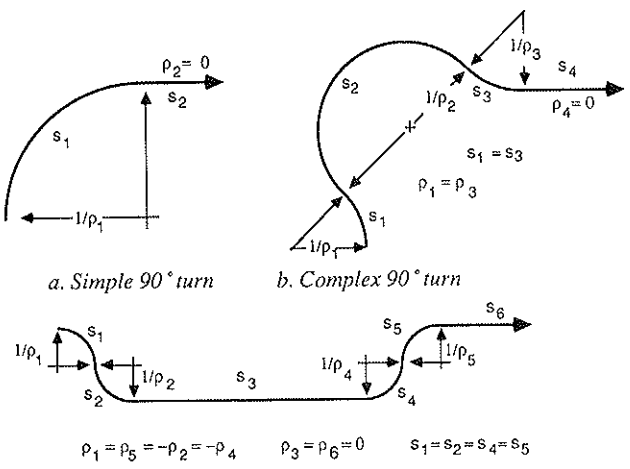

FIGURE 7 Three maneuvers represented as sequences of circular arcs.

Each arc segment in a path is subject to two constraints at the end points to maintain continuity: (a) the end points of consecutive arcs must meet and (b) they must be tangent to each other. As a result of these constraints, each arc can be defined mathematically by two parameters: radius and length. Mathematically, it is more convenient to use curvature--the inverse of radius with units of 1/length--than direct radius (which is infinite for a straight line) or degree of curvature (which has arbitrary units and requires conversion factors). Turns to the right are indicated in this paper as positive curvature, straight lines have zero curvature, and curves to the left have negative curvature. The curvature of an arcs is indicated as ρ, and radius is therefore $1/\rho$.

The second parameter used in the following derivations is arc length, indicated as s. The arc length is used instead of the interior angle because arc length is relevant for straight lines, whereas an interior angle is not.

In addition to the two parameters for each arc, the X-Y coordinates of the first point and the heading angle (ϕ) at the first point can be included for plotting purposes to match the coordinates of the input path to another coordinate system. When these values are specified for the first arc, corresponding coordinates and heading angle can be computed for all subsequent arcs from the conditions of continuity.

Coordinates of a Point on an Arc

To begin the mathematical representation of vehicle offtracking, consider the computation of the x-y coordinates of an arbitrary point on a circular arc. Figure 8 shows a sketch of an arc with curvature ρ and length s. In addition, the coordinates of the beginning point of the arc are given as X_0, Y_0, and the initial heading angle is ϕ_0. At distance s along the arc, the heading angle will be the initial angle plus the angle subtended by the arc. The angle is the product of the arc length and the curvature, and thus

$$\phi = \phi_0 + s\,\rho \tag{6}$$

The coordinates of the end of the arc (for nonzero curvature) can be written as

$$X = X_0 + (1/\rho)\,[\sin(\phi_0 + s\rho) - \sin\phi_0]$$

$$Y = Y_0 - (1/\rho)\,[\cos(\phi_0 + s\rho) - \cos\phi_0]$$

which can be manipulated (using trigonometric identities) to yield

$$X = X_0 + s\,[\text{sinc}(s\rho)\cos\phi_0 - \text{sinc}(s\rho/2)\sin\phi_0 \sin(s\rho/2)] \tag{7}$$

$$Y = Y_0 + s\,[\sin(s\rho/2)\,\text{sinc}(s\rho/2)\cos\phi_0 + \text{sinc}(s\rho)\sin\phi_0] \tag{8}$$

where the function sine curve (sinc) is defined as

$$\text{sinc}(x) \equiv \sin(x)/x \tag{9}$$

Although Equations 7 and 8 are derived for nonzero curvature, they are also valid for straight lines when $\rho = 0$ when they revert to simpler form [if $\rho = 0$, then sinc $(s\rho)$ = sinc $(s\rho/2) = 1$, and sin

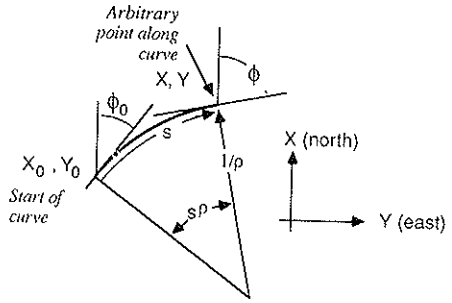

FIGURE 8 Coordinates along a constant-radius curve.

$(s\rho/2) = 0]$. These equations are used in several places in the offtracking simulation to compute new coordinates for various points (Steps b, d, and f in Figure 5).

Rigid Body Rotation

As a rigid body follows a curved path, at any instant it can be characterized by a center of rotation. Figure 3 shows this for a single vehicle unit and also indicates how the center of rotation is calculated for the bicycle model: it is the intersection of the two radial lines (labeled R_1 and R_{in}) that pass through the end points of the bicycle and are normal to the paths followed by those points. Figure 4 shows the special, steady-state case in which both units of a two-unit vehicle have the same center of rotation. In the figure both intersections occur at the same point because this is an illustration of the steady-state case. For transient offtracking, the two centers of rotation change as the vehicle progresses and do not coincide.

For small movements about any given position, the paths of all points on the vehicle are approximately circular, as defined by the instantaneous curvature. Furthermore, the approximation becomes more exact as the distances become smaller. Thus the movements of the axles of the vehicle can be computed using Equations 5-8 if the distance s is small. For most applications, a step interval of several feet is adequate, and an interval of $\Delta s = 1$ ft (0.3 m) is a conservative choice to keep errors negligible.

The method used to compute offtracking can be summarized in two steps, which are repeated as shown by the two inner loops in the flow chart (Figure 5). In the first loop, the curvature at the rear axle is computed for each vehicle unit. In the second step, new positions for each unit are calculated on the basis of an incremental advance of the front axle of the lead vehicle unit.

Computation of Curvature for Vehicle Axles

Figure 9 shows the geometry for an arbitrary vehicle unit (j) hitched to the preceding unit (i) such that

$$j = i + 1$$

Note that two right triangles are formed, with the angles shown defined as

$$\tan \alpha = \lambda_i \rho_i \tag{10}$$

$$\tan \beta = L_j \rho_j \tag{11}$$

Also, the angle β can be written as a function of the heading angles of the two units

$$\beta = \theta_i - \theta_j + \alpha \tag{12}$$

Equations 10-12 can be combined and manipulated to yield

$$\rho_j = \{\tan(\theta_i - \theta_j) + \lambda_i \rho_i]/\{L_j [1 - \lambda_i \rho_i \tan(\theta_i - \theta_j)]\} \tag{13}$$

Even though it is shown for two linked units, Equation 13 also applies to the first vehicle unit where $j = 1$ and $i = 0$. In this case, $\lambda_0 \equiv 0$, and θ_0 and ρ_0 are the current heading angle and curvature of the input path.

Starting with the first vehicle unit, Equation 13 is applied in turn to each vehicle unit to obtain the curvature at the rear axle for that vehicle unit, as shown by the first loop in the flow chart (Figure 5).

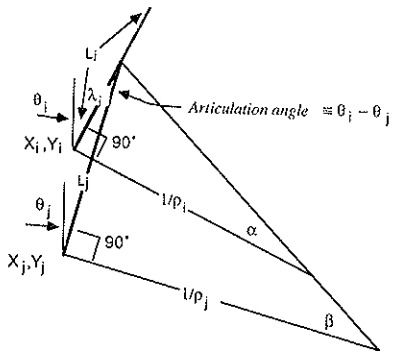

FIGURE 9 Computation of curvature of rear axle.

Updating Vehicle Positions

Equations 5-8 and 13 can be used to compute new coordinates and heading angles for each vehicle unit, if the correct arc distance is known. The incremental distances are not the same for each vehicle unit and will vary during the simulation. Figure 10 shows that the arc length can be defined by a point of intersection of two arcs with known centers and radii.

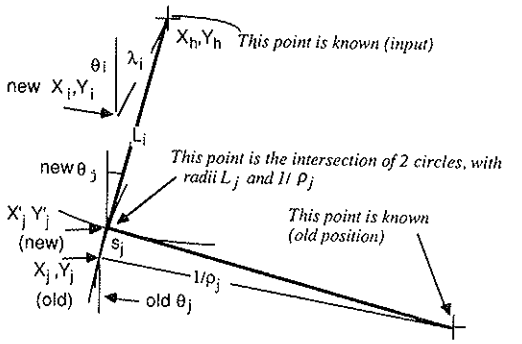

FIGURE 10 Calculation of new position of rear axle.

The coordinates of the new axle position (X_j, Y_j) can be expressed using Equations 7 and 8:

$$X'_j = X_j + s_j[\text{sinc}(s_j\rho_j)\cos\theta_j - \text{sinc}(s_j\rho_j/2)\sin\theta_j \sin(s_j\rho_j/2)\} = X_j + S_j\delta_x \tag{14}$$

$$Y_j' = Y_j + s_j[\sin(s_j\rho_j/2)\,\mathrm{sinc}(s_j\rho_j/2)\cos\theta_j + \mathrm{sinc}(s_j\rho_j)\sin(\theta_j)] = Y_j + s_j\delta_y \quad (15)$$

where X_j' and Y_j' are the new coordinates for the rear wheel of unit j, and

$$\delta_x = \mathrm{sinc}(s_j\rho_j)\cos\theta_j - \mathrm{sinc}(s_j\rho_j/2)\sin\theta_j \sin(s_j\rho_j/2) \quad (16)$$

$$\delta_y = \sin(s_j\rho_j/2)\,\mathrm{sinc}(s_j\rho_j/2)\cos\theta_j + \mathrm{sinc}(s_j\rho_j)\sin\theta_j \quad (17)$$

The coordinates of the front of the unit are the same as the coordinates of the hitch for the preceding unit. Applying Equations 7 and 8 gives the hitch coordinates:

$$X_h = X_i + \lambda_i \cos\theta_i \quad (18)$$

$$Y_h = Y_i + \lambda_i \sin\theta_i \quad (19)$$

The distance between the front and rear of the unit can be calculated from the coordinates (Equations 14, 15, 18, and 19) and must equal the wheelbase (L_j). The Pythagorean theorem gives

$$L_j^2 = (X_j + s_j\delta_x - X_h)^2 + (Y_j + s_j\delta_y - Y_h)^2 \quad (20)$$

Equation 20 can be solved for s_j to yield

$$s_j = (\Delta_x\delta_x + \Delta_y\delta_y - [(L_j^2 - \Delta_x^2 - \Delta_y^2)(\delta_x^2 + \delta_y^2) + (\Delta_x\delta_x + \Delta_y\delta_y)^2]^{1/2})/(\delta_x^2 + \delta_y^2) \quad (21)$$

where

$$\Delta_x = X_j - X_h = X_j - X_i - \lambda_i \cos\theta_i \quad (22)$$

$$\Delta_y = Y_j - Y_h = Y_j - Y_i - \lambda_i \sin\theta_i \quad (23)$$

Equation 21 is not a complete mathematical solution for s_j because the terms δ_x and δ_y, which appear in the equation, are themselves functions of s_j (Equations 16 and 17). However, it becomes a good approximation if a close estimate of s_j is used in Equations 16 and 17. Because the increment Δs used in the computation is small enough for all of the paths to be approximately circular, the change in s_j from one increment to the next is small. Thus the value of s_j that was calculated for the previous position can be used in Equations 16 and 17 to compute δ_x and δ_y, and those values are used in Equation 21 to obtain the new value of s_j.

For the first calculation there is no previous value of s_j to use. However, if the multiple units of the vehicle are lined with each other (all articulation angles at the hitches are zero), then all axles must move the same distance. This orientation is assumed for starting purposes, and therefore each variable s_j is initially equal to the increment of the front axle, $s_0 = \Delta s$.

EXAMPLES

Templates for Two Vehicles

Tables 1 and 2 give the parameters that are used to describe two vehicles of interest: a long (60-ft) tractor-semitrailer and a typical (65-ft) doubles combination. Note that a typical doubles combination is composed of four units: tractor, semitrailer, dolly, and second semitrailer. A triples combination would usually be composed of six units containing the four from the doubles plus an additional dolly and semitrailer. Table 2 includes a negative hitch offset (λ_2). This means that the hitch is behind the effective rear axle for the second unit, the first semitrailer. Figures 11 and 12 show traces of the two points that define a swept path in a right turn: the left front corner of the tractor and the midpoint of the right wheels of the rearmost unit. The path lying 4 ft to the left of the input path is also shown. The input for these two figures is a 41-ft-radius turn (at the vehicle center) followed for angles of 90, 180, 270, and 360 degrees. Although longer, the doubles combination sweeps a narrower path than does the tractor-semitrailer combination. Indeed, the tractor-semitrailer is too long to reach a steady-state condition for a 41-ft-radius turn, and Equation 3 (the SAE formula) has no solution for this vehicle.

Figure 13 shows the type of trajectory that would be predicted for the tractor-semitrailer for a continued turn. The figure is based on the same radius input but continues the turn for three complete circles (1080 degrees). Shortly after 360 degrees, the rear wheels of the trailer have tracked so far inboard that the trailer is actually being pushed backwards as the tractor progresses. As it is pushed backwards, it tracks outward in a diverging path un-

TABLE 1 Vehicle Parameters for Long Tractor-Semitrailer Combination (60 ft Overall, 48-ft Trailer)

Bicycle Model Parameters	Reference Points
n = 2	Left front corner of tractor (Unit 1)
L_1 = 17.5 ft (5.3 m), λ_1 = 2.1 ft (0.6 m)	P_x = 20.5 ft (6.25 m), P_y = -4.0 ft (1.2 m)
L_2 = 40.0 ft (12.2 m)	Left front wheel of tractor (Unit 1), P_x = 17.5 ft (5.3 m), P_y = -4.0 ft (1.2 m)
	Midpoint between right outside wheels of semitrailer (Unit 2), P_x = 0, P_y = 4.25 ft (1.3 m)

TABLE 2 Vehicle Parameters for Doubles Combination (Cab-Over-Engine Tractor, Two 28-ft Trailers, 65-ft Overall Length)

Bicycle Model Parameters	Reference Points
n = 4	Left front corner of tractor (Unit 1)
L_1 = 11.00 ft (3.4 m), λ_1 = 1.8 ft (0.5 m)	P_x = 14.0 ft (4.3 m), P_y = -4.0 ft (1.2 m)
L_2 = 22.8 ft (6.9 m), λ_2 = -2.2 ft (-0.7 m)	Left front wheel of tractor (Unit 1)
L_3 = 6.1 ft (1.9 m), λ_3 = 0	P_x = 11.0 ft (3.4 m), P_y = -4.0 ft (1.2 m)
L_4 = 22.8 ft (6.9 m)	Midpoint between right outside wheels of second semitrailer (Unit 4), P_x = 0, P_y = 4.25 ft (1.3 m)

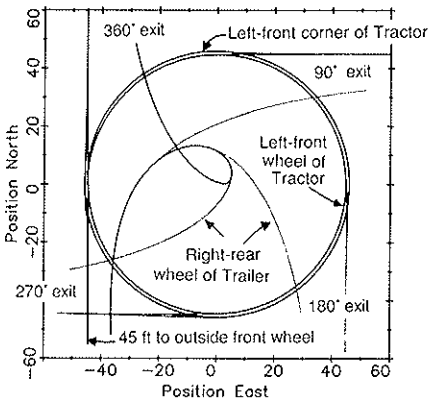

FIGURE 11 Offtracking for a 60-ft tractor-semitrailer vehicle in a 41-ft-radius turn (45 ft to the outside front wheel).

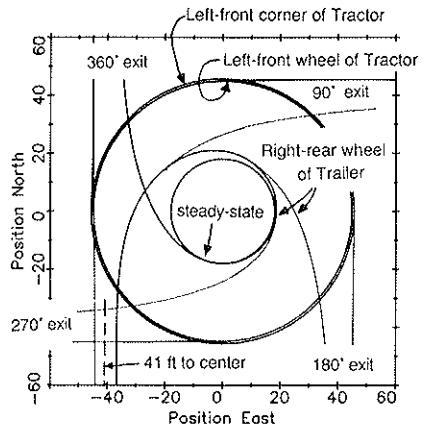

FIGURE 12 Offtracking for a 65-ft doubles combination vehicle in a 41-ft-radius turn (45 ft to the outside front wheel).

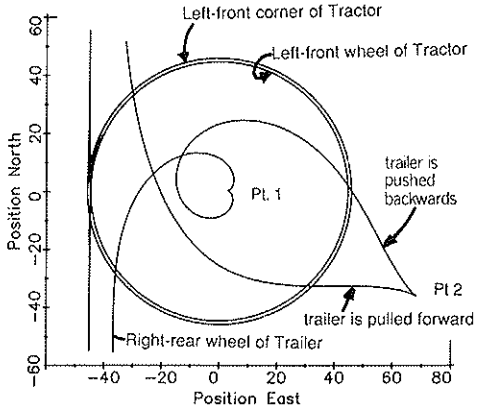

FIGURE 13 Example of a trailer backing up when attempting a continued short-radius turn.

til it is so far outboard that it can go no further. After this point it is once again pulled by the tractor and tracks inboard. The figure demonstrates that the numerical method is stable and versatile, but is also demonstrates a result that could not be obtained with an actual vehicle. To generate the paths shown, the trailer had to pass over the tractor. That is, the articulation angle between tractor and trailer went clear to 180 degrees and continued. Actual tractor-semitrailers are constrained to articulation angles of a little more than 90 degrees, so a real vehicle would have jammed and possibly been damaged if this maneuver had been attempted.

Description of the Apple II Program

A computer program that performs the offtracking computations described in this paper and prepares plots of the paths on an X-Y plotter is available from FHWA. The program was written for the FHWA at the University of Michigan Transportation Research Institute (UMTRI) as part of the project "Impact of Specific Geometric Features on Truck Operations and Safety at Interchanges" (contract DTFH61-83-C-0054). The program runs on an Apple II computer (II+, IIe, IIc) and requires 48k memory and one disk drive. The program is self-contained and relatively user friendly, so that persons who do not have any experience with computers in general (or the Apple II in particular) can use the program without learning much about the operation of an Apple II.

The program allows the user to enter, edit, and save input paths and vehicle descriptions. When a vehicle description and a path description are both chosen, the program simulates the tracking of the selected vehicle as it follows the selected path. The coordinates of the rear axles of each vehicle unit are stored on disk along with the heading angle. Preselected reference points are plotted on the display screen to show the progress of the simulation. Later, the stored data can be used to plot the paths of any arbitrary points in the vehicle.

Scaled hard copies of the vehicle offtracking paths can be obtained in two ways. If a plotter (the Apple X-Y plotter) is available, the program will use it to make scaled ink drawings of the paths traced by any reference points of interest. The plotter can use either paper or transparent material, so it is convenient for making transparent overlay templates. Although the program was developed primarily for use with a plotter, it also allows a dot matrix printer to be used if a "smart" interface card that can control the printer to reproduce the graphic image from the screen is installed in the computer. At the time the program was written there was no software available that allowed detailed graphics covering an entire printed page at once. (The main problem is that a full 8- by 10-in. printer page contains more than 4 million dots and requires more than 400k bytes to store the image.) The hard copy is limited to the information shown on the screen. To make a typical scaled plot, it is necessary to make four hard copies, each showing a different fraction of the entire plot. These can then be taped together, as shown in Figure 14. Thus the trade-off in cost versus performance between a printer and plotter includes both speed and quality. The plotter produces high-quality output in less time than the printer.

The program was developed for a microcomputer instead of a mainframe computer in order to make the program more accessible to state agencies. Most large computers (and many small computers) have special hardware and software for plotting, which means that a program with graphic output will require customization to run on a particular installation. By using an inexpensive and commonly available microcomputer, the problems associated with installation effort and hardware incompatibility are reduced, and they can be completely eliminated by using the supported x-y plotter.

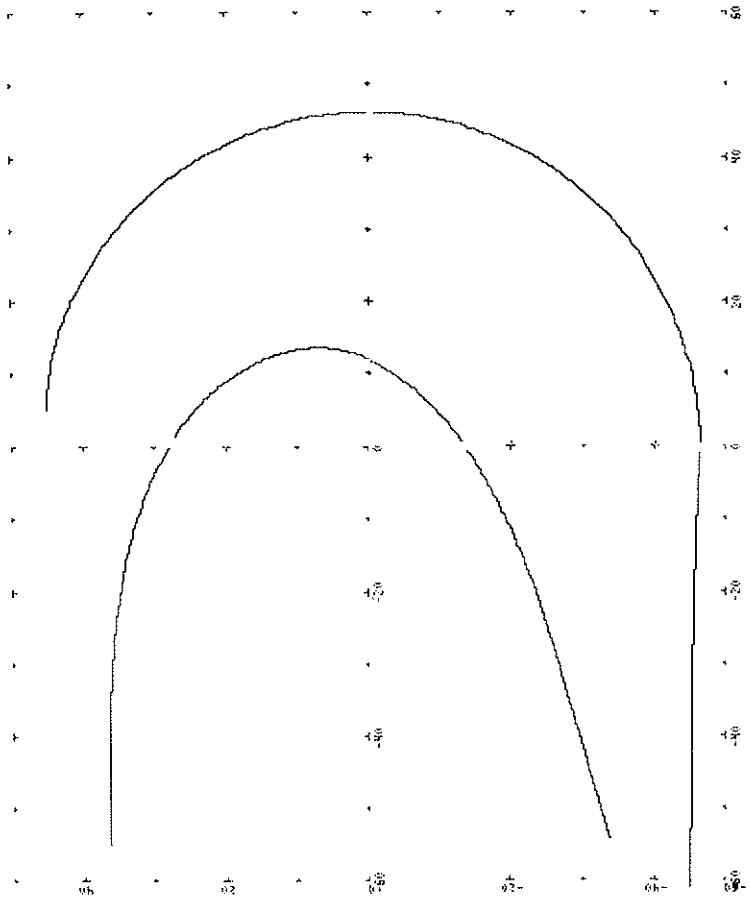

FIGURE 14 Scaled plot created by taping together four images created with a dot-matrix printer and a "smart" interface card.

The main disadvantage of this program is that it is slow to execute. It takes from several minutes to about 20 min to calculate the offtracking paths for a single vehicle maneuver, with the longer times needed for the more complicated vehicles (doubles, triples). It then takes 5 to 15 min to make a plot, as it reads stored data from a disk file. (If the hard copy is made with a dot-matrix printer, the time is multiplied by the number of printed images that must be taped together to obtain the full-scale plot.)

SUMMARY

A review of the methods in use by designers to estimate the offtracking of heavy trucks shows that analytical methods are not available for predicting low-speed offtracking for transient paths. Two graphic methods are used instead: the tractrix integrator (a drafting device) and transparent overlay templates, usually generated with the tractrix integrator. A computer-based method for graphing the complete swept path of an arbitrary vehicle making any type of turn at low speed is described and demonstrated. The computer method is essentially a numerical version of the tractrix integrator, with the improvements that can be obtained using computer graphics equipment. A program that uses this method has been developed for the Apple II computer and is available to the public from the FHWA. When equipped with the appropriate plotting hardware, it produces high-quality scaled drawings of vehicle offtracking. The program can simulate most highway vehicles and handle arbitrarily complex turn geometries. Therefore virtually any geometric design can be evaluated for a particular vehicle of interest.

ACKNOWLEDGMENT

The offtracking computation method described in this paper was developed as part of the FHWA project "Impact of Specific Geometric Features on Truck Operations and Safety at Interchanges" conducted at the University of Michigan Transportation Research Institute (UMTRI) (contract DTFH61-83-C-0054).

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Expected Performance of Longer Combination Vehicles on Highway Grades

K. NABIL A. SAFWAT and C. MICHAEL WALTON

ABSTRACT

Sections 138 and 415 of the Surface Transportation Assistance Act (STAA) of 1982 require the FHWA to report to Congress on the benefits and costs of a national intercity truck route network for the safe and efficient operation of longer combination vehicles (LCVs) such as the double 48-ft and the triple 28-ft combinations. The current (1984) AASHTO criteria for determining critical lengths of grades and climbing lane design for the safe and efficient operation of existing heavy (3-S2) five-axle trucks assume a gross vehicle weight-to-net horsepower (GVW/NHP) ratio of 300 lb/hp to be "representative." The objective of this paper was to investigate the expected performance of LCVs on highway grades and possible impacts on the current AASHTO design criteria. The analysis involved the application of a modified simulation model (used by earlier studies for regular five-axle trucks) under alternative hypotheses about GVW/NHP ratios, rolling resistances, and aerodynamic drag for LCVs operating on different percentage upgrades (1-9 percent grade). The research also included a limited collection of data on GVW and NHP values of actual LCVs. It was found that for LCVs, a GVW/NHP ratio between 300 and 400 would be considered normal, and a ratio above 400 could, occasionally, be observed. It was also found that critical lengths of grades up to 6 percent could be significantly less than AASHTO-recommended values depending on the percentage grade and the LCV's characteristics such as GVW/NHP ratio, rolling resistance, and aerodynamic drag. The expected difference in critical lengths could be as large as 1,060 ft on a 2 percent grade; that is, 44 percent less than the AASHTO-recommended value of 2,400 ft. In order to make specific recommendations with respect to changes in current AASHTO design criteria, actual field data for the operation of LCVs on grades have to be collected and analyzed.

Among the numerous items introduced in the Surface Transportation Assistance Act (STAA) of 1982 is the concept of the longer combination vehicle (LCV) such as the double 48-ft and the triple 28-ft truck combinations. With an overall length of 120 ft, an effective width of 102 in., and maximum axle weight of 20 kips for a single axle and 34 kips for a tandem axle, the gross vehicle weight (GVW) of these LCVs can reach approximately 130,000 lb. Although the STAA cited an upper overall length of 110 ft, fundamental considerations of configuring LCVs identified the length of 120 ft as more appropriate for these units. The current criteria for determining the critical length of grades and for providing climbing lanes, suggested in the recent (1984) AASHTO green book on geometric design policy (1), assume a weight-to-horsepower ratio of a typical 3-S2 heavy truck to be approximately 300 lb/hp. Operational tests conducted by the California Department of Transportation (Caltrans) in 1984 (2) indicate that LCVs are generally slower than are other typical heavy trucks, particularly when the weight-to-horsepower ratios are greater than 350 lb/hp.

The purpose of this paper is to gain more insight into the performance characteristics of LCVs on grades. With this understanding, the objective is then to assess the impacts that the operation of LCVs on grades might have on the current design criteria for determining the critical length of grades and for the provision of climbing lanes.

K.N.A. Safwat, Department of Civil Engineering, Michigan Technological University, Houghton, Mich. 49931. C.M. Walton, Center for Transportation Research, The University of Texas at Austin, 78712.

To achieve this objective, the factors that may influence performance of vehicles on grades were reviewed, and those factors that could be relatively more important for LCVs were highlighted. This is the subject of the second section. In the third section the issue of the prediction of LCV performance on grades is addressed. This involves a discussion of existing approaches, the selection of a particular approach for the study, a detailed description of the selected approach, and the actual application of the method to LCVs. In the fourth section the focus is on the analysis of results for LCVs compared with current AASHTO criteria for the critical length of grade and climbing lane design. The fifth section includes a summary and conclusions.

FACTORS THAT INFLUENCE VEHICULAR PERFORMANCE ON GRADES

Most of the material in this section is extracted and summarized from Walton and Lee (3).

The ability of any vehicle combination to overcome any given grade is directly related to the resultant effect of two principal types of forces. These are the tractive effort forces (i.e., the pulling forces generated through the power train of the vehicle and delivered to the drive wheels) and the tractive resistance forces (i.e., the resisting forces due to inertia, internal vehicle friction, rolling, wind, grade, curvature, etc.). Each of these forces is a function of several factors related to one or more of the four principal components of the transport system. These four components are the vehicle, the roadway, the driver, and the environment. A brief

description of the major influencing factors related to each of these four components follows. The factors that are expected to be relatively more influential in the case of LCVs will be highlighted.

Factors Related to the Vehicle

The vehicle is probably the most important component that may influence performance on grades. Vehicular characteristics that are most likely to affect performance are the gross vehicle weight (GVW), the power train characteristics, and the physical dimensions of the vehicle.

The GVW is no doubt a major factor. As the GVW increases, the rolling, inertial, and grade resistances increase, leading to excessive reductions of speed on grades.

The power train characteristics of the vehicle are also of prime importance. It is the weight-to-horsepower ratio that is considered to be a determining factor as far as the vehicle's performance on grades is concerned.

The physical dimensions of the vehicle that may influence its performance are side area, shape, frontal area, and number and configuration of axles. These features affect the magnitude of air resistance on the vehicle. In addition, the number and configuration of axles may affect the vehicle's inherent resistance. The influence of these features on the vehicle's performance is generally considered to be less significant than is the GVW/NHP ratio. However, in the case of LCVs, these features are expected to play, relatively, a greater role in determining the climbing capability of these particular vehicle types.

Factors Related to the Roadway

The roadway is no doubt an element that has a major influence on the vehicle's performance. The main roadway features of influence are length and steepness of grade, cross-sectional profile, horizontal alignment, and pavement type and condition.

The length and steepness of grade are the most significant factors related to the roadway. The vehicle's deceleration rate is directly dependent on the steepness of grade and the total speed reduction is primarily dependent on the length of grade.

The main cross-sectional variables are the number of lanes, the width of lanes, and the type and width of shoulders. These factors can affect the entry speed at the beginning of a grade, which is a major influencing factor on a vehicle's performance on grades. Significant variability in any of these features along the grade itself can have important effects on the vehicle's performance.

Pavement type and condition can influence operating speed and rolling resistance and, consequently, the vehicle's performance.

Factors Related to the Driver

The performance capabilities of and the ultimate speed at which a heavy vehicle can overcome a grade are, in many ways, dependent on the ability of the driver to coax the maximum pulling force from the vehicle. The training, experience, familiarity with the vehicle, and physical abilities of the driver are the main characteristics related to the driver that are most likely to affect the vehicle's performance.

Factors Related to the Environment

The major environmental factors that can influence the vehicle's performance are atmospheric conditions, traffic conditions, and land use along the roadway.

Atmospheric disturbances such as strong winds, heavy rain, dense fog, high humidity, high or low temperatures, and different altitudes will tend to adversely affect the capability of the driver and the vehicle to operate efficiently on grades. The driver will naturally tend to travel at reduced speeds and the vehicle's engine output will also tend to be reduced.

Heavy traffic volumes, high percentages of trucks and buses, and wide variability of speeds in a traffic stream will certainly have a significant detrimental effect on the performance of vehicles on grades.

As the density of abutting land use increases, the likelihood of interference caused by merging traffic attributable to adjacent activities increases. The natural reaction of the driver in these situations is to be more cautious and to reduce speed.

The relative contribution of each of these factors related to the vehicle, the roadway, the driver, and the environment can be assessed through modeling the interactions among these factors in order to predict the vehicle's performance on grades and performing sensitivity analysis for each factor in the prediction model.

The discussion in the next section should provide more insight into this issue.

PREDICTION OF PERFORMANCE OF LCVs ON GRADES

There are three major approaches to predicting the performance of vehicles on grades. These are actual field testing, econometrics modeling, and simulation modeling.

Actual field testing is no doubt the most satisfactory procedure, but it is rather laborious and expensive to conduct. In addition, the results are applicable only for the given conditions of the tests.

The econometrics modeling approach involves collecting data on actual vehicle performance, vehicle characteristics, the driver, the roadway, and the environment. These data are then used to calibrate an econometric model that relates all relevant factors to the vehicle's performance. This econometric model is then used for prediction. This approach has been used by Walton and Lee (3). They collected an extensive amount of data to study the speeds of commercial as well as recreational vehicles and developed multiple regression models for different vehicle types and sites considered in the analysis. The longest truck combination included in their analysis was the 3-S2 truck (i.e., three-axle tractor and two-axle semitrailer-truck combination). A total of 10 factors were used in their analysis. These are length of grade, percentage grade, approach speed, gross vehicle weight, vehicle horsepower, frontal area, side area, driver experience, age of driver, and age of vehicle. The roadway and environmental conditions were recorded and controlled through the selection of the test sites and times.

The major advantage of this approach is that it allows the prediction of vehicle performance for values of the variables that are different from those observed. It also allows an investigation of the relative importance of each factor. On the other hand, the results, generally, are applicable only within a certain range of values and under environmental and roadway conditions that are more or less

similar to (or at least not significantly different from) those observed. For example, the GVW and the side area of LCVs will certainly be significantly greater than those used by Walton and Lee (3), and, hence, their regression model may not produce sufficiently accurate predictions for LCVs. Nevertheless, the econometrics modeling approach remains a viable alternative.

The third major method for predicting performance of vehicles on grades (i.e., the simulation modeling approach) has been used by SAE (4), St. John and Kobett (5) of the Midwest Research Institute (referred to later as the MRI study), and Abbas and May (6) of the Institute of Transportation Studies at Berkeley (referred to later as the ITS study). The simulation approach focuses on the truck's engine characteristics and decelerations (because of gravity) during gear shifts. It gives explicit consideration to gear shift delays, rolling losses, chassis losses, and aerodynamic losses. The basic assumption of the model is that the engine is employed a varying fraction of the total time.

The simulation method has the advantage of capturing the vehicle's related factors in detail. Aspects of the driver's behavior are taken into consideration in "gear shift delay."

On the other hand, it is not clear how the "other" factors could be considered in the approach. Both the MRI and ITS studies have introduced modifications to the coefficient values of the SAE (4) model in order to validate the model's results with field data. The behavioral implications of these modifications are not apparent.

In this study it would have been desirable to use all three approaches in order to gain the advantages associated with each. However, because of the lack of sufficient field data on the actual performance of LCVs on grades, the approach that relies the least on field data or that is more behaviorally oriented, or both, was selected.

It is clear from the discussion of different approaches that all three methods involve the use of field data. In the econometric approach, field data were used in the calibration of the regression model, and in the simulation model the field data were used in the adjustment of SAE-recommended values of coefficients related to rolling force and aerodynamic drag.

It is not quite clear whether the difference between "calibration" and "adjustment of coefficients" is significant or not. Nevertheless, it may be argued that adjustments in the simulation model's coefficients can be achieved through a deeper understanding of the behavioral implications of the impacts of changes in the vehicle's characteristics on the rolling, chassis friction, and aerodynamic drag forces. In this case the role of field data in the use of the simulation model could be reduced significantly.

As far as the behavioral orientation of different approaches is concerned, it is obvious that the simulation model has a definite advantage. Therefore, in this study, the simulation approach has been selected. A detailed description of the approach is given followed by a discussion of the appropriate values for the model's input variables and parameters for its application to predict LCV performance on grades.

Simulation Model

The model consists of a set of performance equations that depict the capability of the truck along a straight section of roadway with a given gradient under free-flow conditions. The model focuses on the truck's engine characteristics and decelerations during gear shifts. It gives explicit consideration to gear shift delays, GVW/NHP ratios, rolling resistances, chassis friction, and aerodynamic losses. The basic assumption of the model is that the engine is employed a varying fraction of the total time.

The basic performance equation in the model calculates the vehicle's effective acceleration at a given vehicle speed taking into account the acceleration in coasting during gear shift delay. This equation may be expressed as

$$A_e = \bar{A}_p[(\eta \cdot V)/[\eta V + S_p t_s(\bar{A}_p - \bar{A}_c)]), \quad V > V_1 \qquad (1)$$

where

A_e = effective acceleration (ft/sec²);
$\bar{A}_p$ = power-limited acceleration (i.e., with engine employed and vehicle at speed V); the bar indicates the use of average available net horsepower (ft/sec²);
η = parameter dependent on the range of engine speed normally employed; typical values range between 0.33 and 0.43;
V = vehicle speed (ft/sec);
S_p = one times the sign of $\bar{A}_p$ (i.e., +1 or -1);
t_s = gear shift delay (sec);
A_c = acceleration in coasting at a vehicle speed V; the bar indicates the use of average gear ratio (ft/sec²); and
V_1 = maximum speed in lowest speed gear ratio (ft/sec).

In Equation 1, the speed parameter (η) is defined as the ratio between maximum and minimum engine speeds in the operating range minus one. Its typical values vary between 0.33 and 0.43. A value of 0.4 was recommended and used by the MRI study. When the truck speed is less than V_1, the transmission will be in the first gear ratio. In this case the term ηV in Equation 1 is replaced by V_1.

The gear shift delay (t_s) is an important variable in the model. Its value is dependent on the driver's experience and physical condition. A value of t_s = 1.5 sec was used by SAE, MRI, and ITS.

The other two major variables in Equation 1 are the power-limited acceleration and the acceleration in coasting.

The power-limited acceleration is dependent on the GVW/NHP ratio, rolling resistances, aerodynamic drag, chassis friction losses, and highway grade. These factors, except for the gradient, are essentially related to the vehicle's characteristics including its engine, shape, weight, and physical dimensions, and their effects on the vehicle's performance will vary according to several environmental conditions such as temperature and elevation. The performance equation that gives explicit account of these factors may be expressed as

$$\begin{aligned} A_p = {} & C_{pe}\, C_1/[(GVW/NHP_s)\ (1 + C_e)] \\ & - [(C_2 + C_3 V)/(1 + C_e)] \\ & - \{C_{de}\, C_4 V^2/[(GVW/A)\ (1 + C_e)]\} \\ & - \{C_5 R_i/[(GVW/GVW_r)\ (1 + C_e)]\} \\ & - [g(\sin \alpha)/(1 + C_e)] \end{aligned} \qquad (2)$$

where

A_p = power-limited acceleration (ft/sec²);
GVW = gross vehicle weight (lb);
NHP_s = net horsepower at sea level (hp);
V = vehicle speed (ft/sec);
GVW_r = rated maximum gross vehicle weight (lb);
R_i = speed ratio (engine speed/vehicle speed) in the ith gear ratio [rmp/(ft/sec)];

A = projected frontal area (ft^2);
α = grade angle (positive for upgrade);
sin α = percentage grade/100;
g = acceleration due to gravity (32.17 ft/sec^2);
C_{pe} = altitude correction factor converting sea level net horsepower to local elevation, E (ft);
= 1 - 0.00004E (for gasoline engines);
C_{de} = altitude correction factor converting sea level aerodynamic drag to local elevation, E (ft)
= (1 - 0.000006887E) **4.255; and
C_e = correction factor for engine inertia
= 14080/[(GVW/NHP) *V^2].

C_1, C_2, C_3, C_4, and C_5 are coefficients reflecting the influence of different factors on the performance of the vehicle.

Acceleration in coasting (i.e., when the engine is not employed during gear shifts) is mainly influenced by rolling resistances, aerodynamic drag, and grade. In addition, chassis friction losses may be assumed to be 15 percent of the full-power value. The performance equation in coasting is, therefore, given by

$$A_c = -C_2 - C_3V - [C_{de}C_4V^2/(GVW/A)] - [(0.15)\ C_5R/(GVW/GVW_r)] - g \sin \alpha \quad (3)$$

where A_c is acceleration in coasting (ft/sec^2). To apply performance Equations 1-3 to LCVs, appropriate values for the input variables to the model, correction factors, and model coefficients have to be specified. This issue is addressed in the next subsection.

Application of the Simulation Model to LCVs

It is apparent that the results of the simulation model are quite sensitive to the values of its input variables and parameters. The input variables to the model are gear shift delay, gross vehicle weight, net horsepower, vehicle engine speed, frontal area, entry vehicle speed, and highway grade.

The gear shift delay is no doubt an important variable in the model. However, its value is not expected to be significantly different from 1.5 sec. (i.e., the value used in previous studies) in the case of LCVs because this value is related to driver characteristics, which should be more or less similar for professional truck drivers.

The GVW and NHP values of LCVs are expected to be significantly different from those of regular five-axle truck combinations. Table 1 gives typical gross weights and horsepowers of different LCV types [i.e., Rocky-Mountain doubles (RMD), turnpile doubles (TD), and turnpike triples (TT)]. From this table it is obvious that practical maximum GVW is consistently greater than 100,000 lb up to 138,000 lb. The GVW/NHP ratio could exceed 400. A ratio between 300 and 400 would be considered normal. In this analysis GVW/NHP ratios of 300, 350, and 400 were considered. It was assumed that a ratio of 300 corresponds to NHP = 315 hp and GVW = 94,500 lb, a ratio of 350 corresponds to NHP = 315 hp and GVW = 110,250 lb, and a ratio of 400 corresponds to NHP = 330 hp and GVW = 132,000 lb.

The vehicle engine speed varies with the vehicle speed in any given gear ratio. In this analysis a value of 2,000 rpm was used, which should represent an average operating engine speed (2).

The projected frontal area of a vehicle is the maximum cross-sectional area perpendicular to its direction of motion. In the case of LCVs this value was assumed to be 114 ft^2 based on a truck height of 13.5 ft and a width of 8.5 ft.

Entry vehicle speed was assumed to be 55 mph (i.e., the maximum allowable speed on freeways). The highway grade was assumed to vary between +1 and +9 percent. This should include all upgrades that may be encountered in practice. Increments of 1 percent grade are considered. Downgrades were not considered simply because the objective is related to climbing lane design criteria. The model parameters include correction factors and coefficients.

There are three correction factors in the model related to engine characteristics: two for converting sea level net horsepower and aerodynamic drag to local elevation and a third for engine inertia. These factors should not have significant effects on the model's results, particularly when it is assumed that a reasonable value of local elevation would be 1,000 ft above sea level. Therefore the formulas of the MRI study are used in the analysis.

The simulation model includes six different coefficients. These are η, C_1, C_2, C_3, C_4, and C_5 (see Equations 1-3).

The speed parameter (η) is dependent on the operating range of engine speeds in different gear ratios. Its value, as indicated earlier, varies between 0.33 and 0.43. An average value of 0.4 was used in the MRI study. In this study 0.4 was assumed to be a reasonable value to use.

The coefficients C_1, C_2, C_3, C_4, and C_5 account, respectively, for the influence of weight-to-horsepower ratio, speed-independent rolling losses, speed-dependent rolling losses, aerodynamic drag, and chassis friction losses. It should be clear that the values of these coefficients used in previous studies were validated with actual data for the performance of regular truck combinations. Indeed, the MRI study introduced significant modifications to C_3 and C_4 compared with SAE-recommended values so that the model's results may be closer to actual performance data. More specifically, C_3 was reduced to 22 percent of its SAE-recommended value (i.e., reduced from 1.982×10^{-3} to 0.44×10^{-3}) and C_4 was

TABLE 1 Typical GVW and NHP Values for LCVs

Source	LCV Type	Rated hp	NHP	Practical Maximum GVW (lb)	GVW/NHP (ratio)
Roadway (carrier)	RMD				
	TD	365	328.5	138,000	420
	TT	365	328.5	105,000	320
Western Highway Institute	RMD	350	315	100,000	317
	TD	440	396	125,000	316
	TT	350	315	106,000	337
Caltrans	RMD		304/340	107,000	314/351
	TD		340/480	123,000	360/256
	TT		304/340	111,000	365/326
Ryder/PIE (carrier)		350	315	120,000	380

reduced to 72 percent of its SAE-recommended value (i.e., reduced from 0.0317 to 0.0228). What the values of these and other coefficients for LCVs should be is an issue that is yet to be investigated through actual field experimentation with LCVs. Such an investigation is, however, beyond the scope of this paper. Therefore, "reasonable" values of coefficients for LCVs were hypothesized and a sensitivity analysis of the model's predictions under alternative hypotheses was performed.

As obvious as it may be, the model's coefficients for LCVs are expected to be generally different from those recommended by SAE (4) as well as those modified by MRI and ITS. In particular, the coefficients for rolling resistances and aerodynamic drag are expected to increase in order to reflect an increase in these losses for LCVs due to their increased number of tires and axles, additional weight, larger side areas, and increased number of combinations [see Knight (7) for a more detailed discussion of the effects of these factors for rolling and aerodynamic losses.] It was assumed that C_3 (the speed-dependent coefficient for rolling resistances) could be as high as 0.003 compared with the 1965 SAE-recommended value of 0.001982 and the MRI modified value of 0.00044. As far as the aerodynamic drag coefficient (C_4) is concerned, it was assumed that it could reach 0.04 compared with the SAE value of 0.0317 and the MRI modified value of 0.0228. The speed-independent rolling losses coefficient (C_2) was assumed to remain unchanged. The remaining coefficients (C_1 and C_5 for GVW/NHP ratio and chassis friction losses) were also assumed to be unchanged. This assumption of unchanged coefficients reflects the hypothesis that the differences between their values for LCVs and their corresponding current values in the model are expected to be relatively less significant compared with the expected differences in C_3 and C_4. The best way to verify this hypothesis is to perform actual field tests. As indicated earlier, this is not within the scope of this paper. Nevertheless, for the purpose of analysis, the hypothesis appears to be appropriate.

Table 2 gives a summary of all assumptions related to the application of the simulation model to predict performance of LCVs. The results of the application are presented and analyzed in the following section.

ANALYSIS OF RESULTS

The main objective of the analysis is to investigate the possible impacts that the operation of LCVs on grades might have on the current AASHTO criteria for determining critical length of grades and climbing lane design under alternative assumptions for rolling losses, aerodynamic drag, GVW/NHP ratio, and percentage grade.

The recent edition of the AASHTO green book (1) considers three main criteria; these are speed at entrance of grade, allowable speed reduction along the grade, and vehicle gradability. The average running speed is assumed to approximate the entry speed (e.g., a speed of 55 mph on major freeways). Allowable total reduction in speed is assumed to be 10 mph. This value is recommended primarily for safety reasons. In the 1965 AASHTO policy a 15-mph speed reduction was considered allowable, but studies in 1970 indicated that accidents involving trucks with four or more axles were 2.4 times greater for a 15-mph reduction than for a 10-mph reduction (8). The truck gradability criterion is stated as follows: "A loaded truck, powered so that GVW/NHP ratio is about 300, is representative of the size and type of vehicle normally used for design control on main highways." The relationship among speed reduction, percentage grade, and length of grade for an assumed typical heavy truck of 300 lb/hp is shown in Figure 1 (1, Figure III-30).

The four major variables of analysis are the GVW/NHP ratio (300, 350, and 400); rolling losses coefficient (C_3) (0.00044, 0.001, 0.001982, and 0.003); aerodynamic drag coefficient (C_4) (0.0228, 0.0317, and 0.04); and percentage grade (1, 2, 3, 4, 5, 6, 7, 8, and 9). The remaining variables and parameters in the simulation model are set at their appropriate values as explained in the third section and summarized in Table 2. Therefore the analysis involves 3 x 4 x 3 x 9 = 324 simulation runs. The results are summarized in the Appendix. The tables and figures of this section are essentially extracted from the raw data in the Appendix as needed for analysis. A detailed analysis of the major issues under consideration follows.

Impacts of No Change in Coefficients of Both Rolling and Aerodynamic Losses

In this case it was assumed that coefficients of both rolling and aerodynamic losses for an LCV are identical with those for a regular five-axle truck. This situation corresponds to the use of MRI modified values of C_3 and C_4. That is, C_3 = 0.00044 and C_4 = 0.0228. The results [referred to later as CTR

TABLE 2 Summary of Assumed Values of the Variables and Parameters of the Simulation Model for Application to LCVs

Variables and Parameters	Assumed Values in the Application
GVW (lb)	94,500, 110,250, and 132,000
NHP (hp)	315 and 330
GVW/NHP ratio (lb/hp)	300, 350, and 400
Entry speed (mph)	55
Percentage grade (%)	1, 2, 3, 4, 5, 6, 7, 8, and 9
Rated GVW (lb)	120,000
Vehicle frontal area (ft^2)	114
Gear shift delay (sec)	1.5
Average elevation E (ft)	1,000
Acceptable total speed reduction (mph)	10
Engine speed parameter (η)	0.4
GVW/NHP ratio coefficient (C_1)	17,693.5
Speed-independent rolling losses coefficient (C_2)	0.2445
Speed-dependent rolling losses coefficient (C_3)	0.00044, 0.001, 0.001982, and 0.003
Aerodynamic drag coefficient (C_4)	0.0228, 0.0317, and 0.04
Chassis friction losses coefficient (C_5)	0.0035387
Acceleration due to gravity (ft/sec^2)	32.2
NHP correction factor for elevation (C_{pe})	1 - 0.00004E
Aerodynamic correction factor for elevation (C_{de})	(1 - 0.000006887E) **4.255
Engine inertia correction factor (C_e)	14,080/[(GVW/NHP) $*V^2$]

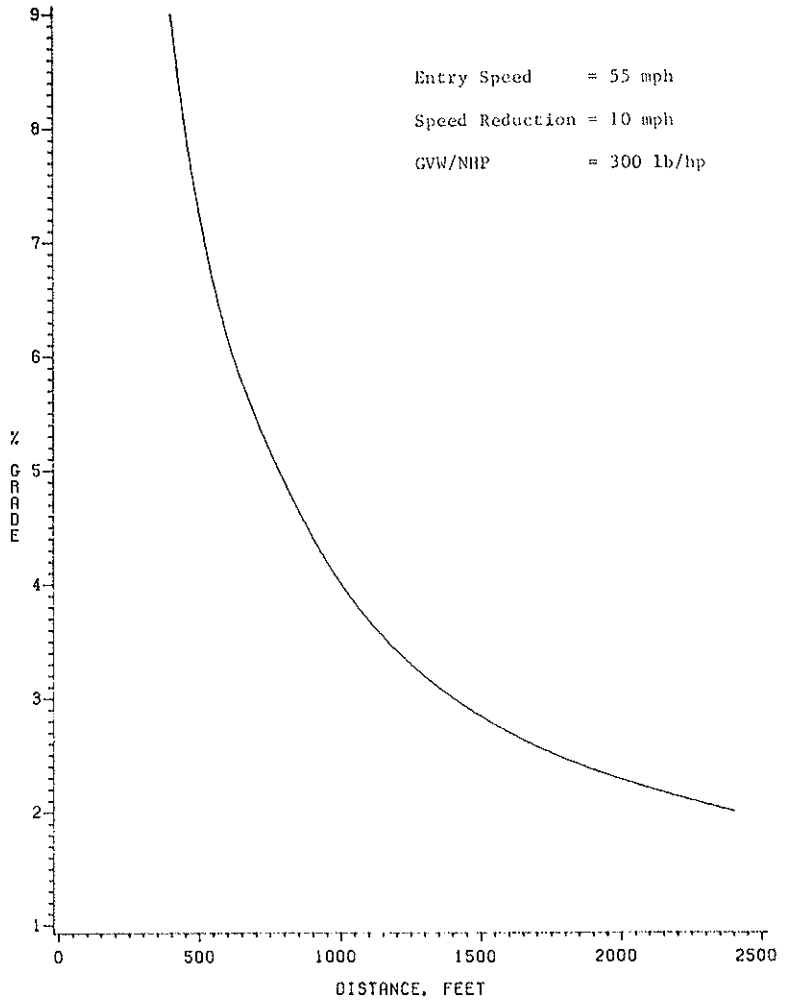

FIGURE 1 AASHTO critical lengths of grade for design assuming typical heavy truck of GVW/NHP = 300.

(i.e., Center for Transportation Research at the University of Texas at Austin)] in this case are expected to be similar to those of MRI but not identical because of differences in GVW, rated GVW, frontal area, and NHP. For the same GVW/NHP ratio, differences between CTR and MRI should be minor, and, hence, CTR results are more or less representative of regular five-axle trucks. This implies that what is essentially investigated are the impacts of differing GVW/NHP ratios on truck gradability compared with AASHTO recommendations at various percentage grades.

Table 3 gives and Figure 2 shows a comparison of critical lengths of grade between CTR and AASHTO under the preceding assumptions for different GVW/NHP ratios and percentage grades. For a GVW/NHP = 300, CTR and AASHTO values are almost identical for grades of 4 percent and greater. This confirms the earlier expectation that CTR results in this case may be representative of regular five-axle trucks. For 3 and 2 percent grades, however, CTR values are greater than AASHTO values by about 3.9 and 10 percent, respectively. This slight overestimation of gradability on lower grades could be related to the higher NHP value used in CTR compared with that of regular trucks. This can be explained by noting that the vehicle's power characteristics become predominant on lower grades because the effects of percentage grade and GVW are decreased. This implies that CTR results are conservative with respect to AASHTO-recommended values. This should, in general, strengthen the analysis and conclusions.

Looking at Table 3 and Figure 2, it is observed that as the GVW/NHP ratio and the percentage grade increase the critical lengths of grade decrease as expected. More important, the differences between CTR and corresponding AASHTO values vary (both in absolute terms and percentage-wise) according to a general trend depending on the GVW/NHP ratio and percentage grade. In general, the range of differences (absolute values and percentages) becomes wider as the percentage grade decreases and becomes narrower as the GVW/NHP increases. In other words, more variability in the results is expected on lower percentage grades and for GVW/NHP ratios closer to 300. The actual difference (in absolute terms and percentage-wise) of course increases as the GVW/NHP ratio gets further from 300 and as the percentage grade decreases. For a 2 percent grade, the critical length of grade could be less than the AASHTO value by as much as 375 ft (for GVW/NHP = 400); that is, a reduction of 16 percent from 2,400 ft. This is a significant difference. For GVW/NHP = 350, the difference could be 174 ft (i.e., a 7.3 percent reduction), which is still considered significant. At GVW/NHP = 300, the CTR value is +247 ft more than the AASHTO value (i.e., a 10 percent increase). This indicates that the difference between CTR and AASHTO values on a 2 percent grade have changed considerably from +247 to -174 ft as GVW/NHP changed from 300 to

TABLE 3 Comparison of Critical Lengths of Grade (ft) Between AASHTO and CRT Assuming No Change in Aerodynamic and Rolling Losses Coefficients (C_3 and C_4), Entry Speed = 55 mph, and Speed Reduction = 10 mph

		CTR Minimum Difference			CTR Average Difference			CTR Maximum Difference				
Grade (%)	AASHTO (ft)	GVW/NHP = 300 C_3 = 0.00044 C_4 = 0.0228 (ft)	Difference Feet	Difference Percentage	GVW/NHP = 350 C_3 = 0.00044 C_4 = 0.0228 (ft)	Difference Feet	Difference Percentage	GVW/NHP = 400 C_3 = 0.00044 C_4 = 0.0228 (ft)	Difference Feet	Difference Percentage	Expected Difference Range (ft)	Expected Difference Range (%)
1		18,000+			7,712			5,472				
2	2,400	2,647	+247	+10	2,226	-174	-7.3	2,025	-375	-15.6	+247 to -375	+10 to -16
3	1,400	1,455	+55	3.9	1,321	-79	-5.6	1,252	-148	-10.6	+55 to -148	+4 to -11
4	1,000	1,006	+6	+0.6	941	-59	-5.9	909	-91	-9.1	+6 to -91	+1 to -9
5	780	769	-11	-1.4	733	-47	-6	711	-69	-8.8	-11 to -69	-1 to -9
6	620	623	+3	+0.5	601	-19	-3	586	-34	-5.5	+3 to -34	+1 to -6
7	520	524	+4	+0.8	506	-14	-2.7	498	-22	-4	+4 to -22	+1 to -4
8	450	451	+1	+0.2	440	-10	-2.2	433	-17	-3.8	+1 to -17	+0 to -4
9	400	399	-1	-0.2	388	-12	-3	385	-15	-3.7	-1 to -15	-0 to -4

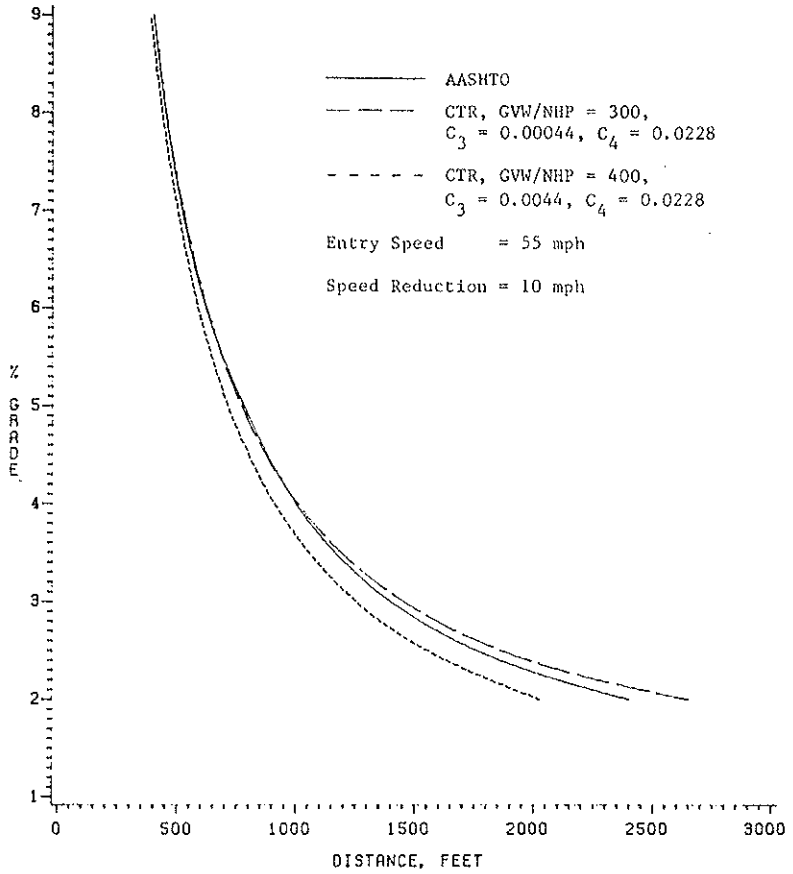

FIGURE 2 Comparison of AASHTO and CTR critical lengths of grade assuming no change in either rolling or aerodynamic losses coefficients.

350 and continued to change, but at a reduced rate, from -174 to -375 ft as GVW/NHP changed from 350 to 400, respectively. The same trend can be observed for other percentage grades as well, but the differences become less significant as the percentage grade increases.

For a 1 percent grade the variability and differences (in absolute values and percentage-wise) are expected to be extremely large. If it is assumed that the AASHTO value for a 1 percent grade is not less than 10,000 ft, it is seen that the difference could be as large as -4,528 ft. This could represent serious problems if the grade actually extended for a few miles. In practice, however, this could be a rare event.

Probably the most important conclusion to be drawn from Table 3 and Figure 2 is that even if it is assumed that LCVs would have the same aerodynamic and rolling losses coefficients as those of regular five-axle trucks, their GVW/NHP ratios, which may normally vary between 300 and 400, could result in significant reductions of critical lengths of grades compared with AASHTO design criteria. These reductions could reach 16 percent (375 ft) on a 2 percent grade, 11 percent (148 ft) on a 3 percent grade, and 9 percent (191 ft) on a 4 percent grade.

Impacts of the Increase in Coefficient of Aerodynamic Drag (C_4) Only

It was assumed that the rolling losses coefficient is unchanged (C_3 = 0.00044) while the aerodynamic

TABLE 4 Comparison of Critical Lengths of Grade (ft) Between AASHTO and CTR Assuming an Increase in Aerodynamic Drag Coefficient (C_4) Only, Entry Speed = 55 mph, and Speed Reduction = 10 mph

Grade (%)	AASHTO (ft)	CTR Minimum Difference: GVW/NHP = 300, C_3 = 0.00044, C_4 = 0.0317 (ft)	Difference: Feet	Difference: Percentage	CTR Average Difference: GVW/NHP = 350, C_3 = 0.00044, C_4 = 0.0317 (ft)	Difference: Feet	Difference: Percentage	CTR Maximum Difference: GVW/NHP = 400, C_3 = 0.00044, C_4 = 0.04 (ft)	Difference: Feet	Difference: Percentage	Expected Difference Range (ft)	Expected Difference Range (%)
1		11,697			5,604			3,888				
2	2,400	2,320	-80	-3.3	2,022	-378	-15.7	1,765	-635	-26.5	-80 to -635	-3 to -27
3	1,400	1,349	-51	-3.6	1,247	-153	-11	1,146	-254	-18	-51 to -254	-4 to -18
4	1,000	955	-45	-4.5	904	-96	-9.6	850	-150	-15	-45 to -150	-5 to -15
5	780	714	-40	-5	707	-73	-9.4	674	-106	-13.6	-40 to -106	-5 to -14
6	620	604	-16	-2.6	583	-37	-6	561	-59	-9.5	-16 to -59	-3 to -10
7	520	509	-11	-2.1	495	-25	-5	480	-40	-7.7	-11 to -40	-2 to -8
8	450	440	-10	-2.2	432	-18	-4	418	-32	-7	-10 to -32	-2 to -7
9	400	388	-12	-3	381	-19	-4.8	374	-26	-6.5	-12 to -26	-3 to -7

drag coefficient (C_4) is increased from its current value of 0.0228 in MRI to 0.04 in CTR. The SAE-recommended value is 0.0317, in the middle between the MRI and CTR values.

In Table 4 and Figure 3 the critical lengths of grade obtained from CTR are compared with those of AASHTO, under the previously stated assumptions, for different GVW/NHP ratios and percentage grades. The CTR values are consistently less than the corresponding AASHTO results, as expected. The discrepancies become more and more significant as the percentage grade decreases. For a 2 percent grade the difference could reach 27 percent, and it is about 16 percent on the average. In absolute terms, the expected maximum difference is 635 ft and the expected average is about 378 ft. These are indeed extremely large differences. For a 1 percent grade (Figure 3) the difference is expected to reach about 10 times that for a 2 percent grade. For higher grades, up to 5 or even 6 percent, significant differences could still be observed. The expected maximum differences are 254 ft (18 percent), 150 ft (15 percent), 106 ft (14 percent), and 59 ft (10 percent) for grades of 3, 4, 5, and 6 percent, respectively. On the average (that is, for GVW/NHP = 350 and C_4 = 0.0317), expected differences are 378 ft (16 percent), 153 ft (11 percent), 96 ft (9.6 percent), and 73 ft (9.4 percent) for grades of 2, 3, 4, and 5 percent, respectively. The differences between CTR and AASHTO results become less significant for grades of 7 percent and more or GVW/NHP ratios closer to 300, or both.

Probably the most important conclusion that could be extracted from Table 4 and Figure 3 is that if it

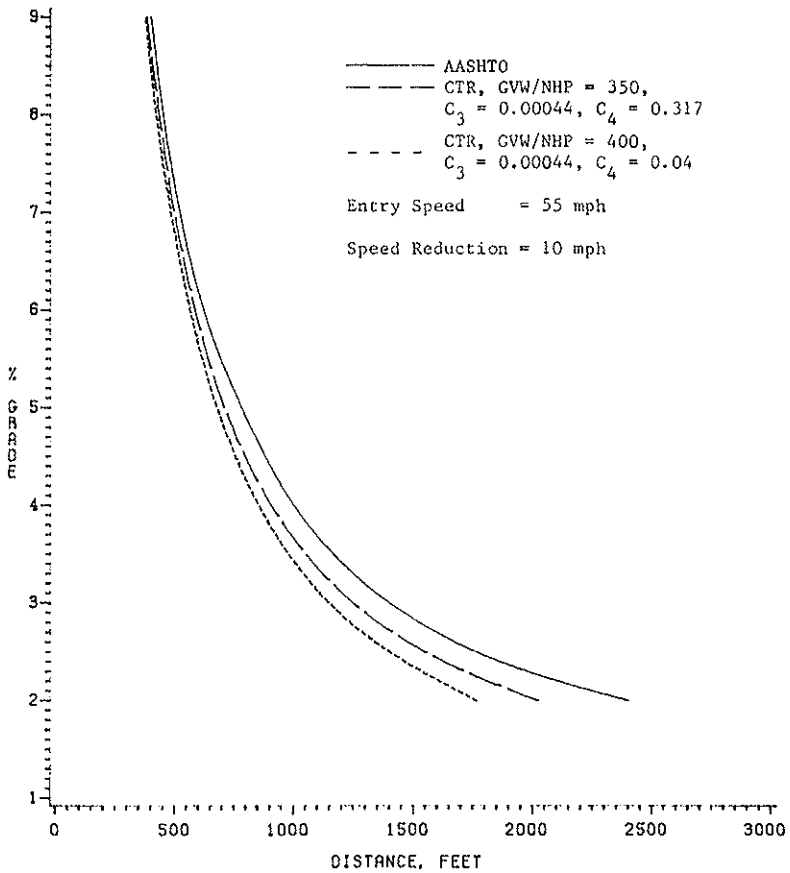

FIGURE 3 Comparison of AASHTO and CTR critical lengths of grade assuming an increase in aerodynamic drag coefficient (C_4) only.

is assumed that the aerodynamic drag coefficient (C_4) for LCVs is increased by about 75 percent of its MRI modified value, gradability results could be quite different from AASHTO design criteria. The reductions of critical lengths of grades could reach 27 percent for a 2 percent grade (an absolute difference of 635 ft). Significant differences could be expected for grades of up to 5 or even 6 percent and for GVW/NHP ratios around or greater than 350.

Impacts of the Increase in Rolling Resistances Coefficient (C_3) Only

In this case it was assumed that the aerodynamic drag coefficient is unchanged (C_4 = 0.0228) while the rolling losses coefficient (C_3) is increased from its current value of 0.00044 in the MRI study to 0.003 in CTR. Within this range a value of 0.001 (between MRI and SAE) and the SAE-recommended value of 0.001982 were considered.

In Table 5 and Figure 4 the critical lengths of grade obtained from the CTR study are compared with those of AASHTO, under the previously stated assumptions, for different GVW/NHP ratios and percentage grades. The trend of the CTR results in this subsection is similar to that in the preceding subsection, as expected. The CTR results are consistently less than those of AASHTO, and the differences between CTR and AASHTO increase as the percentage grade decreases and, of course, as the GVW/NHP ratio gets further above 300. The rate of increase, however, decreases as the GVW/NHP ratio increases.

Compared with those of Table 4 and Figure 3, the

TABLE 5 Comparison of Critical Lengths of Grade (ft) Between CTR and AASHTO Assuming an Increase in Rolling Losses Coefficient (C_3) Only, Entry Speed = 55 mph, and Speed Reduction = 10 mph

		CTR Minimum Difference			CTR Average Difference			CTR Maximum Difference				
		GVW/NHP = 300 C_3 = 0.001	Difference		GVW/NHP = 350 C_3 = 0.001982	Difference		GVW/NHP = 400 C_3 = 0.003	Difference		Expected Difference Range	Expected Difference Range
Grade (%)	AASHTO (ft)	C_4 = 0.0228 (ft)	Feet	Percentage	C_4 = 0.0228 (ft)	Feet	Percentage	C_4 = 0.0228 (ft)	Feet	Percentage	(ft)	(%)
1		15,898			4,091			2,750				
2	2,400	2,397	-3	-0	1,747	-653	-27	1,488	-912	-38	-3 to -912	-0 to -38
3	1,400	1,375	-25	-2	1,156	-244	-17	1,025	-375	-27	-25 to -375	-2 to -27
4	1,000	966	-34	-3.4	857	-143	-14.3	780	-220	-22	-34 to -220	-3 to -22
5	780	747	-33	-4.2	678	-102	-13	630	-150	-19	-33 to -150	-4 to -19
6	620	608	-12	-2	564	-56	-9	531	-89	-14	-12 to -89	-2 to -14
7	520	513	-7	-1.3	480	-40	-7.7	458	-62	-12	-7 to -62	-1 to -12
8	450	443	-7	-1.5	421	-29	-6.4	403	-47	-10	-7 to -47	-1 to -10
9	400	392	-8	-2	374	-26	-6.5	359	-41	-10	-8 to -41	-2 to -10

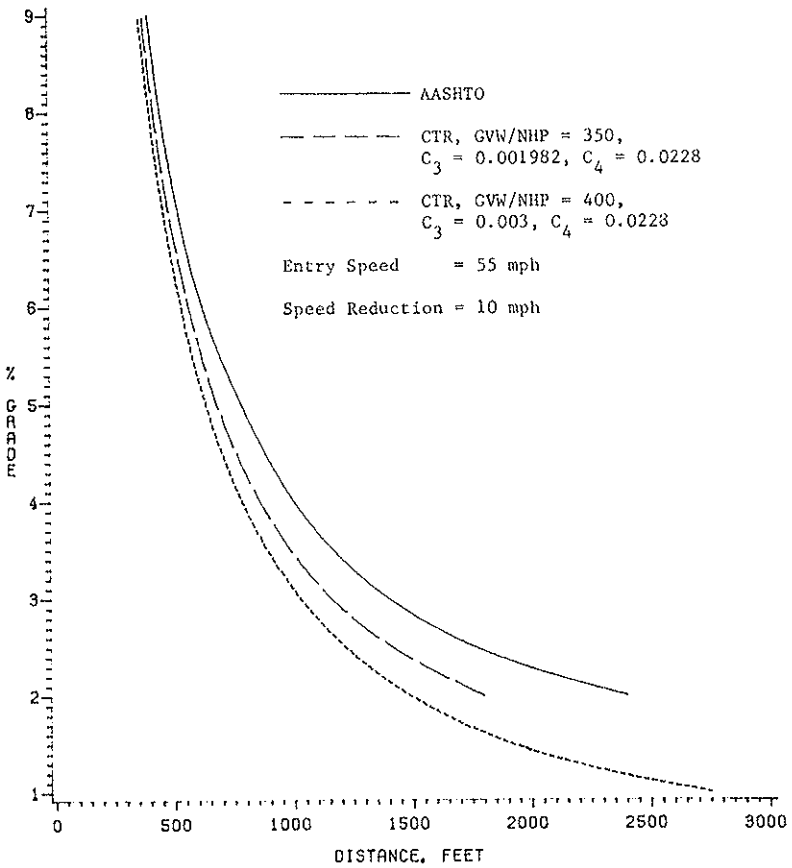

FIGURE 4 Comparison of AASHTO and CTR critical lengths of grade assuming an increase in rolling losses coefficient (C_3) only.

differences in Table 5 and Figure 4 are considerably greater. This is because the assumed values of C_3 in Table 5 imply more variability than do the assumed values of C_4 in Table 4.

Looking at the last two columns of Table 5, the differences between CTR and AASHTO could reach 38 percent (912 ft), 27 percent (375 ft), 22 percent (220 ft), 19 percent (150 ft), and 14 percent (89 ft) for grades of 2, 3, 4, 5, and 6 percent, respectively. On the average (for GVW/NHP = 350 and C_3 = 0.001982), expected differences are about 27 percent (653 ft), 17 percent (244 ft), 14 percent (143 ft), and 13 percent (102 ft) for grades of 2, 3, 4, and 5 percent, respectively. These expected maximum as well as average differences, which range between 912 and 90 ft for grades of 2 through 5 percent and GVW/NHP ratios 400 to 350, are certainly significant and should have strong impacts on the current AASHTO criteria, if it turns out that indeed the rolling losses coefficient for LCVs should be adjusted to the assumed values.

Impacts of Increases in Both Rolling and Aerodynamic Losses Coefficients (C_3 and C_4)

In this subsection it was assumed that coefficients C_3 and C_4 are increased from their current values in the MRI study to reflect the expected increases in aerodynamic and rolling resistances for LCVs compared with regular five-axle trucks. This is the basic underlying assumption of the analysis.

Table 6 gives and Figure 5 shows a comparison of AASHTO criteria and CTR results under alternative

TABLE 6 Comparison of Critical Lengths of Grade (ft) Between CTR and AASHTO Assuming Increases in Coefficients of Both Rolling and Aerodynamic Losses (C_3 and C_4), Entry Speed = 55 mph, and Speed Reduction = 10 mph

		CTR Minimum Difference			CTR Average Difference			CTR Maximum Difference				
Grade (%)	AASHTO (ft)	GVW/NHP = 300 C_3 = 0.001 C_4 = 0.0317 (ft)	Difference Feet	Difference Percentage	GVW/NHP = 350 C_3 = 0.001982 C_4 = 0.0317 (ft)	Difference Feet	Difference Percentage	GVW/NHP = 400 C_3 = 0.003 C_4 = 0.04 (ft)	Difference Feet	Difference Percentage	Expected Difference Range (ft)	Expected Difference Range (%)
1		7,078			3,440			2,288				
2	2,400	2,124	-276	-11.5	1,658	-742	-31	1,342	-1,058	-44	-276 to -1,058	-11 to -44
3	1,400	1,283	-117	-8.4	1,101	-299	-21.4	952	-448	-32	-117 to -488	-8 to -32
4	1,000	919	-81	-8.1	824	-176	-17.6	740	-260	-26	-81 to -260	-8 to -26
5	780	718	-62	-8	659	-121	-15.5	604	-176	-23	-62 to -176	-8 to -23
6	620	590	-30	-5	549	-71	-11.5	509	-111	-18	-30 to -111	-5 to -18
7	520	498	-22	-4	469	-51	-10	443	-77	-15	-22 to -77	-4 to -15
8	450	432	-18	-4	414	-36	-8	392	-58	-13	-18 to -58	-4 to -13
9	400	385	-15	-3.8	366	-34	-8.5	348	-52	-13	-15 to -52	-4 to -13

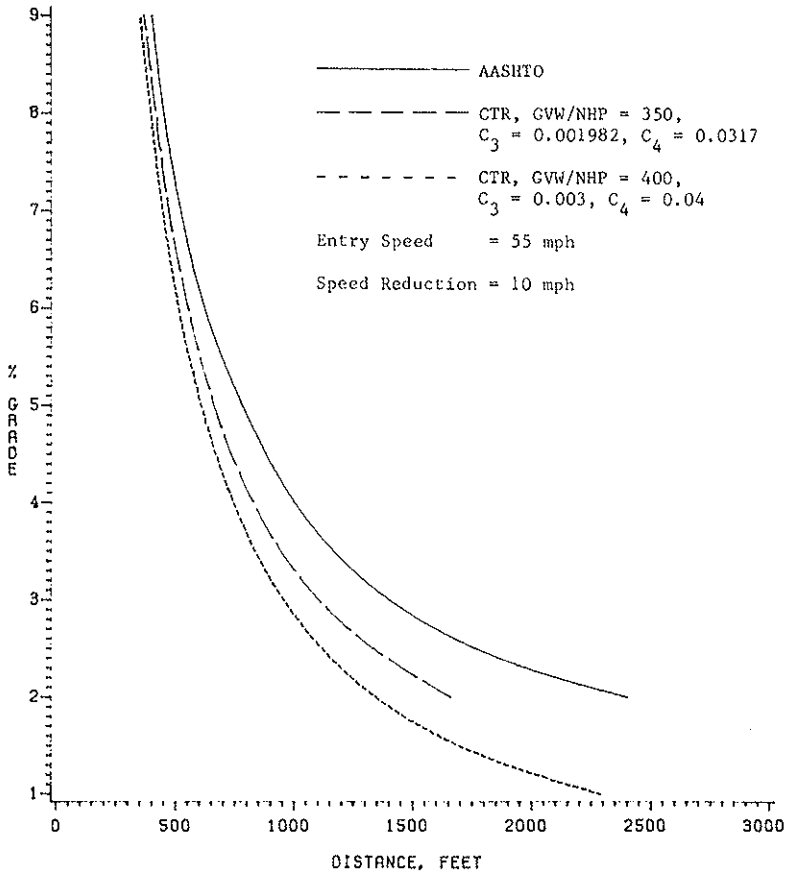

FIGURE 5 Comparison of AASHTO and CTR critical lengths of grade assuming increases in both rolling and aerodynamic losses coefficients (C_3 and C_4).

assumptions for C_3 and C_4 for different GVW/NHP ratios and percentage grades. The trend of CTR results is, again, similar to that encountered in preceding subsections. In this subsection, however, the differences between AASHTO and CTR results are becoming extremely large because both coefficients were allowed to increase simultaneously.

Looking at Table 6, the differences in critical lengths of grades between CTR and AASHTO ranges from 11 to 44 percent (276 to 1,058 ft), 8 to 32 percent (117 to 448 ft), and 8 to 26 percent (81 to 260 ft) for grades of 2, 3, and 4 percent, respectively. This indicates that even the expected minimum differences (for GVW/NHP = 300, C_3 = 0.001, and C_4 = 0.0317) for lower grades are significant. Of course, the expected maximum differences (for GVW/NHP = 400, C_3 = 0.003, and C_4 = 0.04) are quite large for grades of up to 6 percent and are still significant for higher grades. The expected average differences (for GVW/NHP = 350, C_3 = 0.001982, and C_4 = 0.0317) are quite high for grades of up to 5 percent and are still significant for higher grades.

These results indicate that differences between critical lengths of grades for LCVs and AASHTO-recommended values could be as large as 44 percent (1,058 ft) for a 2 percent grade depending on the values of aerodynamic and rolling losses coefficients and GVW/NHP ratios of these LCVs. This "extreme" difference is expected to decrease for grades of more than 2 percent and GVW/NHP ratios below 400.

SUMMARY AND CONCLUSIONS

The current AASHTO (1) criteria for determining critical lengths of grades and climbing lane design for the safe and efficient operation of heavy vehicles assumes a GVW/NHP ratio of a typical five-axle (3-S2) truck to be about 300 lb/hp. Operational tests conducted by Caltrans (2) indicate that LCVs such as double 48-ft and triple 28-ft truck combinations with an overall length of 120 ft and an effective width of 102 in. are generally slower than other typical five-axle trucks, particularly when GVW/NHP ratios are greater than 350.

The main objective of this paper was to gain more insight into the performance of LCVs on highway grades and to investigate the possible impacts that the operation of LCVs on grades might have on the current AASHTO design criteria.

To achieve this objective the factors that may influence performance of vehicles on grades were reviewed and the relatively more important factors for LCVs were highlighted. The issue of the prediction of the performance of LCVs on grades was then addressed. This involved discussion of existing approaches, selection of a particular approach for the study, detailed description of the selected approach, and actual application of the approach to LCVs. The results of the application were analyzed in view of the current AASHTO criteria.

The major conclusions of this paper may be stated as follows:

1. The practical maximum GVW for LCVs is consistently greater than 100,000 lb up to 138,000 lb. NHP ranges between 300 and 330 hp and could exceed 400 hp. A GVW/NHP ratio between 300 and 400 lb/hp would be considered normal and could, occasionally, exceed 400.

2. The larger side area of LCVs should increase their aerodynamic drag. The increased GVW and number of tires for LCVs should increase their rolling resistances. These increased aerodynamic and rolling resistances could be considerable. The rolling losses coefficient (C_3) in the simulation model could increase from its current value of 0.00044 (the MRI modified value) to 0.003. The aerodynamic drag coefficient (C_4) could increase from its MRI modified value of 0.0228 to 0.04. Actual representative values for LCVs can only be obtained from actual field tests.

3. If it is assumed that the rolling and aerodynamic coefficients (C_3 and C_4) are unchanged from their current MRI values for regular five-axle trucks, the results (CTR) of critical lengths of grades (assuming entry speed = 55 mph and speed reduction = 10 mph) at GVW/NHP = 300 are almost identical with the corresponding AASHTO values for grades of 4 percent and more, as expected. For 3 and 2 percent grades, however, the CTR values are greater than the AASHTO values by about 3.9 and 10 percent, respectively. This slight overestimation of gradability on lower grades can be explained by noting that effects of percentage grade and GVW tend to decrease on lower grades, where the effects of the vehicle's power capabilities become predominant, and that the net horsepower used in the analysis (NHP = 315 hp) is indeed higher than the value for regular trucks with the same GVW/NHP = 300 ratio. This implies that CTR results are conservative with respect to AASHTO criteria. This should strengthen the analysis.

4. In the comparison between critical lengths of grades obtained from CTR and the corresponding AASHTO values, a certain trend for the differences depending on the GVW/NHP ratio and the percentage grade was observed. Of course the differences increase as the GVW/NHP ratio increases, but the rate of increase of differences decreases for higher GVW/NHP ratios. As the percentage grade increases, the differences decrease both in absolute value and percentage-wise. In other words, more variability in the results is expected on lower percentage grades and for GVW/NHP ratios closer to 300.

5. Under the assumption of no change in rolling and aerodynamic losses coefficients for LCVs compared with regular five-axle trucks, the critical lengths of grades can still be significantly less than AASHTO criteria. For GVW/NHP = 400, these reductions could reach 16 percent on a 2 percent grade (from 2,400 to 2,025 ft), 11 percent on a 3 percent grade (from 1,400 to 1,252 ft), and 9 percent on a 4 percent grade (from 1,000 to 909 ft).

6. Assuming that the rolling losses coefficient is unchanged (C_3 = 0.00044) while the aerodynamic drag coefficient (C_4) is increased from its current MRI value of 0.0228 to 0.04, more significant differences between CTR and AASHTO could be observed. Expected maximum reductions (for GVW/NHP = 400 and C_4 = 0.04) are 27 percent (635 ft), 18 percent (254 ft), 15 percent (150 ft), 14 percent (106 ft), and 10 percent (59 ft) for grades of 2, 3, 4, 5, and 6 percent, respectively. Expected average reductions in critical lengths of CTR compared with AASHTO (for GVW/NHP = 350 and C_4 = 0.0317) are 16 percent (378 ft), 11 percent (153 ft), 9.6 percent (96 ft), and 9.4 percent (73 ft) for grades of 2, 3, 4, and 5 percent, respectively. In other words, under the previously stated assumptions, gradability results for LCVs could be quite different from those of AASHTO.

7. Assuming that the aerodynamic drag coefficient is unchanged (C_4 = 0.0228) while the rolling resistances coefficient (C_3) is increased from its MRI value of 0.00044 to 0.003, the differences between CTR and AASHTO become more and more dramatic. Expected differences could be as much as 38 percent (912 ft), 27 percent (375 ft), 22 percent (220 ft), 19 percent (150 ft), and 14 percent (89 ft) for grades of 2, 3, 4, 5, and 6 percent, respectively. On the average (for GVW/NHP = 350 and C_3 = 0.001982), expected differences are about 27 percent (653 ft),

17 percent (244 ft), 14 percent (143 ft), and 13 percent (102 ft) for grades of 2, 3, 4, and 5 percent, respectively.

8. Assuming that both rolling and aerodynamic coefficients (C_3 and C_4) are increased from their MRI values to reflect the expected increases in rolling and aerodynamic resistances for LCVs compared with regular five-axle trucks, the resulting reductions in critical lengths of grades could be extremely high. Expected reductions of CTR compared with AASHTO critical lengths could be as high as 44 percent (1,058 ft), 32 percent (448 ft), 26 percent (260 ft), 23 percent (176 ft), and 18 percent (111 ft) for grades of 2, 3, 4, 5, and 6 percent, respectively. Expected minimum differences corresponding to GVW/NHP = 300, C_3 = 0.001, and C_4 = 0.0317 are still significant for lower grades. These differences are 12 percent (278 ft), 8.4 percent (117 ft), and 8.1 percent (81 ft) for 2, 3, and 4 percent grades, respectively.

9. Reductions of critical lengths for grades of 7 percent and greater could be at most 77 ft (15 percent), 58 ft (13 percent), and 52 ft (13 percent) on grades of 7, 8, and 9 percent, respectively.

10. The variability in the results for a 1 percent grade is enormous. Estimated critical length reductions could reach about 75 percent. The absolute values for critical lengths are, however, quite large in most cases. On the basis of the hypothesized results in this paper, as long as the length of a 1 percent grade is less than 2,500 ft, the performance of LCVs should be satisfactory in the majority of situations.

In summary, the CTR results indicate that critical lengths of grades for LCVs could be less than the AASHTO design values by as much as 1,058 ft on a 2 percent grade (44 percent less than the recommended 2,400 ft) for GVW/NHP = 400, C_3 = 0.003, and C_4 = 0.04. This extreme difference is expected to decrease for grades of more than 2 percent, GVW/NHP ratios below 400, C_3 values below 0.003, and C_4 values less than 0.04.

Notice that in the analysis no attempt was made to recommend specific values for "representative" LCVs. Instead, a sensitivity analysis was performed within certain "reasonable" ranges of values for GVW/NHP ratios, percentage grades, C_3 coefficients, and C_4 coefficients. The message conveyed through this analysis is that the operation of LCVs on grades could indeed have serious implications for the current AASHTO criteria for determining critical lengths of grade and climbing lane design. To make specific recommendations in this regard, actual field experimental data on the performance of different LCV types (such as turnpike doubles, turnpike triples, and Rocky-Mountain doubles) on grades have to be collected and analyzed. These field tests may consider, in addition to straight upgrades, operation along loop ramps at major interchanges. Operation on loop ramps is influenced by the combined effects of grade and curvature. Of course, the entry speed at loop ramps is considerably less than 55 mph. This combination of factors could have serious adverse consequences as far as the operation of LCVs is concerned unless appropriate changes in existing geometric design practices, if deemed necessary, are undertaken.

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APPENDIX --SIMULATION RESULTS

TABLE A-1 CTR Critical Lengths of Grades for GVW/NHP = 300

% Grade	GVW = 94500 lb			NHP = 315			GVW/NHP = 300					
	C_3 = 0.00044			C_3 = 0.001			C_3 = 0.001982			C_3 = 0.003		
C_4 →	0.0228	0.0317	0.04	0.0228	0.0317	0.04	0.0228	0.0317	0.04	0.0228	0.0317	0.04
1	--	11697.3	6597.4	15899.0	7078.7	5078.7	6173.2	4556.0	3685.8	4165.1	3391.5	2892.2
2	2647.1	2320.1	2080.1	2397.2	2124.5	1920.8	2056.3	1852.4	1695.8	1791.4	1634.6	1514.0
3	1455.2	1349.1	1264.8	1375.1	1283.2	1206.3	1258.0	1177.6	111.7	1155.7	1086.3	1031.3
4	1006.9	955.5	911.5	966.9	919.1	878.7	908.1	864.3	830.9	853.2	816.4	783.5
5	769.6	740.1	714.3	747.6	718.2	692.5	710.9	685.1	663.0	677.8	652.3	633.7
6	623.3	604.8	586.4	608.6	590.1	571.9	583.1	568.0	549.9	560.9	546.0	531.3
7	524.5	509.8	498.5	513.5	498.9	487.7	495.2	484.0	472.9	480.3	465.8	456.1
8	451.4	440.3	432.7	444.0	433.0	425.4	432.7	421.8	414.3	418.2	410.6	399.9
9	399.8	389.0	381.6	392.5	385.1	377.7	381.6	374.2	366.8	370.6	363.3	359.3

TABLE A-2 CTR Critical Lengths of Grades for GVW/NHP = 350

% Grade	GVW = 350			NHP = 315			GVW/NHP = 350					
	C_3 = 0.00044			C_3 = 0.001			C_3 = 0.001982			C_3 = 0.003		
C_4 →	0.0228	0.0317	0.04	0.0228	0.0317	0.04	0.0228	0.0317	0.04	0.0228	0.0317	0.04
1	7712.5	5604.3	4494.2	5793.7	4550.5	3800.5	4091.9	3440.1	2999.5	3153.7	2755.7	2465.9
2	2266.8	2022.3	1861.7	2048.4	1873.2	1734.5	1797.1	1658.5	1552.2	1593.1	1483.6	1395.8
3	1321.6	1248.0	1185.5	1259.1	1189.5	1130.9	1156.9	1101.6	1050.4	1069.1	1021.2	977.3
4	941.9	904.9	871.7	908.8	872.1	842.5	857.3	824.3	795.0	806.2	780.2	754.5
5	733.3	707.7	689.0	711.4	689.3	670.7	678.4	659.8	641.4	648.8	630.4	612.2
6	601.4	583.2	568.5	586.8	572.0	557.3	564.7	550.0	535.5	542.7	528.1	517.0
7	506.4	495.3	487.6	498.8	487.8	476.8	480.7	469.7	462.1	465.8	454.9	447.4
8	440.3	432.8	425.4	433.0	425.5	418.1	421.8	414.4	407.0	407.4	400.0	396.0
9	389.0	381.7	377.7	381.8	377.8	370.6	374.2	366.9	363.0	363.3	359.4	352.2

TABLE A-3 CTR Critical Lengths of Grades for GVW/NHP = 400

% Grade	GVW = 132,000			NHP = 330			GVW/NHP = 400					
	C_3 = 0.00044			C_3 = 0.001			C_3 = 0.001982			C_3 = 0.003		
C_4 →	0.0228	0.0317	0.04	0.0228	0.0317	0.04	0.0228	0.0317	0.04	0.0228	0.0317	0.04
1	5472.8	4519.6	3888.8	4486.5	3829.3	3369.8	3423.0	3027.8	2733.9	2750.2	2488.6	2288.4
2	2025.4	1882.4	1765.2	1879.2	1754.6	1652.1	1664.0	1565.1	1484.4	1488.7	1411.5	1342.1
3	1252.7	1194.1	1146.2	1194.1	1142.6	1098.5	1106.0	1058.6	1021.7	1025.5	985.3	952.2
4	909.0	876.2	850.3	876.2	846.9	824.5	828.3	802.6	780.5	780.8	758.7	740.1
5	711.6	693.1	674.9	693.1	674.8	659.9	660.3	645.4	630.7	630.9	616.1	604.8
6	586.9	572.4	561.3	572.4	561.2	550.1	550.4	539.3	531.6	531.8	520.8	509.9
7	499.0	488.1	480.6	488.1	480.5	473.1	473.3	465.8	485.4	458.5	451.0	443.7
8	433.1	425.7	418.5	425.7	418.4	414.5	414.6	407.3	403.4	403.5	396.3	392.4
9	385.2	378.0	374.2	378.0	374.1	367.0	367.1	363.3	359.5	359.6	355.8	348.7

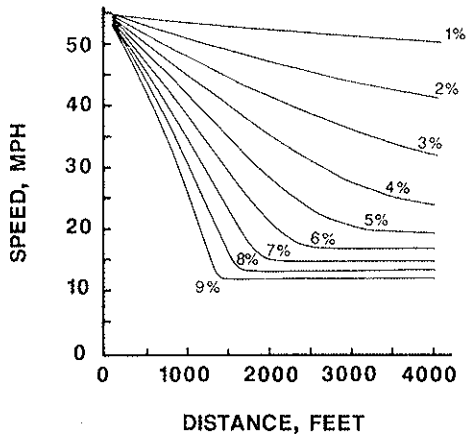


FIGURE A-1 Speed-distance profile for GVW = 94,500 lb, NHP = 315 hp, C_3 = 0.00044, C_4 = 0.0228, and GVW/NHP = 300.

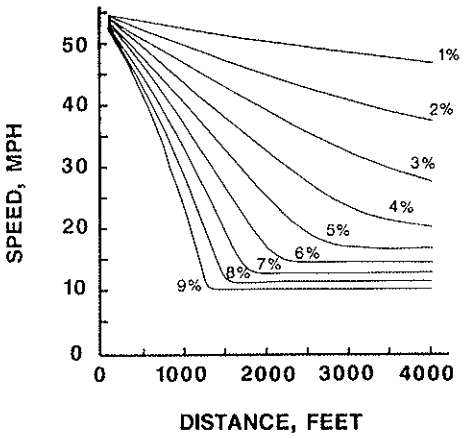


FIGURE A-2 Speed-distance profile for GVW = 110,250 lb, NHP = 315 hp, C_3 = 0.00044, C_4 = 0.0317, and GVW/NHP = 350.

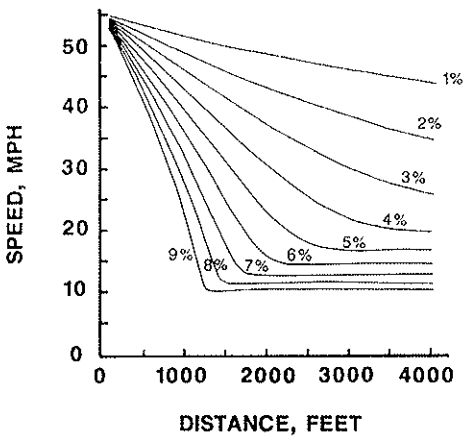


FIGURE A-3 Speed-distance profile for GVW = 110,250, NHP = 315 hp, C_3 = 0.001982, C_4 = 0.0317, and GVW/NHP = 350 (expected average).

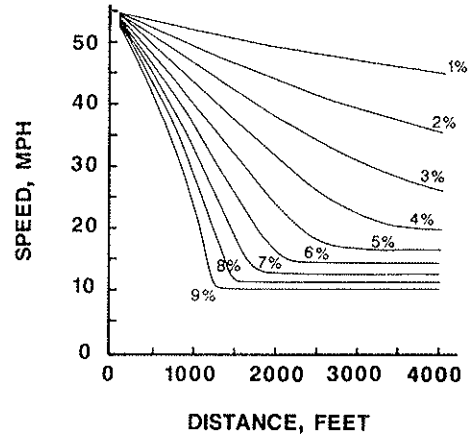


FIGURE A-4 Speed-distance profile for GVW = 110,250 lb, NHP = 315 hp, C_3 = 0.001982, C_4 = 0.0228, and GVW/NHP = 350.

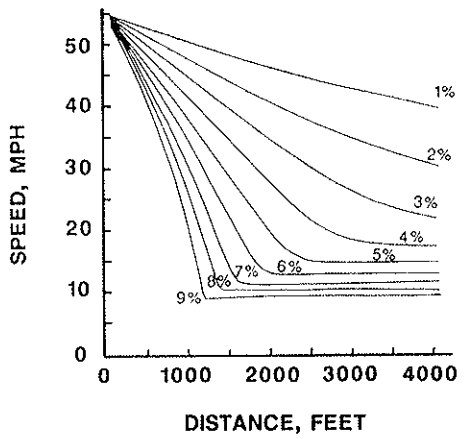


FIGURE A-5 Speed-distance profile for GVW = 132,000 lb, NHP = 330 hp, C_3 = 0.003, C_4 = 0.04, and GVW/NHP = 400 (expected worst).

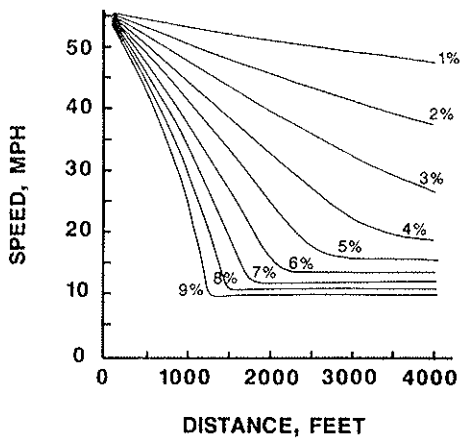


FIGURE A-6 Speed-distance profile for GVW = 132,000 lb, NHP = 330 hp, C_3 = 0.00044, C_4 = 0.0228, and GVW/NHP = 400.

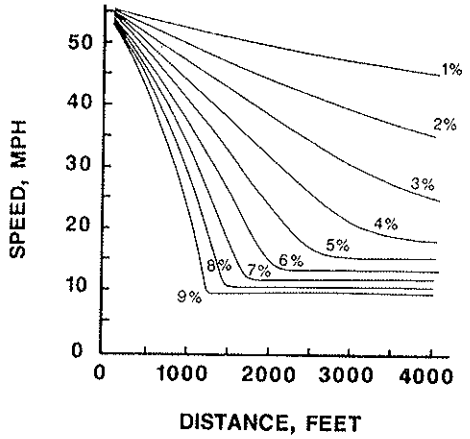


FIGURE A-7 Speed-distance profile for GVW = 132,000 lb, NHP = 330 hp, C_3 = 0.00044, C_4 = 0.04, and GVW/NHP = 400.

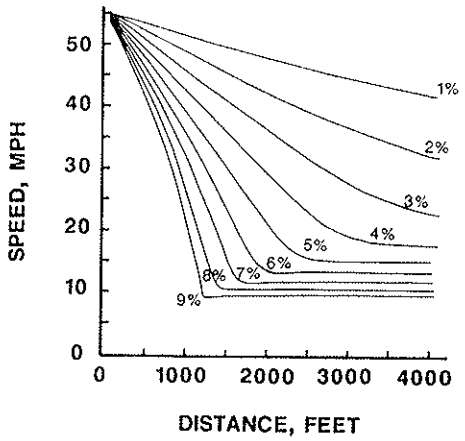


FIGURE A-8 Speed-distance profile for GVW = 132,000 lb, NHP = 330 hp, C_3 = 0.003, C_4 = 0.0228, and GVW/NHP = 400.

Influence of the Geometric Design of Highway Ramps on the Stability and Control of Heavy-Duty Trucks

ROBERT D. ERVIN, CHARLES C. MacADAM, and MICHELLE BARNES

ABSTRACT

A research study is described in which accidents experienced by tractor-semitrailers on expressway ramps were found to depend largely on the interaction between highway geometrics and vehicle dynamic behavior. The accident rates of tractor-semitrailers on expressway ramps in five states were scanned to select 14 individual ramps that exhibited an unusual incidence of serious accidents involving these vehicles. The geometrics of each ramp were fully defined in a computer simulation in such a way that the dynamic behavior of example tractor-semitrailers could be examined. The results of combined study of accident data, simulated vehicle response, and geometric details of ramp design are presented. The findings of the study indicate that the maneuvering limits of certain trucks are quite low relative to those of automobiles so current practice in ramp design leaves an extremely small margin for control of heavy vehicles. The primary design issues are embodied in the nominal side friction factor achieved at each curve, the transition geometry, and the layout and signing of curve segments in order to assure that truck speeds are suitably reduced for negotiating small-radius curves.

The geometric design of highway ramps is guided by the design policy of AASHTO (1). These policies provide specific guidance on the relationships among curve radius, superelevation, transition sections, vehicle speeds, and other details that control ramp design. For a given anticipated ramp layout, there exists a range of variations, which are allowed within the design policy, in each design parameter. In the real world, ramps that are in service around

Transportation Research Institute, University of Michigan, Ann Arbor, Mich. 48109.

the country exhibit even further variations in parameters because they were built before the tighter prescriptions of modern design or because certain physical or economic obstacles made strict adherence to the AASHTO policy unachievable. Accordingly, it is clear that highway ramp design varies widely around the country.

When considering the margins of safety that existing ramps provide for the operation of heavy-duty trucks, it is immediately apparent that the considerations that underlie ramp design recommendations in the AASHTO design manual (1) make little or no allowance for the special requirements of trucks. Indeed, it is clear that the geometric design of ramps is almost exclusively rationalized on the basis of automobile usage. This situation is in distinct contrast with the specific attention that is given to truck requirements in other areas of road design, such as climbing lanes, the width of turning roadways, corner radii at intersections, and certain sight distance considerations. The particular truck requirements of interest here are those that govern the limits of vehicle stability and control. Thus, both because of the variations in design that exist from one ramp to the next and because even the recommended design policies take no particular note of truck stability and control limits, it appears to be reasonable to explore the possible conflicts that trucks may encounter in negotiating highway ramps.

Particular impetus for such exploration is given by the accident record for trucks in general, recognizing for example that the accident file of the Bureau of Motor Carrier Safety (BMCS) for 1980 shows that 9 percent of all jackknife accidents and 16.8 percent of all truck rollovers occur on ramps. Clearly, such percentages are much higher than the fraction of total highway miles represented by ramp sections. The influence of certain of the geometric design variables of ramps on accidents or operational aspects, or both, has been examined by many investigators in the past (2-15), although no one has focused a ramp-accident study specifically on trucks. Nevertheless, some studies (2) have found trucks to be underinvolved in the population of all aggregated ramp accidents relative to their presence in the traffic stream. Such findings, together with the indication in the BMCS data that trucks are overinvolved in loss-of-control accidents on ramps, may suggest that the main problem that trucks experience on ramps is that of controllability, although the potential for collision accidents involving trucks on ramps may be no worse, or even better, than that of other vehicles.

To examine truck controllability problems on ramps, and to relate them to geometric design, a project was conducted by the University of Michigan Transportation Research Institute (UMTRI) under sponsorship of the FHWA. This paper is a report on the prominent findings of that study that serve to identify the special types of conflict that occur.

METHODOLOGY

The study first sought to identify specific examples of highway ramps on which had occurred an inordinate number of loss-of-control types of truck accidents. Because it was determined that national-level accident files do not contain sufficient detail to enable identification of individual ramps, it was necessary to draw from the accident files of selected states in order to identify ramps for study. Because it was not possible to clearly determine accident rates because of a lack of exposure information, a "first cut" in selecting ramps was done on the basis of absolute numbers of truck accidents at individual sites. With the aid of automated data-processing capabilities, a number of states were able both to identify individual, heavily involved ramps and to supply hard-copy reports for each truck accident on the selected ramps. During examination of each of the individual accident reports for each candidate ramp, a total set of 15 ramps was selected. The participating states were California, Illinois, Maryland, Michigan, and Ohio.

Engineering drawings were obtained documenting the geometric design, posted speeds, and traffic control devices at each of the selected ramp sites. The individual accident reports from each ramp were then examined closely to locate the approximate point on the ramp at which the loss-of-control events appeared to be occurring. In general, it was possible to focus attention on a specific curve or transition area on each ramp. The geometric data needed to completely define the curvature, superelevation, and grade of each ramp section of interest were then provided as input to a comprehensive simulation of the dynamic behavior of heavy-duty trucks (16). The simulation model provides a 32-degree-of-freedom representation of a tractor-semitrailer, allowing the full range of steering and braking maneuvers over the three-dimensional roadway. The model is configured such that an active "driver" system steers the vehicle, following the lane centerline with response characteristics that are demonstrably like those of a real driver up to the control limit conditions (17). The validity of the simulation model has been demonstrated in various exercises that compared computed results with experimental measurements from full-scale tests (18-20).

Each of the selected ramps was examined by means of the simulated operation of tractor-semitrailers that were represented in two loading conditions, namely (a) a baseline loading placing the payload center of gravity (CG) at 83 in. above the ground--a value that is thought to characterize a large fraction of typical truck traffic and (b) a loading case with the payload CG at a height of 105 in., which is representative of various specialized tank vehicles as well as van trailers carrying a full cube load of homogeneous freight. The tractor-semitrailers were simulated at various speeds--some cases at the posted advisory speed value and some above--over each ramp. The gross motion response of the vehicle was then interpreted in terms of a likely loss-of-control outcome.

The simulation results, supported by various other research findings that generalize on the dynamic behavior of heavy vehicles, serve to identify certain aspects of ramp geometric design that tend to restrict the margins of safety available for truck operation. Five cases that serve to illustrate the more potentially significant of these aspects of ramp design will be discussed.

ILLUSTRATIVE CASES

Heavy-duty trucks and truck combinations suffer constraints on their maneuvering capability in negotiating ramps as a result of certain size parameters and also because of certain limitations in the mechanical performance of the vehicles and their components. In addition, it may be inferred from reading the hard-copy accident reports that a substantial number of truck drivers tend to take ramps too fast, perhaps because of the desire to keep up speed in anticipation of merging or simply because of a lack of appreciation for the small tolerance that some ramp designs afford for trucks exceeding the advisory speeds.

In the illustrative cases that follow, the cited

"problems" fall either into the category of inherent limitations in truck stability and control qualities or into the category in which truck driver behavior appears to frequently involve peculiar misjudgments. Each case is first characterized by the particular aspect of ramp design that appears to be connected with the truck control problem of interest.

Case 1 (side friction factor is excessive given the roll stability limits of many trucks)

The first case involves the exit ramp that is sketched in Figure 1. As shown, Curve 3 is preceded and followed by spiral transitions and is posted with an advisory speed of 35 mph. The R and J designations indicate the approximate points at which vehicles involved in rollover and jackknife accidents came to rest. At 35 mph, the 342-ft radius of this ramp curve yields a centripetal acceleration of 0.24 g. Although the lead-in spiral is 150 ft long (ample for full attainment of the 0.28 ft/ft superelevation of the curve), the full superelevation is not developed until almost completely through the curve. Thus, at the point of entry of the steady curve, the superelevation level (e) is only 0.03 ft/ft, and the side friction factor (f) at that point is 0.21. Although it is unusual and perplexing to find a spiral transition that provides such an incomplete development of superelevation at the point of curvature, it is general practice on non-spiraled transitions to have achieved only one-half to two-thirds of the full superelevation level at the initial point of curvature.

Shown in Figures 2 and 3 are simulation results that illustrate the dynamic response at 35 and 40 mph, respectively, of a tractor-semitrailer that is loaded with freight in the high CG configuration (payload mass center at 105 in.) and that is operated over the cited curve. The results show that the vehicle at 35 mph experiences a near rollover, with a large amount of load being transferred from the right to the left tires. The transient character of the maneuver is such, however, that the roll response has not fully developed during the brief duration of the peak lateral acceleration level. Thus the vehicle "just squeaks by" at the posted speed by virtue of the relatively short-lived peak demand condition. In the 40 mph case (Figure 3) it can be seen that the tire loads on the right have reached zero at approximately 5.5 sec into the run--at which time the vehicle is approximately 50 ft beyond the leading end of the constant-radius curve. Although the zero-load condition on the tractor's inside wheels signals an imminent rollover, the body of the vehicle would not actually strike the ground for another 100 ft or so.

Although, at first note, it appears surprising that a common commercial vehicle will nearly roll over at the posted ramp speed on a primary U.S. highway, it is instructive to examine the margin of safety that is reflected in the side friction factor that pertains to the cited curve. Shown in Figure 4 is a diagram of the components that make up the instantaneous side friction factor at the advisory speed of 35 mph, plotted as a function of the longitudinal position along the ramp section. The figure presents the centripetal acceleration (e + f), the side friction factor (f), and a suggested "likely" side friction demand curve that is 15 percent above the f curve, reflecting the level of steering fluctuations that has been measured in tests of the normal driving of a tractor-semitrailer through expressway ramps (21). Because superelevation is not fully developed along the spiral transition, the peak side friction factor of 0.21, at the point of

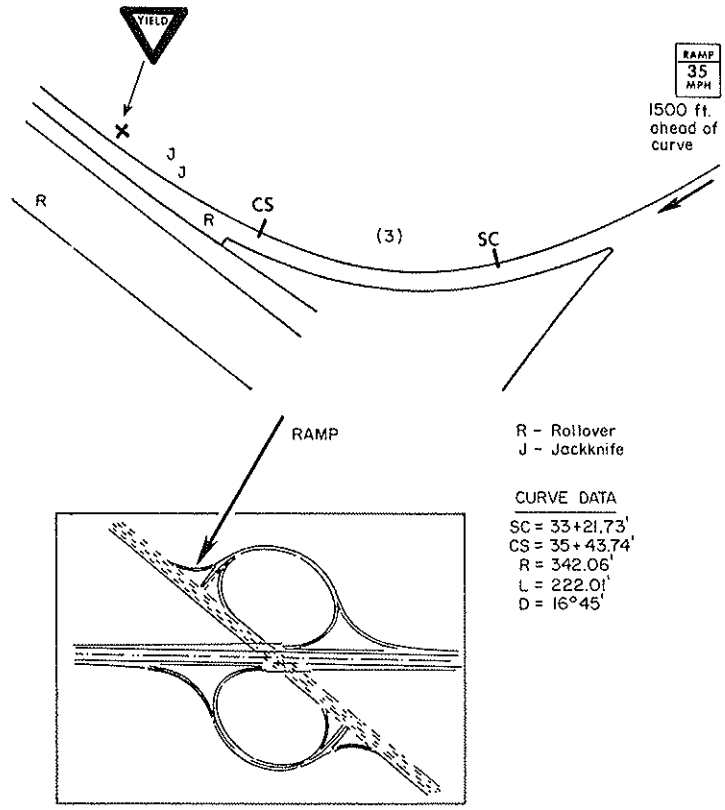


FIGURE 1 Layout of site that poses a challenge to truck roll stability level.

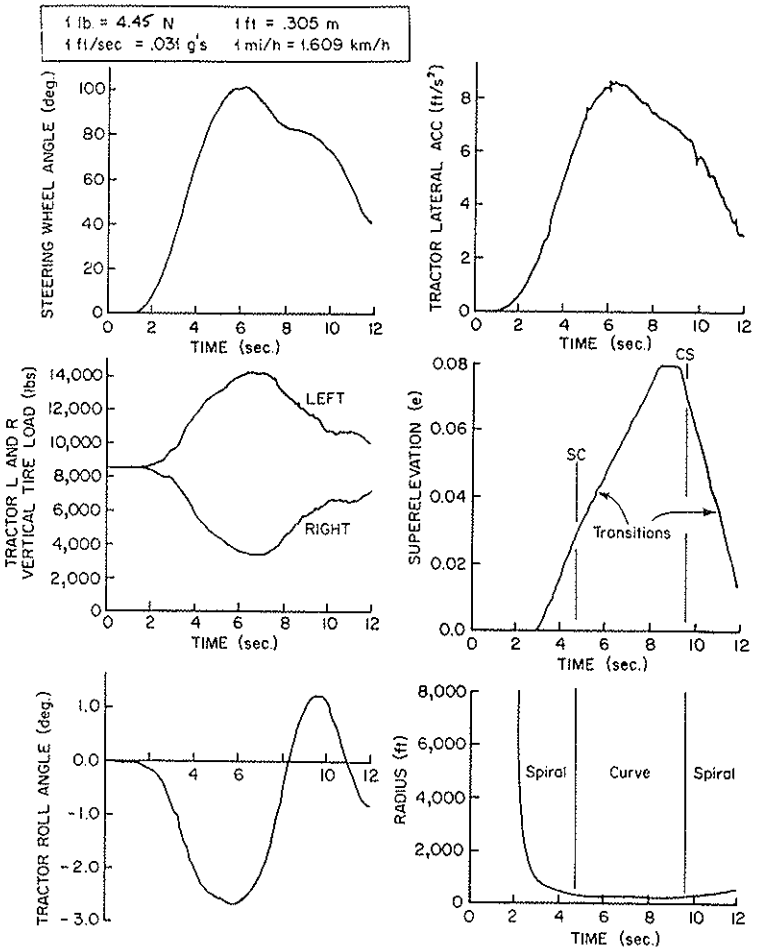


FIGURE 2 Vehicle response variables for travel through the ramp of Figure 1 at 35 mph.

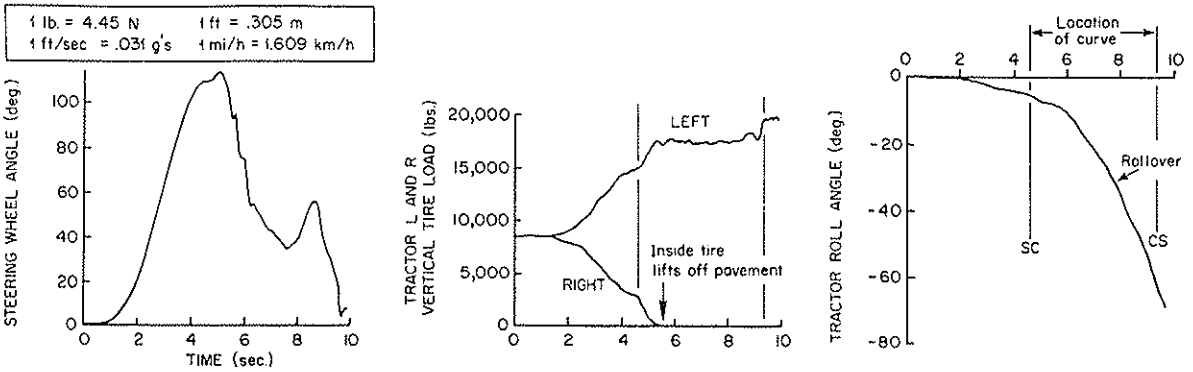


FIGURE 3 Response variables showing rollover at 40 mph.

curvature (SC), corresponds to a demand level of 0.24, allowing for steering fluctuations. This demand level is essentially equal to the steady-state rollover threshold limit of fully loaded tractor-semitrailers that lie at the low end of the stability range of vehicles in common service (22).

To reconcile the clear hazard that such a curve will pose for many heavy-duty vehicles, it is useful to note, first, that at the final superelevation value of 0.08 ft/ft, the curve would be characterized by a nominal friction factor of 0.16. This value is in virtual compliance with the AASHTO recommendation of a maximum of 0.155 for the side friction value in curves posted at 35 mph. The first issue, then, concerns the basic matter of the suitability of a design policy that allows friction factor levels of 0.155 (or 0.16), recognizing that loaded heavy vehicles exhibit static rollover threshold levels as low as 0.24. A full discussion of this matter would require review of (a) the essential basis for the AASHTO policy on side friction factors and (b) the mechanics and operational realities that determine the roll stability levels of heavy commercial vehicles. Although no comprehensive treatise can be attempted here, a minor elaboration on each point is warranted.

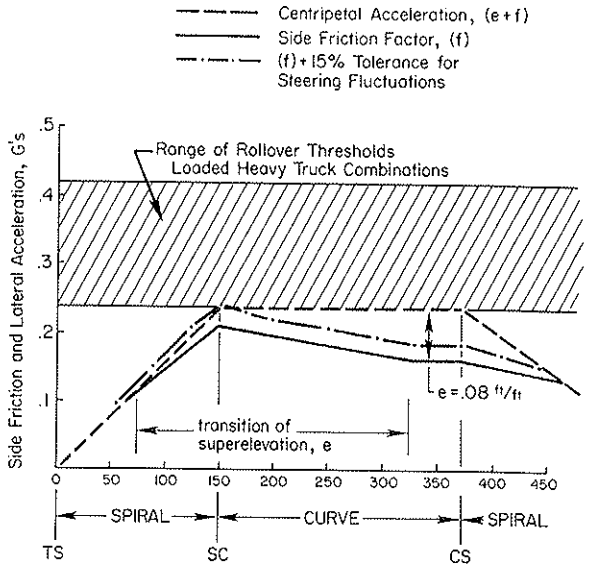


FIGURE 4 Elements of side friction demand compared with range of truck rollover tolerance for ramp curve layout of Figure 1.

The AASHTO policy (1) on side friction factor allowance is clearly based on consideration of (a) the proximity of the friction demand level to the lateral traction limits of automobiles, beyond which "side skidding" may occur and (b) the point of discomfort noted by automobile drivers. It is clear that the maximum recommended values for side friction factor have been set by AASHTO primarily to avoid driver discomfort. It is apparent that this policy intends a substantially larger margin than is achieved with heavy trucks that are at the lower (but by no means rare) end of the stability spectrum. For example, the discussion of the AASHTO policy in the green book (1) indicates that the effective limit condition is established by the maximum side friction capacity of automobile tires (as low as 0.35 at 45 mph) that can be sustained without skidding on wet pavements with smooth treads. Accordingly, the guidelines that limit the design value of side friction factor (to a maximum of 0.17 at 20 mph) appear to reflect a substantial degree of conservatism in behalf of automobiles. Indeed, the design policy for side friction factors has been derived to accommodate the limits of driver discomfort--at which levels the conservatism relative to side skidding is quite generous.

Considering the margin of safety for trucks, however, it is apparent that there also exists a fundamental difference between the respective probabilities that trucks and automobiles will "bump against" their respective maneuvering limits when traversing a demanding ramp. Although, on one hand, an automobile may be constrained by a 0.35 traction coefficient only when (a) smooth tires and (b) a poor pavement texture condition are combined with (c) wet weather, an adversely loaded truck will be continually constrained by its low rollover threshold characteristic as it goes down the road. Accordingly, it can be seen not only that the truck margin of safety on AASHTO-recommended ramps can be exceedingly narrow, in absolute terms, compared with the margins provided for automobiles but also that the risk of loss of control for certain trucks is continual rather than temporally dependent on maintenance factors and weather.

The low stability level of trucks derives, of course, from the height (H) of the center of gravity of the combined payload and tare vehicle relative to the track width (T) and to a host of other sensitivities involving the compliances of tires, suspensions, fifth wheels, and frames (23). Perhaps part of the reason that truck stability limits may have been traditionally overestimated, and thus dismissed in considerations of highway design, is that the vehicle was considered to be effectively rigid in roll, such that the roll stability limit, in g's, would be simply T/2H. If it had been assumed that trucks were as stable as the T/2H figures suggest, with minimum values around 0.45 g's, it would have been reasonable to conclude that the skidding limit of approximately 0.35 for automobiles constituted the effective design condition. Because of the compliant elements in actual trucks, however, rollover occurs at approximately 60 percent of the T/2H values (22). Shown in Figure 5, for example, are five common vehicle loading arrangements with their accompanying roll stability limits. Clearly, a number of common freight loadings render vehicle rollover threshold levels that are quite near the levels of side friction factors that prevail in the Case 1 example.

Although the transition of superelevation in this example is nonideal, and certainly disapproved of by AASHTO as a design practice, the fact that a zero margin of safety exists with some trucks should not be dismissed as attributable simply to the transition anomaly. For the more common cases in which superelevation is transitioned without spirals, the AASHTO-preferred method would have two-thirds of the superelevation achieved at the point of curvature. Even this policy would still allow a side friction factor as high as 0.20 in the transition portion of the curve, thus yielding 0.23 as the effective side friction demand level, allowing for steering fluctuations. Thus it appears that the problem that led to the identification of the Case 1 ramp as heavily involved in truck loss-of-control accidents is (a) understandable in terms of ramp geometry and (b) rather generally anticipated for ramp curves that are built to the limits of the recommended AASHTO practice.

It is also worthwhile to note that AASHTO design policy for low-speed urban streets allows side friction factors up to 0.30! Such a level will surely yield rollover in a large fraction of the population of loaded commercial vehicles.

Case 2 (truckers assume that the ramp advisory speed does not apply to all curves on the ramp)

One aggravating aspect of the truck loss-of-control problem on ramps is that many ramps involve multiple curved segments that have differing side friction factor demands, although only one ramp speed is generally posted. As a consequence, it appears that truckers occasionally assume, at some point along the ramp, that they have now passed the curve or curves that warranted the low value for the posted speed. Subsequently, they begin to speed up in preparation for the merging task, only to find that the remaining curve is at least as demanding of the low advisory speed condition as was the preceding portion of the ramp.

A clear case in point is the ramp shown in Figure 6--a loop with four curves within a partial cloverleaf, rural interchange. The ramp is posted at 25 mph and has two rather sharp curves at either end and two intermediate curves with more moderate

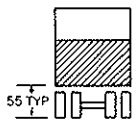
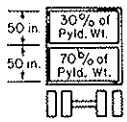
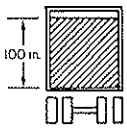
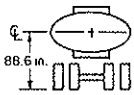
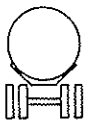
CASE	CONFIGURATION	WEIGHT (lbs) GVW	PAYLOAD CG HEIGHT (in)	ROLLOVER THRESHOLD (G's)
A.	 Full Gross, Medium-Density Freight (34 lb/ft³)	80,000	83.5	.34
B.	 "Typical" LTL Freight Load	73,000	95.0	.28
C.	 Full Gross, Full Cube, Homogeneous Freight (18.7 lb/ft³)	80,000	105.0	.24
D.	 Full Gross Gasoline Tanker	80,000	88.6	.32
E.	 Cryogenic Tanker (He_2 and H_2)	80,000	100.	.26

FIGURE 5 Loading data and resulting rollover thresholds for example tractor-semitrailers at full load.

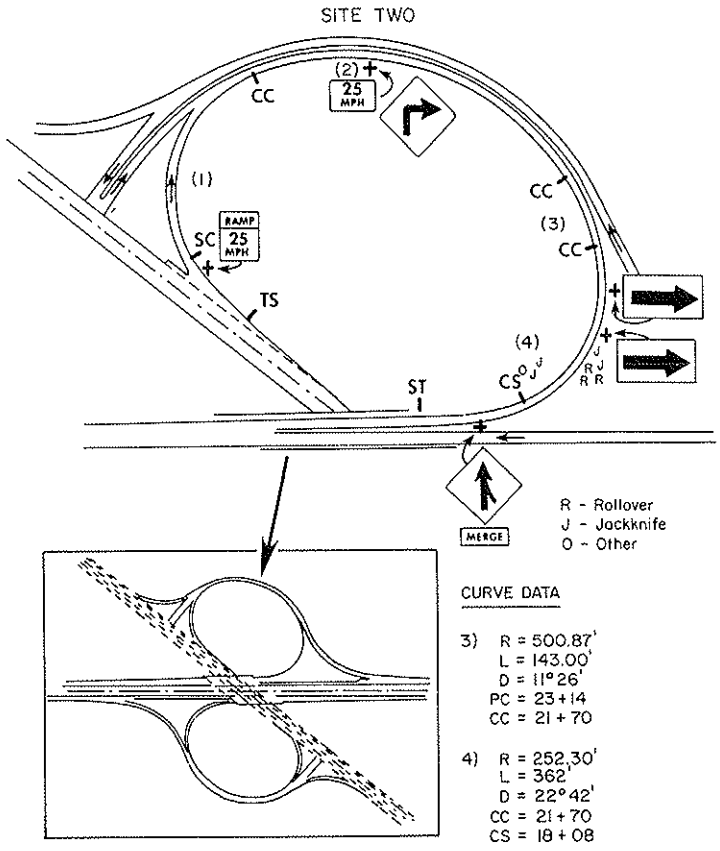


FIGURE 6 Layout of compound curve ramp.

radii. Listed in the following table are the essential data for each of the four curves.

Curve No.	Radius (ft)	Length (ft)	Side Friction Factor
1	250	435	0.09
2	520	993	0.00
3	500	144	0.003
4	252	362	0.09

Spiral transitions to the tangent legs at both ends of this ramp provide that both Curves 1 and 4 are superelevated at 0.08 ft/ft throughout their lengths. Thus the nominal values listed for side friction factor are also the maximum values.

The truck accidents that occur on this ramp are all clustered at the approximate midlength location of Curve 4. Because Curves 1 and 4 are both characterized by identical values of side friction factor, it can only be surmized that truck drivers (a) reasonably satisfy the speed requirements of Curve 1 but then (b) misjudge the continuing need for retaining the low advisory speed while traveling the 1,100 ft through the mild curves (Curves 2 and 3). The analysis shows that a high-CG tractor-semitrailer such as cited in Case 1 would roll over in Curve 4 if the driver permitted his speed to exceed 34 mph.

The number of jackknife accidents reported at this site equals the number of rollover incidents, which suggests that heavy braking is probably being applied when the driver perceives that general loss of control is imminent. Although this site was unusual because the posted speed was mandated by the designs of both the initial and the final curves on the ramp, a number of other problem sites were also identified where drivers apparently lost the conviction that the speed advisory still applied later in the ramp. Again, the trucker is peculiarly vulnerable in the event of such a misjudgment because of the small tolerance that the low-stability vehicle has for increased side friction factors.

Also, it appears reasonable to hypothesize that the very short lengths of acceleration lane available for bringing a fully loaded rig up to speed serve to encourage the driver to achieve as much speed as possible within the ramp before merging. For example, it was noted that a typical 80,000-lb tractor-semitrailer combination powered by a 250-hp engine will require in excess of 5,000 ft to accelerate from a ramp speed of 25 mph to 50 mph (24). Indeed, even the provision of an AASHTO-recommended acceleration lane, 1,100 ft in length (1), does virtually nothing to lessen the truck driver's concerns over merging with minimum disruption of through traffic. Thus, although the truck driver who exceeds the posted ramp speed can be criticized, it appears more realistic to observe that the sum of the highway geometric constraints imposed in this case has "boxed in" the driver and, perhaps, promoted the possibility of misjudgments.

Case 3 (deceleration lane lengths are deficient for trucks, resulting in excessive speeds at the entrance of sharply curved ramps)

The 1965 AASHO blue book (25) gives a definitive background rationale behind the recommended lengths of deceleration lanes. Notwithstanding the careful basis that is developed for designing such lanes to meet the needs and comfort threshold of automobile drivers, both the blue and green book specifications for deceleration lanes place a substantial burden on the stopping capability of many heavy-duty truck combinations. The background figures in the blue book reveal that the "comfortable" level of deceleration for automobile drivers slowing from 55 mph is 0.24 g's. The recommended lengths for deceleration lanes are calculated to allow approximately 3 sec of deceleration of the vehicle in gear, followed by braking at the "comfortable" automobile rate. The blue book does note that trucks require longer stopping distances than do automobiles to decelerate for the same difference in speed but finds longer allowances for deceleration lanes unwarranted because "average speeds of trucks are generally lower than those of passenger cars." Although the green book does not restate the observation concerning truck speeds, the newer recommendations for length of deceleration lane are virtually identical to those in the 1965 policy. Further, it appears reasonable to observe that average truck speeds on U.S. highways today are at least equal to, and perhaps exceed, those of automobiles.

The study of truck accidents on ramps has indicated cases in which the deceleration lengths available for trucks appear to be patently inadequate. The cases in which the problem becomes pronounced are those in which the ramp incorporates a rather sharp curve right at the end of the deceleration lane such that the low value of advisory ramp speed must be achieved very quickly after departure from the through roadway. Shown in Figure 7 is an example of such an exit ramp with a 249-ft radius and a max-

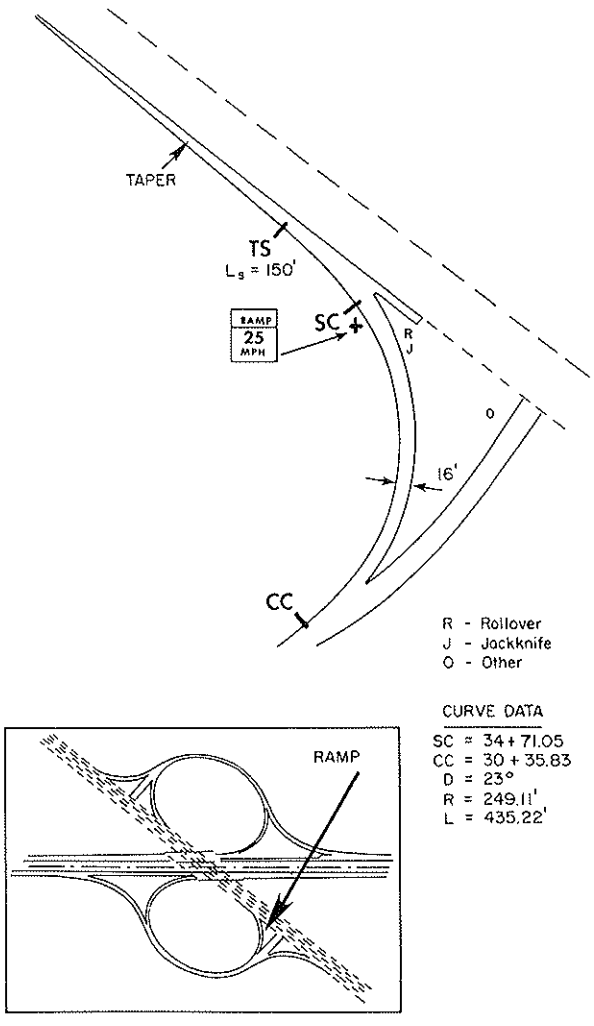


FIGURE 7 Layout of ramp with tapered deceleration lane.

imum superelevation value of 0.08 ft/ft. The side friction factor has a peak value of 0.13 at the advisory speed, given a transition that achieves approximately 50 percent of the full development of superelevation at the point of curvature. The tapered exit begins 375 ft ahead of the point of curvature and thus requires a nominal deceleration of 0.21 g's even if braking begins immediately on entry to the curve. The 0.21-g requirement allows no distance for delay in brake application beyond the leading edge of the taper and assumes that the vehicle will begin decelerating while still placed fully in the through lane. According to the AASHTO recommendations, this deceleration lane is extremely short and provides only approximately 100 ft of roadway that should be "counted" for deceleration in recognition that the acknowledged deceleration lane begins only at the point at which the taper has progressed 12 ft from the right edge of the through lane.

The penalty paid by truckers who fail to achieve the required speed on entering this curve is, of course, most likely to be rollover. The accident data show both rollover and jackknife accidents occurring right at the beginning of the example curve. Of course, the jackknife accidents are seen as simply resulting from the overbraking behavior of truck drivers who are endeavoring to achieve a speed that is low enough to avoid rollover. Simulation results shown in Figure 8 illustrate that a tractor-semitrailer carrying freight at a more or less typical level of CG (payload mass center at 83 in.) passes through the curve easily at 25 mph but barely escapes rollover at 35 mph. Other calculations for the same vehicle with a high CG (payload at 105 in.) show that the rig rolls over quickly if it enters the ramp at 35 mph. Thus there is no question that the deceleration task must be accomplished by most loaded truck combinations if they are to safely negotiate curves that have this degree of demand.

The key issue, then, is the extent to which deceleration requirements of the level represented in this case, and more generally of the level implicit in AASHTO policy, can be reasonably accomplished by heavy-duty truck combinations. There is a great deal of evidence establishing that the braking capability of heavy-truck combinations is quite low (26,27). Even on a dry pavement, a stop at approximately 0.4 g's would be considered a severe braking condition for a heavy truck. The Federal Motor Vehicle Safety Standard 121 that requires a deceleration capability of 0.41 g's for air-braked trucks stopping from 60 mph was seen as imposing a serious challenge to the state of truck braking technology. This standard, applied to stopping on dry pavement, implied a braking efficiency of approximately 50 percent. Further, because trucks suffer from large variability in the effectiveness of the basic brake itself, poor main-

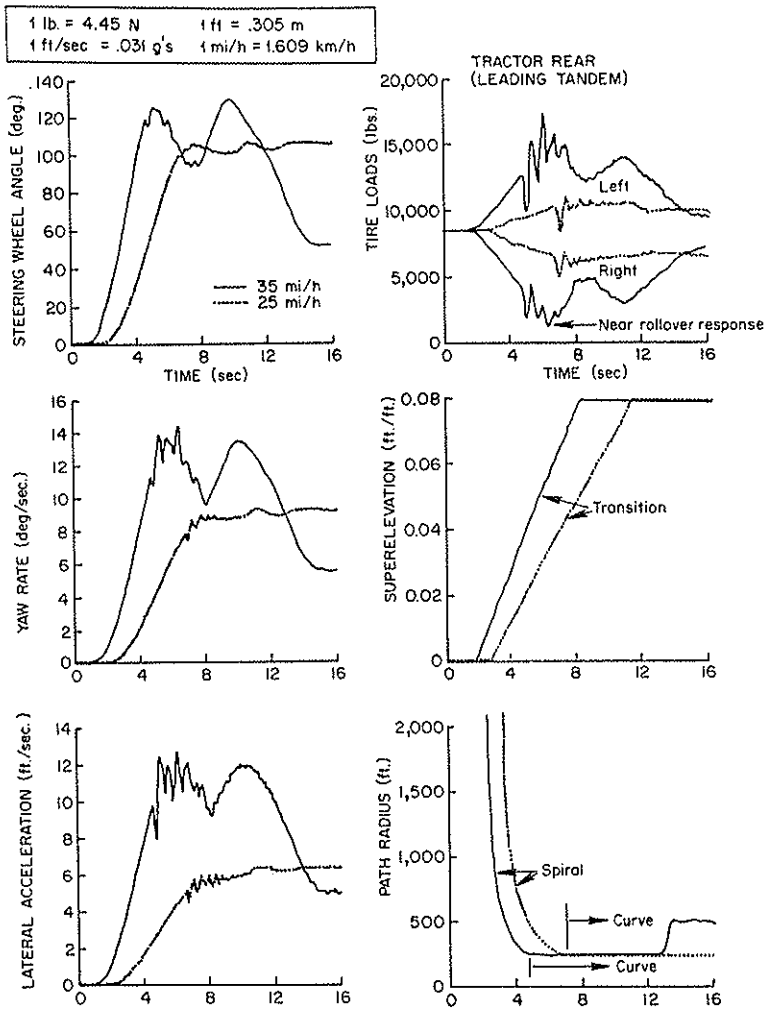


FIGURE 8 Vehicle response on entering the curved ramp of Figure 7 at 25 and 35 mph.

tenance of slack adjustment, and large variations in axle loading depending on the cartage application, levels of braking efficiency even lower than 50 percent are encountered in service.

Under partial loading conditions, a vehicle can exhibit both a low level of roll stability and an extremely poor level of braking capability. In such cases, the unfavorable distribution of axle loads makes it difficult for the truck to decelerate, even though the relatively high CG demands that speed be reduced as required by the curve in order to avoid rollover. Shown in Figure 9 is a plot of the maximum deceleration capability of a doubles combination with a partly loaded rear trailer and a loaded front trailer. To achieve a deceleration level equal to the 0.21-g condition required on the example ramp (with brakes applied right at the beginning of the taper) requires a rather substantial peak tire-road friction level of 0.55. The extremely poor stopping capability of this partly loaded vehicle is attributable to the light load prevailing at the rearmost axle. As braking is increased, the brake torque level applied at that axle quickly arrives at the point of saturating the shear force capability of the lightly loaded tires such that an unstable swinging motion of the second trailer is threatened. Similarly, a tractor-semitrailer with payload only in the front portion of the trailer, or a compartmented tanker with fluid emptied from its rear compartments, would exhibit very poor stopping performance (comparable with that of the example double), while also providing a low level of rollover resistance. Although completely empty truck combinations are also known to be conspicuously poor in braking efficiency, their higher roll stability levels tend to be somewhat compensating (assuming that the driver senses that full deceleration to the value of the posted ramp speed is not so crucial that he is prompted to overbrake).

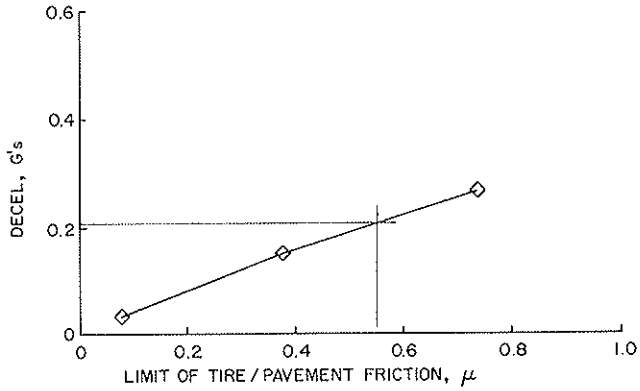


FIGURE 9 Maximum deceleration capability of partly loaded doubles combination as a function of the tire-pavement friction level.

The AASHTO policy for length of deceleration lanes clearly provides for more relaxed braking conditions than those required by the example ramp, although trucks must take liberties with the design relative to the expected usage by automobiles. In particular, the green book requires that deceleration length be measured on tapered exits beginning with the point at which 12 ft of taper is achieved. By this standard, the example ramp would have been constructed with the taper beginning approximately 390 ft sooner than it was. Trucks that begin braking right at the taper of such a deceleration lane would experience only a moderate braking demand. Taking the recommended lengths of deceleration lanes, generally, truck drivers could make a compromise usage of the suggested design by simply applying brakes throughout the available length of the lane thus forsaking the luxury of a 3-sec period for coasting in gear. By this approach, for example, the 490-ft value that the green book recommends for reducing speed from 55 to 25 mph would require a steady deceleration of 0.16 g's--a level that should be reasonably achievable by almost all trucks under most wet and dry conditions.

The primary observation that has been made on the subject of deceleration lanes pertains to the very poor stopping capability of many truck combinations. Clearly, the problem in this regard is analogous to that encountered with regard to allowances for side friction factor. Namely, design specifications that are selected to assure comfortable operation of automobiles pose demands that may challenge the controllability limits of heavy-duty trucks.

Case 4 (lightly loaded truck tires are sensitive to pavement texture in avoiding hydroplaning on high-speed ramps)

Recent findings (28,29) that indicate the potential for hydroplaning with lightly loaded truck tires offer a likely explanation for loss-of-control problems that are seen at certain ramp sites in wet weather. These findings are based on the observation that at the light tire loads associated with empty truck combinations the footprint with which a truck tire contacts the pavement is unusually incapable of expelling water. Accordingly, very lightly loaded truck tires are vulnerable to a pronounced traction deficiency on pavements on which the water cover stands sufficiently above the textural asperities. Because the loss of tire traction on wet surfaces is clearly most pronounced when speed is high, potentially troublesome ramps are categorically those that have large-radius curves such as at interchanges between two high-speed highways. The applicable scenario leading to loss of control involves an unloaded truck combination; a high-speed turn that also poses a substantial side friction demand; and poor pavement texture or water drainage characteristics, or both.

An example ramp site that was found to provide a dramatic illustration of this phenomenon is sketched in Figure 10. The ramp constitutes a nearly steady curve, 2,600 ft in length, which is comprised of two curve segments of 1,400-ft radius with a 290-ft tangent section connecting the two. The entire curved portion of the ramp plus the 290-ft tangent section was superelevated at 0.05 ft/ft, yielding a side friction factor of 0.05 at the special truck advisory speed of 45 mph. The evidence suggests, however, that many truckers simply sustain the 55-mph speed that is posted for other vehicles and thus the trucks experience a side friction factor of 0.09.

Forty-four loss-of-control accidents occurred at this site with tractor-semitrailers during a 2-year period following operating of the new roadway. All 44 accidents occurred when the pavement was wet. The rate of accidents was so great when wet conditions prevailed that a number of the accidents were witnessed by police officers who were on the scene to aid in the recovery of another truck that had lost control. Thirty-two of the accidents at this site involved tractor jackknife, five culminated in rollover, and seven involved other events such as simply running off the road or striking a guardrail. The ramp was resurfaced at the end of this 2-year period with a high-friction bituminous concrete overlay, after which the wet-weather accident problem essen-

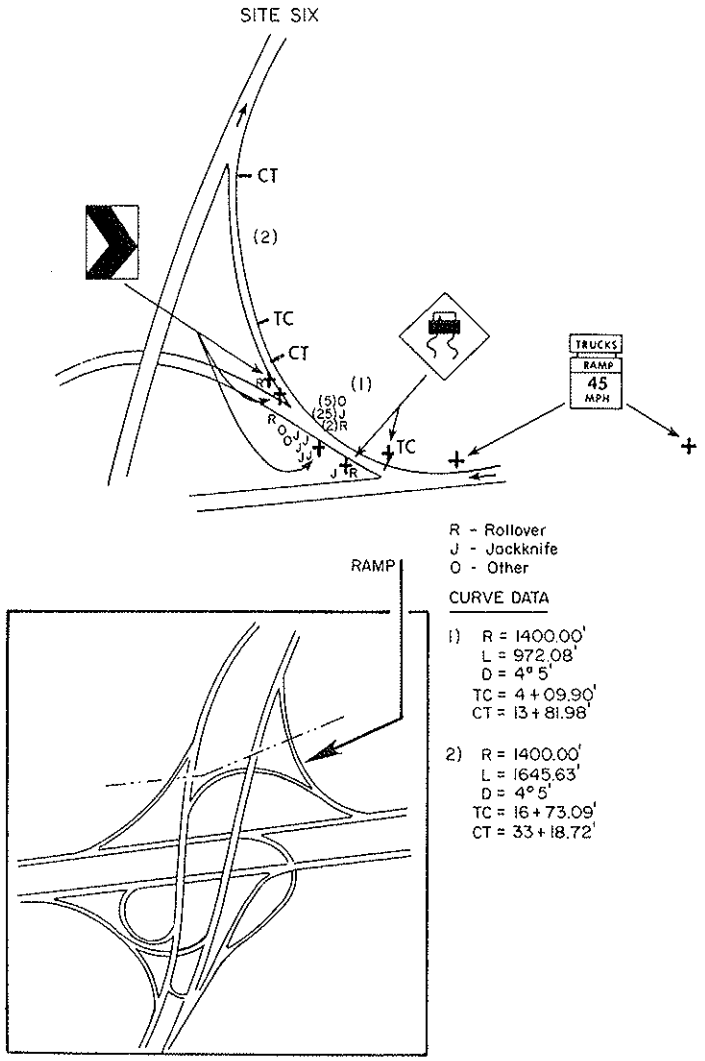


FIGURE 10 Layout of curved ramp site at which numerous loss-of-control accidents occurred with tractor-semitrailers in wet weather.

tially disappeared. Although the police-reported accident forms provided no note of vehicle loading, the large number of loss-of-control incidents that involved running off the road without rollover suggests that many of the semitrailers were lightly loaded or empty.

Shown in Figure 11 are simulation results illustrating the jackknife response of an unloaded tractor-semitrailer running on the example ramp at a constant speed of 55 mph. The conditions producing loss of control in this example involve the assumption of a near-hydroplaning level (mu = 0.12) at the tractor rear and trailer tires compared with a friction level at the front tires of 0.50. This peculiar distribution of tire-pavement friction levels was rationalized on the basis of large differences in tire load among the respective axles and the corresponding implications for friction, considering the potential for strong hydrodynamic influences (28). Static loads on front and rear tires were 4,700 and 1,300 lb, respectively. The simulation results indicate that if the friction levels attain the identified values, the vehicle becomes sufficiently disturbed in traveling over the superelevated tangent portion of the curve that a rapid jackknife divergency is precipitated (on saturating the lateral force output of the tractor rear tires).

Although this example simulation illustrates one possible set of conditions under which accidents such as those reported could occur, it should be recognized that braking and steering inputs could also disturb the vehicle to precipitate the actual jackknife sequence. That the great majority of the jackknifed tractor-semitrailers came to rest on the inside of the turn suggests that the jackknife typically began when tractor drive wheels were locked, following which brakes were released, causing the vehicle to go rapidly in the direction toward which the tractor had begun to rotate--toward the inside of the turn.

The item of general importance illustrated in this case is that heavy-duty vehicles are now known to be unusual in their potential for loss of control on wet pavements. Ramps that impose moderate to large demands for side friction factor while also permitting high-speed travel must be maintained with particular attention to pavement friction level and water drainage in order to safely accommodate lightly loaded truck combinations.

Case 5 (curbs placed on the outer side of curved ramps pose a peculiar obstacle that may trip and overturn articulated truck combinations)

Every truck driver knows that the rear axles on the trailing elements of an articulated truck combina-

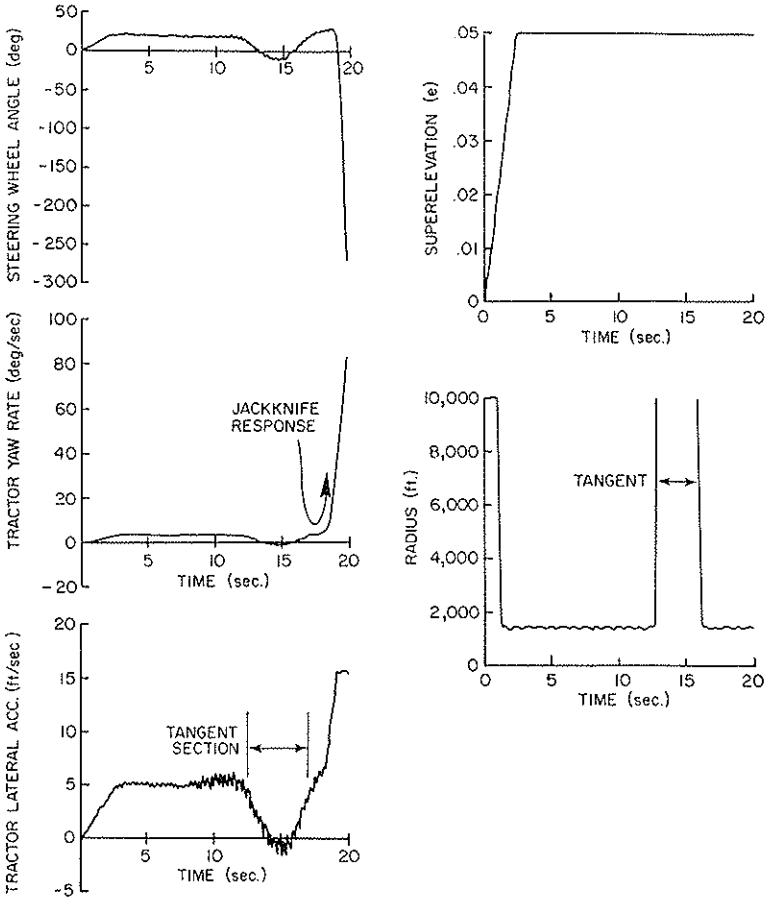


FIGURE 11 Response of tractor-semitrailer traveling at 55 mph without braking through the ramp shown in Figure 10.

tion will track inboard of the path of the tractor during low-speed, tight-radius turning maneuvers. This phenomenon has been called low-speed offtracking and has been recognized as a consideration in highway design for many years. It has been observed in recent years, however, that the trailers in tractor-semitrailer and doubles combinations tend to "fling out" in a turn as the lateral acceleration level increases, such that the rearmost axles may actually subtend paths that are outboard of those traced by tractor axles (30). The magnitude of the outboard offset in wheelpaths can be of the order of 2 to 3 ft in a steady turn (31). The particular safety concern that arises from this behavioral characteristic is that the rearmost axles may strike a curb that is situated, on certain ramps, along the outer side of the curve. Because it is thought that truck drivers are generally unaware of this so-called "high-speed offtracking" phenomenon, the safety problem may be exacerbated by the harmful natural instinct of drivers who may tend to steer close to the outer curb, believing that the trailer axles always tend to go inboard.

As shown in Figure 12, the trailer attitude associated with the outboard offtracking motion is such that the outer trailer tire approaches the curb at a sideslip angle, with the tire pointed away from the curb rather than toward it. Accordingly, the tire tends to resist mounting at the curb face. Although no definitive experiments are known to have been conducted to examine tire force response under such curb contact conditions, it appears certain that large side force levels would be available so that rollover would be a likely outcome.

Shown in Figure 13 is a case in which truck rollover accidents appeared to have involved tripping at an outside curb. The ramp involves two 12-ft lanes that constitute an interchange leg between two urban expressways. The curve radius of 374 ft, together with a superelevation of 0.05 and an original ramp advisory speed of 35 mph, yielded a side friction factor of 0.17. The ramp incorporated a cross-sectional design, as shown in Figure 14, with curbs provided to assist in channeling water drainage. The right curb is a mountable type permitting access by disabled vehicles to a paved right shoulder.

This ramp provides, first, a relatively severe side friction demand together with the curb that is within approximately 20 in. of the lane edge along the outside of the curve. It would appear that truck combinations may have experienced sufficient outboard offtracking of the trailer axles, because of the substantial side friction factor, that the rearmost outer tire struck the mountable curb. Because the sideslipping tire, with its inward orientation, was unable to mount the curb, a lateral force response developed due to the curb contact and thus produced the additional roll moment needed to overturn the truck combination.

The practice of building curbs on the outside of a curved ramp was among the approved design approaches cited in the AASHO blue book (25). Even on loops or direct connection roadways with continuous-curve alignment in one direction, curbs along the outside edge were justified as providing "an effective delineator on the high side of the pavement." In the more recent green book (1), AASHTO policy has apparently changed such that the use of curbs on in-

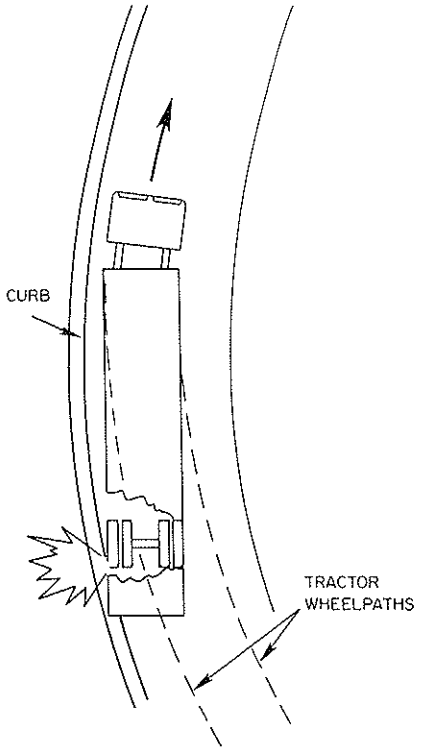


FIGURE 12 Outboard offtracking of semitrailer that leads to contact between trailer tires and an outside curb.

termediate and higher speed ramps is not recommended. Indeed, the green book suggests that curbs be considered only to facilitate particularly difficult drainage situations. It is clear that the use of a curb on the high side of a superelevated curve cannot be rationalized as an aid to drainage.

CONCLUDING REMARKS

The study that led to the findings presented herein examined individual ramps that had been found to have numerous truck loss-of-control accidents. Although, on one hand, a number of these ramps incorporated features that AASHTO policy discourages, it appears that even the current recommendations of AASHTO on geometric design will allow ramps that severely limit the safety margin available to many heavy-truck combinations. Indeed, the most useful aspect of this study, from the viewpoint of the highway design community, may be simply the illustration that truck stability and control levels are low relative to the vehicle control limits that are assumed in geometric design. Although it may be impractical in certain respects to truly design highways so that trucks can be operated as comfortably as automobiles, it does appear rational to suggest that highways be designed so that truckers obeying the posted speeds can be assured of nominally safe travel.

It would also appear beneficial for those maintaining the highway system to examine ramp sites that have frequent truck accidents to determine whether any of the peculiar problems identified here

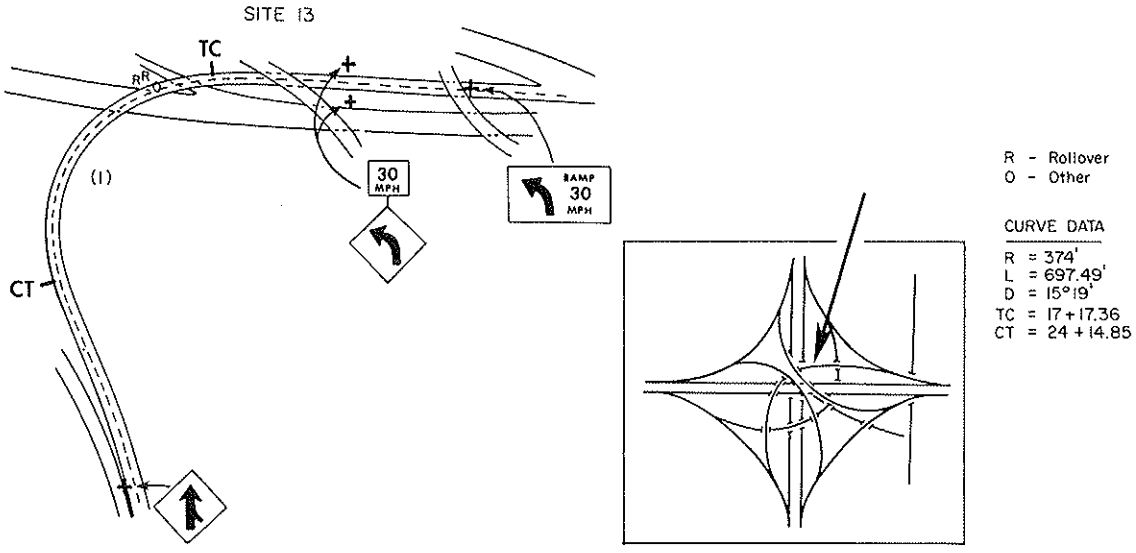


FIGURE 13 Layout of ramp on which curb-contact accidents occurred.

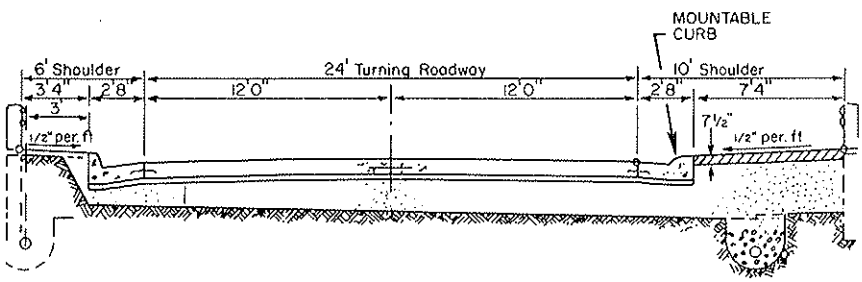


FIGURE 14 Cross section of roadway from ramp site shown in Figure 13.

might apply. Although many of the countermeasures implicit in the discussion here would involve major reconstruction of the ramp, improved speed advisories, resurfacing, and curb removal are also among the actions that can be taken in certain cases.

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Large Vehicles and Roadside Safety Considerations

JARVIS D. MICHIE

ABSTRACT

Because three-quarters of highway traffic is composed of passenger sedans, most current roadside hardware has been designed to interact with this vehicle type because of technical and economic restraints. Recent trends in national data indicate that the percentage of vehicles larger than passenger sedans is increasing. In addition, as a result of the Surface Transportation Assistance Act of 1982, large trucks are expected to become wider and longer. The import of these trends is examined with respect to roadside safety considerations, in particular to the roadside features and hardware that may need to be upgraded.

Until the mid-1970s about 80 percent of all vehicle miles of travel in the United States was done by automobiles; the remaining 20 percent was attributed to (a) motorcycles, (b) buses, (c) large and small trucks, and (d) special vehicles such as concrete trucks. Roadside safety research concentrated primarily on the passenger automobile because it was the principal risk. Specifically accommodating any or all of the remaining 20 percent of the other vehicle types was considered technically and economically questionable. (Even within the passenger vehicle segment of the traffic stream, drastic downsizing has occurred since 1974 and has necessitated design modification to roadside hardware.)

The proportion of vehicles heavier than the 4,500-lb passenger sedan in the traffic stream has increased in the past 10 years with an attendant increase in roadside accidents involving larger vehicles. In response, more roadside safety research has been directed to the large vehicle problem by state and federal agencies. With the passage of the Surface Transportation Assistance Act (STAA) of 1982, there is concern about the effects that the longer and wider trucks permitted by the act will have on roadside safety.

The questions that are addressed here are (a) how serious is the large vehicle-roadside safety problem? (b) is the problem becoming more critical? and (c) what, if anything, can be done to lessen the problem?

BACKGROUND

Although several state and private agencies performed some full-scale crash testing of roadside hardware before 1960, it was in September 1962 with the publication of Highway Research Board Circular 482 (1) that vehicle crash test procedures (and roadside safety research) were formalized. It is noteworthy that a 4,000-lb passenger sedan was indicated as the only test vehicle. In 1974 NCHRP Report 153 (2) presented more in-depth methods of evaluating highway appurtenances by vehicle crash testing and these methods were further refined in 1978 (3). However, only passenger sedans were specified as the test vehicles. It was not until NCHRP Report 230 (4) was published in March 1981 that test vehicles larger than a 4,500-lb passenger sedan were specified; even so, tests with larger vehicles were not considered required experiments but were recommended for use as supplementary experiments.

Irrespective of the lack of standardized crash test procedures, the FHWA in the early 1970s began exploring the technical feasibility of developing longitudinal barriers that would contain and redirect large vehicles. In the period 1972-1976 the collapsing ring bridge railing system was developed and evaluated for school bus, intercity bus, and tractor-trailer rig impacts; gross mass of one test vehicle was 70,000 lb (5). Also in 1976 the concrete median barrier was shown to have the capability of redirecting a 40,000-lb intercity bus (6). Early on, it was recognized that the high-performance barriers would have a premium cost compared with barriers designed to redirect only passenger sedans and could not be economically justified for general use. Instead, application of these special barriers would be limited to a few high-risk sites. A benefit-to-cost method was used in developing a multiple service level approach to warranting bridge rail systems (7).

As shown in Figure 1, traffic volume is the principal warranting factor for the four levels of service based on a benefit-to-cost analysis. Recently, other overriding factors have been proposed for an expanded array of bridge rail systems including the "tall wall" and "super tall wall" developed by Hirsch et al. at the Texas Transportation Institute (TTI) (8,9); sites for such high-performance barriers will probably be justified on the basis of "unacceptable consequences, regardless of the improbable risk of occurrence, of a heavy vehicle and/or its cargo penetrating the bridge rail." Examples of such sites might include a bridge that spans a critical water supply, a petrochemical plant, or a pedestrian mall.

Two large-vehicle accidents occurred in 1976 and focused national attention on the limited collision performance capability of bridge rail systems. The first on May 11 in Houston, Texas, involved a tractor-tanker carrying anhydrous ammonia that penetrated an overpass bridge rail and fell on freeway traffic. The second on May 24 involved a school bus that failed to negotiate an off-ramp curve in Martinez, California, penetrated the bridge rail, and resulted in 28 occupant fatalities. Although the FHWA had recognized the growing need for high-performance barriers, these two incidents focused national attention on large-vehicle safety and galvanized support for accelerated roadside safety research.

Dynatech Engineering, Inc., 301 South Frio, San Antonio, Tex. 78207.

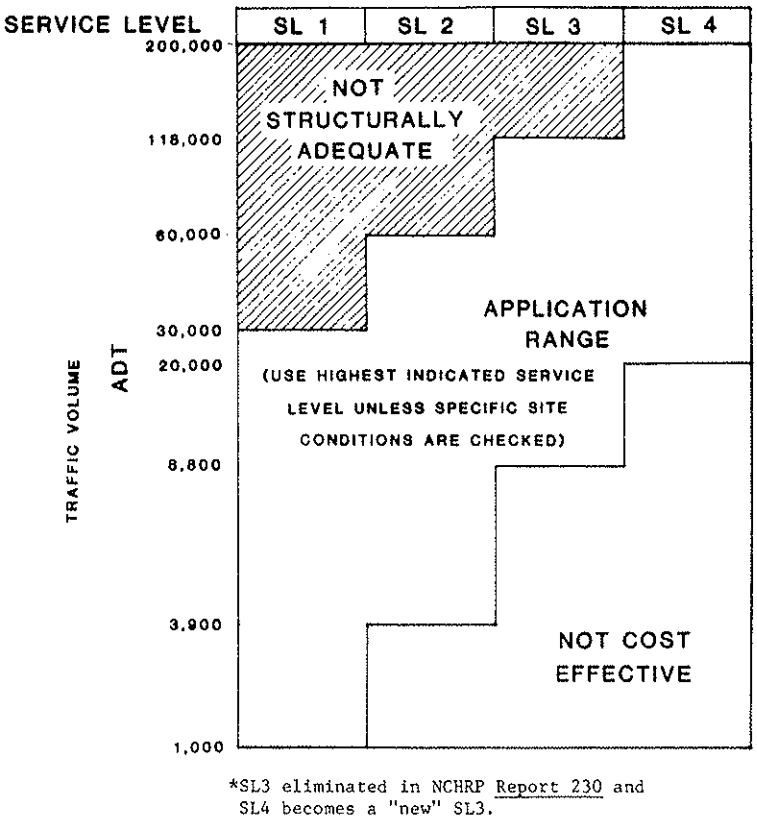

FIGURE 1 Traffic volume and bridge railing service level category summary (7).

TRAVEL GROWTH OF LARGE VEHICLES

During the period of 1970 to 1982 overall vehicle exposure measured in vehicle miles of travel (VMT) grew from 1.12×10^9 to 1.59×10^9 VMT or a 42 percent increase (10). This is shown in Figure 2. The passenger automobile part of this total travel grew from 0.9×10^9 to 1.1×10^9 VMT or 22 percent. The largest growth area was in the vehicle segment denoted as "single unit trucks," which more than doubled from 0.17×10^9 to 0.38×10^9 VMT.

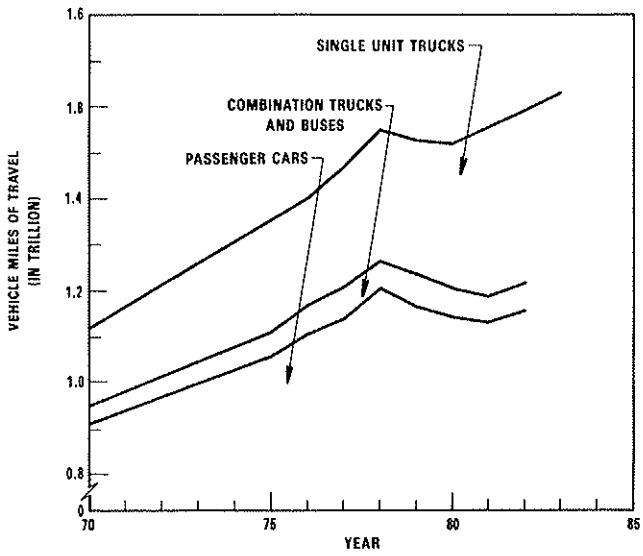

FIGURE 2 Travel growth by vehicle size.

This trend is further analyzed in Table 1. Findings of interest are

- Although passenger automobile travel continued to increase, its percentage of all travel decreased from 78.9 to 72.1 percent;
- Single-unit truck travel increased in both magnitude (i.e., 218.9×10^6 to 376.7×10^6 VMT) and percentage (i.e., 16.5 to 23.6 percent); and
- The combination truck and bus segment exhibited little travel growth and a decrease in percentage of overall travel.

From these statistics, it appears that the single-unit truck is the rapidly growing part of the traffic stream and there is little if any change in the combination truck and bus segment. Even with the effect of the STAA of 1982, the author speculates that the 4 to 5 percent of travel of combination trucks will not change significantly during the next decade. These are, of course, national averages and may not reflect local conditions. Specific routes such as the New Jersey Turnpike are used by a disproportionate amount of truck travel and would not be properly represented by these statistics.

The single-unit truck not the combination truck may represent the most important vehicle with regard to roadside safety. Insight into the type of vehicles that comprise the single-unit-truck segment can be obtained from Table 2 (10,p.17). Of the 2.7 million trucks sold in 1983, about one-half had a mass in the 0- to 6,000-lb range. Although it cannot be deduced from the figures in Table 2, it is judged that about 0.5 million of these vehicles have mass less than 4,500 lb and fall within the passenger vehicle test matrix of NCHRP Report 230. The conventional pickup and van probably represent the major

TABLE 1 Billions of Vehicle Miles of Travel *(10)*

	Passenger Automobile Travel		Single-Unit Truck[a] Travel		Combination Truck and Bus Travel		All Motor Vehicle Travel	
Year	VMT	Percentage	VMT	Percentage	VMT	Percentage	VMT	Percentage
1975	1,050.5	78.9	218.9	16.5	60.7	4.6	1,330.1	100.0
1980	1,129.9	74.3	324.6	21.3	66.4	4.4	1,520.9	100.0
1982	1,148.9	72.1	376.7	23.6	66.9	4.2	1,592.5	100.0

[a]Principally vehicles weighing less than 10,000 lb also denoted as "light trucks and vans."

TABLE 2 Retail Sales of New Trucks by Franchised Dealers of U.S. Manufacturers *(10)*

	Year					
Gross Vehicle Weight	1978	1979	1980	1981	1982	1983
0-6,000 lb						
Utility	79,588	74,878	50,842	36,389	50,735	253,823
Car-type pickup	83,522	77,094	49,696	37,080	25,305	26,170
Compact pickup						
Domestic	–	78	25,525	59,431	359,177	433,167
Import	140,736	225,410	228,878	159,551	95,277	55,143
Van	126,072	110,393	78,871	74,983	74,546	67,299
Mini van	–	–	–	–	–	18
Conventional pickup (includes extended and crew cabs)	904,002	783,035	544,959	520,180	485,977	445,370
Station wagon (truck chassis)	–	–	–	–	–	8,394
Mini passenger carrier	–	–	–	–	–	8,174
Passenger carrier	472	439	6,446	8,333	10,608	16,364
Total 0-6,000 lb	1,334,392	1,271,327	985,217	895,947	1,101,625	1,313,922
6,001-10,000 lb						
Utility	275,790	205,181	107,541	70,938	76,457	84,493
Van	471,334	331,848	172,045	168,469	207,466	311,207
Van cutaway chassis	76,277	43,797	19,918	21,662	30,951	45,228
Conventional pickup (includes extended and crew cabs)	1,171,257	884,551	545,720	468,730	484,909	573,918
Station wagon (truck chassis)	100,395	73,294	38,807	37,564	54,517	68,844
Passenger carrier	6,398	4,792	65,917	56,964	74,992	76,985
Multi-stop	38,193	30,816	24,867	25,622	31,771	45,924
Total 6,001-10,000 lb	2,139,644	1,574,279	974,815	849,949	961,063	1,206,599
10,001-14,000 lb	73,119	15,408	3,510	748	1,062	145
14,001-16,000 lb	5,792	2,686	195	12	9	2
16,001-19,500 lb	2,699	2,952	2,309	1,916	1,434	1,159
19,501-26,000 lb	155,616	145,977	89,764	71,993	44,214	46,532
26,001-33,000 lb	41,032	49,623	58,436	51,402	62,488	59,383
33,001 lb and more	161,608	173,543	117,270	100,334	75,777	81,647
Total	3,913,902	3,235,795	2,231,516	1,972,301	2,247,672	2,709,389

part of vehicles in this group with mass greater than 4,500 lb. Even so, most of these vehicles would be at least marginally addressed by the NCHRP Report 230 procedures.

Vans and conventional pickups comprise a large part of the 1.2 million vehicles in the 6,001- to 10,000-lb mass range. It is unknown what part of the 200,000 odd vehicles with mass greater than 10,000 lb is combination trucks; regardless, it is less than 10 percent of the total 2.7 million vehicles.

The most important factors are that (a) about one-half of the truck population (i.e., that which weighs less than 6,000 lb) is at least grossly addressed by current NCHRP Report 230 test conditions; (b) another 45 percent of the total truck population weighs between 6,000 and 10,000 lb and is composed chiefly of conventional pickups and vans; and (c) the remainder, less than 10 percent of all trucks, have mass that extends from 10,000 lb to more than 33,000 lb. This last segment will include the new wider and longer vehicle provided by STAA of 1982 although it will be several years before there are significant numbers in the vehicle fleet.

SAFETY ASSESSMENT

Roadside safety research addresses mainly the single-vehicle, ran-off-the-road accident scenario. This scenario begins with an inadvertent encroachment and concludes with either an unreported "drive-away" or a reported accident. Inadvertent encroachments have been the subject of extensive research in the past 20 years; findings indicate that highway geometrics (e.g., curves, grade, number of lanes) and traffic volume are the two main factors that affect the number of errant vehicles that leave the traveled way. With regard to traffic volume, accident statistics indicate that the number of each type of vehicle involved in roadside collisions is roughly proportional to its portion of the traffic stream.

An analysis of highway accidents for each major vehicle type is presented in Table 3 (10-12). Vehicles in accidents and vehicles in fatal accidents are compared with billion miles of travel for each vehicle type. Numbers of fatal accidents are reported events whereas the National Accident Sampling System (NASS) accident numbers are projected to a national basis from a scientifically controlled sample of 15,000 events. Table 3 includes multiple- as well as single-vehicle events. Findings of interest are that automobiles are overrepresented in accidents and underrepresented in fatal accidents. Light trucks and vans are underrepresented in both accidents and fatal accidents. Buses are representative in both. Heavy trucks are representative in acci-

TABLE 3 1982 Data on Accidents by Vehicle Type Compared with Exposure *(10-12)*

Vehicle Type	Exposure		NASS-Projected Vehicles in Accidents		Vehicles in Fatal Accidents	
	Billion Vehicle-Miles	Percentage	No. (1,000)	Percentage	No.	Percentage
Passenger automobiles	1,133.9	71.2	7,715.0	78.1	33,955	60.4
Motorcycles	15.0	0.9	177.0	1.8	4,420	7.9
Special vehicles and unknown			17.0	0.2	2,884	5.1
Buses	6.6	0.4	51.0	0.5	286	0.5
Light trucks and vans[a]	376.6	23.7	1,571.0	15.9	10,057	17.9
Heavy trucks[b]	60.3	3.8	344.0	3.5	4,588	8.2
Total	1,592.5	100.0	9,875.0	100.0	56,190	100.0

dents but overrepresented in fatal accidents. The seriousness of heavy-truck accidents may be attributed to the mismatch of the large truck mass compared with the smaller vehicle mass of other traffic, to the propensity of large trucks to jackknife, and to the longer distance required to decelerate heavy trucks.

A further analysis of types of vehicles in accidents is given in Table 4. The 9.8 million accidents that were extrapolated by NASS in 1982 are summarized by single-vehicle and miltiple-vehicle types, and then the single-vehicle accidents are examined for noncollision, fixed object, and other object. With regard to single-vehicle, fixed-object accidents, automobiles and heavy trucks are slightly overrepresented and light trucks and vans are underrepresented. It is noted that rollover or overturn accidents involving heavy trucks as well as jackknifing (i.e., other noncollision) are overrepresented with respect to exposure measure. With the projected increase in the number of double- and triple-trailer combinations that result from the STAA of 1982, the author speculates that these heavy-truck rollover and jackknifing types of accidents will increase. Moreover, the seriousness of these accidents in terms of property damage, injuries, and fatalities will probably also increase. On a national scale where heavy trucks represent only 3.8 percent of the traffic stream, it may not be cost-effective to provide high-performance roadside safety design to accommodate special requirements of the large mass vehicles. On the other hand, on specific routes where heavy-truck traffic greatly exceeds the 3.8 percent national average, the highway design engineer can and should take measures to minimize the occurrence and consequences of roadside excursion events.

As an independent check on the findings for light trucks and vans, insurance claim frequencies were examined for 1981-1983 for vans, pickups, and utility vehicles and these claim frequencies are shown in Table 5 (13).

It is clear that vans, pickups, and some utility vehicles are not involved in as many accidents as is the traffic fleet in general. The reason for this underinvolvement is not clear, but it may be attributable to travel patterns and driver profiles associated with this type of vehicle. Thus it is seen that while the volume of light truck and van traffic is increasing, this segment is relatively safe and is underinvolved in accidents.

ROADSIDE DESIGN REQUIREMENTS FOR LARGE VEHICLES

In some cases vehicles larger than passenger sedans exhibit more demanding performance requirements for roadside appurtenances. In other cases, roadside interactions with these larger vehicles are less critical.

Specifically, breakaway structures such as sign and luminaire supports, which are usually designed for small automobile impacts, cause a lesser velocity change in the larger mass vehicles and are therefore less hazardous from that standpoint. On the other hand, the sign blank missile hazard to truck occupants may be another problem. Mounting height of the sign blank should be developed with regard to truck compartment geometry as well as to impact trajectory after passenger automobile impacts. These safety considerations are in addition to sign visibility and readability, which are also a function of mounting height.

Crash cushions are generally designed for two

TABLE 4 1982 Data on Single- and Multiple-Vehicle Accidents by Vehicle Type *(10,11)*

Vehicle Type	Exposure		Single Vehicle											
			Noncollision											
	Billion Vehicle-Miles	%	Rollover/Overturn		Other[a]		Fixed Object[b]		Other Object[c]		Multiple Vehicle		Total Accidents	
			No.	%	No.	%	No.	%	No.	%	No.	%	No.	%
Passenger automobiles	1,133.9	71.2	77,150	39.6	77,150	51.2	694,350	77.5	231,450	81.9	6,634,900	79.5	7,715,000	78.1
Motorcycles	15.0	0.9	49,560	25.5[d]	1,770	1.2	19,470	2.1	7,080	2.5	99,120	1.2	177,000	1.8
Special vehicles			170						340	0.1	16,490	0.2	17,000	0.2
Buses	6.6	0.4			500	0.3			1,500	0.5	49,000	0.6	51,000	0.5
Light trucks and vans[e]	376.6	23.7	47,130	24.2	47,130	31.3	141,390	15.8	31,420	11.0	1,303,930	15.6	1,571,000	15.9
Heavy trucks	60.3	3.8	20,640	10.6	24,080	16.0	41,280	4.6	10,320	3.5	247,680	2.9	344,000	3.5
Total	1,592.5	100.0	194,650	100.0	150,630	100.0	896,490	100.0	282,110	100.0	8,351,120	100.0	9,875,000	100.0

[a] Jackknifing of combination units, explosions, immersion, gas inhalation, etc.
[b] Buildings, bridge abutments, poles, trees, etc.
[c] Animals, trains, etc.
[d] Motorcycle overturning accidents are different in nature from rollover of other vehicles because of the inherent instability of two-wheeled vehicles.
[e] Vehicles less than 10,000 lb.

TABLE 5 1981-1982 Insurance Claim Frequency for Light Trucks and Vans *(13)*

Make	Relative Claim Frequency	Exposure (vehicle-years)
All passenger automobiles	100	4,696,446
All vans	64	133,267
Small pickups	86	245,250
Standard pickups	60	380,858
Small utility vehicles	97	24,434
Intermediate utility vehicles	58	51,081
Large utility vehicles	43	10,837

conflicting conditions: softness and stroke efficiency. For softness, a low-level interaction force must be maintained to protect occupants in small-vehicle collisions. For larger automobiles, the crash cushion must have sufficient stroke to absorb the kinetic energy yet be compact in size to adapt to most sites. Crash cushions are generally staged with a soft nose and a crush stiffness that increases along its length. Labra (14) examined the feasibility of extending crash cushion design capability to include large vehicles. Two vehicle properties limit this application. First, semitractor-trailer rigs are inherently unstable vehicles and will readily jackknife after even a minor collision or sudden maneuver. Second, cargo restraints, especially for flatbed trailers, are designed for braking forces (about 1 g) and are inadequate for normal crash cushion forces of 8 to 10 g's. Under crash cushion collision conditions, a cargo would readily break loose from the tie-down restraint and move forward crushing the driver cab. For these reasons, it is judged impractical to develop crash cushions for very large trucks. On the other hand, it would be practical to develop crash cushions for light trucks and vans with mass of up to 10,000 lb.

A roadside feature that is specially designed for very large trucks is the escape ramp. These features are situated at the bottoms of long, steep inclines where trucks are likely to loose their brakes and require emergency assistance in stopping. Several techniques have been successfully used among which are elongated beds of loose gravel and a reverse incline. These designs are contained in current standard design specifications and will not be discussed further here.

In the past 15 years research has been directed to longitudinal barriers designed to contain large 80,000-lb vehicles. Such barriers are not insignificant because they must accommodate kinetic energy levels 40 times that of small 2,000-lb passenger sedans. Two principal factors govern performance of a longitudinal barrier: height to interact with a substantial structural element of the vehicle and structural strength to sustain the impact force. It is noted that the tractor-trailer rig has two separate components that must be redirected. Barrier height must be sufficient to interact with major structural elements of both the tractor and the trailer. For van-type trailers, a height of 5.5 ft has been shown to be adequate. On the other hand, the midheight of a tanker trailer is about 84 in., and an adequate barrier height is about 90 in. Hirsch (9) has recently developed and demonstrated two high-performance bridge rail systems to contain and redirect 80,000-lb tractor-trailers. Hirsch determined that critical barrier loading occurs when the rear tandem axles of the tractor rotate into the barrier with a 50-ms peak acceleration of 5.5 to 6.0 g's. Coupled with local vehicle mass of 34,000 lb, the applied horizontal loading is about 200,000 lb. It is speculated that the 200,000-lb force will not be exceeded by the longer and wider vehicle permitted by the STAA of 1982.

With the exception of the concrete safety shape (i.e., New Jersey) barrier and the recently developed SERB system, most current guardrail and median barrier operational systems cannot contain or redirect large trucks and buses including the wider and longer vehicles that are being introduced into the traffic stream. Benefit studies reveal that high-performance longitudinal barriers are generally too costly for highways with only 3.8 percent heavy-vehicle traffic but may be justified for those sites where the truck traffic exceeds 25 percent of the total traffic stream.

SUMMARY

Key findings developed in this paper with regard to large vehicles and roadside safety are

1. Travel growth
   - Single-unit truck travel is increasing both in VMT and as a percentage of all VMT.
   - Combination truck and bus travel is static and is decreasing as a percentage of all VMT. Local traffic properties may differ markedly from these national averages.
   - A large part of the single-unit truck segment is composed of pickups and vans that weigh less than 10,000 lb. Only about 8 percent of all 1983 truck sales were trucks weighing more than 10,000 lb.
2. Accident experience
   - Light trucks and vans are underrepresented in (a) total, (b) single-vehicle, (c) single-vehicle and fixed-object, and (d) fatal accidents. On the other hand, the number of noncollision rollovers or overturns is representative of the total traffic mix.
   - Heavy trucks are representative in (a) total, (b) single-vehicle and fixed-object, and (c) multivehicle accidents but overrepresented in (a) overturn or rollover, (b) jackknifing, and (c) fatal accidents.
3. Roadside design requirements
   - Breakaway structures such as signs and luminaire supports do not pose a severe hazard to the large vehicle if the sign blank missile hazard is properly treated.
   - Crash cushions are not technically feasible for heavy trucks. However, designs to accommodate light trucks (i.e., up to 10,000 lb) should be considered.
   - Longitudinal barriers such as bridge rails, guardrails, and median barriers are being designed to accommodate the largest vehicles but are relatively expensive and therefore sites must be carefully selected.
   - Shoulder sideslope may need to be examined with regard to truck overturns and rollovers.

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Longitudinal Barriers for Buses and Trucks

T. J. HIRSCH

ABSTRACT

In May 1976 two significant accidents occurred involving traffic bridge rails. An ammonia truck in Houston, Texas, struck a bridge rail leaving 11 dead, 73 hospitalized, and causing 100 other injuries for a total of 184 casualties. In Martinez, California, a school bus struck a bridge rail and left 29 dead and 23 injured. As a result of these accidents, an extensive effort has been made to develop longitudinal traffic barriers or rails capable of restraining and redirecting buses and large trucks. The results of 34 crash tests conducted using automobiles and mostly buses and trucks on 16 different traffic rails were obtained from the references. Vehicles represented are 4,500-lb passenger automobiles, a 4,000-lb van or light truck, 20,000-lb school buses, 32,000- to 40,000-lb intercity buses, and 40,000- to 80,000-lb tractor-trailer trucks. Results of these crash tests are summarized. Theory and crash test results are presented to demonstrate the magnitude of the impact forces these traffic rails must resist and how high they must be to prevent vehicle rollover. Typical designs of longitudinal barriers that have been successfully crash tested in accordance with recommended procedures are presented.

In May 1976 two significant accidents involving traffic rails occurred. An ammonia truck in Houston, Texas, struck a bridge rail and fell on traffic below leaving 11 dead and 73 hospitalized and causing 100 other injuries for a total of 184 casualties. In Martinez, California, a school bus struck a bridge rail and fell upside down leaving 29 dead and 23 injured. As a result of these accidents, an extensive effort has been made to develop longitudinal traffic barriers or rails capable of restraining and redirecting buses and large trucks.

Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843.

Before 1956, when the Interstate Highway Act was passed by Congress, most highway bridges crossed rivers, streams, or other natural features. Few highways had traffic lanes divided or separated by median barriers. Longitudinal barriers such as bridge rails, median barriers, and guardrails were designed only to restrain and redirect passenger automobiles. It was the general attitude that buses and trucks were driven by trained, skilled, professional drivers, and sensational traffic barrier accidents with buses and trucks were rare.

Since 1956 tens of thousands of miles of divided traffic lane Interstate highways, urban expressways, and freeways have been built. Most of the bridges

(1) on these systems are grade separation structures that cross other densely populated traffic lanes. In addition, with the demise of railroads and the increase in school busing, there has been a significant increase in the number of buses and trucks on the roadways. Consequently, the number of sensational bus and truck accidents involving longitudinal barriers has increased. Many highway engineers now believe that there are selected locations where barriers capable of restraining and redirecting buses and trucks are needed.

A search of the recent literature (1972 to 1985) yields 14 references to 34 crash tests into longitudinal traffic barriers that were conducted essentially in accordance with current recommended practice (2). These crash tests used automobiles, vans, buses, and trucks ranging in weight from approximately 4,000 to 80,000 lb. In general, the passenger automobile and van tests were conducted at 60 mph at a 25-degree angle into the longitudinal barriers. The school and intercity buses weighed from 20,000 to 40,000 lb, and tests with these vehicles were conducted at 60 mph at a 15-degree angle into the longitudinal barriers. The tractor-trailer trucks weighed from 40,000 to 80,000 lb and were crash tested at 50 mph at a 15-degree angle into the barriers. A summary of these vehicle crash test results is presented in Table 1.

These crash test results and some elementary theory (17-19) are presented to demonstrate the magnitude of the impact forces these longitudinal traffic barriers must resist and also how high these barriers must be to prevent vehicle rollover. In addition, typical designs are presented in Figures 1-3 of longitudinal barriers that have been successfully crash tested in accordance with current recommended procedures (2). The costs per foot of length shown on Figures 1-3 would be typical of Texas and are for comparison only.

BASIC MOTOR VEHICLE AND BARRIER PROPERTIES TO BE CONSIDERED

Most current longitudinal traffic barriers (guardrails, bridge rails, and median barriers) are designed only to restrain and redirect passenger automobiles ranging in weight from 1,700 to 4,500 lb. The recommended strength test (2) is for a 4,500-lb automobile to be redirected at 60 mph and a 25-degree angle impact. Figure 1 shows some basic properties of these automobiles and two common and effective longitudinal barriers that can restrain and redirect them. These automobiles have centers of gravity (CGs) ranging from 18 to 24 in. above the roadway. The 27-in.-high standard guardrail and 32-in.-high concrete safety shape are strong enough to redirect the automobiles and high enough to prevent rollover. These barriers exert a redirecting and stabilizing force on the fenders, tires, and door panels of the impacting car, as shown in the figure. The approximate cost per foot of these traffic barriers is shown for comparison purposes.

Figure 2 shows some basic properties of buses (school and intercity) and two traffic rails that have restrained and redirected them. School buses (66 passenger) generally weigh from 20,000 to 26,000 lb loaded. Intercity buses (45 passenger) generally weigh from 32,000 to 40,000 lb loaded. The CG of these buses ranges from 46 to 58 in., with an average of about 52 in. The two minimum height rails that have prevented these buses from rolling over under 60 mph, 15-degree angle impact are the two shown with heights of 38 and 42 in. The approximate cost per foot of the barrier is shown for comparison purposes.

Traffic rails 32 in. and 34 in. high have consistently produced rollover with buses at this speed and angle of impact. The significant redirection force from these barriers is delivered to the bus through the front and rear tires and axles. The largest impact force reported in Table 1 occurs when the rear tires and axle strike the barrier.

Figure 3 shows some basic properties of van and tank-type trucks and some longitudinal barriers that have restrained and redirected them. These trucks weigh from 25,000 lb empty up to 80,000 lb when fully loaded (21). The CG of an empty truck can be about 45 in., and a fully loaded truck could have a CG of from 60 to 78 in. Figure 3 shows three distinct locations or heights where a longitudinal barrier can effectively push on a van or tank truck to redirect it. A 42-in.-high barrier can push on the 42-in.-high tires (and axle). For a van-type truck, the floor system from 48 to 54 in. high is capable of receiving a significant redirecting force. Above this height the van truck generally has a very thin weak sidewall that is not capable of receiving much redirecting force.

A tank truck can receive a redirecting force through the tires up to 42 in. high and then another redirecting force at about 84 in. high into the central area of the usually circular tank. A traffic rail element between approximately 42 and 78 in. usually has nothing to push against.

The 42-in.-high concrete parapet barrier shown redirected without rollover an 80,000-lb van truck with a 65-in.-high CG. A similar truck with a 78-in.-high CG rolled over the 42-in.-high barrier (5). All these tests are nominally at 50 mph and 15-degree angle impact.

The 50-in.-high combination barrier (concrete parapet with metal rail on top) restrained and redirected an 80,000-lb van truck with a 66-in.-high CG. The truck rolled over on its side. However, it did not go over the bridge rail, and the truck remained on the simulated bridge. This was considered a successful test for a truck. A rollover would not be acceptable for a passenger automobile or a bus.

The 54-in.-high combination bridge rail shown smoothly restrained and redirected an 80,000-lb van truck with a 64-in.-high CG (no rollover).

STRENGTH REQUIREMENTS OF LONGITUDINAL BARRIERS

A relatively simple method of predicting the impact forces on a longitudinal barrier is the equations presented in NCHRP Report 86 (22).

Figure 4 shows a vehicle striking a longitudinal traffic rail at an angle (θ). From this illustration of the impact event it can be shown that the average lateral vehicle deceleration (G_{lat}) is

$$\text{avg } G_{lat} = [V_I^2 \sin^2(\theta)]/[2g(AL \sin(\theta) - B[1 - \cos(\theta)] + D)] \quad (1)$$

If the stiffness of the vehicle and rail could be idealized as a linear spring, the impact force-time curve would be in the shape of a sine curve; then the peak or maximum lateral vehicle deceleration (max G_{lat}) would be

$$\max G_{lat} = (\pi/2)(\text{avg } G_{lat}) \quad (2)$$

The lateral impact force (F_{lat}) on the traffic rail would then be equal to the lateral vehicle deceleration times the vehicle weight, thus

$$\text{avg } F_{lat} = (\text{avg } G_{lat})W \quad (3)$$

and

TABLE 1 Summary of Vehicle Test Results

Author	Test No.	Test Condition Vehicle-CG (in.) and Vehicle Weight-Speed-Angle (lb-mph-degrees)	Max. Avg. 0.05-sec Force: Load Cells (kips)	Max. Avg. 0.05-sec Force: Accelerometer (kips)	Height of: Resultant (in.)	Height of: Barrier (in.)	Barrier and Remarks
Noel *(3,4)*	3451-32	Plymouth-22 4,680-52.9-15	52.1	43.6	21.2	42	Concrete wall; smooth redirection
	3451-36	Plymouth-22 4,740-59.9-24	59.9	69.6	21.9	42	Concrete wall; smooth redirection
Buth *(5)*	4798-7	Dodge van-30 3,983-59.2-24		20.0		27	W-beam guardrail, 2.34-ft deflection; van rolled over 270 degrees
Buth *(4)*	3451-9	School bus-50-43[a] 19,940-55.2-15		83.7		27	Reinforced W-beam bridge rail; redirection, front axle stripped[a]
	3451-10	School bus-50-43[a] 20,010-52.0-13.3		70.1		27	Reinforced W-beam bridge rail; redirection, front axle stripped[a]
	3451-11	Intercity bus-46 31,880-58.4-16		63.8		27	Reinforced W-beam bridge rail; redirection, bus rolled over
	3451-4	School bus-50 19,760-59.8-14.3		102.8		30	6- x 8-in. steel tube on 9-in. concrete parapet; redirection, bus rolled over
	3451-23	School bus-50 19,920-57.3-14.8		97.6		32	4-in. aluminum rail on 18-in. concrete parapet; redirection, bus rolled over
Davis *(6)*	3080-1	School bus-51.5 20,270-61.6-15		120.0		32	CMB concrete; bus rolled over
	3115-1	School bus-51.5 19,990-60.9-16		120.0		32	CMB concrete; bus rolled over
Ivey *(7)*	3825-8	School bus-50 20,000-57.7-15		106.0		32	CMB concrete parapet; bus rolled over
Kimball *(8)*	RF-26	School bus-55 23,000-57.1-14.7		89.7		32	Thrie-beam bridge rail; bus rolled over
	RF-27	Scenicruiser bus-56 40,000-59.7-17.6		No data		32	Thrie-beam bridge rail; bus rolled over
Davis *(6)*	8307-1	Scenicruiser bus-50 40,020-54-16.2		150.0		32	CMB concrete; redirection
	8307-3	Scenicruiser bus-50 40,030-54-14		170.0		32	CMB concrete; redirection
Buth *(5)*	4798-12	Scenicruiser bus-56 39,970-59.6-14.5		179.9		34	Thrie-beam median barrier; bus rolled over
Kimball *(8)*	RF-28	Scenicruiser bus-56 40,000-56.3-14.5		164.0		38	Thrie-beam bridge rail; smooth redirection
Hirsch *(9)*	230-3	School bus-50 19,690-54.4-15		96.5		42	Concrete parapet and metal rail; smooth redirection
Noel *(3,4)*	3451-34	School bus-50 20,030-57.6-15	73.8	82.2[b]	32.7	42	Concrete wall; smooth redirection
	3451-35	Intercity bus-46 32,020-56.9-15.7	211.2	220[b]	28.4	42	Concrete wall; smooth redirection
Hirsch *(9)*	230-5	Intercity bus-46 32,080-61.1-15		105.9		42	Concrete parapet and metal rail; smooth redirection, 44-in. rail deflection
Kimball *(10)*	BR-8	School bus-53± 19,000-60.9-13.9		74.1		59	Collapsing ring bridge rail
	BR-11	Intercity bus-53± 40,000-54.2-15.1		88.0		59	Collapsing ring bridge rail, 35-in. permanent deflection
Bronstad *(11)*	TTR-2	School bus-53± 20,000-55.2-13.7		80.0		60	Thrie-beam bridge rail; bus rolled over, rail deflection
	TTR-3	School bus-53± 20,000-53.9-15.3		70.0		60	Thrie-beam bridge rail; good redirection
Ivey *(12)*	3825-17	Ford truck-58 18,240-60.1-15		153.0		32	CMB concrete; truck rolled over
Davis *(6)*	8307-2	Tractor-trailer van-60 40,030-53-15		110.0		32	CMB concrete; truck mounted and straddled CMB
Hirsch *(13)*	CMB-7	Tractor-trailer van-55 48,800-44.7-15		143.2		32	CMB concrete parapet; smooth redirection
Buth *(5)*	4348-2	Tractor-trailer van-78 80,180-52.8-15		194.0		42	Concrete parapet CMB type; truck rolled over
	4798-13	Tractor-trailer van-65 80,180-52.1-16.5		108.5		42	Concrete parapet CMB type; redirection
Hirsch *(14)*	416-1	Tractor-trailer van-66 80,080-48.4-15		188.0		50	Concrete parapet CMB type and metal rail; truck rolled over
Hirsch *(15)*	230-6	Tractor-trailer van-64 79,770-49.1-15		200.5		54	Concrete parapet and metal rail; smooth redirection
Kimball *(10)*	BR-14	Tractor-trailer van-50 40,000-57.3-15.6		117.0		59	Collapsing ring bridge rail, 10-ft deflection; truck rolled over, defective rail
Hirsch *(16)*	911-1	Tractor-trailer tank-72 80,120-51.4-15		188.6		90	Concrete parapet; smooth redirection

[a] In Tests 3451-9 and 10 the school bus had a CG of 50 in. before impact. During impact with the 27-in.-high rail, the front axle was knocked out from under the bus and the front end of the bus dropped 24 in. The CG was almost instantly lowered 7 in. down to 43 in. before the rear axle impacted the rail. This unusual behavior had a significant stabilizing influence on the bus.

[b] Corrected for shifting load.

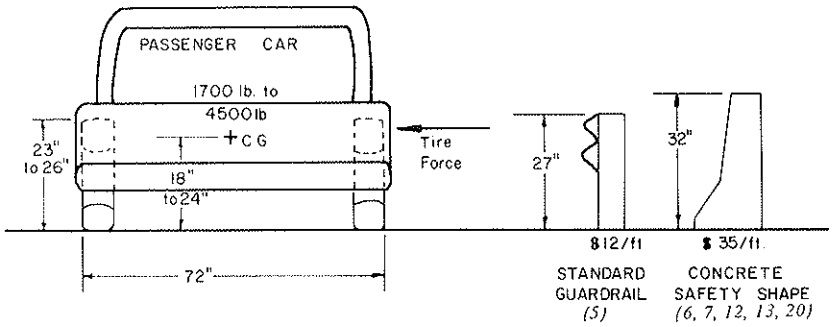


FIGURE 1 Basic properties of passenger automobile and effective longitudinal barriers.

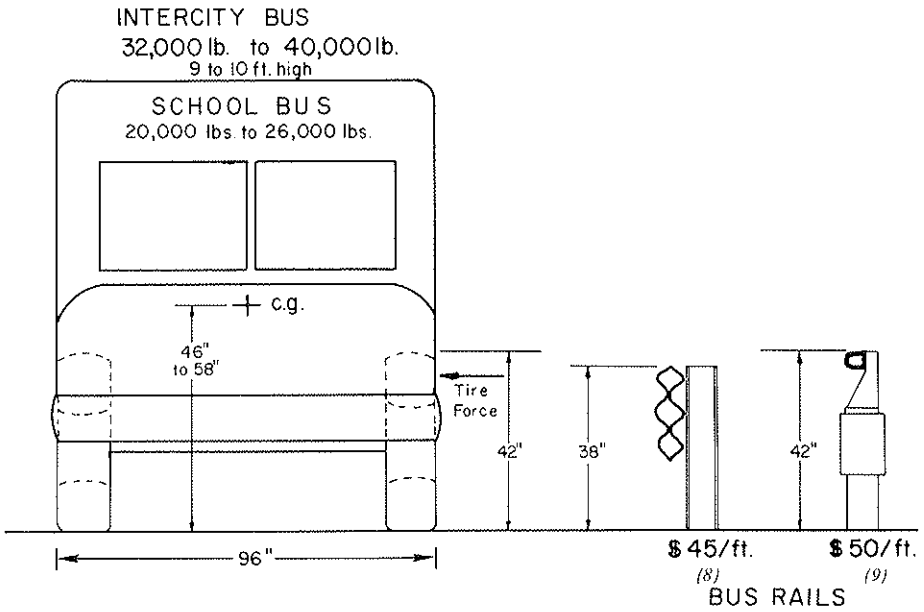


FIGURE 2 Basic properties of buses and two effective longitudinal barriers.

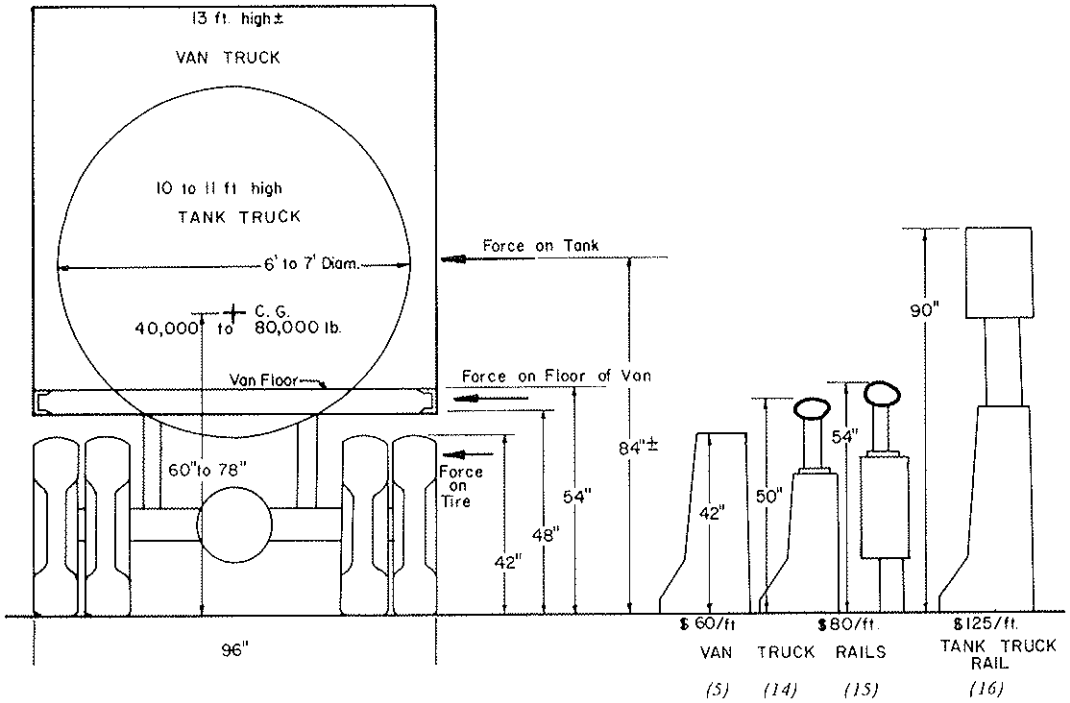


FIGURE 3 Basic properties of tractor-trailer trucks (van and tank types) and some longitudinal barriers that have restrained and redirected them.

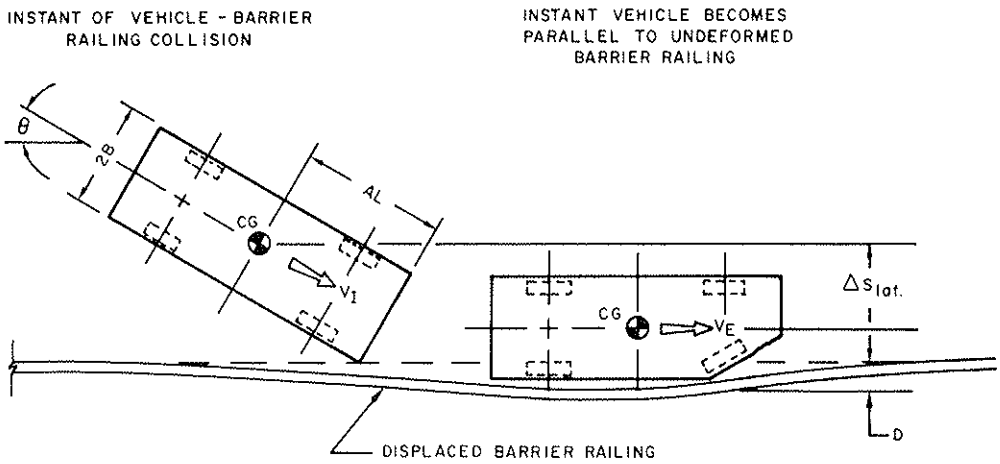


FIGURE 4 Mathematical model of vehicle-barrier railing collision *(22)*.

$$\max F_{lat} = (\pi/2)(\text{avg } F_{lat}) \quad (4)$$

The longitudinal forces on the rail could be determined by multiplying the lateral forces times the coefficient of friction (μ) between the vehicle and the rail. The symbols used are defined as follows:

L = vehicle length (ft),
2B = vehicle width (ft),
D = lateral displacement of barrier railing (ft) assumed to be zero for rigid rail,
AL = distance from vehicle's front end to center of mass (ft),
V_I = vehicle impact velocity (fps),
V = vehicle exit velocity (fps),
θ = vehicle impact angle (degrees),
μ = coefficient of friction between vehicle body and barrier railing,
a = vehicle deceleration (ft/sec^2),
g = acceleration due to gravity (ft/sec^2),
m = vehicle mass (lb-sec^2/ft), and
W = vehicle weight (lb).

Equations 1-4 express average vehicle decelerations as a function of (a) type of barrier railing, rigid or flexible; (b) dimensions of the vehicle; (c) location of the center of mass of the vehicle; (d) impact speed of the vehicle; (e) impact angle of the vehicle; and (f) coefficient of friction between the vehicle body and the barrier railing. When computed deceleration values from these equations were compared with full-scale vehicle crash test data, it was found that these equations predict the behavior of standard-sized passenger automobiles to an accuracy of ±20 percent. Such a comparison is remarkable when the simplicity of the model and the difficulties involved in acquiring and reducing data obtained from full-scale dynamic tests are considered.

These equations were used to compute the lateral impact forces a vehicle would impose on a rigid traffic rail or bridge rail (Figure 5). For articulated vehicles like tractor-trailer trucks, only the tractor is considered to strike the traffic rail. The rear axles of the trailer and the load they are supporting are not considered. Numerous crash tests have shown that the big impact force is delivered by the rear tandem axles of the tractor.

Table 1 and Figure 5 present some actual measurements (from load cells) of impact forces during crash tests. Table 1 also gives some estimates of impact forces determined from accelerometers located on the vehicles. These estimates of impact forces from accelerometer readings were made in the following manner:

1. For the passenger automobiles, vans, school buses, and Ford trucks, the accelerometers were located near the CG of the vehicle. The impact forces were obtained by multiplying the maximum average 50-ms acceleration in g's by the total weight of the vehicle.
2. The impact forces for the intercity and scenicruiser buses were obtained as described in Step 1 except for the two tests (tests 8307-1 and 3) by Davis (6). For those two tests, the accelerometers were located over the rear axles and thus the maximum average 50-ms acceleration in g's was multiplied by the weight on the rear axles only.
3. Impact forces for all the articulated tractor-trailer rigs were obtained from accelerometers located on or near the rear tandem axles of the tractor. The maximum average 50-ms acceleration in g's was multiplied by the weight on the rear tandem axles only to obtain the recorded maximum forces.

When these maximum 50-ms forces from the crash tests with buses and trucks striking at nominally 60 mph and 15 degrees are compared with those predicted by Equation 4, they appear to be about 78 percent higher. Some reasons for this could be (a) buses and trucks have a greater wheelbase length, (b) the payload is a larger percentage of the total load and shifts during impact, (c) tractor-trailers are articulated, and (d) these test results are the maximum average 50-ms impact forces whereas the theory is an idealized sinusoidal maximum force that occurs during a time period of 200 ms or more.

HEIGHT REQUIREMENTS OF LONGITUDINAL BARRIERS

In the previous section data were presented on the magnitude of the lateral impact forces to which a longitudinal barrier would be subjected. Although a barrier must be strong enough to restrain and redirect a vehicle, it must also be high enough to prevent the vehicle from rolling over it.

Figure 6 shows a rear or front view of a vehicle striking a longitudinal rail. The force (F_{lat}) is the resisting force of the rail that would be located at the centroid of the rail member or top of a concrete parapet. The height (H) of this resisting force is defined as the effective height of the rail. For example, the top of a standard 12-in.-deep W-beam guardrail is mounted 27 in. high in Texas; however, its effective height (H) would only be 21 in.

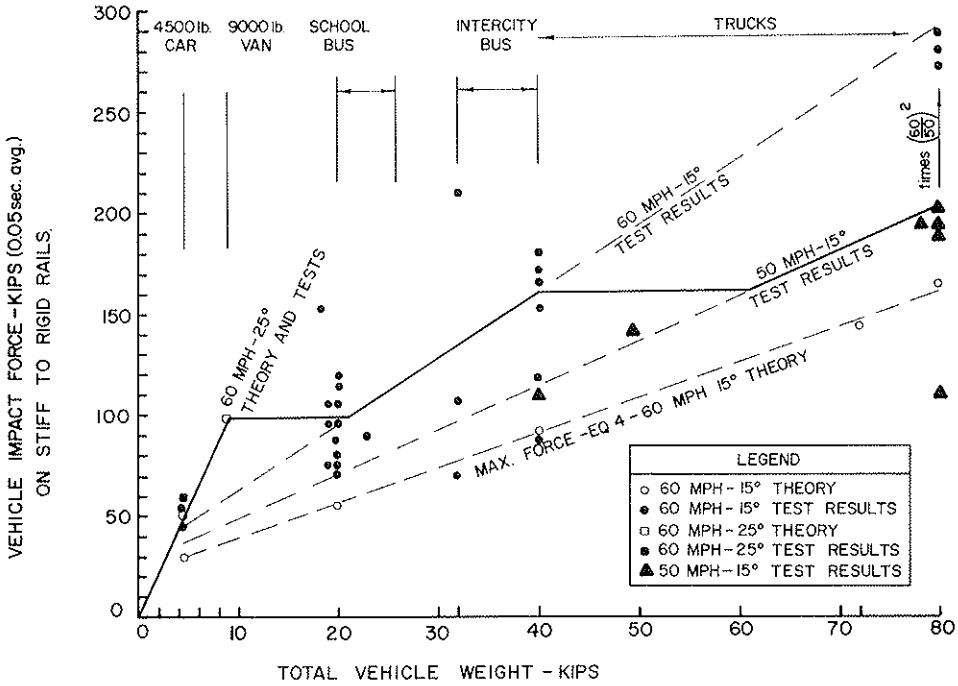
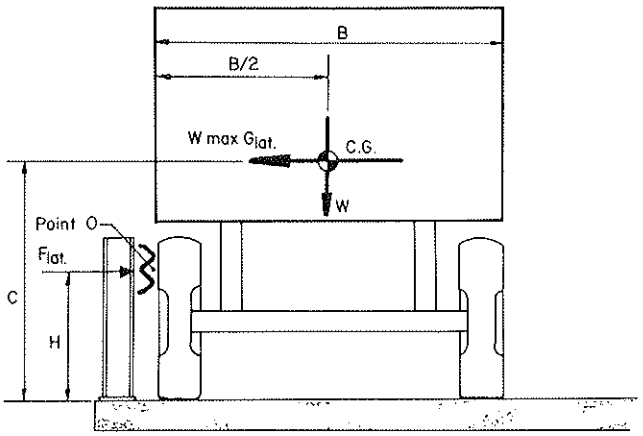


FIGURE 5 Comparison of vehicle impact forces and total vehicle weight, theory and test results for stiff rails.



W = weight of vehicle

max G_{lat} = max lateral deceleration of vehicle from Eq 2

C = height to vehicle c.g., in.

H = effective height of barrier rail, in.

O = center of overturning rotation located at centroid of rail or top of concrete parapet

B = width of vehicle, in.

$F_{lat.}$ = resisting railing force located at effective rail height

$$Mo = W \max G_{lat.}(C - H) - WB/2 = 0$$

$$H = \frac{\max G_{lat.} C - B/2}{\max G_{lat}} \qquad (Eq.\ 5)$$

FIGURE 6 Approximate analysis of bridge rail effective height required to prevent vehicle from rolling over rail.

In many cases the CG of an impacting vehicle may be much higher (C) than the effective height (H) of the rail. The vehicle does not necessarily roll over the rail in this case because a stabilizing moment equal to the weight of the vehicle (W) times one-half the width of the vehicle (B/2) is also acting on the vehicle. Equation 5, shown in Figure 6, indicates the approximate effective height required for a bridge rail to prevent a vehicle from rolling over it. This effective height is a function of the maximum lateral impact deceleration of the vehicle, the height of the CG of the vehicle, and the width and length of vehicle in this simplified mathematical model.

Figure 7 shows a comparison of the required effective height of a longitudinal rail to the CG height for five selected design vehicles. From Figure 7 it can be seen that to prevent a large passenger automobile with a CG of from 20 to 24 in. from rolling over the rail, an effective height of from 16 to 21 in. is required. As mentioned previously, the standard guardrail has an effective height of 21 in. To prevent a school bus with a CG of from 50 to 55 in. from rolling over, the rail would require an effective height of from 38 to 42 in. An intercity bus would require rails of similar effective heights. A large van tractor-trailer truck would require a rail with an effective height of from 50 to 54 in.

SUMMARY AND CONCLUSIONS

The information presented in this paper has shown that longitudinal barriers (guardrails, median barriers, and bridge rails) can be designed and constructed to restrain heavy vehicles such as buses and trucks. Figure 5 indicates the magnitude of the impact forces that these barriers must resist. These forces are for fairly stiff to rigid longitudinal barriers. To redirect a 20,000-lb school bus at 60 mph and a 15-degree angle, the barrier should resist

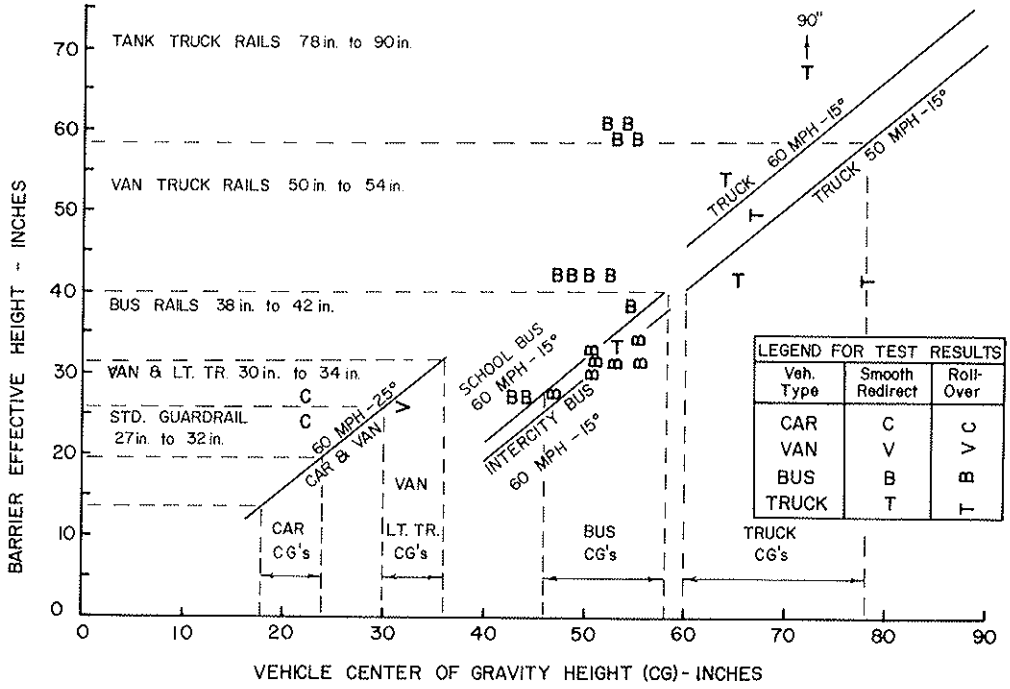


FIGURE 7 Comparison of required barrier height and vehicle CG, theory and test results.

about 100,000 lb of force. To redirect a 40,000-lb intercity bus at 60 mph and a 15-degree angle, the barrier should resist about 165,000 lb. To redirect an 80,000-lb tractor-trailer at 50 mph and a 15-degree angle, the barrier should be capable of resisting about 190,000 lb. Barriers similar to those shown in Figures 2 and 3 have demonstrated this. For precise design details of these barriers, the appropriate references should be consulted.

Figure 7 indicates that to redirect school and intercity buses without rollover, such barriers should be about 38 to 42 in. high. School buses are more vulnerable to rollover than are intercity buses. Figure 7 also indicates that van-type trucks need a barrier from 50 to 54 in. high to minimize rollover at 50 mph and 15-degree angle impact. Tank-type trucks need a barrier from 78 to 90 in. high to prevent rollover at the same speed and angle.

The tests conducted so far indicate that barriers with a vertical face on the traffic side are much better for resisting vehicle rollover. Barriers similar to the 54-in.-high combination rail shown in Figure 2 are an example. On the other hand, the sloping-faced concrete safety shape assists vehicles to roll over. For example, the 42-in.-high concrete safety shape in Figure 2 permitted the vehicle to roll 24 degrees before it contacted the top of the barrier. The 50-in.-high combination rail in Figure 2 permitted the impacting truck to roll 11 degrees before it contacted the upper steel rail.

ACKNOWLEDGMENTS

The author wishes to acknowledge the Bridge Division and District Engineers of the Texas State Department of Highways and Public Transportation for supporting and encouraging much of this research. The FHWA of the U.S. Department of Transportation has also supported this work.

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Traffic Control Device Problems Associated with Large Trucks

DAVID JAY SCHORR

ABSTRACT

The changing pattern of traffic and increased truck volumes and sizes are resulting in blockage of road signs. The inability of drivers to see advisory and warning signs will result in an increasing number of accidents leading to a growing number of law suits with the states as defendants. There are some guidelines that engineers can use, but a general solution is not available at this time.

How often do you find that your view of the road ahead is suddenly obliterated by a truck pulling into the lane in front of you? Then you look in your rear view mirror to find yourself sandwiched between two units with a third passing to your left, and, in the congestion and confusion, you miss an important directional or advisory sign. How many people realize that when they pull out to pass a truck, they may also be cutting off their view of all signs for the next 1/4 mi? And who of us can read a sign more than 1/4 mi away?

Forensic Engineering Services, 1603 Old York Road, Abington, Pa. 19001.

There are potential accident situations developing as a result of the presence of more trucks on the road. Think of drivers misreading, misinterpreting, or missing a sign altogether because of total or partial blockage and then overreacting or overcompensating, or both, in an effort to recover from the situation in which they find themselves. They miss a ramp, pass the intersection at which they should have turned, are in the wrong lane for through traffic, do not see a stop sign, or are confronted with a sudden traffic pattern change. The legal ramifications for the political entity that is responsible for the roadway could be devastating.

Ours is a society that believes that if there is a problem, the solution is to sue. For a plaintiff

to get a case into court, he need only show that the defendant--state, city, or borough--knew that conditions existed that might potentially result in an accident. This places the question of actual negligence in the hands of the jury. Unfortunately, the question is all too often decided by emotions, how much sympathy the attorney can generate for the injured party, not on the factual merits of the case. The bottom line is that the cost to the governing body could be astronomical.

What can be done to correct the problem and stay the inevitable onslaught of law suits and who should do it are the current critical issues. But first, engineers must recognize the problem. Trucks driving in convoy, whether by design or coincidence, clustering as they follow and pass each other are becoming an ever more common sight on highways. Trucks involved in interstate and intercity transport frequently dominate the sight lines on expressways and major roadways. On rural roads traffic movement is often restricted and limited by truck movements, and urban traffic is even more frequently dominated by truck movements and loading and unloading patterns.

The problems that these trucks and other large vehicles create for the effectiveness of current road signs and signals are at best difficult to correct. In reality there are an almost infinite number of problems, most without any complete solution (1-6). The solution could be as simple as removing all trucks from streets and highways. On the other hand, the solution could be as complicated as having to treat each and every sign and signal as a separate engineering problem. Each sign and signal and its location would be given individualized attention, and the final recommendation as to location, height, size, and color would depend only on the ability of the driver to see the sign; there would be no other applicable standards. There may not be a topic for which the advice that is given to the engineer in the standards, to use good engineering judgment, is more significant.

The complexity of the problems is the focus of this paper. An effort is made to alert the reader to some of the many factors of which the engineer needs to be aware and to some of the effects each factor may have on others. No effort is made to recommend a general solution to the problems for, indeed, there may not be one. The factors that affect visibility and comprehension of road signs and signals are many and what is of even more concern is that they are ever changing. Even the most obvious conditions that affect sight lines, such as truck size (height and length) and driver's eye height, have become variables.

There are at least six major categories that must be considered in each evaluation of visibility:

1. Trucks,
2. Automobiles (or other vehicles conveying passengers),
3. Roads,
4. Atmosphere,
5. Human factors, and
6. Signs.

Each of these is a variable with numerous subfactors that require understanding and evaluation (Table 1).

It is doubtful that there is any engineer, researcher, or psychologist with experience in this field who could not add to this list or propose a reasonable and meaningful further breakdown of subfactors, or both. It also becomes obvious that there is no way that the engineer concerned with the problem will be able to account for each and every factor let alone the infinite combination with which the designer will be confronted.

TABLE 1 Visibility Evaluation Factors

Major Factor	Subfactor
Trucks	Size, length, height, width
	Number of units using the road
	Frequency of use
	Speed
	Position
	Following distance (number in a row)
	Passing
	Performance characteristics
Other vehicles	Type
	Driver's eye height
	Vehicle visibility
	Windshield size and angle
	Side windows
	Driver's position
	Visibility angle through windows
	Position of units relative to the truck
	Following
	Passing
	Performance characteristics
Road	Type
	Expressway or other high-speed multilane
	Rural: two lane, three lane, one way, two way, sidewalk
	Urban: parking lanes, sidewalks, buildings, setbacks, overhead wires, crossovers
	Geometry
	Curve: left, right, degree of
	Grade: up, down, percentage, crest, valley, on a curve, and at what point
	Number of lanes--width
	Median strip
	Width
	Type
	Roadside conditions
	Shoulder type and width
	Other recovery areas, if any
	Blockage
	Fixed objects and their location
	Seasonal (trees)
	Lighting (if any)
	Type, location, intensity
	Shadows (fixed objects and others)
	Shadows (seasonal, trees)
	Construction and maintenance zones
Atmosphere	Day- or nighttime
	Weather
	Rain
	Snow
	Fog
	Background color (sky, clouds, etc. affected by geometry and weather)
Human factors (driver)	Performance
	Behavior
	Physical condition
	Eye range (full and effective)
	Peripheral vision
Signs	Location
	Side
	Overhead
	Size
	Color
	Height (sign and post)
	Distance of setback
	Illumination
	Type
	Shape

If any attempt is to be made to analyze the problem, the engineer should consider the road system as composed of at least three categories of road:

1. Expressways and other multilane high-speed highways,
2. Rural roads, and
3. Urban streets.

The special conditions created by construction and maintenance zones should be considered as a separate problem.

The factors listed in Table 2 indicate that the situation generated on expressways is the least com-

TABLE 2 Road Evaluation Factors

Category	Problem	Factor
Open roads (expressways and other multilane high-speed highways)	Passing	Position of trucks
		No. of trucks
		Roadside signs
		Overhead signs (see Figure 3)
	Following	Distance
		Height of truck
		Driver's eye height
		Roadside signs
		Overhead signs
Rural roads	Problems of open road	
	Other conditions	Limited sign locations
		Sharper grades and curves
		Cross traffic
		Pedestrian traffic
		Greater need for signs
		Increased roadside activity
Urban streets	Problems of rural roads	
	Intensification of conditions	Limited sign location
		Sharper grades and curves
		Cross traffic
		Pedestrian traffic
		Greater need for signs
		Increased roadside activity
	Additional conditions	Need for new type of signs
		Small informational signs (street names, no-parking, etc.)
		Higher traffic volumes
		Congestion
		Parking
		Loading and unloading activity
		Traffic control devices
Construction and maintenance zones	Problems of road where zone exists	
	Additional conditions	Limitations caused by the activity
		Change in traffic pattern
		Need to advise
		Limitations of placing temporary advisory signs, channeling devices, traffic control devices
		Constant change

plex. This is due in part to the wider right-of-ways, limited access, and minimal activity allowed on these roads. There are also other advantages: at least two lanes in each direction, built-in periodic overhead sign supports (bridges) that can be used when needed, and two shoulders and off-road recovery areas on which signs can be erected. If all variations in unit size are disregarded, driver's eye height and limitations on sight range due to physical limits of the vehicle design (windshield size and shape, etc.) and human factors, the study can be reduced to its simplest form.

PASSING

Consider a full-sized passenger vehicle passing a 50-ft tractor-trailer. Place the automobile so that the front is adjacent to the rear of the trailer (Figure 1). This condition only limits the passing driver's view of roadside signs for a distance of about 150 ft. Because off-road signs would be visible for quite some distance to the same driver in his following position (before he pulled out to pass), it is unlikely that an observant driver would miss seeing any approaching signs. However, if the first truck is following a second truck (same size) and is within 63 ft of that unit, the obliterated sight distance is increased more than threefold to more than 455 ft. What causes the real problem is a potential third unit ahead of the second truck. If the spacing between the second and third units is 192 ft or less, the obstructed sight distance is extended and can be as much as 0.2 mi. This number and spacing of tractor trailers on roadways is already common. (Observations and an attempt to photograph were made on the Pennsylvania Turnpike, US-81 between the Pennsylvania Turnpike and Hagerstown, and PA-132.) Any roadside signs in that area become totally useless to the passing driver. The effect can be further exacerbated if there are other trucks in the sequence or if the units are longer than 50 ft (i.e., 55-ft or double trailers, both of which are common on expressways today). In such situations, there appear to be five obvious solutions:

1. Use overhead signs in the passing lane,
2. Repeat the signing,
3. Require greater spacing between trucks,
4. Restrict passing, and
5. Place signs high enough to be seen over the top of a trailer.

Only Solutions 1 and 4 can provide reasonable assurance that a sign will be seen in adequate time for a driver to properly react to its message and Solution 4 is impractical and unrealistic on a multilane highway. Placing an additional sign is not a guarantee because it too can be missed in another passing maneuver although the probability of seeing the second sign is greatly increased. With regard to spacing, maintaining a safe distance appears to be the most sensible approach, but this is something over which the traffic engineer has no control. It is a matter of law enforcement, and restrictions are not likely to be effective. Elevating the signing so that it can be observed over the top of the trailers is shown in Figure 2. For eye height of a driver in a full-sized automobile (3 ft 9 in.) and a trailer height of 12 ft, the required sign height is dependent on the angle of observation over the top left edge of the trailer. The heights shown are also a function of the position of the sign relative to the edge of the road. The elevations shown are for the bottom of any sign. If a sign were placed just off the shoulder or 10 ft from the outside edge of the right lane, it would have to be 34 ft above the roadway to be completely readable. It is interesting to note that, in this situation, adjustment for truck height or driver's eye height is not as critical as might be expected. A 3-in. adjustment only changes the sign height requirement by 1 ft.

FOLLOWING

Following a truck presents no significant difficulty in seeing off-road signs on an expressway but does place a limitation on seeing overhead signs (and signals on other roadways). If, for illustration, a speed of 50 mph is assumed and the following vehicle is dropped back five car lengths (95 ft), an overhead sign with a 16-ft clearance cannot be fully seen until the trailing car is within 141 ft or 1.92 sec of the sign (Figure 3). On the surface, this does not look like too bad a situation, but two factors make it significant. First, it involves the uncontrollable situation of spacing or following distance. Second, should the following take place in the passing lane under the passing conditions explained previously, it eliminates overhead signing as an effective way of providing information to drivers of vehicles in the passing mode.

The information in Table 2 makes it obvious that no matter how complex any solution may be for an expressway, rural roads, urban streets, and construc-

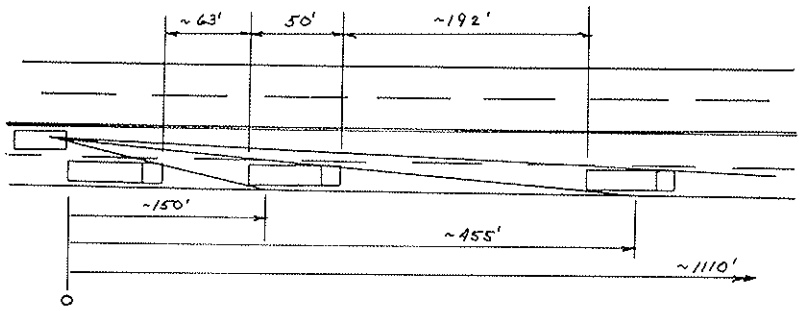

FIGURE 1 Horizontal sight line--passing mode.

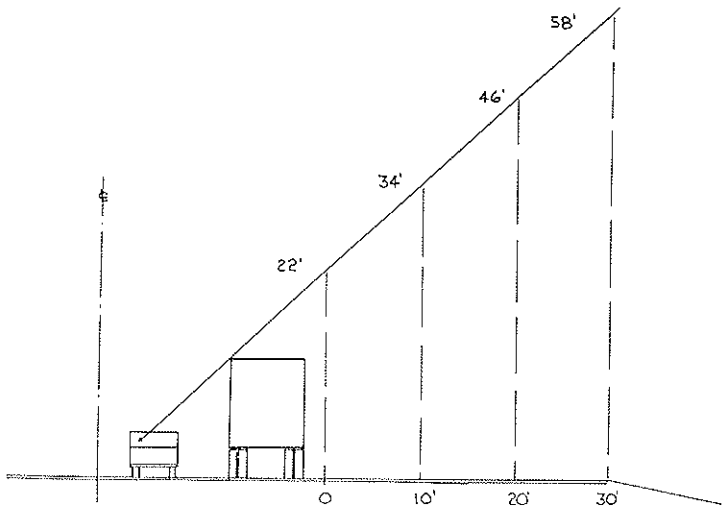

FIGURE 2 Vertical sight line--passing mode.

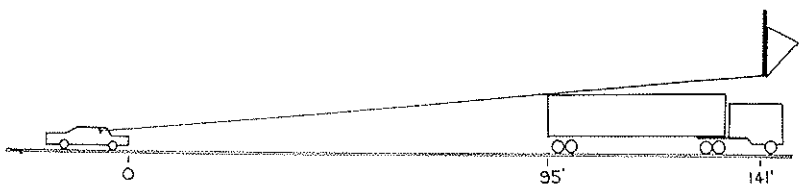

FIGURE 3 Vertical sight line--following mode.

tion zones present an even greater number of limitations and hence reduce the probability of finding a general solution to almost zero. Add to this the effect of the atmosphere and nighttime driving and engineers are faced with an almost impossible task of establishing an effective set of general standards to satisfy all conditions.

There is no way that the situation can correct itself. Eye heights are dropping because of the reduced size of automobiles. Trucks are getting longer and to a limited extent taller and wider by virtue of carrying oversized loads. The number of trucks is growing and clustering is increasing. Engineering logic says to do the best that can be done, that nothing is ever 100 percent sure. But as William A. Silver once said, "Engineering is logical but the law is the law and law is not logical." The present condition is sure to result in an increase in the number of highway accidents and, with the elimination of sovereign immunity to protect the states, an increase in law suits that name states as primary defendants.

Lawyers will quote from the engineering standards that say that signs should be placed and be of a proper size and color to be readable at a great enough distance to allow a driver ample time and distance to interpret and react to the message. They will say that the signs were placed to advise or warn drivers and that by placing them in a position that allowed a truck to obstruct the view of the automobile operator (hence depriving him of the advice and warning he had the right to expect) the state placed the driver in danger. Hence, the state knew of a potential hazard, or it would not have placed the sign there in the first place, and then left the driver vulnerable by allowing the sign to be blocked by a truck. With the image of the state as the bad guy and the members of the jury placing themselves in the position of the confused driver who suddenly found himself in an accident because he missed a warning sign, a good plaintiff's attorney will get the jury to render a verdict based on emotion, not on engineering logic and the dictates of prudent driving.

The lawyer would further argue that, by virtue of the developed standard practice of placing signs,

the driver has the right to expect to be advised of and warned about hazardous situations and changes in traffic patterns. Hence the state is negligent in not having placed signs properly, which resulted in violation of the driver expectations and an accident.

What can the engineer do to better protect the public, the state, and the engineer? There may not be an answer, and, if there is one, this author does not yet know of it. Several guidelines, however, are worth noting:

1. Repeat signs whenever practical and if possible use them in combination, roadside and overhead.
2. Educate the driving public placing emphasis on the need to maintain a suitable spacing not just for stopping, which is where the big push has been, but also for observing and reacting to signing.
3. Increased effort to enforce good driving with legal criteria, not just a recommendation, for vehicle spacing.
4. Document all design and sign placement with a permanent record to show that a study was conducted and the installed system was the best possible.
5. Do a site inspection and check for any unusual condition that did not show up on paper (e.g., need for added height for a traffic signal to be seen when installed just beyond the crest of a grade or moving a sign so it precedes a driveway that is used by trucks that would block it while waiting to enter the traffic stream).

The cost of repeating, educating, enforcing, documenting, and inspecting for a year may indeed be less than the cost of legal defense of a single law suit or being found liable, or both.

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Start-Up Accelerations of Heavy Trucks on Grades

THOMAS D. GILLESPIE

ABSTRACT

The acceleration performance of heavy trucks starting on grades represents an important boundary consideration in highway design. Trucks generally possess the lowest levels of acceleration performance. This, in combination with their length, makes them the highway vehicle class that requires the greatest time to proceed across intersections. Especially at railroad-highway grade crossings, truck performance establishes bounds on the timing requirements for warning devices. Guidelines on truck acceleration performance on level grades have been established in the past for use in highway design. However, the new size and weight allowances warrant review of these guidelines and present the opportunity to consider the influence of grade on performance. The performance bounds for truck acceleration depend on both the truck properties and the driving techniques used by the driver. The application of some "rules of thumb" for driving and knowledge of truck power train design provide a basis for a first-order estimate of the start-up performance range expected on various grades. The analysis is applied to the problem of clearance times at rail-highway grade crossings where regulations mandate travel in the start-up gear and the time-distance relationships are thus determined by the gear required for starting on the grade. The analysis finds that attainable speed decreases with increasing grade and affects the clearance times that should be allowed.

The acceleration performance of heavy trucks represents an important boundary consideration in highway design. Trucks generally possess the lowest levels of acceleration performance, which, in combination with their length, makes them the highway vehicle class most likely to impede other traffic. Truck acceleration performance affects highway design in a number of areas:

1. Need for climbing lanes on long upgrades,
2. Lengths of acceleration lanes at traffic merge areas,
3. Sight distance and signal timing at traffic intersections, and
4. Clearance times at rail-highway crossings.

A truck's ability to accelerate from a full stop and clear an intersection is the key interest in the last two categories. Yet, in many situations, the truck cannot accelerate continuously; it is constrained by regulations or grade conditions to traverse the intersection at the limiting speed of the starting gear.

For example, federal, state, and Bureau of Motor Carrier Safety (BMCS) regulations require vehicles transporting passengers and hazardous materials to stop at rail-highway grade crossings regardless of the type of warning device present. The regulations then require the vehicle to proceed through the crossing in a gear that allows the vehicle to complete the crossing without a change of gears. This practice results in the vehicle negotiating the crossing at the speed limit of the starting gear, thus increasing exposure time in the hazard zone.

Similar situations may arise at an upgrade intersection where the truck must stop. The low gear required for starting on steep grades does not allow the attainment of sufficient speed to permit a shift to a higher gear to be accomplished without the vehicle again slowing to a stop. Thus the driver must proceed in the starting gear to a point where the grade diminishes. Though infrequent, these conditions create hazards by obstructing traffic and presenting longer exposure times in intersections.

Predicting truck clearance times in intersections requires an understanding of the mechanics of the start-up process and how it is influenced by grade. The objective of this paper is to present an analysis of these mechanics and apply the methods to the problem of predicting truck clearance times at rail-highway grade crossings. The analysis is limited to heavy highway trucks typified by the 80,000-lb tractor-semitrailer.

MECHANICS OF TRUCK START-UP

The start-up acceleration process for heavy highway vehicles involves two phases of operation--the start-up mode in which the clutch is being engaged and the full-throttle acceleration mode from the point of full clutch engagement until maximum engine speed is reached in that gear. In normal situations, a gear shift would occur and the truck would continue to accelerate. However, at rail crossings or upgrade intersections it may be necessary for the vehicle to proceed in the start-up gear while it clears the crossing or intersection. For heavy highway vehicles, predominantly powered by diesel engines, the maximum speed is controlled by the gear selected and the governed speed of the engine. Unlike passenger automobiles, heavy trucks are routinely driven with the engine at or near its maximum governed speed.

The start-up mode involves the least time and distance; therefore, it has little direct influence on the time required to traverse an intersection. Indirectly, the practices that are used may have significance in that the selection of the gear in which the vehicle is started affects travel speed through the crossing. The low gear ratios on tractor-trailers may include "deep reduction gears" intended for use in terminal operations or when the

Transportation Research Institute, University of Michigan, Ann Arbor, Mich. 48109.

vehicle is off the road. The choice of the lowest gear may limit the vehicle to an unnecessarily low speed for the duration of the travel, and the gear chosen may be so low as to be an unreasonable choice by the driver.

In the acceleration phase the full torque output of the engine is applied to accelerating the rotating components as well as the vehicle itself. Characteristically, in the lower gears, the effective inertia of the rotating components may be as large as, or larger than, the translation inertia of the vehicle. The effective inertia of the rotating components is dependent on the square of the gear ratio; thus selection of an unnecessarily low gear will adversely affect the acceleration phase of the travel.

Finally, during travel at top speed in low gear, which represents the majority of the time required for clearing the crossing, the specific choice of gear, and the associated top speed, will directly influence the time consumed.

Selection of Starting Gear

Little quantitative information is available to aid in identifying the normal practices of drivers in selecting a gear and actuating the clutch from a full stop in tractor-trailer vehicles. At best the practices that would be used by a conscientious driver can be described as releasing the clutch while the engine is at, or near, idle speed without allowing the engine to stall then fully applying the throttle while the vehicle is accelerated. Klokkenga (1) suggests that the gear selected for start-up should be the highest gear (lowest numerical ratio) for which the steady-state gradability of the vehicle exceeds the local grade by 12 percent. According to knowledgeable engineers in companies that are manufacturers of heavy-truck clutch and transmission components, a good rule of thumb for engine output torque during start-up is 500 ft-lb. The gear selected must be of a high enough ratio to allow complete engagement within a period of 1 to 2 sec without pulling the engine to a speed of much less than 500 revolutions per minute (rpm). In general, this requirement is in consonance with the 12 percent reserve suggested by Klokkenga.

The gearing of the transmission is not the only reduction in the driveline; there is also gearing in the rear axle. In addition, the maximum speed of the engine, which is fixed by the governor, will directly affect the maximum speed possible in a gear. Although these factors appear to present additional variables of choice in the analysis, rules of thumb can again be applied to help rationalize a selection. The rationale derives from the fact that the overall gearing in the majority of trucks is selected in conjunction with the governed speed of the engine and the tire size to produce a maximum speed of about 60 to 65 mph. Rarely would the selection yield a maximum speed of 55 mph or less. Accordingly, gearing for much higher speeds is not common except in some of the specialty (owner-operator) trucks used in the West. Inasmuch as the top speed is normally associated with direct drive in the transmission (a 1:1 input-to-output speed ratio), the maximum speed possible in the other gears of the transmission can be readily estimated by dividing 60 mph by the numerical ratio. For example, a low gear ratio of 10:1 will have a top speed of 60/10 or 6 mph. Ratios for the lowest gears commonly used in tractor-trailers range from 7.5:1 to 15:1. Maximum speeds for these two ratios are, respectively, 8 and 4 mph.

The gear ratio necessary to start up on different grades while satisfying the previously mentioned criteria can be determined by writing Newton's second law for the truck. Neglecting aerodynamic drag because of the low speed, the governing equation is

$$F_x = W\,(a/g + G + C_r) \tag{1}$$

where

F_x = tractive force at the ground from the drive wheels,
a = vehicle acceleration,
g = gravitational acceleration,
G = grade, and
C_r = coefficient of rolling resistance.

The drive force comes from the engine and can be related to engine torque by

$$F_x = TN_tN_rn_tn_r/R \tag{2}$$

where

T = engine torque,
N_t, N_r = transmission and rear axle gear ratios,
n_t, n_r = transmission and rear axle drive efficiencies, and
R = radius of drive wheels.

In clutch engagement the transmission gear ratio selected for Equation 2 must be high enough that the acceleration in Equation 1 is sufficient to achieve a vehicle speed that synchronizes with the engine idle speed (nominally 500 rpm) in a period of about 1 to 2 sec. The parameters N_r/R in Equation 2 simply relate engine speed and forward speed in direct drive (high gear). Substituting ϕ_m/V_m for these parameters and combining the equations yields a quadratic equation for the transmission ratio required by the start-up conditions:

$$N_t^2 - \{[W(G + C_r)V_m]/Tn_tn_r\phi_m\}N_t - (WV_m^2\phi_s/\phi_m^2t_sgTn_tn_r) = 0 \tag{3}$$

where

t_s = time for clutch engagement,
ϕ_s = engine speed at synchronization,
ϕ_m = maximum governed engine speed, and
V_m = maximum governed vehicle speed with $N_t = 1$.

This equation can then be solved to find the "best" ratio for the start-up gear at any grade condition. Figure 1 shows the solution as a function of grade with positive values for upgrade conditions and negative values for downgrades. Solutions for both a 1- and a 2-sec clutch engagement time are shown. For this plot a maximum vehicle weight of 80,000 lb has been assumed, and the other parameter choices are as listed in Table 1.

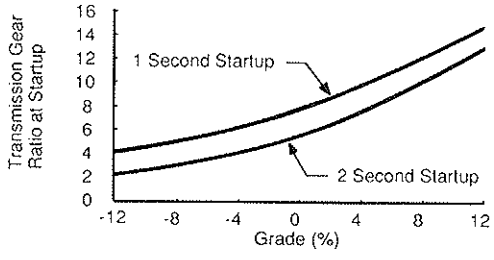

FIGURE 1 Start-up gear ratio predictions for various grades.

TABLE 1 Parameters for Calculation of Start-Up Gear

Symbol	Meaning	Value
W	Gross vehicle weight	80,000 lb
C_r	Coefficient of tire rolling resistance	0.0041
V_m	Maximum speed in direct drive	60 mph
T	Engine torque during start-up	500 ft-lb
n_t	Transmission efficiency	0.90
n_r	Rear axle efficiency	0.85
ϕ_m	Maximum engine speed	2000 rpm
ϕ_s	Engine speed at full engagement	500 rpm
g	Gravitational constant	32.2 ft/sec^2
t_s	Time of clutch slip during engagement	1, 2 sec

These predictions of the starting gear ratio align well with real practice and support the constraint assumptions from which they are derived. For the most common upgrade conditions, normally limited to the 0 to 6 percent range on main highways, Figure 1 indicates that the gear of choice would not have to exceed a ratio of 9:1. The first gear on many modern trucks intended for highway use has a ratio of 7.5 to 8. Thus they can readily start up with only 1 to 2 sec of clutch slip on grades of up to 4 percent and with only slightly greater abuse on grades of up to 6 percent. On steeper grades a lower gear is used if available or, if a lower gear is not available, the driver may typically try to avoid coming to a full stop.

On downgrades the gear ratios drop well below the 7.5 to 8 range available in first gear. This simply reflects the reality that under these circumstances the vehicle can be started in a higher gear (lower numerical ratio).

On the assumption that the maximum road speed is 60 mph, the maximum speed in the start-up gear can be determined. The value is simply 60 divided by the numerical ratio. By defining the best start-up gear ratio as that required for the 1-sec clutch engagement, the maximum speed in the start-up gear can be predicted for upgrade conditions as shown in Figure 2. For level conditions the speed is approximately 8 mph but may drop as low as 4 mph on steep grades

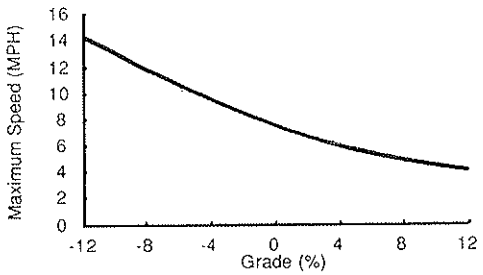

FIGURE 2 Maximum speeds in the start-up gear required for 1-sec clutch engagement.

where the highest available reductions (a 15:1 first gear) would be used.

Full-Throttle Acceleration

When clutch engagement is complete, routine driving practice with a heavy truck involves full-throttle acceleration up to maximum engine speed. At that point the governor will cut back on the amount of fuel supplied to prevent the engine from going above its rated speed. These characteristics for a typical diesel engine (1) are shown in Figure 3. Note that over most of the operating speed range (600 to 2000 rpm) this engine's characteristics are close to those of a constant torque model. Some other modern diesel engines have a "torque rise" with decreasing speed that is nearly equivalent to constant power.

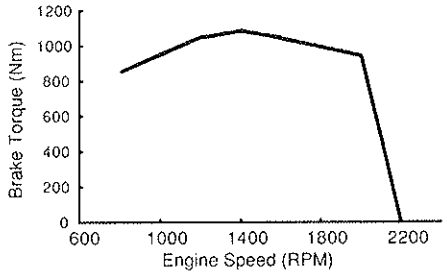

FIGURE 3 Torque characteristics of a typical truck diesel engine.

To precisely calculate acceleration during this period of full-throttle application, more comprehensive equations, which take into account the rotating inertias of the drive system, must be written. Instead of taking this laborious route, a computer simulation available at the University of Michigan Transportation Research Institute (UMTRI) has been used to study this phase of truck acceleration.

Among the computer programs developed over the years at UMTRI for simulating various aspects of heavy-truck performance is a Truck Acceleration Performance program that operates on the IBM-PC desktop computer (2). The program calculates, as a function of time, the speed of a truck encountering arbitrary grades at full throttle. A typical application is to calculate the change in speed as a truck encounters a grade in the road and thus to generate a speed-distance or speed-time profile. Time-based integration is performed using the mathematical equations that include the drag effects on the vehicle from tire rolling resistance, aerodynamic drag, and grade. Tractive force from the engine is calculated as a function of its torque output, gear ratios, driveline efficiencies, tire radius, and other appropriate factors. Algorithms are included that select the highest possible gear at all times, determine appropriate shift points, and account for the loss of engine effort during the shift periods.

The program was used to calculate truck accelerations during a start-up maneuver as discussed in the preceding section. Specifically, when started from zero speed, an engine torque value of 500 ft-lb is applied to the driveline until such time as the vehicle speed increases to match a 500 rpm engine speed. Thereafter, a wide-open throttle condition that allows the vehicle to accelerate to the governed speed of the engine is assumed. For these calculations the engine was assumed to be a constant torque source equivalent to 300 hp at a governed speed of 2100 rpm (typical values for a tractor-trailer of the assumed type). The tires were assumed to be of the radial type with a rolling coefficient obtained from SAE recommendations. For these low speeds the aerodynamic parameters are not important and were simply set at typical values for van-type trailers. The gross combination weight was 80,000 lb.

Start-up simulations were conducted using the first gear ratios indicated in Figure 1 for the 1-sec clutch engagement time on different grade conditions. When maximum speed has been reached in a gear, in the case in which the driver cannot shift, the time-distance values beyond this point can be readily calculated from

$$dX = V_{max}dt \qquad (4)$$

where

dX = incremental distance traveled,
V_{max} = maximum velocity in that gear, and
dt = incremental time consumed.

The calculated time-distance plots for grades of from 0 to 10 percent are shown in Figure 4. The initial start-up and acceleration phase occurs at the far left of the figure (covering no more than the first 25 ft). In this region performance varies little with grade. However, because of the different limiting speeds on each grade, the performance curves begin to diverge significantly when maximum speed has been reached. Note that the distance traveled during start-up and acceleration is relatively small, from 6 to 20 ft, which is only a fraction of the length of the truck. The majority of the travel distance (and time) required to cross and clear an intersection is covered while the truck is running at constant speed, and the primary variable controlling this performance is the ratio of the start-up gear. Thus the exact shape of the curves in the initial phase of acceleration is of little significance. The constant-speed region of the time-distance curves represents the area of primary interest.

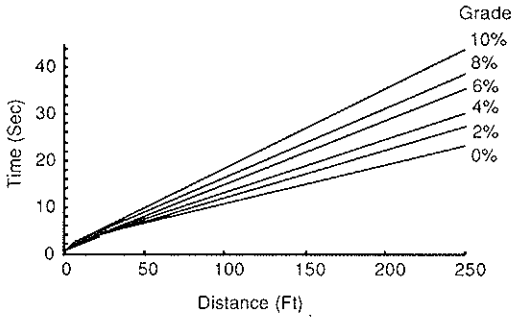

FIGURE 4 Time-distance plots calculated for various grades.

If the time-distance lines are extrapolated back to zero distance, they all intercept the time axis at approximately 3 sec or less. Consequently truck performance can be easily characterized by

$$\text{Time (sec)} = \text{Distance}/V_{m1} + 3 \text{ (sec)} \quad (5)$$

where V_{m1} is the maximum speed in the start-up gear as shown in Figure 2. The reasonableness of the time constraint on clutch engagement can be seen more directly by considering what happens if only one gear ratio is used on all grades. Simulated starts were performed using only a 12:1 low gear ratio on 0, 5, 10, and 13 percent grades. The results are shown in Figure 5. The initial curve in each of the plotted lines represents the clutch engagement segment along with acceleration to maximum engine speed. For 0 and 5 percent grades, the clutch is engaged within the first few seconds of the start-up process. Note that this cannot be seen readily on the plots but is obtained from the calculations in the computer simulation. Therefore the 12:1 ratio is a reasonable gear selection for starting on those grades. At the extreme of the 13 percent grade, the clutch must slip for more than 5 sec, which would be considered a very severe start for the vehicle. Thus a still lower gear would be selected by an experienced driver if it were available. Many truck transmissions do not have a gear

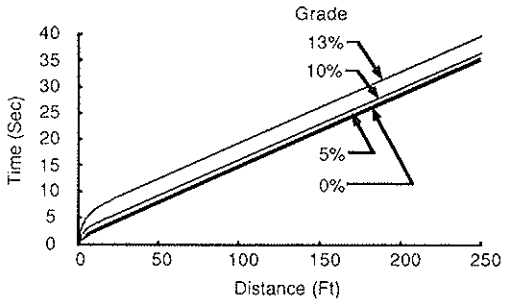

FIGURE 5 Simulated start-up with a 12 to 1 gear ratio on various grades.

lower than the 12:1 ratio, in part because a 13 percent grade situation is infrequent. However, if such a grade were encountered, most drivers would compensate either by stopping at a different point, where the grade was less severe, or by not coming to a complete stop in order to avoid having to overwork the clutch.

PREDICTIONS OF RAIL-HIGHWAY CLEARANCE TIMES

A practical application of this analysis lies in estimating the clearance times for various tractor-trailer combinations under differing grade conditions at rail-highway grade crossings. This issue was brought up in a recent project conducted by Goodell-Grivas, Inc., "Consequences of Mandatory Stops at Railroad-Highway Crossings" (3). At such crossings the vehicle is required to stop before the crossing and then proceed in a low gear until the crossing is cleared. The highway between and in the near vicinity of the tracks, as shown in Figure 6, represents a hazard zone where collision with a train is a risk. The time interval from vehicle

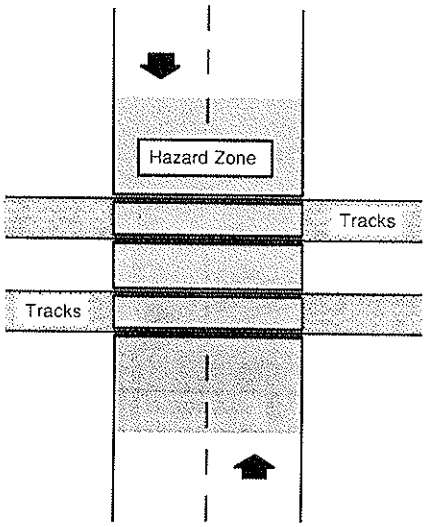

FIGURE 6 Hazard zone at a rail-highway grade crossing.

start-up until the rear of the unit clears the hazard zone is therefore a key variable in properly timing warning devices and establishing necessary sight distances.

By using Equation 4 the problem of estimating clearance times is reduced to a decision about what

TABLE 2 Clearance Time (sec) for 65-ft Tractor-Semitrailer

	Length of Hazard Zone (ft)								
Grade (%)	35	45	55	65	75	85	95	105	115
0-2	11.5	12.4	13.2	14.1	14.9	15.8	16.6	17.5	18.3
3-5	14.4	15.5	16.6	17.7	18.9	20.0	21.2	22.3	23.5
6-10	16.6	18.0	19.4	20.7	22.1	23.5	24.8	26.2	27.5
11-13	20.0	21.8	23.5	25.2	26.9	28.6	30.3	32.0	33.7

TABLE 3 Clearance Time (sec) for 70-ft Doubles

	Length of Hazard Zone (ft)								
Grade (%)	35	45	55	65	75	85	95	105	115
0-2	11.9	12.8	13.6	14.5	15.4	16.2	17.1	17.9	18.8
3-5	14.9	16.1	17.2	18.3	19.5	20.6	21.8	22.9	24.0
6-10	17.3	18.7	20.0	21.4	22.8	24.1	25.5	26.9	28.2
11-13	20.9	22.6	24.3	26.0	27.7	29.4	31.1	32.8	34.5

TABLE 4 Clearance Time (sec) for 115-ft Triples

	Length of Hazard Zone (ft)								
Grade (%)	35	45	55	65	75	85	95	105	115
0-2	15.8	16.6	17.5	18.3	19.2	20.0	20.9	21.8	22.6
3-5	20.0	21.2	22.3	23.5	24.6	25.7	26.9	28.0	29.1
6-10	23.5	24.8	26.2	27.5	28.9	30.3	31.6	33.0	34.4
11-13	28.6	30.3	32.0	33.7	35.4	37.1	38.8	40.5	42.2

constitutes a reasonable value for the maximum speed in the start-up gear. From Figure 4 is it evident that clearance times can vary over a substantial range depending on the starting gear selected. On shallow grades any of the gears could be selected depending on driver choice. On steeper grades there are fewer choices for a reasonable gear. Hence it is only possible to estimate a range of clearance times that reflects the variations in driver practices.

The times required for semitrailers, doubles, and triples were estimated for the Goodell-Grivas study using an analysis similar to that presented here. Assuming 80,000-lb gross vehicle weights and a 300-hp engine, the clearance times for the three vehicle combinations given in Tables 2, 3, and 4 were obtained. It should be noted that overall vehicle length is the only distinction among semitrailers, doubles, and triples that is relevant to this analysis. That is, the acceleration and speeds achieved by the vehicles are not affected by the configuration because they are all assumed to be at maximum gross weight. It should also be noted that the power reserve of the engine is adequate in every grade condition to reach governed speed. Although in reality the maximum speeds possible will be slightly reduced on higher grades (perhaps by a factor of a few percent) because of engine governor characteristics, this effect was neglected in the analysis.

The shortest times for each of the grade ranges in the tables can be interpreted as reasonable estimates for typical vehicles and driver practices on the indicated grade.

The longest times, listed for grades of from 11 to 13 percent, not only apply to rail-highway crossings with those grade conditions, they may also be interpreted as the prevailing clearance times for that portion of the truck population with gear ratios of approximately 15:1 available (and presuming the drivers proceed through the crossing in the lowest gear).

Clearance times that are best to use in any particular application, of course, should be selected with knowledge of the consequences. The maximum times shown in the tables (for the 11 to 13 percent grades) are suggested as the most conservative choice, regardless of the grade at a crossing, for design of warning devices. Though the choice will be conservative in comparison with the performance of a majority of the tractor-trailers encountering any given rail crossing, it will accommodate the slower vehicles that exist within the overall truck population.

COMPARISON WITH EXPERIMENTAL DATA

Data were collected at three locations in Michigan for comparison with the predictions. All of the locations were at zero grade because of the difficulty of finding crossings that were on roadway grades. Observations of time versus distance were made of a total of 77 tractor-trailers that came to a complete stop before the crossing. There was no knowledge of which vehicles were empty or loaded or of gross vehicle weight. In addition, no doubles or triples were included in the sample.

The data are compared with the predictions for tractor-semitrailers on the 0 to 2 percent grade in Figure 7. The experimental data are quite scattered, reflecting the difference capabilities of each vehicle and the different practices of each driver. The majority of the experimental points properly fall below the prediction, which is an estimate of the upper bound on the clearance times. The points below the prediction line represent vehicles loaded to less than the maximum permitted weight assumed in the calculations or started from higher gears, or both. Lower weight allows better performance and thus shorter clearance times. The points that fall above the prediction line would represent trucks started in gears that are lower than necessary or drivers who are more casual in their start-up practices.

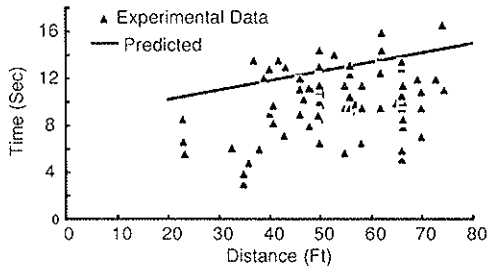

FIGURE 7 Comparison of experimental and predicted time-distance relationships on a level grade with tractor-semitrailers.

CONCLUSION

The agreement seen in Figure 7 indicates that nominal predictions of truck start-up performance can be made from the analysis presented. Because trucks and driver practices differ, the performance is variable. However, the predictions from the analysis capture approximately 90 percent of the vehicles and at that level provide a reasonable estimate of maximum clearance times required. Experimental data were only available for level grade crossings, so the accuracy of the predictions for steeper grades cannot be assessed.

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California Design Practice for Large Trucks

EARL ROGERS

ABSTRACT

Highway design engineers have long been concerned about the wide offtracking characteristics of large trucks. With the enactment of the Surface Transportation Assistance Act (STAA) of 1982, a truck longer and wider than ever before was allowed on the Interstate and qualifying primary system known as the designated system. Following the passage of the 1982 STAA, the California State Legislature changed state laws to comply with federal truck regulations on the designated system. The new state law prescribes access to the system. Service access and terminal access are separately defined. The former is handled by the State Department of Transportation. Local agencies are responsible for the latter. California has adopted an Interstate design vehicle based on dimensions spelled out in the 1982 STAA. A computer program is now available for generating offtracking plots. As a tool for highway design engineers a set of truck-turn templates has been prepared. Design practice is evolving. Current practice requires highway designers to use the Interstate truck-turn templates on all new or upgraded interchange projects. Some exceptions to the current practice are allowed. On 3R projects at designated service access points large trucks are accommodated if the work can be done at reasonable cost with no extra right-of-way. The answer to who bears the cost of retrofitting interchanges and upgrading local roads for terminal access is also evolving. The most likely arrangement will probably be shared cost with both public and private funding.

Highway design engineers have long been concerned about the offtracking characteristics of large trucks. With the enactment of the Surface Transportation Assistance Act (STAA) of 1982, a truck longer and wider than ever before was allowed on the Interstate and qualifying primary system known as the designated system.

California has traditionally controlled offtracking by limiting the maximum kingpin-to-rear axle dimension. Currently, California law places a 38-ft limit on the kingpin dimension except on the desig-

Geometric Design & Standards Branch, Office of Project Planning & Design, California Department of Transportation, 1120 N Street, Sacramento, Calif. 95814.

nated system where the combination of a 48-ft-long semitrailer and an unlimited vehicle length have disrupted the controls on offtracking. What does this mean for design engineers? At the very least it means that ramp intersections will have to be redesigned with wider flares; electroliers, signs, and signal poles will have to be moved; loop ramps will require widening; curbs and gutters will have to be replaced; and drainage inlets will need to be reset.

California has built about 6,000 freeway ramps on the Interstate system alone. If all of those ramp intersections were to be fixed and if all of the loop ramps were to be widened, the estimated cost would exceed $100 million.

The extra 6 in. of width allowed by the new law has also contributed to the severity of offtracking. Moreover, in the big cities California has restriped many miles of freeways using shoulders to provide an extra traffic lane and reducing the lane width to 11 ft. An 8.5-ft-wide truck must now operate with less maneuvering space in the narrow lanes.

ACCESS TO THE DESIGNATED SYSTEM

After enactment of the 1982 STAA, a bill was introduced in the California legislature to make state laws conform to federal laws for trucks using the designated system. This legislation was required of the states in order to avoid losing federal highway funds. The California bill that was signed into law by the governor in June 1983 also dealt with the question of access to the system, dividing access into two parts.

Service access is permitted for fuel, food, lodging, and repairs provided those services are within 1/2 mi of an interchange.

Terminal access, on the other hand, places no limits on the distance between terminal and interchange. "Terminal" is somewhat broadly defined as a facility at which freight is consolidated to be shipped; or where full-load consignments may be offloaded; or at which vehicle combinations are regularly maintained, stored, or manufactured.

Service access is handled by the California Department of Transportation (Caltrans) with the concurrence of local agencies. An interchange where service is currently available is reviewed for big truck accessibility. If fix-up work, such as minor paving or moving signs, can be done inexpensively, it may be handled by minor contract, or it may be incorporated into a 3R pavement rehabilitation project. Service access signs are placed on the freeway in advance of the interchange making it legal for a big truck to exit or enter the freeway without being cited. Figure 1 shows a service access sign.

Terminal access is treated differently. Local agencies bear the primary responsibility for terminal access routes. Instead of placing a limitation on the distance from the designated system to a terminal, California reviews each route for safe operation on a case-by-case basis. Terminal access routes originate as a request from the terminal operator to the local agency. Figure 2 shows a terminal access sign.

INTERSTATE DESIGN VEHICLE

Since enactment of the 1982 STAA, Caltrans designers have been using two different design vehicles. The Interstate design vehicle is for use on the designated system, which now includes 4,200 centerline miles of Interstate and non-Interstate freeway and some conventional highway.

The off-Interstate design vehicle shown in Figure 3 is the model used for the remainder of the state highways in California, about 11,000 centerline miles.

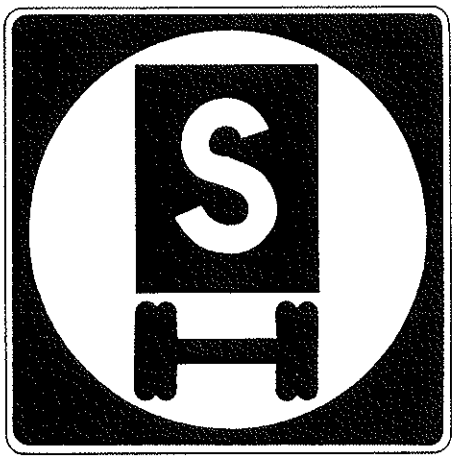

FIGURE 1 Interstate truck service access sign (blue on white).

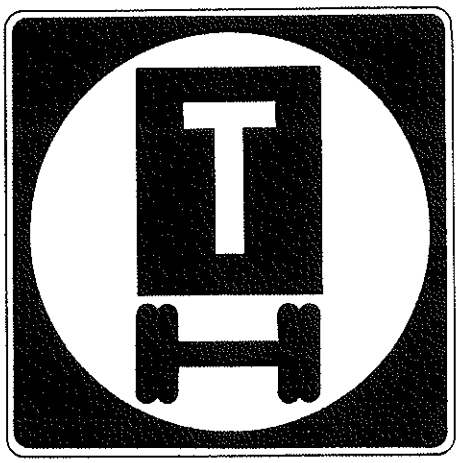

FIGURE 2 Interstate truck terminal access sign (blue on white).

Figure 4 shows the dimensions of the Interstate design vehicle, a hypothetical tractor-semitrailer combination that is being used in California for the design of interchanges on the designated system. The 48-ft length and the 8.5-ft width of the semitrailer are the only dimensions spelled out in the 1982 act. All other dimensions are assumed.

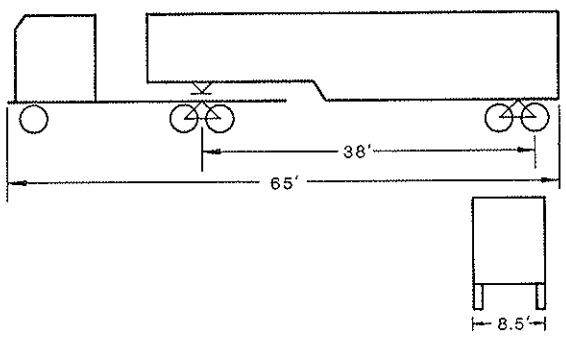

FIGURE 3 1983 California off-Interstate design vehicle.

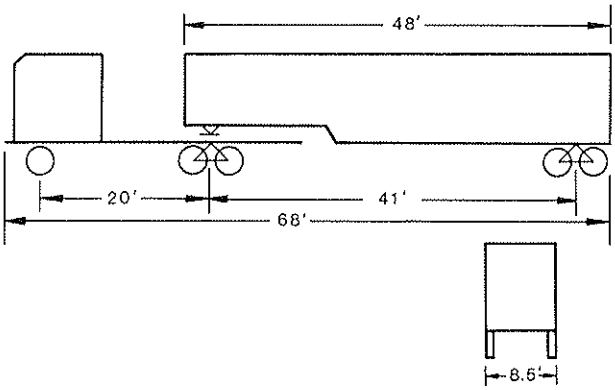

FIGURE 4 1983 Interstate design vehicle.

The Interstate design vehicle became the basis for truck-turn templates developed by Caltrans in early 1983 following passage of the 1982 STAA. The original work on the templates was done using a drafting tool (tractrix integrator) that draws an inked trace of the turning path of a tractor-semitrailer combination on a sheet of vellum to a predetermined scale. The job took many months to complete because the initial graphic work had to be drawn to a large scale, digitized, run through a computer smoothing routine, and finally drawn at a reduced scale on an automated plotter.

In November 1983 Caltrans ran field tests using a tractor with a wheelbase of 15 ft 6 in. and a semitrailer kingpin-to-rear axle dimension of 40 ft 6 in. These dimensions, somewhat less than those of the Interstate design vehicle, yielded swept widths that were about 5 percent less at maximum offtracking than the results of the graphic plots.

More recently Caltrans has been using a computer program that was originally developed by the University of Michigan Transportation Research Institute in cooperation with FHWA (see Vehicle Offtracking Models by M.W. Sayers in this Record). Caltrans added a number of enhancements and adapted the program to run on an IBM 360 driving a Calcomp or Xynetics automated plotter. The computer program will generate offtracking plots for virtually any vehicle combination in a fraction of the time previously required. The computer-generated plots show good results compared with those of field tests, hand-drafted graphic plots, and the SAE formula. The tractrix integrator and hand-drafted graphic solutions to offtracking problems have become history.

Figures 5 and 6 show the Interstate truck-turn templates for a 50- and a 60-ft turning radius. Figure 7 is a tabulation of loop ramp widening needed to accommodate the Interstate design vehicle.

DESIGN PRACTICE IN CALIFORNIA

The shortest horizontal curve radius necessary for the design vehicle to stay within a 12-ft traffic lane while turning through 180 degrees of central angle is about 300 ft. In other words, all offtracking will take place within the 12-ft lane provided the outside wheel of the steering axle is crowding the outside lane line. On the assumption that a large truck should not cross a lane line, especially a centerline, when traveling around a curve, and allowing for some margin of error, a 400-ft minimum curve radius was established for the designated system. Certain routes on conventional highways have been deleted from the designated system because of the 400-ft radius rule.

At freeway off-ramps the wide pavement area needed for truck turns at the crossroad intersection has raised some safety and operational questions. The wide pavement area makes sign placement difficult, increases the chance of wrong-way moves because the off-ramp looks more like an on-ramp, and requires longer pedestrian travel distance. Despite these concerns, current practice requires highway designers to apply the Interstate truck-turn templates on all new construction or major modifications to interchange and intersection projects. However, cost, right-of-way, environmental sensitivity, local agency desires, and the type of community be-

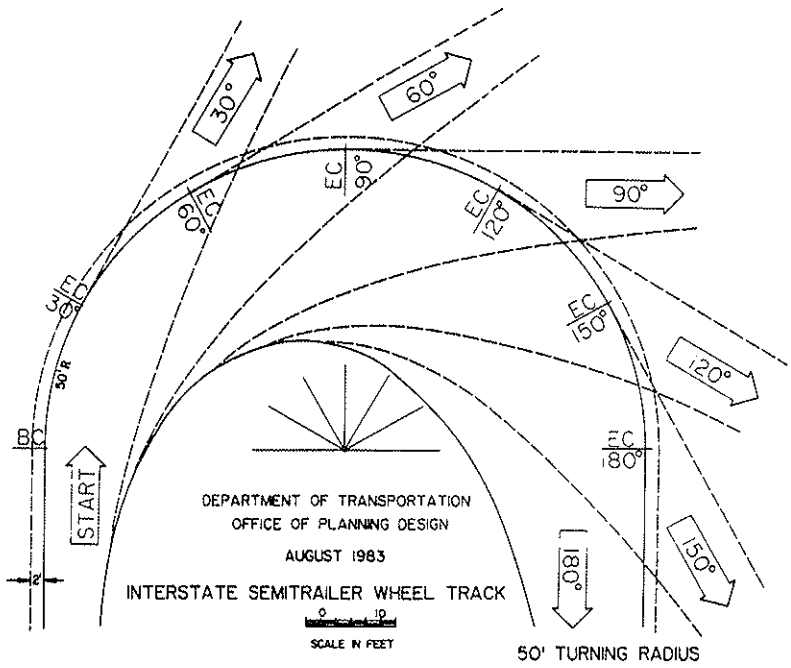

FIGURE 5 Interstate truck-turn template for 50-ft turning radius.

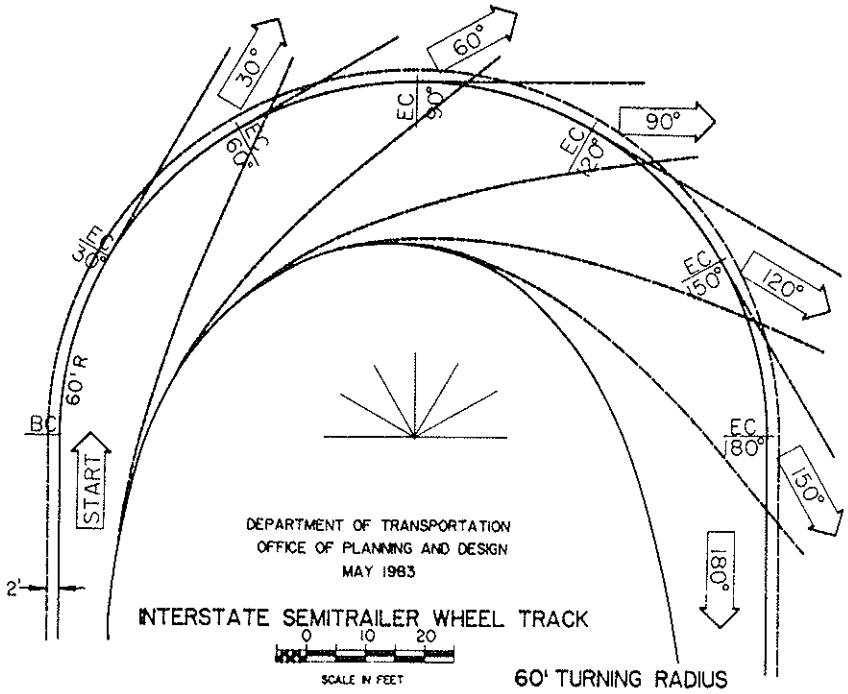

FIGURE 6 Interstate truck-turn template for 60-ft turning radius.

LOOP RAMP WIDENING			
Ramp Radius	Widening	Lane Width	Lane Plus Shoulder
120'	6'	18'	26'
150'	4'	16'	24'
180'	3'	15'	23'
210'	2'	14'	22'
250'	1'	13'	21'
300'	0'	12'	20'

FIGURE 7 Loop ramp widening needed to accommodate the Interstate design vehicle.

ing served are factors that will, on occasion, require exceptions to the current practice.

Who bears the cost of retrofitting interchanges and upgrading local roads for terminal access is unclear at this time. The most likely procedure will be a specific cost determination for each route with the state, the local agency, or the private sector paying all or a share of the cost.

On 3R-type projects at both service and terminal access points, modifications may be made to accommodate large trucks if the work can be done at reasonable cost with no taking of extra right-of-way.

To date only a handful of terminal access routes have been requested, and most of these are for military reservations like Fort Ord in the Monterey area and Vandenberg Air Force Base in Santa Barbara County. As more and more terminal access routes are proposed, it is expected that a good-faith effort will be made by the participants to reach an agreed-on sharing of costs.

CONCLUSIONS

California's design practice for large trucks is still evolving. Caltrans and the local agencies are reluctant to undertake expensive retrofitting of freeway interchanges and street intersections for the sole purpose of big truck access when other serious operational improvements are begging for money. All parties are still waiting to see just how the cost sharing for terminal access routes will shake out.

Except for signs and minor improvements for service access, no major construction work, such as widening, moving drainage inlets, extending pipes, or moving traffic signals, has yet been done. It is not entirely clear how such projects should be funded and whether they should compete with other operational improvement projects for federal and state dollars.

Swept Paths of Large Trucks in Right Turns of Small Radius

J. R. BILLING and W. R. J. MERCER

ABSTRACT

When a large truck makes a turn of large radius, the driver may make a steady steering input and the swept path of the vehicle through the turn may be computed by an offtracking procedure. However, when a large truck must make a turn of small radius, such as a right turn at an urban intersection, the driver must devise a more complex steering input that minimizes intrusion of the vehicle into the space of other vehicles and also keeps the trailer vehicle units from encroaching on the curb. Determination of the steering input necessary for such a turn is defined as a steering-path problem. In this paper is described a purely geometric approach to the solution of the steering-path problem that results in steering inputs and swept paths typical of those observed in real turning maneuvers by large trucks. The method has been implemented as a computer program for IBM mainframe computers.

When a large truck makes a turn, the strategy used by the driver depends on some relationship among the turn geometry, the overall vehicle length, and the length and turning properties of the individual vehicle units. For purposes of discussion, only 90-degree turns of circular arc will be considered, such as the situation in which a two- or four-lane two-way road intersects with a four-lane two-way road (Figure 1). Such a turn would often be made after a complete stop, but that is not relevant to this discussion. The method to be presented is, however, quite general and may be applied in other situations.

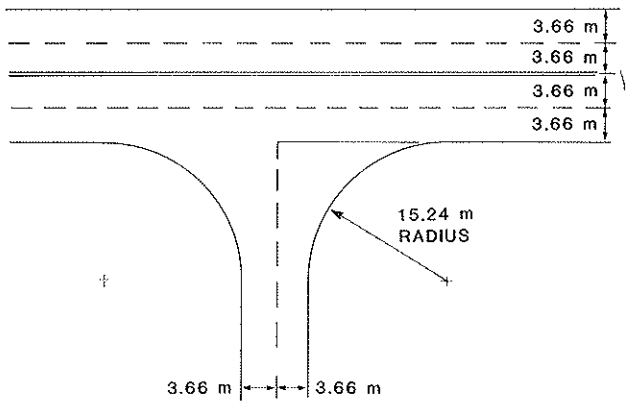

FIGURE 1 Typical intersection.

When a truck turns left at the example intersection, the driver may start from a central position in the entry lane and maneuver to terminate in the curb lane of the exit roadway, as shown in Figure 2. Such a turn typically would have a radius of 25 m or more. The driver simply steers such that the tractor

Automotive Technology Office, Technology and Energy Branch, Ontario Ministry of Transportation & Communication, 1201 Wilson Avenue, Downsview, Ontario M3M 1J8, Canada.

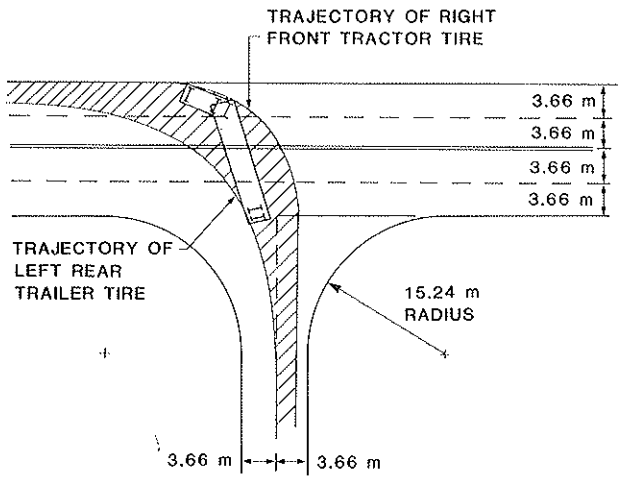

FIGURE 2 Left-turn trajectory of tractor-trailer at typical intersection.

steering axle follows a circular arc. The remainder of the vehicle combination follows, and its trajectory can be computed by the standard methods of offtracking (see "Vehicle Offtracking Models" by M.W. Sayers in this Record). There is generally enough space in the roadway intersection that the offtracking of the vehicle does not interfere with other traffic or obstructions such as curbs or islands. Such turns are straightforward because the turn radius is sufficiently large in comparison with the size of the truck and the roadway width that a steady steering input by the driver is all that is necessary to make the turn.

An entirely different situation arises, however, when the truck turns right at the same intersection, as shown in Figure 3. The driver will usually move as far as possible to the left in the entry lane to increase the available radius. This radius could be further increased by moving even more to the left, if traffic would permit. [Such a move is considered hazardous because of the possibility that a following vehicle (cycle, motorcycle, or small car) also

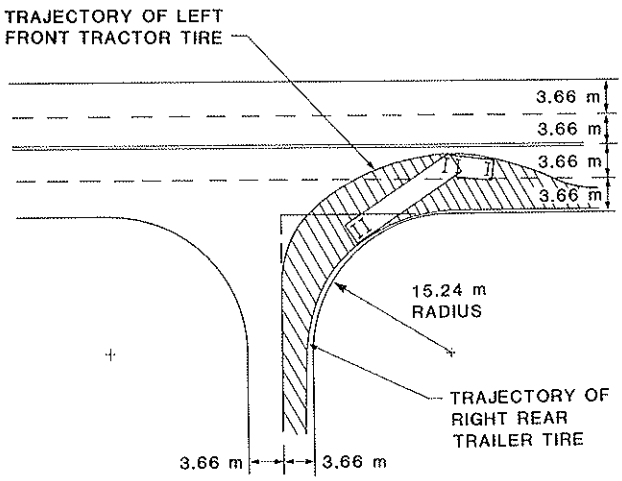

FIGURE 3 Right-turn trajectory of tractor-trailer at typical intersection.

intending to make the right turn might be tempted to pass inside the truck and would be trapped between the truck and the curb as the truck proceeded through the turn.] The truck driver negotiates the turn using a steering input that is intended to minimize intrusion of the tractor into lanes other than the exist lane and to keep trailing vehicle units from running over the inside curb. The steering input demands of the driver may be rather complex, especially for multiply articulated vehicle combinations--doubles and triples. The driver essentially has to solve an optimization problem, minimizing intrusion into other lanes while subject to the constraint of not running over the curb.

There is evidently a great difference between the steering input made by a driver in a right turn and that made in a left turn. This difference arises from the much larger left-turn radius, due to the available space of the oncoming traffic lane, that allows the driver to constrain the steering axle to follow a circular arc so that the rest of the vehicle will follow. For a right turn the driver generally constrains the rear axle of the vehicle to follow some path that comes close to but does not impinge on the curb, for at least some critical portion of the turn.

For a left turn the swept path of the vehicle can be determined in a straightforward manner by an offtracking procedure. However, for the small-radius right turn, no such procedure has been available. The steering-path problem is therefore defined as the determination of the steering input a driver must provide to make a right turn with the rear of the vehicle following the curb as closely as possible. This problem is unconstrained in the sense that the driver is free to make the turn subject only to the limitations that encroachment on the curb and movement to the left outside the entry lane are not permitted. A constrained steering-path problem, in which the driver would also be required to avoid obstacles such as other vehicles or islands, might be defined. This is considered much more complicated and is beyond the scope of this paper.

The steering-path problem may be addressed in several ways. It might for instance be treated as a multivariable optimization problem, a boundary value problem, or a feedback control problem. The latter might indeed be a rather instructive approach. A purely geometric approach, which depends on the assumption of a path for the rear unit of the vehicle traversing the curve, is described next.

STEERING-PATH PROBLEM

The steering-path problem is the determination of the driver's steering input in a turn when the rear of the vehicle tracks around a curve. If a path is assumed for the track of the turn center of the rear unit of the vehicle around the curve, the problem may be solved purely geometrically.

First, some definitions are necessary. A vehicle unit is a component of a vehicle that may steer or articulate relative to an attached vehicle unit. The tractor steering axle is, by this definition, considered a vehicle unit, and it tows the body of the tractor. Trailers and conventional converter dollies are also vehicle units, as are any self-steering axles attached thereto. A vehicle combination is a vehicle composed of a number of vehicle units, with the steering axle following some path prescribed by the driver and subsequent vehicle units in tow. An offtracking procedure is a computational algorithm by which the trajectory of the towed units of a vehicle combination may be obtained when the steering axle follows a prescribed path.

For purposes of this paper, the turning properties of a vehicle combination will be determined by following the common practice of using a zero-width "bicycle model" of the vehicle, in which each vehicle unit has an equivalent wheelbase or distance from its hitch point to turn center (see paper by M.W. Sayers in this Record). When a vehicle is driven at low speed along a prescribed path, the steering axle follows that path and at every point is tangential to it, if the small slip effects present are ignored. An offtracking procedure then permits the trajectory of the towed units of the vehicle to be determined. The offtracking procedure used in this paper is the method of pure pursuit. As the ith unit of a vehicle combination moves in small steps along the path, the problem is to find the position and orientation of the towed unit. This is done on the geometric assumption that the turn center of the towed unit ends on a line joining its new hitch point location and its previous turn center location. Other more sophisticated procedures are available and could as readily be used.

Now consider the steering-path problem. The key to this is the realization that, at every point through the turn, the turn center of the last unit of the vehicle combination is tangential to the specified path. At first, therefore, the steering path may be generated by an offtracking procedure with the vehicle reversed through the turn, as shown in Figure 4 for the rear trailer of a vehicle combination. This leads immediately, however, to three problems: First, if a towed unit, which in reverse tracking becomes a towing unit, has the hitch ahead of the turn center of the unit it is towing in reverse, then that unit is unstable and will simply perform a pirouette as shown in Figure 5. This usually arises for the tractor because of the location of the fifth wheel. This problem is overcome by making a shift of hitch point so that stability is obtained. Usually the shift required is small, no more than a few centimeters. The second problem is that units towed in reverse, and particularly the tractor, tend to cut inside the specified path initially as turning in reverse is commenced. This is actually what is required to back around a corner but is unrepresentative of a right turn. This problem is cured simply by modifying the path so that the tractor is not permitted inside the curb, as shown in Figure 6. A smooth transition may be inserted between the reverse offtracking and the modified paths. When the modified path has been established, the vehicle is driven forward to generate a swept path using an offtracking procedure. This may result in a minor

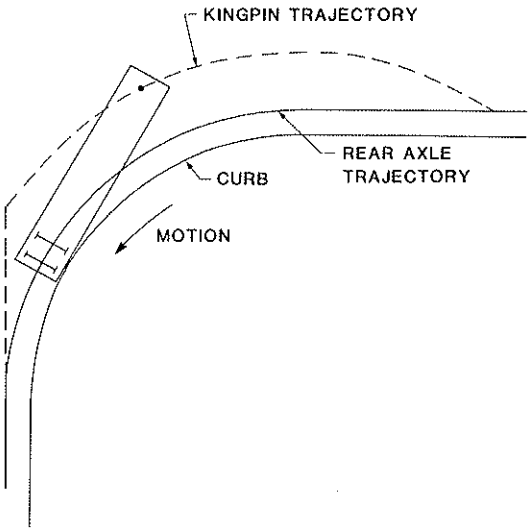

FIGURE 4 Locus of trailer kingpin as vehicle unit is backed with axle tangential to a specified curve.

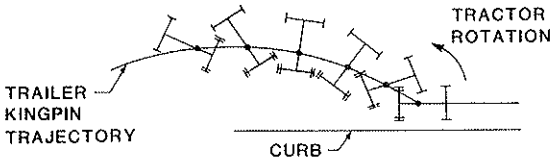

FIGURE 5 Tractor pirouette.

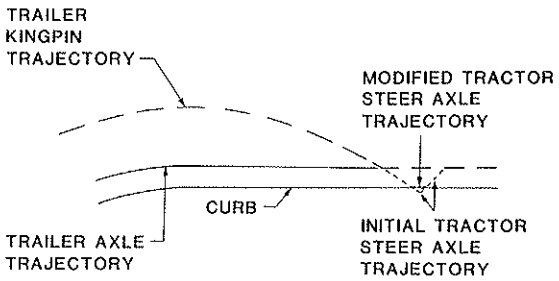

FIGURE 6 Modification of tractor path necessitated by reverse offtracking.

incursion (a few centimeters) of the the vehicle inside the specified path. The final problem arises during this forward offtracking step. For some combinations large steering angles, which exceed the steering angle limit, may be required. When the steering limit is reached in the forward offtracking phase, that steering angle must be held and the vehicle made to proceed forward in a steady turn until that turn trajectory meets the desired trajectory when a smooth transition is arranged onto the desired trajectory.

COMPUTER PROGRAM

The steering-path method outlined has been programmed in FORTRAN for an IBM 308X-series computer. The program proceeds in the following steps:

1. Define a coordinate system origin and define 251 rays at 1-degree angles from +10 to +260 degrees, as shown in Figure 7.
2. Define the curb as a 90-degree circular arc of given radius at the points where the curb meets the rays, also shown in Figure 7.
3. Specify the path of the centerline of the rear unit of the vehicle, either as a 90-degree circular arc or as a set of points measured from test data. In the former case, data points are defined on the rays. In the latter case, the given points are fitted by a series of cubic splines and data points are generated on the rays by solution for the point of intersection of each ray with the appropriate cubic spline curve.

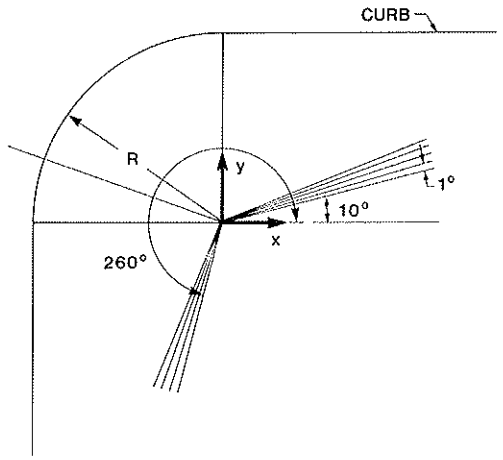

FIGURE 7 Coordinate system.

4. Define the geometric data for the vehicle.
5. Position the rear of the vehicle on the final (10-degree) ray, and perform the rearward offtracking procedure until the rear of the vehicle reaches the 260-degree ray.
6. Correct any inward incursion of the tractor inside the specified path, as described earlier.
7. Compute the required steering angle for the corrected path by using a cubic spline curve fit to the 251 data points of the path, which gives directly the tangent to this path at each point.
8. Wherever the steering angle exceeds the vehicle steering limit, produce a circular arc of the minimum steering radius until it meets the corrected path and arrange a suitable transition where the paths meet.
9. Drive the vehicle forward using the offtracking procedure along the modified path from Step 8 and develop the actual steering angle using the procedure described in Step 7.
10. From the hitch position of each towing unit, the orientation of each towed unit, and the edge geometry of each towed unit, compute a vehicle swept path as the innermost and outermost limits where each vehicle edge at each step of the offtracking procedure meets each of the rays. Then compute the clearance between the inside of the vehicle and the curb.
11. Print the results and store them for plotting.

The program was written to develop the method described in this paper. It is not considered suitable for highway geometric design purposes. It could be relatively easily modified in various ways, such adding curb- or path-generation methods to represent particular highway geometric design standards, building in the dimensions of standard vehicles used in highway geometric design, or including other off-

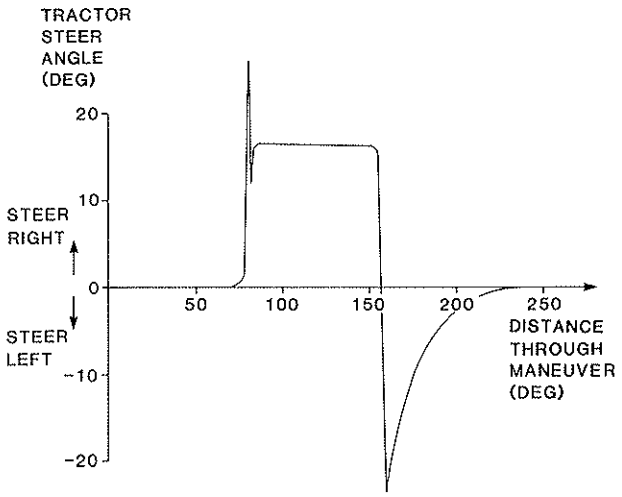

FIGURE 8 Tractor steering angle history in right turn of 9.144-m radius.

tracking procedures. The program takes only a few seconds to run. However, because of the accuracy necessary in this type of geometric computation, double-precision arithmetic is used throughout. Many large arrays are also used to simplify programming. The program is therefore not readily transferable to a microcomputer, though undoubtedly it could be with some modification to the flow of computation and some careful evaluation of the actual number of rays required for particular vehicles. The program will deal with vehicles of up to seven units, which is the conventional triple. It will not at present handle a multibranched chain of vehicle units, though again, that might not be a difficult modification.

Figure 8 shows the steering angle history of a tractor in combination with a 13.7-m (45-ft) semitrailer computed by this program in a 90-degree right turn of 9.144-m radius. This is clearly different from the steady value used in conventional offtracking procedures. It is of interest that the program of Sayer could be used to determine the steering path in a right turn by suitable choice of steering input segments through a series of iterations.

The major limitation of both the method and the program is that the user must specify the path of the rear of the vehicle. This path may not always be easily determined. Subtle variations in this path may also cause significant effects on the duration and magnitude of excursion out of the exit lane. Nevertheless, this method does reproduce the characteristic trajectory of the right turn, which is not readily possible with a direct offtracking procedure.

CONCLUSIONS

A method has been developed that permits computation of the swept path of a vehicle combination of arbitrary configuration as it makes a right turn of small radius. This method, the steering-path method, requires a good estimate of the path of the rear of the vehicle if the swept path is to be realistic. Such as estimate may, in some cases, be relatively easily obtained. The method has been programmed in FORTRAN for a large-scale IBM computer system.

The work to date demonstrates that there is a direct computational method for estimating the swept path of a truck combination in a small-radius right turn, a situation for which an offtracking procedure is often inappropriate. When fully developed, the method may be of interest where extended length combinations are required to travel to urban areas that have highway geometrics of a bygone standard.

Consideration of Larger Trucks in Pavement Design and Management

JOHN M. MASON, Jr., and VERONICA S. DRISCOLL

ABSTRACT

Common pavement design methods (empirical and theoretical) and axle load equivalency factors are reviewed. Research on techniques for modeling new truck configurations permitted by the 1982 Surface Transportation Assistance Act is summarized. A synthesis of various pavement management system methods is provided along with two case study examples of the impact of heavy truck loads and the use of double-bottom trailers.

The Surface Transportation Assistance Act (STAA) of 1982 permitted longer, wider, and heavier trucks to operate on the Interstate system and on the primary system designated by the Secretary of Transportation. An initial step toward understanding the impact of this new traffic on roadway pavements requires a knowledge of various pavement design methods.

One of the highest priority needs in pavement design is for data to support future evaluations. In addition to the fundamental pavement structural relationships, the effects of increased loadings on pavement performance and deterioration must be investigated. Composition of the vehicular fleet, axle configurations, weight distributions, tire construction, and magnitude of tire pressures are changing rapidly and are expected to have a significant impact on the rate of highway deterioration (1).

In 1981 the Transportation Research Board prepared a proposal, which was subsequently funded by the FHWA, to do a study entitled the Strategic Transportation Research Study (STRS). The results of the TRB Committee's efforts were reported in Special Report 202, "America's Highways--Accelerating the Search for Innovation." The highway portion of the STRS is currently the Strategic Highway Research Program (SHRP). A major component of the SHRP is the study of long-term pavement performance in the United States. This ambitious undertaking is expected to continue for 20 years. Anticipated data collection includes information on loading, environment, material properties and variability, construction quality, and maintenance levels in pavement distress and performance. The objectives are to evaluate existing design methods, improve design methodologies and strategies for rehabilitation of existing pavements, and improve design equations for new and reconstructed pavements (1).

Given these considerations, the purpose of this paper is to underscore the need to provide an overview of current pavement technology. Conclusions regarding the effects of larger trucks on highway pavements can only be drawn from a perspective of the dilemma associated with establishing a long-term pavement data bank. Among the specific concerns that need to be addressed is the ability to accurately collect and maintain traffic and weight data from which the effects of loading can be determined.

Texas A&M University System, College Station, Tex. 77843.

Traffic is incorporated in design methods primarily through repetitions of an 18-kip equivalent single axle load (ESAL). Conversion of mixed traffic consisting of various axle loads and configurations to an 18-kip ESAL is accomplished through the use of axle load equivalency factors. The most commonly used equivalency factors are the empirical values derived by AASHTO (2). Researchers have attempted to establish theoretical equations to replicate the AASHTO values and to model axle loads and configurations not included in the original AASHTO data base. Treybig (3) has developed a set of equivalency factors for use with flexible pavement design. Sharma et al. (4) have developed equivalency factors for both flexible and rigid pavement designs.

The empirical pavement design methods reviewed in this paper are generally based on the widely used AASHO Road Test results. Boussinesq theory is the basis for elastic layer analysis and is the cornerstone of theoretical pavement design. The theoretical methods identified in this paper include those set forth by the Asphalt Institute (5), Monismith (6), Shell (7), Chevron (8), and Chua and Lytton (9).

At first it may appear that these two approaches are distinctly different. Actually, the design methods vary from pure "field" experience to detailed finite element analysis techniques. As a result it is not uncommon to obtain different answers (pavement thicknesses) from different design methods using identical input factors (2).

Pavement management systems (PMSs), which assess and predict roadway conditions and rank maintenance scheduling in priority order, are valuable tools for calculating the impact of new truck traffic characteristics. Currently implemented PMS methods, including their respective procedures for calculating traffic impact, are reviewed in this discussion.

Also reviewed are studies that investigate specific topics related to the STAA. Included are reports on oil field traffic, double bottoms, and productivity savings.

PAVEMENT DESIGN METHODS

The evaluation of the effects of heavier, wider, and longer trucks is usually accomplished through the use of standard pavement design equations. An understanding of these design methods is therefore necessary to ensure the proper assessment of the impact of these vehicles. Every rational pavement design method consists of (a) a theory to predict failure

or a specific distress parameter or parameters, (b) an evaluation of pertinent material properties, and (c) a relationship between the magnitude of the parameter in question and failure at a specific performance level (2). Both empirical and theoretical procedures are explained.

Empirical Design Methods

AASHTO

The AASHTO pavement design procedure (2) is centered on the idea of performance as the failure criterion. Performance is defined as the ability of a pavement to satisfactorily serve traffic over a period of time. The performance of a pavement at any point in time is measured by the present serviceability index (PSI). PSI is calculated using a regression equation that considers the following distress variables: longitudinal roughness, rut depth, cracking, and patching. A damage equation is used to estimate the number of 18-kip ESALs necessary to obtain a specific value of PSI. The number of axle load applications, however, is a function of pavement structure, terminal PSI value, environmental factors, and subgrade characteristic value. The depth of each layer, the actual design, is then obtained through a regression equation that uses the structural value of the pavement.

Modifications to AASHTO Method

The AASHTO method has been implemented for many years. Alterations, proposed by Lytton et al. (10) for flexible pavements and Darter as cited by Lytton et al. (10) for jointed concrete pavements, exist with respect to the shape of the damage equation. To satisfy both the inherent boundary conditions and the experimental evidence, the equation has been revised to yield an S-shaped curve. The AASHTO design equation is of the form:

$$g = (P_i - P)/(P_i - P_t) = (N/\rho)^{\beta} \quad (1)$$

where

g = damage function that begins at 0 and becomes 1 when $P = P_t$,
P_i = initial serviceability index,
P = present serviceability index,
P_t = terminal serviceability index,
N = number of 18-kip ESALs, and
ρ, β = constants that depend on the pavement structure and the load acting on it.

The equation used by Darter for describing the long-term performance of jointed concrete pavements is of the form:

$$(P - P_f)/(P_i - P_f) = 1/\{e^{\beta[(N/\rho) - 1]} + 1\} \quad (2)$$

where P_f is the asymptotic value of serviceability index that the performance equation approaches.

According to Lytton, the long-term performance of flexible pavements is described by the equation:

$$(P - P_f)/(P_i - P_f) = 1 - e^{-(\rho/N)^{\beta}} \quad (3)$$

Equations 1 and 2 are compared in Figure 1 for an 8-in.-thick jointed concrete slab. Figure 2 is a comparison of Equations 1 and 3 for a flexible pavement section (seal-coated pavement) with a structural number of approximately 1.0. The graphs illustrate the more accurate modeling of field data by the S-shaped curves of Equations 2 and 3.

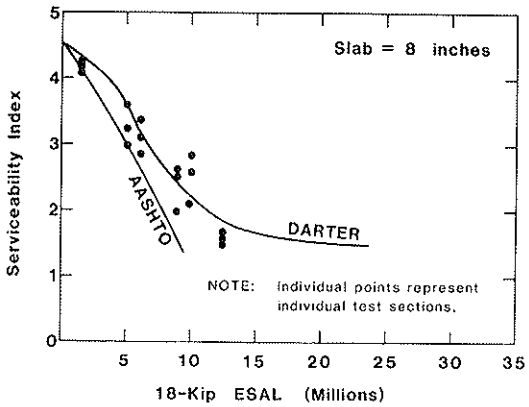

FIGURE 1 Comparison of original AASHTO performance equation, Darter's new performance equation, and actual performance data for 8-in.-thick jointed concrete slab.

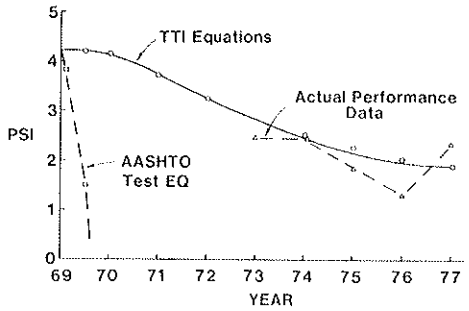

FIGURE 2 Comparison of AASHTO performance equation, Lytton's new performance equation, and actual performance data for a flexible pavement.

Theoretical Design Methods

A significant advancement in flexible pavement design was the introduction of mechanistic design methods that employ the Boussinesq theory for calculating stresses, strains, and deflections. The Boussinesq theory is only directly applicable to one-layer systems; however, adaptations of the theory are used in analyzing multilayer systems. The latest development in pavement design is the incorporation of finite element analysis. Primary distresses considered in mechanistic approaches include permanent deformation, caused by vertical compressive strain at the subgrade surface, and cracking, caused by horizontal tensile strain in the asphalt layer. Various methods, using different material characterizations and distress equations, have been proposed by the Asphalt Institute (5), Monismith (6), Shell (7), Chevron (8), and Chua and Lytton (9).

Asphalt Institute

The Asphalt Institute method for heavy wheel loads (5) incorporates a multilayer elastic theory to design full-depth asphalt pavements. The horizontal tensile strain is not considered; therefore the design is based on limiting the subgrade vertical strain. The asphalt thickness is a function of the

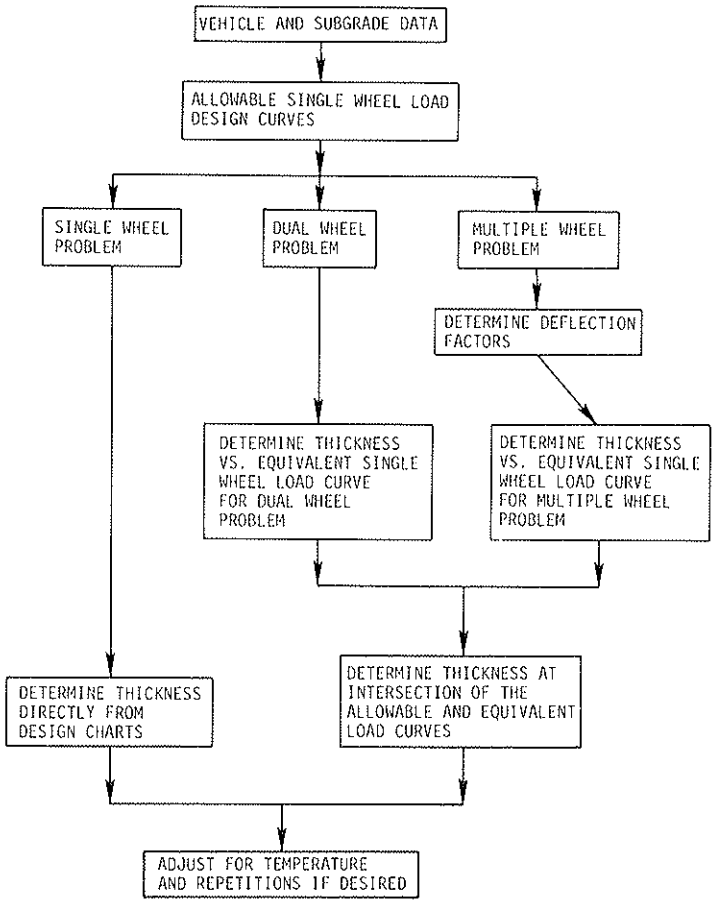

FIGURE 3 Flow diagram for the Asphalt Institute pavement design method for heavy wheel loads.

subgrade strength and the contact pressure of the load. Figure 3 shows this procedure.

Monismith

Monismith (6) incorporated the original Shell nomograph, by van der Poel, in his procedure as a means of calculating the bitumen stiffness given time of loading, temperature, and penetration index. A second nomograph allows the determination of the asphalt mix stiffness given bitumen stiffness and percentage voids in the mineral aggregate. Other inputs to the Monismith method include the average asphalt temperature, the average vehicular speed, the number of standard axles, and the subgrade elastic modulus. Figure 4 shows Monismith's methodology.

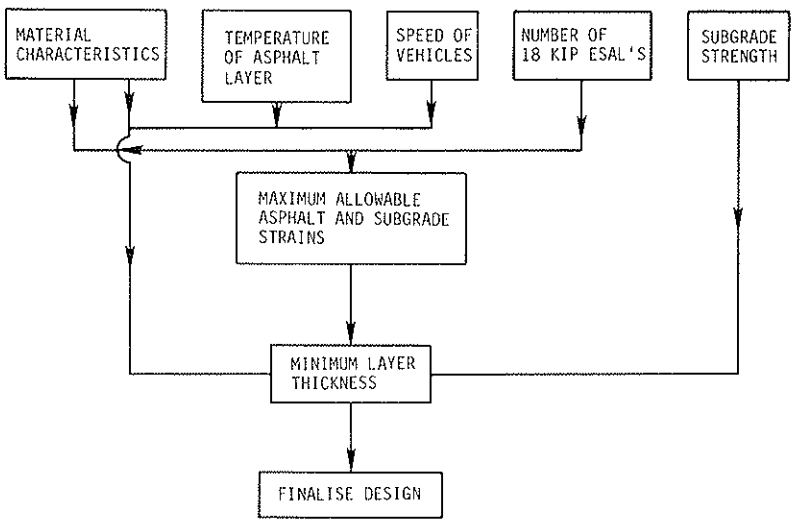

FIGURE 4 Flow diagram for the Monismith asphalt pavement design method.

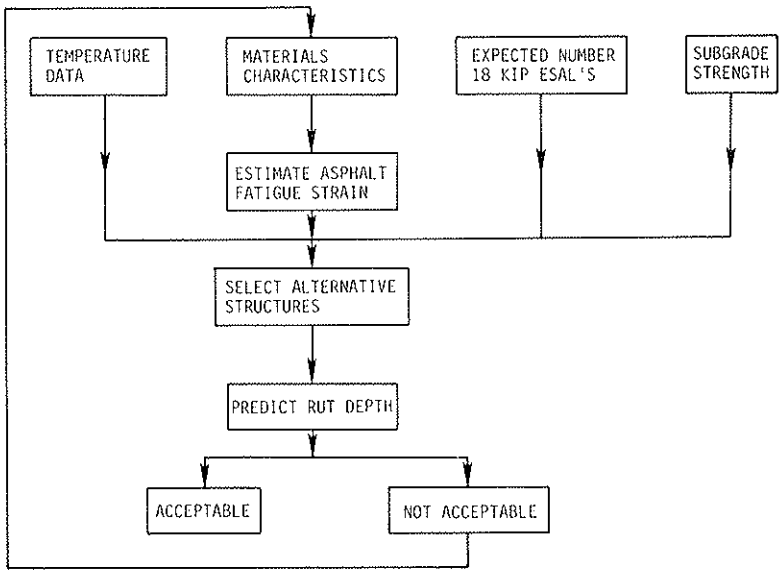

FIGURE 5 Flow diagram for the Shell asphalt pavement design method.

Shell

Extension to a three-layer linear elastic system is possible with the Shell method (7). An updated version of the Shell nomograph allows the determination of the asphalt mix stiffness given the percentage volume of mineral aggregate, the bitumen stiffness, and the percentage volume of bitumen. The BISAR computer program is used to obtain the limiting strain values and the corresponding number of 18-kip ESALs. Figure 5 shows the Shell analytical procedure.

Chevron

The Chevron method (8) uses a two-layer elastic structural model. The contributing factors include the number of 18-kip ESALs, the subgrade strength, the modulus of rupture of the asphalt, and the cure state of the asphalt. Figure 6 is a flow chart that illustrates this method.

Chua and Lytton

Chua and Lytton (9) calculate the number of passes of a specific load that causes a critical rut depth. The procedure can be used iteratively to obtain a pavement structure that will suffer a specific rut depth for given traffic conditions. The load-deflection relationship is described by a hyperbolic stress-strain curve for repetitive loading. This relationship combined with the ILLI-PAVE finite element program, which simulates deflection basins, results in rut depth histories for given pavements.

LOAD EQUIVALENCY FACTORS

The traffic factor included in each of the preceding pavement design methods is an integral component of the calculation of pavement life spans. With respect to the design of highway pavements, the traffic impact is normally incorporated through ESALs. (Figure 7). The damage effects of all vehicle types in the traffic stream are converted through the use of equivalent axle load factors to relative damage caused by a standard vehicle. The end result is the

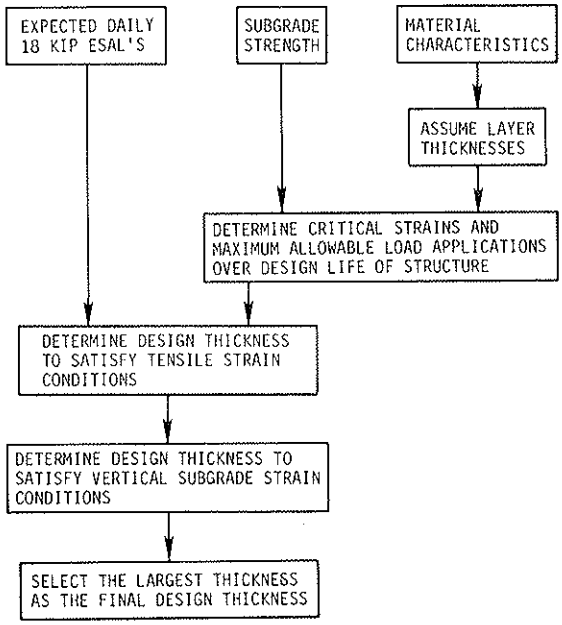

FIGURE 6 Flow diagram for the Chevron asphalt pavement design method.

computation of the number of axle load applications that a pavement is designed to withstand in its lifetime. The values used as the equivalency factors therefore constitute a critical step in the pavement design process. Tables 1 and 2 give the equivalent axle loads calculated using the AASHTO, Monismith, and Shell equivalency procedures for the same situation. The total number of 18-kip (80-kN) ESALs in Tables 1 and 2 are AASHTO, 1,443; Monismith, 1,675; and Shell, 1,501. Monismith's values differ from those of AASHTO by +16 percent, and the Shell values differ from those of AASHTO by +4 percent.

AASHTO

The most widely used equivalent axle load factors are those developed from the original AASHO Road

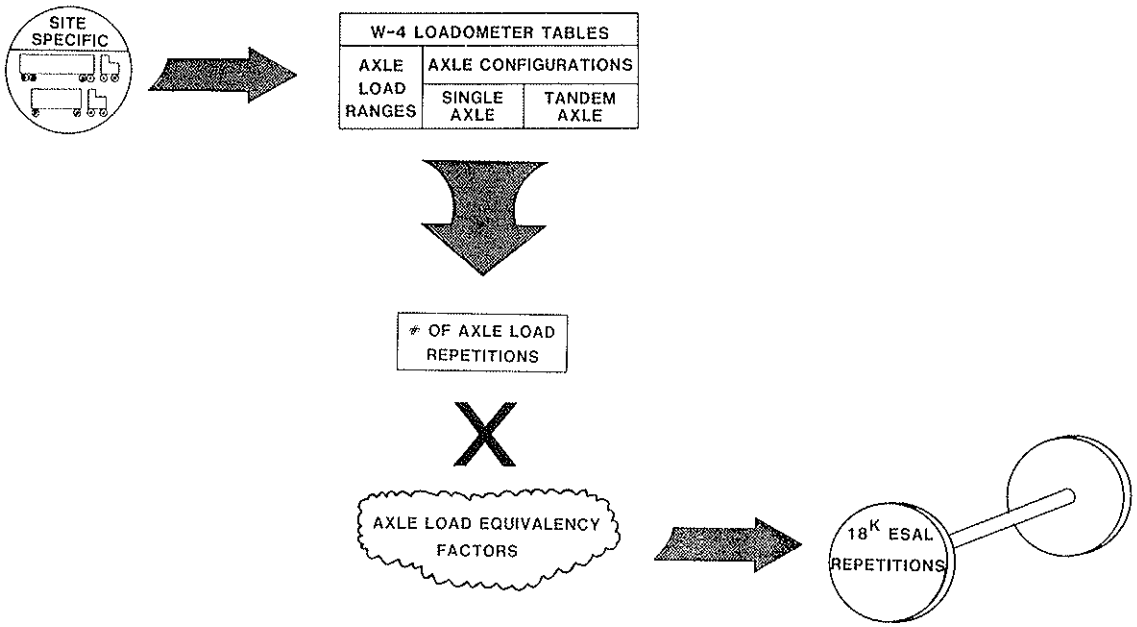

FIGURE 7 Reduction of traffic data to equivalent axle loadings.

TABLE 1 Contrast of Equivalent Axle Loads Calculated Using the AASHTO, Monismith, and Shell Equivalency Procedures for Single Axles for Hypothetical Pavement Problem in Which SN = 3.0, p = 2.5.

Axle Load		No. of	Equivalency Factors			Equivalent 18-kip (80-kN) Axle Loads		
(kips)	(kN)	Axles	AASHTO *(2)*	Monismith *(6)*	Shell *(7)*	AASHTO	Monismith	Shell
2	8.9	500	0.0003	0.0002	0.0001	0.15	0.1	0.05
6	26.7	500	0.02	0.012	0.011	10	6	5.5
10	44.5	1,000	0.12	0.096	0.086	120	96	86
14	62.3	300	0.40	0.37	0.33	120	111	99
18	80.0	200	1.00	1.00	0.90	200	200	180
22	97.8	100	2.17	2.23	2.01	217	223	201
26	115.6	10	4.31	4.36	3.93	43.1	43.6	39.3
Total						710.25	679.7	610.85

TABLE 2 Contrast of Equivalent Axle Loads Calculated Using the AASHTO, Monismith, and Shell Equivalency Procedures for Tandem Axles for Hypothetical Pavement Problem in Which SN = 3.0, p = 2.5.

Axle Load									Equivalent 18-kip (80-kN) Axle Loads		
Tandem		Single		No. of Axles		Equivalency Factors					
(kips)	(kN)	(kips)	(kN)	Tandem	Single	AASHTO *(2)*	Monismith *(6)*[a]	Shell *(7)*[a]	AASHTO	Monismith	Shell
4	17.8	2	8.9	20	40	0.01	0.0002	0.0001	0.2	0.008	0.004
12	53.4	6	26.7	300	600	0.02	0.012	0.011	6	7.2	6.6
20	89.0	10	44.5	500	1,000	0.16	0.096	0.086	80	96	86
28	124.5	14	62.3	800	1,600	0.55	0.37	0.33	440	592	528
36	160.1	18	80.0	150	300	1.38	1.00	0.90	207	300	270
Total									733.2	995.208	890.604

[a]One tandem axle is considered to be two single axles.

Test pavement design procedure (2). In response to a 1982 study, 43 state transportation agencies stated that they used the AASHTO guide in determining wheel-axle load equivalencies (11,pp.1-4). This procedure computes the number of axle load repetitions to failure for the pavement being designed. The number of repetitions is a function of pavement rigidity, load characteristics, and terminal serviceability value. The load characteristics consist of the magnitude of the axle load and the axle configuration (single or tandem). The actual equivalency factor (F_j) is given as the ratio of the number of repetitions to failure for a standard 18-kip single axle load ($N_{f_{18}}$) to the number of repetitions to failure for the given axle load and configuration (N_{f_j}). This ratio has been defined as a regression equation that includes the variables of axle load (L_1), axle configuration (L_2), and pavement characteristics (G, β, a, b):

$$F_j = N_{f_{18}}/N_{f_j} = [(L_1 + L_2)^a/(18 + 1)^a][10^{G/\beta}/(10^{G/\beta_j})L_2^b] \quad (4)$$

Values of the equivalent axle load factor have been tabulated as computed functions of the structural number (flexible pavements), the pavement thickness (rigid pavements), the terminal serviceability (P_t), the axle load, and the axle configuration (2).

Asphalt Institute

The Asphalt Institute pavement design method for heavy wheel loads (5) incorporates traffic data as equivalent single wheel loads rather than as equivalent single axle loads. This method is typical of airport pavement design procedures on which the design methodology is based. The standard highway pavement design procedure set forth by the Asphalt Institute, however, uses the AASHTO equivalency factors.

Monismith

Monismith's procedure defines the load equivalency factor (EF_W) in terms of axle loads:

$$EF_W = (W/80)^4$$
$$= 2.44 \times 10^{-8} W^4 \quad (5)$$

where

EF_W = axle load equivalency factor,
W = any particular axle load (kN), and
80 = standard axle load (kN).

The 80-kN standard axle load is roughly equivalent to the 18-kip standard axle load of the AASHTO design (6).

Shell

The Shell design procedure also stipulates the use of ESALs through an equation nearly identical to Monismith's:

$$n = 2.2 \times 10^{-8} L^4 \quad (6)$$

where n is axle load equivalency factor and L is other axle load (kN). The standard axle consists of two dual 20-kN wheels with contact stresses of 600 kN per square meter and a loaded area radius of 105 mm. This relationship is based on the AASHTO equivalency factors (7).

Chevron

Traffic is reduced to 18-kip ESALs for the Chevron pavement design procedure. A particular formula for calculating the 18-kip ESAL is not given, thus allowing the designer to use his own judgment in choosing an equivalency definition.

Chua and Lytton

The procedure of Chua and Lytton does not include load equivalency factors. Individual traffic loads are directly incorporated and the resulting rut depths are calculated (9).

Recent Developments in Equivalency Factors

A significant problem arises when an attempt is made to use the AASHTO or related equivalency factors for situations that do not fall within the scope of the AASHTO experimental data. An example of this conflict is the evaluation of new or unique truck axle configurations. Extrapolation of the AASHTO equivalency factors for these new trucks is not adequate, and therefore new approaches are necessary.

A fundamental relationship for the equivalency factor was devised by Treybig (3). This relationship results in factors similar to the AASHTO factors for identical situations, but it also provides for the calculation of factors for axle loads and configurations not represented by the original AASHTO equivalency factors. The equation for the equivalency factors [$F(X_n)$] (3,p.36) is

$$F(X_n) = [\varepsilon_1(X_n)/\varepsilon(18_S)]^B + \sum_{i=1}^{n} \{[\varepsilon_{i+1}(X_n)] - [\varepsilon_{i-i+1}(X_n)]/\varepsilon(18_S)\}^B \quad (7)$$

$$B = \log F(X_S)/\log [\varepsilon(X_S)/\varepsilon(18_S)] \quad (8)$$

where

$F_i(x_n)$ = equivalency factor for axle configuration n of load x,
$\varepsilon(18_S)$ = maximum asphalt strain or subgrade vertical strain for the 18-kip ESAL,
$\varepsilon_1(X_n)$ = maximum asphalt strain or subgrade vertical strain under the leading axle or axle configuration of load x,
$\varepsilon_{i+1}(X_n)$ = maximum asphalt strain or subgrade vertical strain under axle i + 1 of axle configuration n of load x,
$\varepsilon_{i-i+1}(X_n)$ = maximum asphalt strain or subgrade vertical strain in the critical direction between axles i and i + 1 of axle configuration n of load x.
$\varepsilon(X_S)$ = maximum asphalt strain or subgrade vertical strain for an x-kip single axle load, and
$F(X_S)$ = AASHTO equivalency factor for an X-kip single axle load.

This equation should only be applied to pavements that are similar to those of the AASHO Road Test with respect to material properties and thicknesses. Also, this relationship is only applicable to flexible pavements; a similar relationship derived for rigid pavements did not correlate well with the AASHTO values.

The trend toward theoretically based equivalency factors was continued by Sharma et al. (4). Their method converts mixed traffic with single or tandem axles and dual tires or single tires of various widths to equivalent 18-kip dual-tire single axle load applications (Figure 8). Two separate sets of equivalency values were computed, one for flexible pavements and another for rigid pavements.

For flexible pavements the calculation of equivalent wheel load factors began with elastic layer theory to calculate maximum horizontal strains. Next, the number of axle load repetitions until failure was determined using fatigue analysis. The equivalent wheel load factors were then computed for single tires (widths = 10, 12, 14, 16, and 18 in.) on single axles to allow conversion to 18-kip dual-tire (width = 10 in.) single axle loads. Both the flexible and the rigid pavement equivalent wheel load factors were verified by field studies (4).

Rigid pavement procedure entailed the use of a finite element analysis, ILLI-SLAB, to calculate maximum flexural stresses. Warping stresses are then added to the flexural stresses; the combination is then used to calculate the number of axle repetitions to failure using fatigue analysis. Finally,

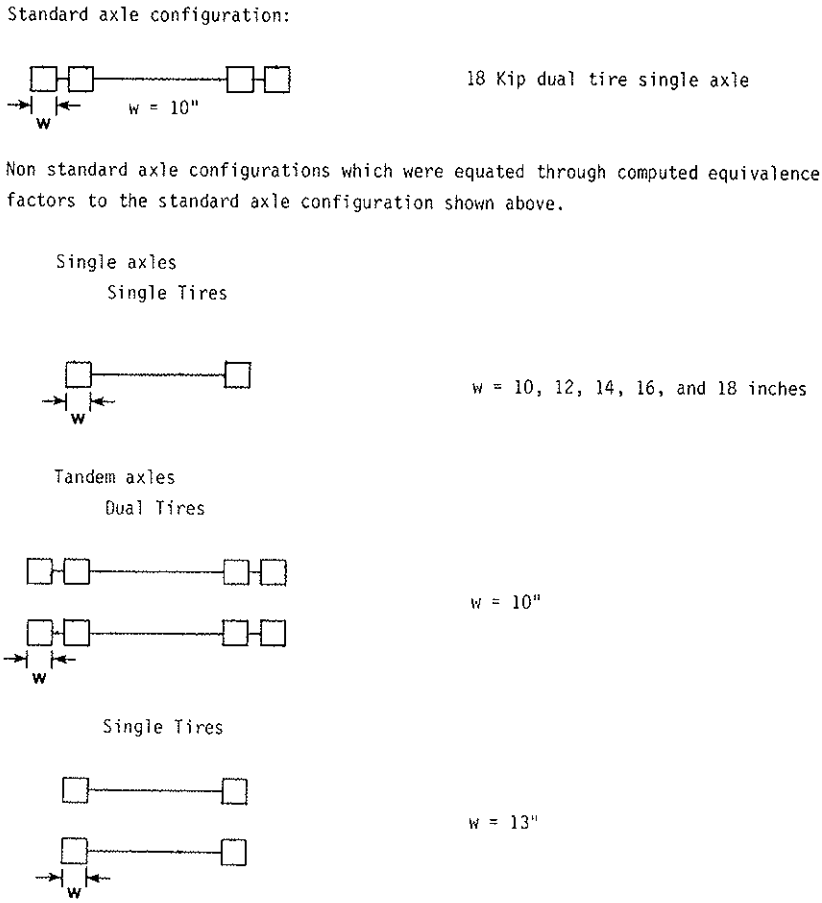

FIGURE 8 Axle configurations examined by Sharma, Hallin, and Mahoney *(4)*.

equivalent wheel load factors were developed for single tires (widths = 10, 12, 14, 16, and 18 in.) on single axles, dual tires (width = 10 in.) on tandem axles, and single tires (width = 13 in.) on tandem axles as conversion factors to 18-kip dual-tire (width = 10 in.) single axle loads (4).

PAVEMENT MANAGEMENT SYSTEMS

If the various design theories are correct in assuming shorter life expectancies and increased distress levels for pavements subjected to heavier, wider, and longer trucks, then the ability to monitor these pavements becomes essential. Pavement management systems (PMSs) are techniques or methodologies used to assess the condition of a current pavement network, predict the location of future distresses, and rank the scheduling of necessary maintenance in order of priority. Fiscal restraints and responsibilities support the implementation of a PMS to ensure the efficient use of money and materials.

Pavement management systems are necessarily tailored to each agency's needs and desires. The level of comprehensiveness varies greatly. Current systems range from those that are primarily visual and subjective to empirical models that estimate various pavement distresses and related serviceability. In general, the effects of truck traffic are included through fixed percentage increases in the number of 18-kip ESAL repetitions. Seasonal variations and subgrade condition and composition are also incorporated in most current PMS procedures. Several examples illustrate the implementation of a PMS.

Arvada, Colorado

The city of Arvada, Colorado, implemented a method of monitoring and evaluating the present condition of the pavement network in order to identify and recommend immediate and future corrective measures (12). A visual inspection of the network is made to note and rate various types of pavement distress. Ride quality is determined and the condition of structural appurtenances is also recorded. Individual deduct values are determined for each distress noted, and a pavement condition rating score (PRS) is calculated. A computerized decision tree is then used to obtain the optimum rehabilitation techniques and associated costs. Finally, a priority value is calculated as a function of cost, length of pavement, average daily traffic, PRS, and presence of industrial or commercial vehicles (trucks). No distinction is made with respect to type of trucks involved, axle loadings, or axle configurations.

Alberta, Canada

The PMS used by the province of Alberta, Canada, is an empirically based procedure that incorporates pavement performance prediction models to identify both current and future needs (13). Field measurements are first obtained. Then these measurements are used as input for several regression equations to determine three indices: a riding quality index (RQI) represents the roughness of the pavement; the structural ability of the pavement to withstand traffic is based on a structural adequacy index

(SAI); and severity and extent of surface distress are recorded as a visual condition index (VCI). The overall quality of the pavement is represented by the pavement quality index (PQI), which is a function of RQI, SAI, and VCI. Rehabilitation needs are then established for each index. The inclusion of truck traffic is accomplished in the calculation of SAI and is based on the number of 18-kip ESAL repetitions. Approximate axle load equivalency factors are therefore a necessary requirement.

Texas Flexible Pavement Damage Functions

Texas flexible pavement damage functions also rely on an estimate of 18-kip ESAL repetitions (10). The Texas method requires the input of the average daily traffic count, the percentage of trucks, the flexible base thickness, the subgrade Atterburg limits, the maximum Dynaflect deflections, and climatic data. The number of 18-kip ESAL repetitions is calculated and used as input to several pavement distress equations. Pavement distress equations have been developed to examine rutting, flushing, alligator cracking, raveling, and longitudinal cracking. A pavement score ranging from 0 to 100 is then obtained with a value of 35 defined as "failure." The distress types deemed most significant at the time of failure are identified. Appropriate rehabilitation strategies can then be recommended to remedy the condition.

TRUCK IMPACT STUDIES

Oil Field Traffic

The usefulness of the Texas pavement distress methodology was demonstrated in a study conducted for the Texas State Department of Highways and Public Transportation (14). This study illustrated the effects of oil field truck traffic on low-volume, surface-treated flexible pavements. A computer program was created that estimates the service life of thin surface-treated pavements serving both oil field traffic and original "intended-use" traffic. In addition to the Texas flexible pavement distress equations, the program also determines a pavement serviceability index based on the standard AASHTO 18-kip ESAL equivalency factors.

Double Bottoms

Impacts of the 1982 STAA permitting larger trucks are difficult to ascertain. This point is evident in a study by Tobin and Neveau (15) who investigated the effects of tandem trailers (double bottoms). The assumptions on which the study was based are critical in that the presence of double bottoms could either increase or decrease the number of axle loadings and correspondingly the amount of pavement deterioration. If the freight tonnage were to remain constant and be carried via double trailers rather than single trailers, then the number of axle loadings would be smaller because there would be fewer tractors. Shipping via doubles, however, is less costly per freight unit than shipping via singles. Therefore the allowance of doubles could result in greater freight tonnage and, hence, more trucks and more axle loadings.

The AASHTO 18-kip ESAL equivalency factors were used to model the truck axles and obtain pavement deterioration rates. Study results indicate that, in the short term (10-year span), the impact of tandem trailers appears to be negligible with respect to maintenance costs. For the long term (20-year span) no clear relationship could be identified between maintenance costs and pavement deterioration rates. The ambiguity lies in the various accompanying factors including percentage of trucks, type of maintenance, and maintenance scheduling.

Productivity Savings

Economic implications of the 1982 STAA for governing entities with respect to pavement management must also be viewed from the perspective of increased productivity. Although the STAA permitted larger trucks, it also provided for increased taxes to be levied on the trucking industry. Nevertheless, the U.S. Department of Transportation estimated a net productivity savings for the trucking industry of $3.24 billion. The American trucking industry, however, calculated a net productivity savings of $829 million to be realized from the time of passage of the bill until 1985 (16).

SUMMARY

The effects of heavier, wider, and longer trucks permitted by the 1982 STAA are not well established at this time. Various pavement management systems are, however, being used to monitor roadway systems and will provide insight into the contribution of traffic to pavement failure. Each system discussed in this paper relies primarily on conversion of the traffic data to 18-kip ESALs through AASHTO load equivalency factors.

Axle load equivalency is the fundamental concept through which mixed traffic is transformed for use in pavement design and pavement management. This traditional methodology is also being used to measure the effects of new truck sizes and axle configurations. Most widely used is the AASHTO conversion to 18-kip ESALs (11).

Extrapolation of the AASHTO 18-kip equivalency factors for the new axle configurations of larger trucks is not possible because of limitations of the empirical data on which the existing factors are based. Various attempts have been made by Treybig (3) and Sharma et al. (4) to establish sets of theoretically based equivalency factors that would be capable of modeling the heavier loads and various axle configurations permitted by the 1982 STAA.

Research is limited in the area of load equivalency factors. If the axle load equivalency concept continues to be applied in analysis, then additional efforts will be necessary to determine the proper values for implementation. Changes, such as those brought about by the 1982 STAA, require continued investigations to more closely identify, assess, and predict the impacts of longer, wider, and heavier trucks.

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TRB's Study of Twin-Trailer Trucks

ROBERT E. SKINNER, JOSEPH MORRIS, and STEPHEN GODWIN

ABSTRACT

The Surface Transportation Assistance Act (STAA) of 1982 legalized the nationwide use of twin-trailer trucks on Interstate highways and other designated primary routes. In this paper will be reviewed what is known to date about the effect this legislation has had on the trucking industry--who is using these vehicles, where, and for what purposes. This information, coupled with earlier research findings concerning twins and other heavy trucks, will be used as the basis for a brief discussion of the likely effects of twins on the design, maintenance, and operations of highway facilities. Specific topics will include road geometry, pavements, bridges, and traffic capacity. Throughout, references will also be made to other new trucks legalized by the 1982 STAA--the 48-ft single-trailer truck and 102-in.-wide trucks.

Twin-trailer trucks--truck tractors pulling two trailing units with individual lengths of 27 to 28 ft--have been operating in the United States for more than 35 years, but their operation has been confined principally to the far West. In the Surface Transportation Assistance Act (STAA) of 1982, the Congress required states to permit the operation of twins, as well as longer semitrailer trucks (with trailer lengths of at least 48 ft) and wider semitrailers (up to 102 in.), on Interstates and primary routes designated by the Secretary of Transporta-

Transportation Research Board, 2101 Constitution Avenue, N.W., Washington, D.C. 20418.

tion. These changes in truck size limits were intended to increase productivity in the motor carrier industry to at least partly offset increased taxes and fees enacted at the same time.

The 1982 STAA also directed that the National Research Council's Transportation Research Board monitor the effects of twin-trailer truck use on highways and highway safety. A special study committee, appointed by the National Research Council, developed a study design and began its work in June 1984. The study will be completed in June 1986.

In developing the plan for its work the study committee decided to include a thorough review of prior studies and research dealing with twins and, for the most part, rely on the continuing data collection efforts of other organizations to monitor the short-term effects of twins. This approach was selected because the committee chose to examine a broad range of possible effects including industry use and economics, safety, vehicle performance, highway design and maintenance, traffic operations, and highway administration. The study committee has completed its review and critique of prior studies and is in the midst of assembling data and information on the first 2 1/2 years of nationwide use of twins.

The purpose of this paper is to share preliminary study findings about the motor carrier industry's response to the availability of twins and other STAA vehicles on a nationwide basis. These findings address the following key questions:

- What types of firms are purchasing these vehicles?
- For what purposes?
- What are the specific advantages that these trailers offer?
- How do new equipment decisions vary by region?

Questions such as these are particularly important because their answers can be used to speculate about the long-term role and use of twins and other STAA vehicles in the United States. The use level of these vehicles affects the scope and magnitude of virtually all impacts of interest, from highway accidents to pavement wear.

Before specific industry responses are discussed, the structure of the U.S. motor carrier industry is reviewed and the characteristics of twins and their relative advantages and disadvantages are outlined. With this background established, pre-1983 use of twins and what is known so far about post-1983 use are examined.

STRUCTURE OF THE MOTOR CARRIER INDUSTRY

The motor carrier industry is highly varied; five carrier types predominate: firms that provide shipping to the public for a fee (the common carriers), firms that ship goods to specific companies under contract (the contract carriers), independent owner-operators, companies that are not in the primary business of trucking but that instead have fleets to move their wholesale or retail goods to the points of sale (private carriers), and carriers that operate solely within state lines (intrastate and local carriers).

These definitions have been used since the Interstate Commerce Commission (ICC) began regulating the industry in the mid-1930s; however, the definitions overlap considerably. Although the deregulation of the motor carrier industry that began in 1980 has further blurred the distinctions between industry segments, the available information about the industry is still classified according to the traditional definitions (Figure 1).

The ICC regulated trucking according to how firms sold their service to the public, the types of commodities that they shipped, and the routes on which they moved. Private carriers, because they were not primarily in the business of trucking, were not regulated. Although included in the definition of the for-hire industry, movers of certain goods, particularly raw agricultural commodities, were also largely exempt from ICC regulation. Companies that only operated within a single state were completely exempt from ICC regulation.

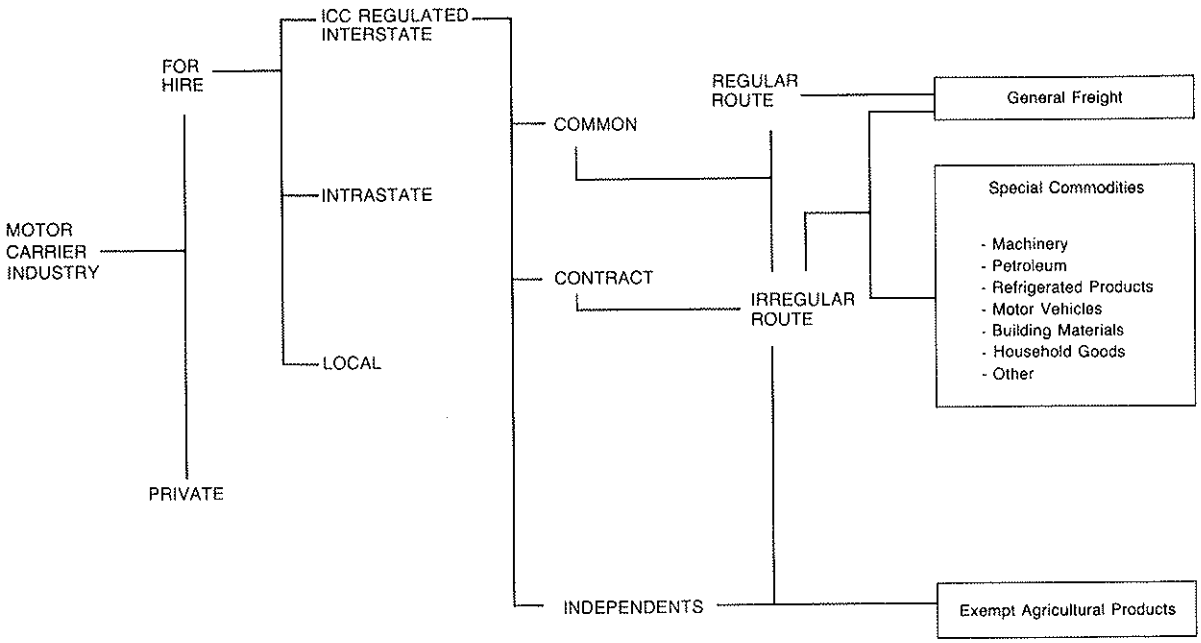

FIGURE 1 Structure of the motor carrier industry.

Unregulated Carriers

The unregulated carriers accounted for the majority (60 percent) of intercity tonnage and total revenues in 1983 (Table 1). Many national wholesale and retail stores, leasing companies, large grocery chains, utilities, governments, and oil companies own and operate private fleets. These fleets vary widely in size, from fewer than five tractor-semitrailer combinations in many fleets to Ryder Truck Rental's more than 20,000 vehicles (5). Although this category also includes independent owner-operators, the private fleets account for the lion's share of travel by unregulated carriers. Indeed, according to the estimates in Table 1, private carriers account for 40 percent of all combination-truck travel.

Regulated Carriers

In the regulated segment of the industry, there are firms that offer shipping to the public according to established rates (common carriers) and others that move goods for individual companies only under contract (contract carriers). Contract carriers mostly transport goods classified by the ICC as special commodities. These goods tend to require a specific type of tractor-trailer combination and include, for example, automobiles, petroleum, and refrigerated products. In addition, these goods move from the factory, or point of origin, directly to the destination in what are referred to as "truckload" lots. This simply means that the carrier picks up a single shipment and moves it directly to its destination. Contract carriers may account for as much as one-third of the combination vehicle miles of travel (VMT) of regulated carriers and about 15 percent of all combination VMT (Table 1).

Common carriers transport special commodities and truckload freight also, but the majority of the largest common carriers handles less-than-truckload (LTL) shipments of general freight. The ICC classifies LTL freight as those individual shipments weighing less than 10,000 lb not as the extent to which a trailer is filled.

At terminals from which goods are headed in the same direction, individual LTL shipments are loaded into trailers and then transported to other terminals for distribution to the final destination. Common carriers transport most of the regulated freight; they travel about twice as much and own twice as many vehicles as do contract carriers (Table 1).

The common carrier segment of the industry is characterized by a few giants surrounded by hundreds of medium-sized companies and tens of thousands of small firms. The top 10 revenue earners for 1983 were responsible for one-quarter of the total revenue of the entire regulated motor carrier industry. The share of earnings increased to 40 percent for the top 50 revenue earners and to just under 50 percent for the top 100 revenue earners (6,p.65).

Impact of Deregulation on Industry Structure

The Motor Carrier Act of 1980 lifted many of the regulatory constraints on the industry. Among the more important changes, the ICC has (a) made entry into the industry relatively simple, (b) allowed private fleets to operate more like for-hire carriers, (c) expanded the classification of exempt commodities, and (d) made it easier for carriers to add service to new points or drop existing service points in their networks.

Although it is too early to assess the full consequences of deregulation, several observations can be made about the experience during the first 5 years. First, despite a severe industrywide recession in the early 1980s, tens of thousands of new firms have sought and received ICC operating rights, mostly for common carrier service. Second, distinctions among major industry segments have been further blurred as firms have begun to offer services outside their traditional areas. Third, as existing firms have sought new markets and new firms have entered the trucking industry, competition has intensified and rates have stabilized or dropped. Finally, larger trucking firms have emerged from the recession of the early 1980s more quickly than small and medium-sized firms, and a number of the larger common carriers are pursuing expansion plans and making major equipment purchases.

CHARACTERISTICS OF TWINS

With the motor carrier industry in a period of unprecedented competitiveness and cost consciousness, the nationwide legalization of twins and other large trucks by the STAA of 1982 has provided new oppor-

TABLE 1 Motor Carrier Industry Freight, Travel, and Revenues: Regulated and Unregulated Segments, 1983

Industry Segment	Intercity Tonnage[a]		Intercity Ton-Miles[b]		Combination VMT[c]		Combination Vehicles[d]		Revenues[e]	
	Millions of Tons	Percentage	Billions of Ton-Miles	Percentage	Billions of Miles	Percentage	Thousands of Vehicles	Percentage	Billions of Dollars	Percentage
Regulated carriers										
Common	NA	NA	NA	NA	19.1	33.2	278.4	26.2	27.7	25.3
Contract	NA	NA	NA	NA	8.8	15.3	124.7	11.7	18.8	17.2
Subtotal	756.0	39.4	227.6	41.3	27.9	48.5	403.1	37.9	46.5	42.5
Unregulated carriers										
Private	NA	NA	NA	NA	24.6	42.8	595.8	56.0	NA	NA
Exempt	NA	NA	NA	NA	4.9	8.5	65.6	6.2	NA	NA
Subtotal	1,138.0	60.1	323.4	58.7	29.6	51.5	661.3	62.1	62.9	57.5
Total	1,894.0		551.0		57.5		1,064.4		109.4	

Note: NA = not available; VMT = vehicle miles of travel.

[a] Tonnage by mode from Transportation in America (1,p.7).
[b] Ton-miles by mode from Transportation in America (1,p.6).
[c] Total travel estimated by FHWA (2, Table VM-1). Share of travel based on distribution by carrier type reported by Census Bureau (3, Table 7).
[d] Total combination vehicles estimated by FHWA (2). Share based on distribution by carrier type reported by Census Bureau (3).
[e] Revenue from Transportation in America (1,p.4). Share of revenues between common and contract carriers from 1982 Financial Analyses of the Motor Carrier Industry (4, Figure 1).

tunities for increased productivity, but these opportunities are not the same for all segments of the industry. Because of the cargo they carry and the nature of their operations, some carriers will be more attracted to twins than others. In this section a summary is given of the characteristics of twins and their potential advantages and disadvantages compared with the semitrailer trucks that they typically replace.

Typical Pre-1983 Twin-Trailer Trucks

The twin-trailer truck most widely used in the United States before 1983 consists of a two-axle tractor drawing two single-axle semitrailers, each 27 ft long (Figure 2). The semitrailers are coupled by a single-axle converter dolley: a short frame mounted on an axle with a hook-and-eye connection to the front trailer and a fifth-wheel connection to the rear trailer.

The overall length of this truck is 65 ft, the maximum legal length in 25 of the 36 states where the vehicle was permitted before 1983 (the other 11 had longer maximums). Apparently the 27-ft length of each trailer and the typical use of short-wheelbased tractors of the cab-over-engine style was dictated by this common 65-ft limit. The width of the vehicle (excluding certain projections such as rear-view mirrors) is 96 in., the legal maximum on all roads in 42 states (7) and the federal maximum on Interstates (8) before 1983.

Other Pre-1983 Double-Trailer Trucks

A variety of other combinations with two trailers was in use before 1983. These include turnpike doubles--nine-axle vehicles with twin 40- or 45-ft trailers, a length of approximately 100 ft, and a maximum weight of up to 105,500 lb (legal, at least on some roads, in 14 states in 1983)--and the Rocky-Mountain double--a tractor pulling a standard length semitrailer plus a shorter second trailer, with seven or eight axles and an overall length of about 85 ft (legal in 11 states in 1983). The 1982 act had no effect on the legality of these longer or heavier doubles because it provided only for double trailers each 28 ft or less in length that are subject to the same 80,000-lb weight limit applied to single-trailer trucks.

FHWA vehicle classification count data (9) show a small number of six-axle double-trailer combinations that are identical to the twin trailer except that they have three axles on the tractor. This configuration is likely to be used increasingly while new twins are being introduced because most tractors in fleets that did not employ twins before 1983 have three axles.

Twins Legalized by the 1982 Act

The Congress in 1982 permitted twins that had trailers up to 28 ft long, unlimited overall length, and a width of 102 in. on Interstates, the federally designated network, and state-selected access roads. Figure 3 shows typical dimensions of these vehicles and of the 48-ft semitrailer combination also legalized. The overall length of the twin is at least 67 ft because the spacing between units and the tractor length of the 65-ft pre-STAA twin were already at a minimum. Twins with conventional (rather than cab-over-engine) tractors, in which the engine is under a hood forward of the cab, may be 2 to 5 ft longer.

Advantages of Twins

To trucking firms, twins can offer two primary advantages relative to semitrailer trucks--greater cubic capacity and greater operational flexibility.

Greater Cubic Capacity

The 28-ft by 102-in. twin has 31 percent more volume capacity than the standard pre-1983 single trailer and 16 percent more than the 48-ft by 102-in. single trailer that Congress also permitted in 1982 (Table 2).

The same federal gross weight limit of 80,000 lb applies to both twins and semitrailers on the Interstates. However, twins are easier to load to the maximum overall limit without exceeding federal individual axle load limits than are five-axle semitrailers, which require careful balancing of the axle loads to maximize the weight legally carried. On semitrailers 48 ft by 102 in. "cubing out" occurs; that is, the entire volume capacity of the trailer is used before the gross weight limit is

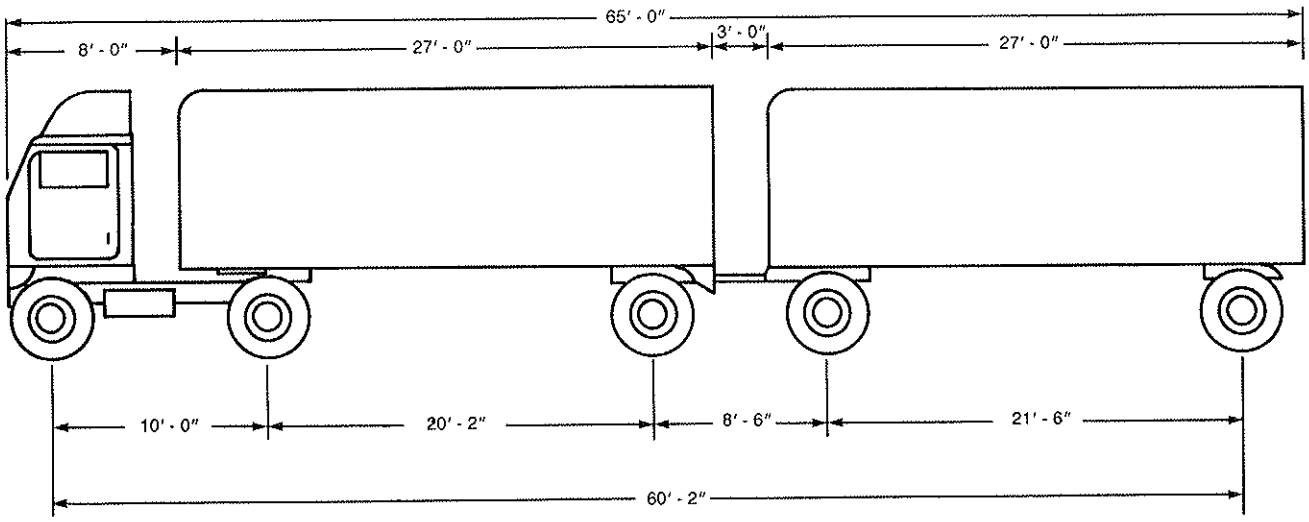

FIGURE 2 Typical dimensions of pre-1983 five-axle double-trailer combination.

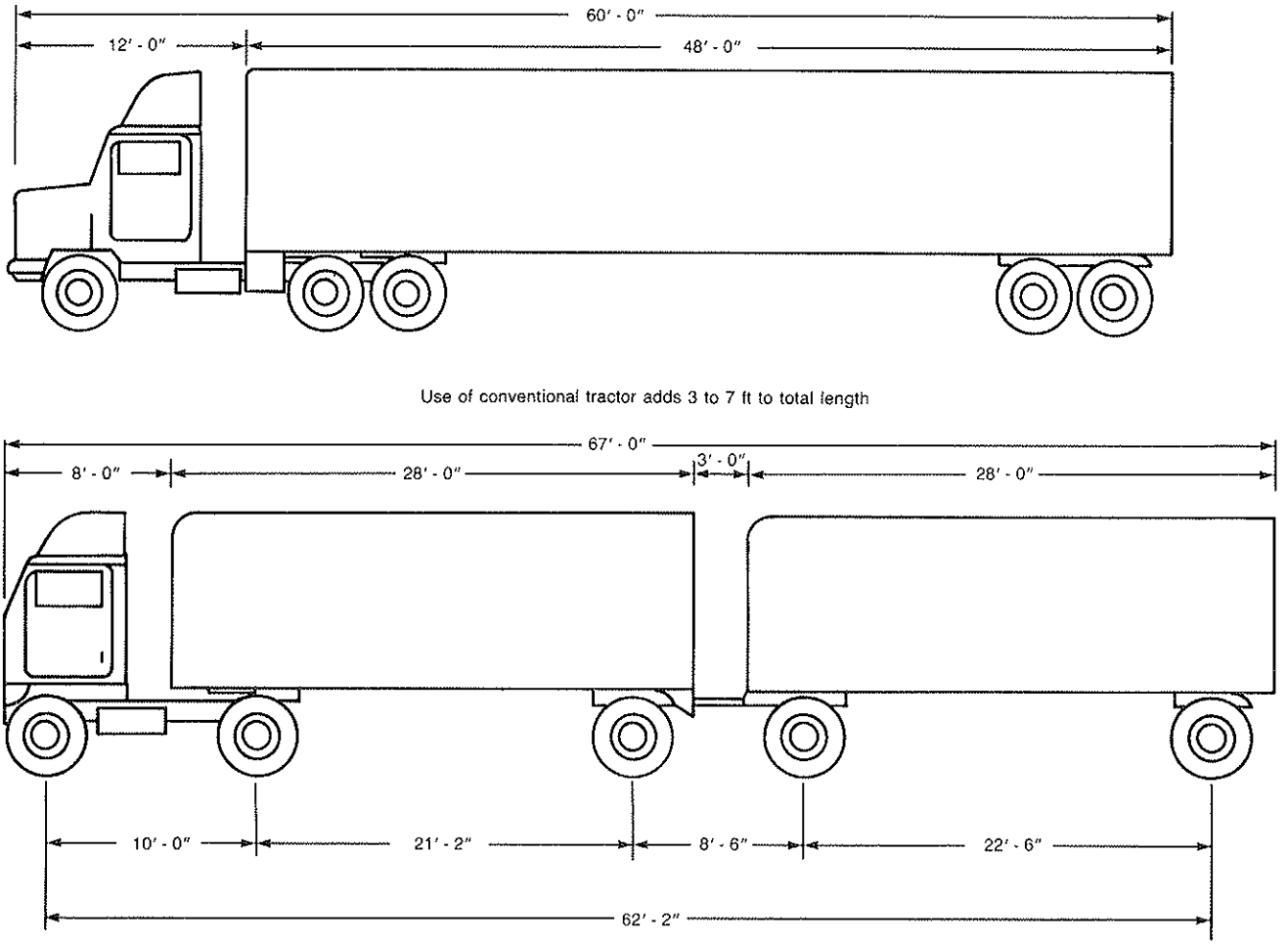

FIGURE 3 Typical dimensions of doubles and singles permitted under STAA of 1982.

TABLE 2 Typical Inside Dimensions and Volumes of Dry Van Trailers

Type and Exterior Dimensions	Inside Dimensions			
	Width (in.)	Height (in.)	Length (in.)	Volume (ft^3)
Semitrailer				
45 ft x 96 in.	93	108	533	3,098
48 ft x 102 in.	99	108	569	3,520
Twin				
27 ft x 96 in. (each)	93	108	317	3,685 (pair)
28 ft x 102 in. (each)	99	108	329	4,070 (pair)

reached at freight densities below 14.2 lb/ft^3. However, twins can carry the maximum weight with cargo as dense as 12.3 lb/ft^3.

Thus carriers with cargoes of relatively low density can carry larger loads with twins than with the semitrailers they replace, even if that semitrailer is the new 48-ft by 102-in. type. For example, one of the biggest common carriers in the country has an average freight density of 11.5 lb/ft^3. Carriers such as this are taking advantage of this added cubic capacity to reduce their line-haul costs, which are about half of all costs to LTL common carriers (10).

Greater Operational Flexibility

For carriers that transport mixed cargoes over large networks with many pickup and delivery points, twins can reduce the number of times that freight must be handled--unloaded and reloaded--on its journey.

For example, consider the intercity movement of LTL freight. As shown in Figure 4, LTL shipments transported in semitrailers must be handled several times between the points of origin and destination. Twin trailers offer an opportunity to bypass some of the normal handling steps in a hub-and-spoke LTL operation. A standard semitrailer might arrive at an intermediate terminal in Charlotte, North Carolina, for instance, and have part of its load removed for transport to Atlanta, Georgia, with the balance bound for Columbia, South Carolina. The space in the trailer bound for Columbia might be filled with other shipments to Columbia and the shipments bound for Atlanta would be consolidated with others in a different trailer. In contrast, a twin trailer arriving at that same terminal could bypass the break-bulk operation. The shipments in the trailer bound for Atlanta would not have to be unloaded; instead, the trailers would simply be separated. In addition to labor cost savings, time savings would be realized because the shipments bound for Atlanta could be dispatched immediately. Further cost savings could be realized if a 28-ft semitrailer was used to pick up the freight from one metropolitan area, was then attached to a tractor and another twin for the line-haul portion of the trip, and was ultimately used as the vehicle for delivery. In this case the freight would be handled only at pickup and delivery. Reduced handling means reduced terminal and break-bulk costs, which account for roughly 20 percent of LTL common carrier costs.

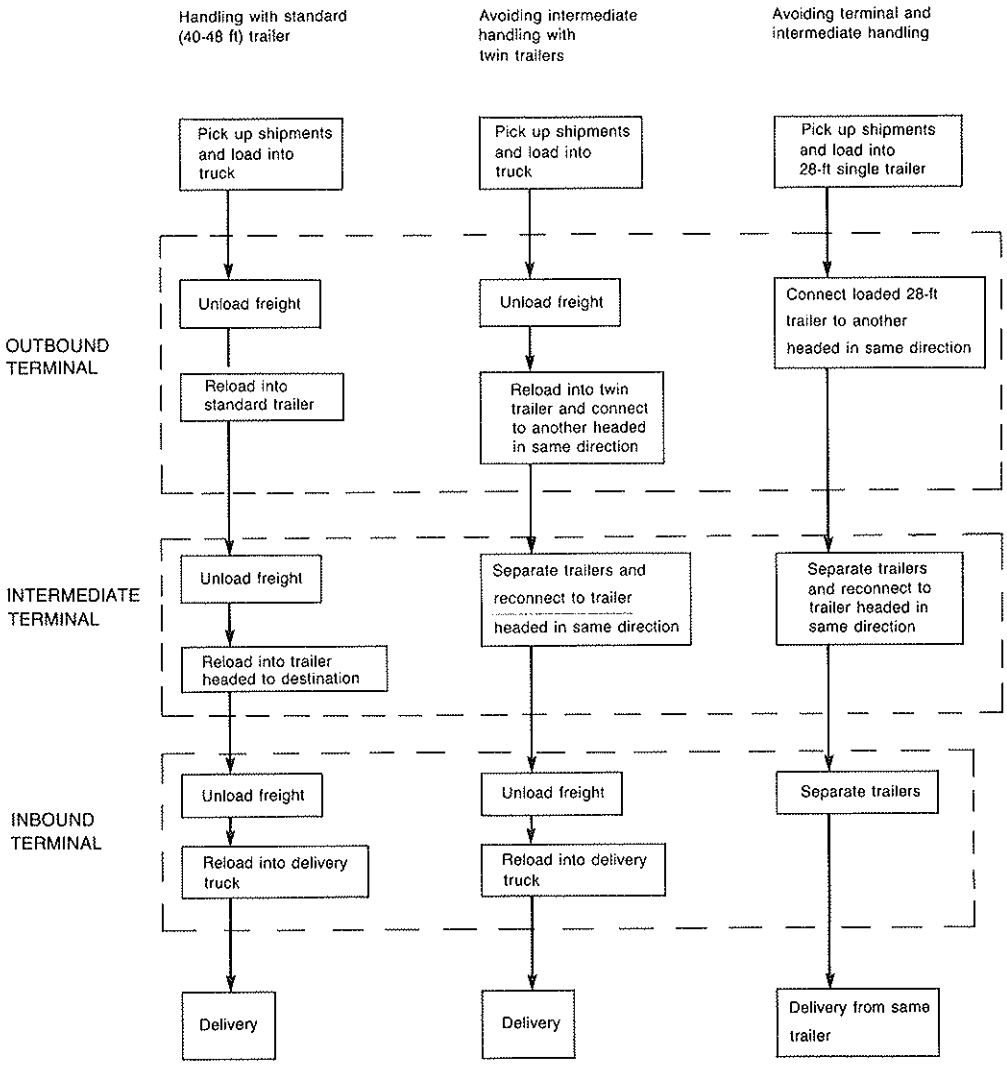

FIGURE 4 Freight handling eliminated by use of twin trailers.

Disadvantages of Twins

Twin trailers also have some disadvantages. A pair of 28-ft trailers plus the dolly sell for about 6 to 7 percent more than a 48-ft semitrailer. The additional size and the dolly also increase the tare weight by about 3,000 lb, thus reducing the potential shipment weight. For terminal-to-terminal freight operators, twins can increase the amount of vehicle handling required at the terminal. The trailers have to be separated, and if both are to be unloaded, additional labor is required to back the trailers up to the loading dock or move them around the terminal. In addition, the new twins can introduce some problems in keeping the fleet in balance when inbound freight tends to be LTL and outbound freight tends to be TL.

PRE-1983 USE OF TWINS

The use of twin trailers has been a western, and primarily a California, phenomenon. Although twins were legal in 37 states in the early 1980s, in only 9 western states (Arizona, California, Idaho, Montana, Nevada, Oregon, Utah, Washington, and Wyoming) and one state outside the West (Nebraska) did they account for 4 percent or more of total combination traffic. In four states where twins were legal at the time (Delaware, Louisiana, Maryland, and Mississippi), none was observed in the FHWA vehicle classification counts before 1983.

Although historical data on twin use provide some guide to future users, national statistics can be misleading. Statistics on twin travel are dominated by California, where use of twins permitted carriers of all types to take advantage of the maximum vehicle weights. Because of the axle weight limits and spacing of axles, a pair of twin trailers could effectively carry a few thousand pounds more cargo than a single trailer. In addition, California produce farmers found twins particularly suited to some of the special characteristics of their harvesting operations. Judging from accident statistics (11), as much as one-half of all twin traffic occurred in California before 1983. Twins registered in California accounted for an even larger share of twin-trailer traffic (3). Because under the STAA of 1982 twins now have the same gross weight limits as singles, nationwide pre-1983 experience is not necessarily a good basis from which to predict future twin traffic.

To examine regional variations in the pre-1983 use of twins and isolate the California experience, the study has tabulated by regional (California, other western and mountain states, and eastern and

central states) data from the Annual Truck Weight Study (9) conducted by the states and reported to FHWA, supplemented by Bureau of Motor Carrier Safety (BMCS) accident data (12) and the Truck Inventory and Use Survey of the Bureau of the Census (3). In all tabulations, twins were compared with three-axle-tractor two-axle-semitrailer combinations, the alternative vehicle for nearly all twin applications. Key findings are as follows:

- Industry class. ICC-regulated for-hire carriers (contract and common carriers) operated 93 percent of twins in the eastern and central states before 1983 compared with 63 percent of five-axle single-trailer combinations. The ICC-regulated firms accounted for 74 percent of twin use in the other western and mountain states but only 25 percent in California. Virtually all ICC-regulated twin mileage is produced by common carriers. Nearly one-half of the California twins and one-fifth of those in other western and mountain states were privately operated. In 1983 BMCS data, large private Interstate twin users included retailers and producers of food and forest products.
- Cargoes. Operators outside California predominantly used twin trailers for general freight and small-package cargoes. Cargoes on twins were much more highly concentrated in these commodity categories than were those on semitrailer combinations. In California, in contrast, the commodities carried by twins were as varied as those in semitrailers; the largest category was agricultural and food products.
- Trailer body types. Examining trailer body types gives another indication of the users of twins. As the previous tabulations would suggest, twins in the eastern and central states and other western and mountain states were mainly enclosed dry vans, whereas twin flatbeds and bulk commodity carriers (hoppers and tank trailers) were common in California. There was no appreciable traffic of twin refrigerated vans or furniture-moving vans in any region.

In summary, in the eastern and central regions before 1983, most twins carried general freight and were operated by common carriers. In California, twins were used for purposes as diverse as those of semitrailer trucks. In other western and mountain states, twin use patterns reflected the spillover of California twin traffic but were closer to the patterns of the eastern and central regions.

POST-1983 INDUSTRY USE OF TWINS

Because of their light-density cargoes and complex networks with multiple terminals and break-bulk facilities, common carriers of LTL general freight appear to be the industry segment most able to take advantage of the capacity and operational characteristics of twins. Because of this, pre-1983 studies of truck size and weight changes generally identified this segment as the primary user of twins if they were to be legalized on a nationwide basis (as they were by the STAA of 1982 (13,p.III-1). Moreover, the pre-1983 experience, except in California, is consistent with this expectation.

Nevertheless, there are a number of uncertainties and unresolved questions. To what extent will twins be adopted for use by LTL common carriers? How does the availability of wider and longer semitrailers affect industry equipment choices? Are there other segments of the industry besides LTL common carriers that will adopt the use of twins? Is increased capacity or greater flexibility in handling and routing cargo the key factor in choosing twins? To begin answering such questions, the TRB study has examined post-1983 trailer sales statistics and interviewed trailer manufacturers and a number of carriers. The preliminary findings of these activities are presented in the following paragraphs.

Trailer Sales

For analyzing the industry response to the availability of new truck configurations, trailer sales statistics are limited and can be misleading--the characteristics of trailers purchased in any given year do not necessarily represent the desired or ideal mix of trailer sizes. Instead they reflect the immediate equipment needs of motor carriers that are in a position to acquire new equipment. Quite possibly these carriers initially acquire newly available equipment in proportions beyond those planned for their overall long-term inventories simply because none of this equipment is on their current inventory.

Nevertheless, trailer sales statistics are the first place that changes in equipment choices by the industry would become apparent. Statistics on trailer sales by size are compiled periodically by the Truck Trailer Manufacturers Association (TTMA) (14). Their 1984 survey of van trailers took place over a period of 9 to 15 months after the effective STAA date at a time when the industry was starting to rebound from its recession and many uncertainties about the extent of the designated network had been resolved. Trailer sales in 1984 were nearly twice the 1983 levels (15).

Compared with 1982 survey results, the 1984 TTMA survey revealed a major shift in the characteristics of new trailer sales, indicating that the STAA of 1982 is having an effect on industry equipment choices (Table 3). The most dramatic shifts concerned van trailer widths and the longer semitrailers. About 70 percent of all 1984 sales were of trailers 102 in. wide, up from nearly zero in 1982. The 45-ft semitrailer dropped from three-quarters of the van market to 15 percent as the market share of the 48-ft semitrailer grew from nearly zero to more than one-half of new trailer sales. Twins (27- and 28-ft lengths) also increased their market share, but more modestly, from 8 to 22 percent.

TABLE 3 Van Trailer Sales by Size (*14*).

Dimension	Percentage 1982	1984
Length		
More than 48 ft	2[a]	1
Exactly 48 ft	2[a]	56
Exactly 45 ft	75	15
27-28 ft	8	22
Other	15	5
Width		
Exactly 96 in.	99.7	29.5
Exactly 102 in.	0	70.3
Other	0.3	0.2

[a]Includes all trailers with lengths greater than 45 ft.

Although these changes are significant and demonstrate that industry is beginning to use twins and the other new vehicle types, the trailer sales statistics alone do not reveal what motor carrier types purchased the new 1982 STAA trailer types, why, or how long this trend will continue.

Interviews with Trailer Manufacturers

As part of the study, staff members have interviewed eight large trailer manufacturers, which collectively sell equipment to motor carriers based throughout the United States. Twin trailers currently account for 5 to 30 percent of their market, and 48-ft semitrailers account for about 30 to 75 percent, figures that are generally consistent with the trailer sales figures discussed earlier. The manufacturers provided their assessment of the current and future market characteristics for twins, 48-ft-long semitrailers, and 102-in.-wide semitrailers.

Twins

Most of the manufacturers agreed that the primary market for twins is LTL common carriers, and so far the larger common carriers have accounted for the bulk of twin purchases. Virtually all orders are for van trailers, and regionally the sale of twins has been strongest in the Midwest and in the southeastern states. In the Northeast, sales of twins have been sluggish, and the manufacturers cite small terminals in congested urban areas and shorter trip lengths as underlying factors that diminish the advantages of added cubic capacity and operational flexibility of twins. In the West, sales have not greatly increased because many pre-1983 twins are still in service. In addition to common carriers, the manufacturers report scattered sales to private and contract carriers, mostly serving industries, such as food and retail store chains, that move low-density commodities to many distribution points.

So far, the manufacturers report that carriers are attracted to twins because of the added cubic capacity; operational flexibility will not be a real factor, they believe, until carriers have had more experience with twins.

The trailer manufacturers disagree about the long-term outlook. Some believe that twins are most advantageous to the large LTL common carriers who are currently buying them and expect that the surge in twins sales will end shortly. Others expect that twin sales will continue to be strong as smaller LTL common carriers and more specialized contract and private carriers adopt their use.

Semitrailers 48 ft Long

Manufacturers report that the 48-ft semitrailer is becoming the industry standard. All types of carriers are purchasing them, especially truckload contract and private carriers. Although longer semitrailers are now legal in many states under grandfather clauses, the trailer manufacturers expect that sales of those longer than 48 ft will be confined to specialized users, such as can manufacturers, and will account for a tiny share of the market.

Semitrailers 102 in. Wide

Manufacturers report that the predominant width for new twin trailers is 102 in. For other trailer lengths, including the 48-ft semitrailers, it is a common but not an overwhelming choice. Some industries, such as food store chains, prefer the 96-in. width because the pallets for their commodities are designed for this width. Overall, however, manufacturers expect 102-in.-wide trailers to become increasingly popular.

Carrier Interviews

As of June 1985, study staff had interviewed four LTL common carriers with predominantly eastern and midwestern operations that range in size from 600 to 3,000 line-haul tractors in service, two California-based carriers, and one transcontinental carrier. Although a more reliable picture of the industry will be available when more interviews have been completed, the results of the early interviews are generally consistent with one another and consistent with the findings of the trailer manufacturer interviews.

Twins

All the carriers interviewed are heavily integrating twins into their operations--eastern and midwestern firms report that twins currently account for 20 to 40 percent of their line-haul vehicle miles and that in 5 years they expect this figure to be 50 to 80 percent. Although all of the carriers would be considered large, size was not so much a factor in the selection of twins as were network characteristics. Twins accounted for more than 85 percent of the fleet for the California-based and nationwide carriers.

Most of the carriers pull twins occasionally with three-axle tractors, but the incidence of the resulting six-axle twins will decline as new two-axle tractors are acquired. All the carriers interviewed are ordering identical drive trains for new tractors regardless of whether they are intended to pull twins or semitrailers.

Three of the four eastern carriers report that their major use of twins is on high-volume routes between break-bulk facilities, which takes advantage of the higher cubic capacity of twins to reduce line-haul truck miles. Although they expect to take advantage of the routing flexibility of twins, this will require substantial modification of operating practices, which they believe cannot occur until they have more experience with twins and more twins are available. One regional carrier, however, with few high-density routes, decided to adopt the use of twins primarily because of the potential for improved operating flexibility.

The use of twins to maximize flexibility and minimize capital cost is common among regional LTL carriers in the West. These carriers do not organize their terminals on the hub-and-spoke pattern characteristic of eastern and nationwide firms. Instead, they load directly to the individual terminals in their networks. With few lines having high volumes of freight and with terminals spaced much farther apart than in the East, the western carriers find the twins essential to serving their market. For example, it is quicker to load a 28-ft trailer than a longer semitrailer. Because shippers in the West expect overnight delivery within 500 mi, this ability to load and dispatch is essential for scheduling. In addition, the use of the single 28-ft trailer for pickup and delivery eliminates much of the need for an additional fleet of straight trucks.

Regionally, the carriers indicated that use of twins has been somewhat curtailed in the Northeast and a few southeastern states because of limitations of the designated network and access to the network. From an overall perspective, however, they report that the effect of these limitations is slight because twin operations have so far been concentrated on Interstate routes between break-bulk facilities. Use in the West has barely been affected, with the exception of increased purchase of 102-in.-wide twins.

Semitrailers 48 ft Long

None of the carriers interviewed plan to use 48-ft semitrailers for their LTL operations, though they do plan to continue using some 40- to 46-ft semitrailers. Those that also have truckload carrier subsidiaries indicated that these carriers were starting to use 48-ft semitrailers.

Semitrailers 102 in. Wide

All carriers are now ordering 102-in. twin trailers exclusively. Because they generally will not be ordering longer semitrailers (40 to 46 ft) for some time, the width of those longer semitrailers is not known.

SUMMARY OF FINDINGS ON INDUSTRY USE OF TWINS AND OTHER STAA VEHICLES

The review of the pre-1983 use of twins and post-1983 experience, as observed through preliminary trailer sales statistics, trailer manufacturer interviews, and LTL motor carrier interviews, suggests the following findings:

- Pre-1983 use of twins was concentrated in western states. Except in California where earlier gross weight advantages had made twins appealing for a variety of users, LTL common carriers were the most frequent users of twins. Because of their relatively low-density freight and complex networks with many terminals and intermediate break-bulk facilities, LTL common carriers can take advantage of the added cubic capacity of twins and the added routing flexibility that is provided by separating freight into two units that can be easily divided.
- The 1982 STAA has had significant effects on the motor carrier industry's equipment decisions. These effects include the increased use of twins; in 1984 twin-sized trailers accounted for nearly one-quarter of all van trailer sales, up from less than 10 percent in 1982. Even more striking is the shift in new trailer purchases to 102-in.-wide and 48-ft-long semitrailers. In 1984 trailers with 102-in. widths accounted for about 70 percent of all van sales, and the 48-ft semitrailer became the most popular van trailer length, accounting for 56 percent of van sales.
- Large LTL common carriers are the primary new users of twins. Eastern and midwestern carriers interviewed report that twins already account for 20 to 40 percent of their line-haul vehicle miles and that in 5 years they will account for 50 to 80 percent. If these percentages hold nationwide, by 1990 twins carrying LTL freight will account for 8 percent of all combination-truck traffic. This could increase during the longer run. Common carriers in the West often have fleets that consist totally of twins; thus midwestern and eastern carriers may ultimately use twins more than they now expect.
- There are scattered instances of other (non-LTL) carriers that have begun using twins. For the most part this use is related to industires that move low-density cargoes to numerous distribution or outlet points, such as food stores and retail chains. Although it is too early to tell how many businesses of this type will adopt the use of twins, the available evidence indicates that such use will develop slowly and not be a major contributor to total VMT by twins.
- Among new users of twins, the primary motivation so far has been added cubic capacity. LTL common carriers are concentrating twins on high-volume routes between break-bulk facilities to produce an immediate reduction in line-haul vehicle miles. Later, after they have had more experience with twins and have larger fleets of them, these carriers hope to achieve further operational efficiency by exploiting the routing flexibility, and twins will begin running more frequently on non-Interstate highways.
- Common carriers in the Midwest and Southeast are quickly adopting the use of twins, but carriers in the Northeast are not. Reasons for the limited use include shorter line-haul trip lengths, which reduce the cost saving from the added cubic capacity; smaller terminals in congested urban areas, which lack the added space needed to maneuver twins; fewer primary highways on the designated network, which makes routes for twins less direct; and some problems in gaining access to terminals.

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Truck Accident Studies

LAWRENCE E. JACKSON

ABSTRACT

Data compiled for most accident reporting systems are typically the result of police accident investigations. Police officers usually have neither the time nor the experience to conduct in-depth accident investigations or collect the necessary data, when trucks are involved, that will allow examination of the relationships between trucks and the roadway environment. When accidents involve multiple deaths or numerous injuries, special police agencies or accident investigation teams may devote the resources necessary to examine truck-roadway environment relationships. Microscopic data, including specific accident investigations, are examined to determine problem areas and to identify vehicle characteristics. Then macroscopic studies and nationwide accident statistics are analyzed to define the potential scope of problems related to trucks and roadway environment. Truck accidents involving runaways, intersections, grade crossings, pavement roughness, barriers, overturns, and wet pavement are examined.

This paper is a compilation of the data on and analysis of many of the in-depth multidisciplinary, heavy-truck accident investigations that have been conducted by the National Transportation Safety Board (NTSB). In particular accidents that involved the truck, its design or operation, and the relationship with the roadway environment were selected for further elaboration. A review was made of the literature and of several national accident data files in order to describe the potential magnitude of problems highlighted during the in-depth investigations.

This methodology is used because most accident data files do not contain sufficient data to allow examination of the human, vehicle, and roadway environmental factors that are involved in an accident. In addition, most accidents are investigated by individuals who either do not have the necessary experience or do not have the time that is required to fully investigate a heavy-truck accident. Sometimes police investigations appear to place too much emphasis on human failure so that blame can be assessed and a citation issued. Often unwarranted citations are issued to truck drivers after accidents for speeding or speed excessive for conditions simply because of the amount of damage caused by the heavier truck. To calculate speeds of trucks involved in accidents, complex analyses are required that often use equations that police are unfamiliar with or incapable of using. There are a lot of factors that may not be accounted for in police investigations, such as tire capability, braking efficiency, and weight shift. Some police officers will not highlight a defect or failure of the roadway environment even if they recognize it because of jurisdictional pressure to avoid liability.

For a similar reason, carriers may not highlight vehicle deficiencies in their report of accidents to the Bureau of Motor Carrier Safety (BMCS). Thus accident files such as the National Highway Traffic Safety Administration's (NHTSA's) Fatal Accident Reporting System (FARS) and the BMCS accident files that are commonly referred to because they are "the best files we have" may be biased because of financial responsibility or limited in scope because of investigative experience and may not provide important information necessary for analyzing complex interrelationships to determine design criteria. In addition, there are usually limited measures of exposure by vehicle type and load configuration that can be correlated with accidents by type of truck and load configuration. Other in-depth accident investigation teams such as California's Multidiscipline Investigation Team (MAIT), Virginia's Crash Team, and NHTSA's National Accident Sampling System (NASS) teams analyze accidents in depth. Only the NASS accidents are computerized, and those accidents may still not provide sufficient data because of lack of heavy-duty experience or evidence that is destroyed or prohibited because of civil or criminal litigation.

Bureau of Technology, National Transportation Safety Board, U.S. Department of Transportation, Washington, D.C. 20594.

ROADWAY ENVIRONMENT DESIGN TO PREVENT ACCIDENTS

In the past, traffic engineers often designed for 85 percent of the vehicles. As an example, speed limits were posted on the basis of the 85th percentile vehicle. However, in the past, trucks accounted for only 10 percent of the traffic on many roads.

There are specific highway segments that carry a disproportionate amount of heavy-vehicle traffic. At some of the more recent truck accident sites the NTSB investigated, the average daily traffic (ADT), type of road, truck percentage, and truck-involved percentage were as given in Table 1.

One recent report (6) stated that 20 percent of the vehicles on the highway are commercial vehicles. Approximately one-half of these are the common tractor-semitrailers. In an unpublished paper, cited in The Influence of Roadway Surface Discontinuities on Safety (6), an author suggests that trucks will make up 34 percent of the vehicle population, are increasing precipitously in number, and are increasing in size and weight as fast as the technical, economic, and political climates will allow.

Future highway designs for high-volume truck routes will warrant special designs for trucks to provide for safety of motorists. As will be discussed later, the highway design may have to reflect

TABLE 1 Characteristics of Sites of Truck Accidents

ADT	Type of Road	Trucks (%)	Truck-Involved Accidents (%)	Reference
4,400	S.R.	17		(1)
8,871	U.S.	17.8	15.4 semitrucks	(2)
13,000	U.S.	26		(3)
3,500	U.S.	41	21 heavy	(4)
2,650	U.S.	20.9	39.1 truck combinations	(5)

the vehicle conditions and operating characteristics of trucks unless vehicle conditions are improved through state inspection of trucks and drivers. Currently, BMCS is funding a Motor Carrier Safety Assistance Program (MCSAP) in an attempt to improve truck conditions.

Designs and maintenance programs in the future will warrant escape ramps; special traffic control devices; wider, less sharp turning radii; longer acceleration and deceleration ramps; stronger and taller barriers; and better maintained, higher friction surfaces.

CURRENT INVOLVEMENT OF TRUCKS IN ACCIDENTS

The 1983 FARS data indicate that there were 3,301 fatal accidents (out of a total of 37,971 fatal accidents) involving "trucks with trailers." The first harmful event, in these 3,301 accidents, was a collision with another vehicle 72.1 percent of the time. The other leading first harmful events included collisions with pedestrians (8.0 percent), guardrails (2.7 percent), trains (0.5 percent), and overturns (5.4 percent). About 19.5 percent of fatal collisions occur on curves; 17.0 percent on wet pavement; 5.3 percent on snow, slush, or ice; and 78.2 percent where there are no traffic controls. (For comparison, FARS indicates that 39.3 percent of all fatal accidents involved collisions with motor vehicles in transport, 16 percent involved pedestrians, 5.3 percent involved guardrails, 1.1 percent involved trains, 6.5 percent were overturns, and 15.1 percent occurred on wet pavement.)

In 1983 motor carriers subject to the BMCS regulations reported (7) 31,628 accidents ($2,000 or more in property damage). These accidents resulted in 2,528 fatalities, 26,692 injuries, and $342,900,000 in property damage. In 82.1 percent of the accidents the type of truck involved was a tractor-semitrailer. Other types of vehicles involved included single trucks (9.6 percent), tractor-full-semitrailer (3.7 percent), tractor with bobtail (3.2 percent), and truck with full trailer (0.7 percent). Of the accidents involving collisions, 58.1 percent of the accidents and 62.6 percent of the fatalities involved collisions with automobiles. About 17.1 percent of the collisions involved other commercial trucks. Collisions with fixed objects account for 9.7 percent of all truck accidents. Noncollisions account for 25.8 percent of all truck accidents including overturns (8.7 percent), run-off-the-roads (7.7 percent), and jackknives (7.1 percent). Only 0.46 percent of the accidents (145 accidents) involved collisions with trains, which were reported to have resulted in $2,768,000 in property damage. The motor carriers reported that only 5 percent of the accidents involved mechanical defects of which 2 percent were brakes, 1 percent wheels and tires, and 2 percent others.

CHARACTERISTICS OF TRUCKS

Before specific types of accidents are discussed in depth, vehicle factors that are common to many trucks should be examined. These marginal vehicle factors often combine with marginal roadway environmental and human factors and result in an accident. Characteristics of trucks that differ from those of automobiles include, but are not limited to, tires, brakes, height of center of gravity, acceleration and deceleration characteristics, and length and weight. Some of these characteristics are related to each other (e.g., tires and acceleration or tires and braking).

Truck Tires

Truck tires are usually designed for mileage and unfortunately sacrifice traction to achieve longer wear. Typically, the rubber compound is made of hard material that provides less adhesion for stopping. As examples, tests conducted on wet asphalt in which the ASTM skid trailer obtained a value of 0.60 resulted in a corresponding friction value of 0.50 with a truck tire at the same speed. On a wet portland cement concrete surface with an ASTM value of 0.35, a truck tire would be expected to have a friction value of about 0.23. Truck tire traction could be expected to be about 65 to 85 percent of that of an automobile. There are numerous studies (8,9) that relate truck tire traction to ASTM numbers or automobile traction. Figures 1 and 2 show examples of some of the data. One study (8) stated that "Although it is difficult to compare traction measurements made on different pavements at different times, the difference in traction performance between truck tires and passenger car tires in traction performance is very clear."

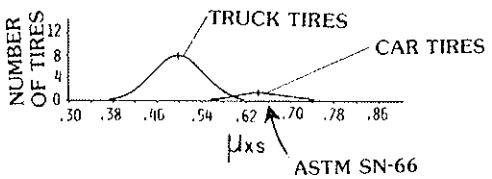

FIGURE 1 Locked-wheel braking on wet asphalt—trucks versus automobiles (8).

Brakes

Truck stopping is further depreciated if brake adjustments are not made at regular intervals. As shown in Figure 3, for one particular type of brake, brake efficiency usually deteriorates rapidly as the slack adjustment begins to exceed 2 in. The maximum available stroke on many large brakes is 2.5 in. Many truck mechanics and drivers state that the front brakes of tractors should be backed off to allow steering in emergencies because a locked wheel slides straight. It is not unusual at the scene of an accident involving a truck to find front brakes on the front wheels either backed off completely or even "capped" to immobilize them. In addition, an accident investigator or vehicle inspector can find other brakes out of adjustment.

Owner-operators of tractors often lease or use company trailers. These truck drivers tend to use trailer brakes in order to save their tractor

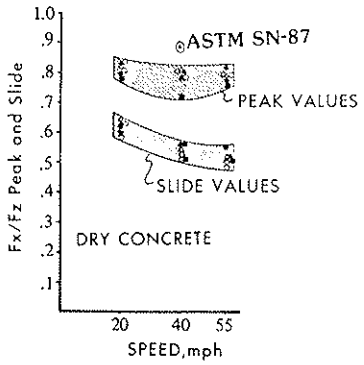

FIGURE 2 Truck tire friction on dry pavement (9).

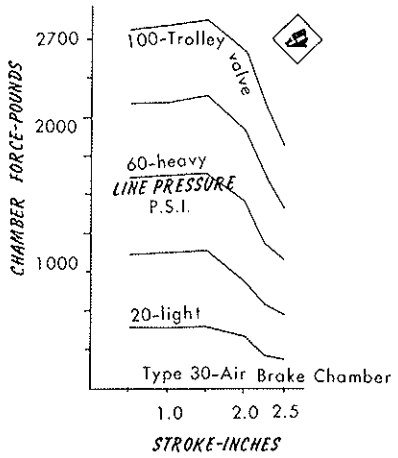

FIGURE 3 Brake efficiency—force versus stroke.

brakes. Lessors may not maintain trailers because of the lack of time the trailer is in their facility, the length of the lease, substantial travel before coming back to the home terminal, or the desire to turn the trailer over to another lessee to make higher profits. Often an investigator will find deficient brakes on trailers.

In BMCS field surveys in 1982, 33,174 vehicles were inspected and 32,510 had violations of the Federal Motor Carrier Safety Regulations (FMCSR). There were 12,564 violations discovered that resulted in a vehicle being placed out of service (38 percent of the vehicles). Brakes accounted for 37 percent of all defects, and 5,946 vehicles (17.9 percent) were placed out of service (10). Another small study examined 190 units to determine the benefits of automatic slack adjusters compared with manual slack adjusters. This study (11) found that one or more brakes exceeded recommended maximum adjustments in 47 percent of the cases with manual slack adjusters and in 42 percent of the cases with automatic slack adjusters. In 15 percent of the trucks with manual slack adjusters and 9 percent of the trucks with automatic slack adjusters the vehicles were placed out of service.

Another study (12) "estimated that more than half of all air braked vehicles have at least one brake out of adjustment and that approximately a fourth of all vehicles have 40 percent or more of their brakes out of adjustment." Many trucks have poorly maintained brakes and the highway engineer should consider designing for actual conditions if improvement in brake conditions is not obtained through other means.

Besides creating longer stopping distances, trucks with deficient brakes create other problems. One phenomenon created by a truck with improperly functioning brakes is weight shift (Figure 4). Weight shift is the transfer of weight from one axle to another. As an example, when an automobile is braked suddenly the front end dips down because of additional loading on the springs and shocks. If a vehicle has brakes that retard equally on all of its wheels, weight shift can be ignored. During braking of a tractor or trailer the lack of brakes on an axle in front of the center of gravity causes weight shift to axles that cannot dissipate energy. As an example, a bobtail tractor without front brakes might only have an effective friction value of 0.34 compared with 0.40 assuming even weight distribution. This is an additional reduction of braking efficiency of about 15 percent due to weight shift (13).

On steep long downgrades improperly adjusted brakes place higher demands on functioning brakes. This results in a buildup of heat in the brakes and the brakes may begin to fade, which can result in a runaway truck. When the pavement is wet, improperly adjusted brakes can result in uneven braking that will cause a truck combination to jackknife.

The California Highway Patrol (CHP), the Bureau of Motor Carrier Safety, and the National Transportation Safety Board have investigated numerous accidents that were caused by deficient brakes. Former California Highway Patrol leaders have helped to form the Commercial Vehicle Alliance (CVA). The CVA and the BMCS are now promoting a national inspection program (MCSAP) that emphasizes checking of brakes and drivers' hours of service (for fatigue) because these are the two most common casual factors highlighted by inspections and accident investigations.

Overturns

The center of gravity of loaded tank trucks, flatbed trucks loaded with high loads, or concrete trucks may approach 70 to 80 in. off the ground. These vehicles may overturn when centrifugal forces on the vehicle exceed 0.24 to 0.45 g's. In addition, tank trucks may experience liquid surge and trucks carry-

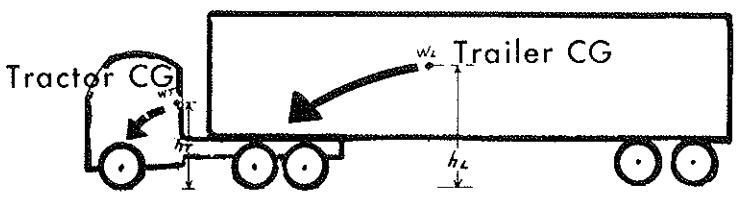

FIGURE 4 Weight shift.

ing meat may have swinging meat, which add additional unstable forces that tend to turn vehicles over.

Acceleration

The acceleration capability of empty trucks may be as slow as 2.0 ft per second per second. Loaded trucks will be even slower, especially on hills. This creates problems for truckers turning onto high-speed roads from intersections where acceleration lanes, if they exist, were probably designed for automobiles. If intersections are tight, trucks may have to turn into the high-speed lanes. At rail-highway grade crossings some trucks are required to stop and are prohibited from changing gears. Depending on grade and load, a truck may be restricted to first or second gear. This may translate into a maximum speed of 5 to 10 mph. This increases exposure time significantly, especially if two or three tracks are involved.

Other characteristics identified during NTSB accident investigations included overhanging loads that swing into opposing lanes on turns. Finally, the additional load on trucks compared with automobiles increases kinetic energy of the vehicles to 20 times that of automobiles used to test barriers and renders most guardrails ineffective.

NTSB INVESTIGATIONS

In the past, because of limited staff, NTSB has investigated only the more spectacular highway accidents. Many of these accidents involved large trucks and resulted in large amounts of property damage or numerous fatalities. The accidents NTSB investigated were not representative of any population, except perhaps to show the extent of the most serious accidents. When several accidents of any one type are viewed together, patterns begin to emerge.

Runaways

In the late 1970s NTSB investigated many runaway accidents involving trucks that had lost brakes because of heat fade or deficient brakes (14-17). Usually when a truck runs away the driver tries to change gear and misses, leaving the runaway truck in neutral. In four of the accidents grades were 1 to 4 mi long and from 1 to 10.6 percent. In three of the cases about half of the brake adjustments exceeded 2.125 in. In the other accident brakes on a trailer were jury rigged with nails and wire.

Roadway designers have addressed many problem locations by adding truck pullout areas and escape ramps. The only areas where problems may still exist are short steep grades approaching urban areas. NTSB highlighted this problem to the FHWA that determined that crash attenuators or other devices that do not require a long runout are not feasible. This may be a problem because many small urban areas developed near water and are located at the bottom of steep hills or because urban areas expanded to include steep hills. In this scenario trucks could run away striking automobiles that might be stopped for a traffic signal at the bottom of the grade. It is hoped that pedestrians, patrons of adjacent businesses, and residents will not become involved.

Recently, in June 1985, a truck in Van Buren, Arkansas, missed two signs (another large sign had been removed by a sewer contractor) and ran away down a 1,900-ft-long hill on which trucks were prohibited. The truck struck a station wagon and pushed it through a guardrail and two historic brick buildings, after which the wagon ignited. Nine people were killed and three buildings burned to the ground. At this location, standard signing has not worked.

Intersections

Intersections, especially those near some industrial areas, are another problem area for trucks. Trucks not only have deceleration problems, discussed previously, they also have acceleration and turning problems. NTSB investigated an accident in the timber belt that involved a truck transporting 80-ft-long pine logs that was making a turn into a pulp mill (18). As the truck turned right onto the cross-road, the rear of the logs swung out into the opposing lane and ripped through an oncoming school bus. Three students were killed. Special precautions are warranted for trucks near some industrial areas such as timber operations, hazardous material users, steel fabricators, or other large-commodity users or producers.

In another accident (19) an empty truck turned a corner and in 927 ft had accelerated to about 42 mph when it was struck in the rear by a bus. The highway had a speed limit of 55 mph and the bus was going 5 to 10 mph over the speed limit when the collision occurred. Speed differential of vehicles has been cited in numerous studies as contributing to conflicts that often result in accidents. At this accident site the acceleration lane was marked to be 345 ft long; the original design was for 575 ft; and AASHTO standards at the time of the accident called for a 900-ft-long acceleration lane. This empty truck on a 0.8 percent upgrade could accelerate to only 42 mph in 1,927 ft. The particular state where the accident occurred allows vehicles to drive on the shoulder, but a better solution may be longer acceleration lanes.

Grade Crossings

Rail-highway grade crossings are a special type of intersection. Although a few highway departments do not pay much attention to grade crossing accidents because they represent only about 1.3 percent of the nation's fatalities on highways, grade crossing fatalities represent more than half the fatalities to the railroad industry (575 versus 498) (20). A recently investigated accident (21) highlighted the problems at crossings for trucks such as lowboys, "Nu-Car" carriers, or house trailers that may become hung up due to the profile of the crossing. This type of accident appears to be occurring more frequently as roads and rails are raised during periodic maintenance. Recently an FHWA-sponsored committee that assessed grade crossing research needs highlighted this problem as a high priority. A proposal to study this problem is currently being considered by the NCHRP.

The NTSB recently investigated several accidents that involved trucks at crossings that were equipped with crossbucks only. Audibility tests showed that in most large trucks the horn of a high-speed train could be heard only 1 to 2 sec before impact. In several accidents the large side mirrors of the truck blocked the driver's view of the approaching engine--the most conspicuous part of the train. In another accident (22) a truck approaching a crossing at 25 mph needed 104 ft to stop short of the crossing, but the driver could not see the crossing until 88 ft before the crossing.

In two recent accidents both trucks had front

axles with inoperable brakes and the rear-axle trailer brakes of one of the trucks probably were not working. If the truck drivers had seen the trains they might have been hampered severely in trying to stop the trucks, especially the driver of the truck loaded with gravel on a timber bridge deck on a 9 percent downgrade approaching the crossing. This 1984 accident alone resulted in $3 million in property damage, more than what carriers reported to BMCS in the 145 train-involved accidents in 1983.

The NTSB previously published a study (23) that described some of the problems of trucks transporting hazardous materials. These vehicles are required by the BMCS to stop before every grade crossing even when lights are not flashing at those crossings equipped with flashing signals. Trucks that stop cannot shift gears when crossing the tracks, which limits top speed to 5 to 10 mph. In one accident (24) on a 5 percent upgrade, the manufacturer calculated that it would have taken the loaded truck 23 sec to cross the single tracks. High-speed trains cannot be detected easily when they are half a mile away and drivers have to make a decision, especially if visibility is restricted by weather, vegetation, or buildings. At active crossings some signals provide only 20 sec of clearance time before the arrival of a train. This has resulted in collisions and in trucks breaking gates when the gates descend on tank semitrailer manhole covers and are pulled forward by the moving truck.

Goodell-Grivas, Inc., has recently completed a study (25) for FHWA that addresses many of these issues. This study recommends that trucks and school buses not be required to stop at active crossings unless the devices are flashing. Active devices are recommended for installation near hazardous material depots and storage facilities, and research is needed to determine the adequacy of the 20-sec advance warning for double- or triple-bottom tractor-trailer combinations.

Trucks are a serious concern at grade crossings because of many of the problems previously cited. The 1983 data of the Federal Railroad Administration (FRA) indicate that 31 percent of all grade crossing accidents involve trucks. About 7.5 percent of the grade crossing accidents involve "truck-trailers." NHTSA-FARS data indicated that 4.2 percent of the fatal accidents at grade crossings involved combination trucks, 2.5 percent involved "other trucks," and 23.4 percent involved pickups. These two data bases may not be comparable because the FRA's data involve many injury accidents. Drivers of large trucks are more likely to survive a grade crossing accident than are pickup drivers if the rear of the truck or the trailer is struck.

Pavement Roughness

NTSB has investigated two truck accidents that may have been related to pavement roughness. One accident (26) involved a 1- to 1.25-in. depression that had been dug out and replaced. When the truck rode over the depression the tractor's tandem equalizer beam failed and the truck overturned on a guardrail that punctured the gasoline tank and resulted in ignition. In another accident (27) a truck broke its right bogie leaf spring assembly about a mile or two after running over a rough section of pavement. This pavement may have helped strain the spring to such an extent that any minor pavement irregularity could have resulted in the fracture of the spring. When the spring broke, the truck, which included a tank semitrailer, went into an uncontrollable left turn and overturned on a concrete median barrier. The high center of gravity of the trailer and the broken spring in combination with the concrete median barrier enhanced the probability of overturn.

In one recent report, cited in The Influence of Roadway Surface Discontinuities on Safety (6), it was stated that

> Perhaps the one area of possible influence that has not been well addressed in the literature is the significance of special wavelengths of road roughness to which trucks may be sensitive. It is known among experienced truckdrivers that certain long wave undulations, as typified by pavement settlements in bridge approach areas, may be peculiarly difficult to negotiate with commercial vehicles, particularly tractor-semitrailers. These features tune to the low-frequency rigid-body bounce and pitch modes of these vehicles. Because the drivers are located near the extremities of the vehicle (far from the center of gravity), large displacement vertical and fore-aft motions can be imposed on the driver, thus complicating the task of maintaining control when negotiating these road features. There is anecdotal evidence that truck drivers have experienced control problems reflecting on safety due to these effects, but there has been no known effort to compile statistics quantifying the magnitude of this particular problem. Unfortunately, available accident data are not specific enough in their recorded detail to provide that answer.

Barriers

In two accidents (27,28) trucks climbed over or overturned on concrete barriers. In one accident (27) the truck climbed over the barrier at each successive joint as the wheels broke through the barrier that was not reinforced through the joints. In the other accident (28) a full truck-trailer's tank trailer flipped over the 32-in.-high barrier. From other accidents (29) it has become clear that trucks "blow-through" guardrails and many steel bridge rails. Work being conducted at the Texas Transportation Institute (30) to design barriers capable of restraining trucks for use in selected locations is promising.

On the basis of the testing this author has seen and studied, he would encourage the use of longer test sections to determine what occurs during secondary and tertiary impacts against the barrier. The author is also concerned that the typical concrete median barrier section may have a tendency to break or dislodge the tractor's front axle, which will disable the truck's steering and stability. Although the guardrail usually provides little protection to trucks, a recent (March 1985) low-speed accident involving a school bus overturn on an 8 percent upgrade in North Carolina could have been prevented if a guardrail had been in place in front of the 63 percent slope that has a 24-ft drop.

Overturns

The FARS indicates that 5.4 percent of accidents involving trucks with trailers are overturns. The motor carriers reported to BMCS that 8.7 percent of accidents are overturns. The NTSB has investigated numerous accidents (31-33) that involved tank trucks and loaded flatbeds overturning on curves. In one accident (31) a driver of a propane truck traveling at 25 mph flattened a 119-ft-radius curve to 184 ft.

When the truck driver took corrective action to avoid an oncoming vehicle the truck overturned. In another accident (32) a gasoline truck traveling 55 to 60 mph overturned on a curve. The only speed guide was a 50-mph speed limit sign. This truck driver was also taking the curve a little wide and sharpened the turn as the truck approached another vehicle, only to overturn.

In Denver, the inexperienced driver of a semitrailer truck, which was carrying Navy torpedoes, going from one Interstate to another missed a 25-mph advisory speed and other visual cues and overturned at 42.5 mph (33). A California study (34) examined 131 tank truck accidents and found that tank trucks have three times the rate of overturns of other trucks for fatal and injury accidents and six times the rate of other trucks for property damage accidents. About 50 percent of the accidents were on curves or ramps, and tank truck accident rates were twice as high as those for other trucks at night. Two-thirds of the speed-related accidents involved overturn on a curve where the speed of the vehicle was 55 mph or less. Only 14 percent of the overturn accidents involved speeds greater than 55 mph.

Several publications by Erwin (35,36) give an indication that some trucks overturn at 0.24 to 0.45 g's. Researchers and AASHTO (37) often believe that the side friction values of 0.12 to 0.30 that are used for roadway design are sufficient. Some even state that motorists will accept a higher level of discomfort on low-speed streets with intersecting traffic, perhaps as high as 0.30 g's at 20 mph.

As cited in the NTSB investigations, truck drivers take curves flatter initially and tighten up the radius of the curve later. This phenomenon was observed in studies involving passenger vehicles by Glennon (38) in the early 1970s. Glennon's data for a 7-degree curve (818-ft radius) show that the average minimum radius driven was 691 ft and the 85th percentile radius was 645 ft, 21 percent sharper than what was designed. Data on the radii truck drivers use on curves on ramps need to be collected.

Not only do some trucks have a low threshold for overturn and drivers turn curves sharper than the design for the curve, but trucks are also susceptible to yaw divergence or instability. "Yaw divergence will lead to rollover in the absence of corrective steering action or reduced speed. Yaw instability manifests itself as the tendency of a vehicle's heading to diverge or increasingly point away from the direction of travel" (39).

Griffith and Gillespie (6,p.38) state that

> By the nature of the way in which the load is carried, and the way in which the roll resistance is shared among axles on commercial vehicles, their turning performance is most often limited by loss of cornering force on the rear axles of a truck or tractor. When this occurs, spin-out follows, with a subsequent risk of rollover. The loss of cornering force is, in part, a function of the road surface and its friction level. In pure cornering maneuvers, the threshold of instability occurs at rather moderate slip conditions (3 to 5 degrees of slip angle), where the cornering force properties are much more dependent on the stiffness of the tire carcass than on the tire-road coefficient of friction. However, when braking is also combined with cornering, brake slip at the rear wheels will contribute to loss of cornering force and subsequent jackknife. Consequently, the potential for this type of accident is greatest when the vehicle is unloaded or when the tire-road coefficient of friction is low.

This phenomenon will occur at about 0.20 g's if the center of gravity is about 80 in. and the speed is 40 mph. The greatest deterrent to yaw instability is superelevation, which eliminates the problem under normal conditions (39).

The author's concern in this area is whether roadway designers can provide enough margin of safety to truck drivers with curve advisory signs. In addition, the author is concerned that at intersections and on merges roadway designers may taper down or completely eliminate superelevation at critical locations, perhaps where the tank truck is beginning to turn a sharper radius than that which was designed.

Wet Pavement

The 1983 FARS data indicate that trucks are slightly overrepresented in accidents on wet pavement compared with all other vehicles (16.7 versus 14.4 percent). Trucks tend to be susceptible to jackknifing on wet pavement because of lower lateral resistance. In 1977 the Safety Board investigators suspected that an empty truck might have hydroplaned before striking a van (4), but researchers claimed that trucks could not hydroplane because of the high air pressure in their tires. Recent research (W.B. Horne, Tractor-Trailer Jackknifing on Flooded Pavements, working paper for TRB Committee A2B07, January 1985, and 40) indicates that trucks can hydroplane and that the old formula should be adjusted to account for the pattern of truck tires. A truck tire at 40 to 100 psi will hydroplane at between 50 and 60 mph. Typically, at more than 55 mph a truck tire will not dynamically hydroplane without combining effects of viscous dynamic hydroplaning. The combined effects decrease cornering ability. When vehicles encounter flooded surfaces, drivers must react with proper steering input quickly and accurately and correct when coming out of the flooded surface, otherwise directional control will be lost.

Forces due to flooding can easily approach 750 lb at 60 mph when there is ponding. This type of condition can create large turning moments that can jackknife a vehicle.

The NTSB has investigated two accidents involving empty trucks on wet pavement. In one accident (4) the roadway had an inconsistent crown with a flat spot 50 to 100 ft before impact on a 3.7 percent upgrade. In this accident the truck driver lost control. In a recent accident (2) involving an empty truck, unbalanced braking caused jackknifing to occur when the truck driver hit the brakes. For this accident the University of Michigan Transportation Research Institute's T3DRS:V1 simulation was used to examine the braking of the truck on wet pavement. A truck can jackknife fully in less than 5 to 6 sec on wet pavement. The model also indicated that trucks with balanced brakes jackknife when braked because of the tractor's proportioning valve or the 2 percent cross slope, or a combination of the two. The model showed that the truck would not have jackknifed on a high-friction surface even when the surface was wet.

NASS data were analyzed to determine if empty trucks are more susceptible to accidents on wet pavement. Unweighted and weighted samples indicated that empty trucks tend to be in accidents on wet pavement more often than loaded trucks. Table 2 gives the NASS data for the weighted samples.

Past studies (41,42) have also highlighted that trucks are involved more frequently in accidents on wet pavement. On the basis of 1977 BMCS data one researcher (41) noted that wet and snowy pavements raised the accident rates of all trucks on all

TABLE 2 Involvement of Trucks by Load in Accidents on Wet Pavement

Load Weight (lb)	No. of Accidents on Wet Pavement	Other Accidents	Percentage on Wet Pavement	Loaded or Unloaded
0-2,500	17,321	62,668	21.65	Unloaded
2,600-10,000	4,073	19,010	17.65	Loaded
10,100-30,000	3,869	22,030	14.94	Loaded
30,100-80,000	4,331	26,270	14.16	Loaded
80,100 or more	36	1,392	2.52	Loaded
Unknown	13,140	89,732	14.64	
Total (average)	42,770	221,102	(16.21)	All

roads. Road surface condition was found to accentuate the effect of day or night such that wet snowy roads at night often had a particularly serious effect on singles and especially on doubles. The other study (42) indicated that the probability of occurrence of jackknifing before an accident, compared with the probability of its nonoccurrence, is about ten times greater on a wet road than on a dry road. Another study (6,pp.38-39) stated:

> The effect shows up in the accident statistics such as the 1980 FARS data for tractor-trailers and doubles. Taking the 10,000- to 30,000-lb weight as indicative of unloaded vehicles, and the 50,000- to 70,000-lb weight as typical of loaded vehicles, the statistics can be summarized as follows:
>
> 1. On dry pavements jackknife is involved in about 7 percent of all fatal accidents of loaded combination vehicles and about 10 percent of those for unloaded vehicles, and
> 2. On wet, snowy, or icy roads the jackknife involvement increases to nearly 17 percent for loaded vehicles and 28 percent for unloaded vehicles.
>
> Thus from the standpoint of tire-road friction coupling, it is concluded that the safety performance of large commercial vehicles is uniquely critical on roads contaminated with water, ice, or snow. The threat to large vehicles under these conditions arises from the potential for loss of control, thus leading to more severe accidents; even at low speeds including jackknife or rollover accidents.

Trucks are overrepresented in accidents on wet pavement. Further study using computer models should be conducted to determine the effects of cross slopes and marginal levels of friction. Both the vehicle and the surface components of tire traction and braking should be improved.

Miscellaneous

Many reports are contradictory and results are confusing, often because vehicles are lumped together. For example, one study (43) indicated that tanker trucks had low accident rates when empty. The higher center of gravity and the potential to overturn when loaded may cause this result. Another study (44) stated that certain attributes of combination trucks might create a high risk especially when traveling empty. Still another study (45) stated that empty combination trucks, particularly empty doubles, had a substantially higher accident involvement rate than did loaded combinations. Accident types, exposure data, and conditions should be separated and studied individually. In addition, better statistical control is needed to avoid contradictory results.

Another study (46) suggested that truck drivers represent a more experienced segment of the driving population and therefore may react differently in potentially hazardous situations and thus may not warrant 2.5 sec of perception and reaction time. In the truck accidents NTSB investigated, cumulative or short-term fatigue, experienced by long-distance truck drivers who may be bored or tired, indicates a need for longer reaction times.

NUMEROUS FACTORS

One of the first accidents the author investigated involved a gasoline truck (high center of gravity), a 720-ft-long 12.6 percent downgrade, and a curve of 117- to 100-ft radius at the bottom of the grade followed by a railroad track with activated flashers and stopped vehicles. The truck overturned and burst into flames. The driving task was perhaps too great, and an accident was imminent. Accidents often occur when vehicle, human, and roadway environmental factors are deficient or marginal.

CONCLUSIONS AND RECOMMENDATIONS

1. Future heavy-truck populations on some specific routes will warrant additional roadway designs for trucks.
2. Trucks often have deficiencies that may have to be accounted for in roadway and vehicle designs because of the frequent occurrence of such deficiencies, including
   - Truck tires provide low friction levels;
   - Brakes, especially front brakes, are not functional in 30 percent or more of the vehicles inspected depending on the study cited; and
   - If trucks have nonfunctioning brakes, the load may undergo a weight shift to those wheels that would further depreciate braking capability.
3. Good solutions for runaway trucks on short steep grades approaching populated areas with limited land for escape ramps have not been developed.
4. Intersections adjacent to special industries such as wood or steel mills may need to have special turning ramps to eliminate load entry into adjacent lanes.
5. At grade crossings trucks have special problems and crossings may have to be designed for trucks near some industries. Trucks have audibility, visibility, low clearance, acceleration, and exposure problems at grade crossings.
6. Pavement roughness may cause uncontrollable loss of truck steering due to broken springs, flat tires, or other components.
7. Most barriers placed before the early 1970s are ineffective in redirecting a truck.
8. Trucks overturn frequently and may turn over when centrifugal loads exceed 0.24 to 0.45 g's if the driver can drive at the design radius of the curve. Often a driver creates a sharper radius.
9. Trucks are overrepresented in accidents on wet pavement and tend to jackknife under many conditions, such as unbalanced braking, lack of brakes, or on low-friction surfaces, and can hydroplane when lightly loaded.

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The opinions, findings, and conclusions expressed in this paper are those of the author and not necessarily those of the National Transportation Safety Board. The author is responsible for the facts and the accuracy of the data presented.

Accident Data Needs for Truck Safety Issues

HUGH W. McGEE

ABSTRACT

Debates about changes in federal truck size and weight limits have emphasized safety as a major issue, and in all cases it has been found that adequate information concerning the safety implications of the proposed changes has been lacking. Although size and weight issues have recently dominated FHWA's concerns about truck safety, there are many questions and issues related to large truck safety on the highway that are still unresolved. In this paper are discussed preliminary findings that lead up to a study plan of data needs necessary to address large truck safety issues in a systematic manner. First, the critical truck safety issues that need to be resolved so that FHWA and the states can make better informed decisions about truck operation restrictions or modifications to the highway system are identified. Next, data elements required to analyze these issues are identified. The ability of existing data bases to provide these elements is discussed as are alternate methods for collecting nonavailable data.

To resolve an issue, test a hypothesis, or merely go on a problem-searching expedition, data to analyze are needed. These data must be of the right kind and in sufficient quantity to permit statistically valid analyses.

It is also true, or at least it should be, that data are collected for a specific purpose. That is, the data are, or are anticipated to be, used to develop statistics that are analyzed to present trends, identify problems, develop relationships, and perform evaluations.

In summary, data requirements are dictated by current or anticipated issues that need to be resolved. More specific to the theme of this symposium, truck data needs are, in part, a function of truck safety issues. This should be an obvious point, but it is one that is too often overlooked in data collection systems.

In this paper are presented what this author believes are the minimum truck data that are required for addressing key truck safety issues, particularly those relevant to the highway community. The paper is focused on accident and exposure data required for truck accident studies. It is recognized that there are other truck issues and, therefore, data elements that are important to the highway community. These are not discussed here.

TRUCK SAFETY ISSUES

The first step in identifying truck safety data needs is to define what the truck safety issues are or are likely to be. More specifically, what are the truck safety issues that can be addressed through traditional accident analyses?

To identify these issues, representatives of the various operating offices of the FHWA (e.g., Office of Traffic Operations, Office of Highway Planning) were interviewed. Additional input came from the literature (i.e., what issues were being raised and evaluated by others). Finally, a panel of researchers experienced in truck accident studies offered their opinions regarding truck safety issues.

These activities resulted in the identification of 66 issues. However, many of these were interrelated and some were not resolvable through traditional accident analyses. A list of those issues that are considered to be the highest priority truck safety issues follows.

- What is the safety record of various truck types and what variables influence their safety?
- What is the relationship of gross weight to truck safety?
- What is the relationship of truck length (or trailer length) to truck safety?
- What is the relationship of the type of highway to truck type?
- Where do truck accidents occur on various highway types and does this vary by truck type?
- How is truck safety affected by critical geometric elements such as lane width, shoulder width, degree of curvature, grade, and so forth?
- What is the relationship of traffic volume (and truck volume) to truck safety?
- In what type of accidents are different types of trucks involved?
- Are restrictions of trucks by lane or time of day effective safety measures?
- What is the incidence of drivers under the influence of alcohol, drugs, or fatigue in truck accidents?
- Are various types of barrier systems (e.g., guardrail, concrete safety shape, impact attenuators) effective in reducing truck accident severity?

The order of listing in no way signifies the order of priority. It should be emphasized that these issues are, for the most part, highway oriented and therefore within the interests of FHWA and, presumably, state highway departments. No doubt there are other valid issues that are of high priority to other organizations and agencies.

Also, except for a few specific ones, these issues tend to be global issues for which more specific subissues could be formulated. Indeed, the first issue could be considered an "umbrella" issue for nearly all of the others listed. This is so because in order to determine what variables affect truck accident rates, consideration must be given to various characteristics of the truck, the driver, the highway, and the environment.

Bellomo-McGee, Inc., 410 Pine Street, S.E., Vienna, Va. 22180.

GENERAL DATA REQUIREMENTS

After what were believed to be the critical safety issues related to truck safety were identified, the next effort was to identify the data elements that would be needed.

In any evaluation of highway safety using accident rates as a measure, there are two types of data. The first, of course, is the accident data themselves. Depending on the issue, various data on the accident may be required. These may range from a simple count of accidents involving a certain vehicle type to a specific aspect of the accident (e.g., when it occurred, type of accident, actions before the accident, driver condition).

Accident measures are typically expressed as accident rates (i.e., accidents per mile of highway or more commonly accidents per vehicle miles traveled). The denominator that provides the rate calculation is typically expressed as the exposure value. Hence, the second principal type of data is the exposure of the vehicles.

Exposure data are important because they are crucial to calculating the actual likelihood of an accident. To be meaningful, the exposure data must be related to the variable (issue) being evaluated. For example, if an accident rate for double-trailer trucks with a van trailer configuration is being sought, then the volume of these trucks over the study section is needed as well as the number of accidents.

With regard to the first issue listed previously (i.e., what is the safety record of various truck types and what variables influence their safety?), if a researcher were asked what factors are likely to influence truck accident rates, he would likely identify quite a few. This is what was done by a panel of five experts in the field of accident research.

The following subsections give the factors, hence the types of data, that it was thought necessary to consider in addressing this basic issue. The factors are grouped into truck factors, driver factors, highway factors, traffic factors, and environmental condition factors. Collectively there are 27 factors, not including possible subcategories.

Truck Factor Data Elements

1. Type
2. Number of axles
   - Tractor
   - Trailer
3. Trailer type
4. Cab type
5. Cargo type
6. Width
7. Length
   - Overall
   - Tractor
   - Trailer or trailers
8. Weight
   - Gross
   - Net cargo
9. Trip type
10. Carrier type
11. Condition of vehicle equipment

Driver Factor Data Elements

1. Age
2. Driving experience
   - Trucks in general
   - Particular truck
3. Hours of service
4. Driving record
5. Driver training

Highway Factor Data Elements

1. Highway type
   - Function
   - Access control
   - Number of lanes
   - Divided or undivided
   - Design speed
2. Geometric elements
   - Curve
   - Grade
   - Passing or no-passing zone
   - Interchange
   - Intersection
   - Work zone
   - Lane width
   - Shoulder width
3. Location
   - Urban or rural
   - State

Traffic Factor Data Elements

1. Volume
   - Average daily traffic
   - Hourly traffic
2. Level of service
3. Truck volume
4. Percentage of trucks
5. Speed

Environmental Factor Data Elements

1. Temporal
   - Season
   - Time of day
2. Pavement conditions
3. Light conditions
4. Visibility conditions

If it is truly believed that all of these factors affect truck accident rates, albeit to varying levels of significance, then, to be statistically accurate, an experimental design that would ensure that there is enough of a sample (in this case accidents and exposure) to establish a reliable estimate of each specific cell accident rate, should be developed. Clearly, the sample size requirements would be quite large and probably unattainable within reasonable periods of time and with available resources.

Consequently, to reduce the data collection task to a manageable level, some judgments must be made about those variables of primary interest and those that can be accepted a priori as insignificant or simply ignored. For example, what if it were true that trucks with cab-over-engine tractors experienced a higher injury rate than cab-behind-engine tractors? What if it were also true that double-trailer trucks had a much greater percentage of cab-over-engine tractors than single-trailer trucks? Then, assuming all other factors were accounted for, if doubles had a higher accident rate than singles, it could be attributed, at least in part, to the tractor type rather than to the trailer configuration. This would have been indeterminable if the cab type had not been included as a data element and considered in the analysis.

It is these possible relationships that argue for more rather than fewer factors being included in the experimental design and, hence, data elements. Still, resources and time are limited, so consideration must be given to reducing the number of factors (variables) and strata within a factor.

A list of factors was developed that should be considered a minimum. These factors and the strata assigned for the factors (variables) dictate the data elements. In the next few sections these factors are discussed in more detail.

SPECIFIC DATA ELEMENTS

Truck Type

One of the most critical issues is the ability to differentiate the safety of various truck types. Types of trucks can be described in many ways depending on the specific issue at hand. Indeed, a truck could be classified according to its

- Number of axles,
- Number of trailers,
- Trailer type,
- Tractor (cab) type,
- Weight, and
- Length and width.

Here, truck type is considered to be the general description of the truck as determined by the configuration of the power unit (tractor) and the cargo unit or units. It is therefore the lowest order or, expressed another way, the least specific classification of trucks. Under this assumption the following truck types are of concern for safety issues.

1A. Single-unit truck--all trucks with the cargo unit and tractor on a single frame having two or more axles with at least six tires (2-0 and 3-A).

1B. Single-unit truck with trailer--a single-unit truck pulling any type of trailer (2-1, 2-2, 2-3, 3-2, 3-3).

2. Tractor-semitrailer (semi)--a truck combination consisting of a tractor with two or more axles and a semitrailer with one or more axles (2S-1, 2S-2, 3S-1, 3S-2).

3. Tractor-semi plus full trailer (double)--a truck combination consisting of a tractor with two or more axles, a semitrailer with one or more axles, and a full trailer with one or more axles (2S1-2, 2S2-2, 3S1-2, 3S2-2).

or

3A. Turnpike double--three-axle tractor and two two-axle semitrailers each 40 to 45 ft long coupled by a two-axle dolly.

3B. Rocky-Mountain double--a three-axle tractor, a two-axle 40- to 45-ft semitrailer, a one-axle dolly, and a second 27- to 28-ft single-axle semitrailer.

3C. Twin-trailer truck--a double trailer truck with a two- or three-axle tractor and two single-axle semitrailers, each usually 27 or 28 ft long, coupled by a single-axle dolly.

4. Tractor-semi plus full plus full trailer (triple)--a truck combination consisting of a two- or three-axle tractor, a semitrailer with one or more axles, and two full trailers with one or more axles each.

Note that this classification scheme yields four, five, six, or seven truck types, depending on the level of detail.

The smallest strata would allow distinction of four truck types:

1. Single-unit truck (straight),
2. Tractor-semitrailer (single),
3. Tractor-semitrailer-full trailer (double), and
4. Tractor-semitrailer-full trailer-full trailer (triple).

The largest strata classification would distinguish between single-unit trucks with and without a trailer and also would establish three separate types of doubles: the so-called turnpike double, the Rocky-Mountain double, and the twin-trailer double. These three types of doubles are different enough in terms of their configuration and operation that they should be evaluated separately.

The obvious truck characteristic missing from this classification is the number of axles. This is so because it was believed that the number of axles does not significantly affect safety. If this premise is accepted, there is no reason to be able to distinguish the number of axles in either the accident or the exposure data.

For accident data, the truck classifications recommended can only be discerned from the Bureau of Motor Carrier Safety (BMCS), Fatal Accident Reporting Systems (FARS), and National Accident Sampling System (NASS) data bases. Only five states currently have an accident report form that can distinguish between a single- and double-trailer truck type. Consequently, to determine truck type in accident involvement will require a special data collection effort.

On the exposure side, truck classifications are established on the basis of the number of axles and trailers, so it is possible to distinguish among straight trucks, and single- and double-trailer combinations. However, the different types of doubles (i.e., western versus Rocky Mountain versus twin trailer) cannot be distinguished by current traffic counting systems.

Truck Length

The relationship of truck length to truck safety still remains an unresolved key issue. On the basis of safety, just how long can trailers or the total tractor-trailer or trailers combination be allowed to be? There are valid arguments for evaluating both trailer length and overall length, but it is believed that overall length is the more relevant highway safety issue. The only exception to this statement is, perhaps, the specific issue of turning trucks and offtracking. Longer trailers and more specifically longer wheelbases are more critical than is overall length.

Assuming that overall length is accepted as the key variable, it must be possible to distinguish, as a minimum, total truck length in both the accident and the exposure data. For accident data, overall truck length is available from the BMCS and the NASS data base. However, none of the states currently records either overall truck length or trailer length on their police accident report form.

For exposure, truck length is not generally available from truck classification or weight surveys. Hence, this requires a special data collection effort. The technology for identifying vehicle length is still developing and therefore not yet being used to any significant degree.

Truck Weight

Maximum allowable gross and axle weight is certainly an issue related to pavement and bridge structure performance. Weight is also a critical safety issue. Hence, it is a necessary data element for a comprehensive analysis of truck safety.

Gross weight of trucks involved in accidents is available from both the BMCS data and the NASS data base. The problem with the BMCS weight data is that they are self-reported and therefore susceptible to underreporting for overweight trucks. None of the states reports gross weight on police accident report forms.

Gross weight exposure data are available from truck weigh stations for some classes of roads. However, the weights obtained from these are often not representative of the lower and overweight stratum because drivers of overweight trucks, aware that the weigh scales are open, bypass them by using alternate routes. Also, trucks are sometimes allowed to pass by the scales if it is observed that they are empty.

The technology for portable and weigh-in-motion devices is improving, which should make it more feasible to collect reliable weight data for a variety of highway types.

Trailer Type

The relative safety of trucks with different trailer (cargo) types was not identified as a high-priority issue. Still it is a required data element for the following reason. There are numerous types of trailer configurations for both straight trucks and tractor-semitrailer combinations. However, for double- and triple-trailer combinations, there are relatively few trailer types, primarily limited to enclosed vans, with some tankers, bulk commodity, and automobile trailers. An analysis of singles versus double-trailer combinations would be more reliable if similar trailer combinations were compared. This would ensure that any effect due to trailer type is controlled.

To do this it is necessary to identify trailer type in the accident and exposure data collection system. There are numerous trailer types, so to minimize the classification strata, the following classification scheme is suggested.

1. Van--cargo is completely hidden from view; cargo unit has solid top, sides, front, and rear.
2. Tank, liquid carrier--may have different configurations but contains a liquid substance.
3. Platform--flat cargo-carrying unit with no sides or top structure.
4. Bulk commodity--loose or semiloose solids carrier (e.g., agricultural products, cement) has sides but no hard top.
5. All other cargo body types.

Essentially four distinct trailer types are established with all others grouped in a fifth class.

For accident data, trailer type is available from the BMCS data but the classifications are not the same as suggested here. This is true of the NASS data base as well. Cargo or trailer type are not identified on any state accident reporting system.

For exposure data, there are no currently available trailer type classification counts. Unfortunately, this is one truck characteristic that cannot be recorded automatically and requires manual observation.

Type of Operator

A factor that is believed to be related to truck safety is operator type. By this is meant the classification of the truck driver by employment status. It has been hypothesized that owner-operators are overinvolved in truck accidents compared with employees of either common or private fleet operators. If this is true, this factor should be considered in the design of any analysis of accident rates by truck type.

There are basically two classes of operators: (a) owner-operators who own the tractor and possibly the trailer and (b) employed drivers who are hired to operate rigs owned by someone else. This is one of the more difficult data items to acquire. It is not available from BMCS, FARS, or NASS and none of the states records this on the police accident report form. Consequently, it has to be obtained from supplemental investigations, such as a mail or phone survey as was done by the University of Michigan's Transportation Research Institute.

Exposure data for this variable are likewise not readily available and are not being collected in any data collection system. These data, too, will require supplemental surveys conducted on the road at weigh stations, truck stops, rest areas, and other places.

Driver Age

The age of the driver has been found to correlate with accidents in general, and this appears to carry over to truck drivers as well. Older, more experienced truck drivers have a better accident rate than young, inexperienced drivers. If this is true, age may be an influencing variable in the issue of the relative safety of singles versus doubles because it has been claimed that drivers of double-trailer trucks are generally the older and more experienced drivers. If so, age should be considered in the analysis.

The age of the driver is an easily obtainable data element for the accident data component. It is available from the police accident report. However, obtaining truck-type mileage by driver age will require special field surveys.

Highway Type

From the perspective of the highway engineering community, a key issue is to be able to identify the relationship of truck safety to highway type. It has long been recognized that accident rates vary by highway type as well as other influencing variables, so it can be expected that trucks experience different accident rates on different highway types, and, perhaps, this difference varies among the truck types.

There currently is no formal "highway type" classification. Highways can be classified by any number of factors including

1. Function,
2. Access control,
3. Number of lanes
4. Divided or undivided,
5. Lane or pavement width, and
6. Design speed.

To develop a highway type classification system, the percentage of mileage and vehicle miles traveled for highways defined by the first four of the factors noted previously was determined. From that analysis the following highway types were established.

Urban

1. Interstates and other freeways and expressways, more than two lanes, divided, full access control.
2. Interstates and other freeways and expressways, more than two lanes, divided, partial access control.
3. Other principal arterials, two lanes, undivided, no access control.
4. Other principal arterials, more than two lanes, divided, no access control.
5. Other principal arterials, more than two lanes, undivided, no access control.
6. Minor arterials, two lanes, undivided, no access control.
7. Minor arterials, more than two lanes, divided, no access control.
8. Minor arterials, more than two lanes, undivided, no access control.
9. Collectors, two lanes, undivided, no access control.

Rural

1. Interstates, more than two lanes, divided, full access control.
2. Other principal arterials, two lanes, undivided, no access control.
3. Other principal arterials, more than two lanes, divided, full access control.
4. Other principal arterials, more than two lanes, divided, partial access control.
5. Other principal arterials, more than two lanes, divided, no access control.
6. Minor arterials, two lanes, undivided, no access control.
7. Minor arterials, more than two lanes, divided, no access control.
8. Major collectors, two lanes, undivided, no access control.
9. Minor collectors, two lanes, undivided, no access control.

This classification yields nine types of urban highways and nine types of rural highways.

SUMMARY

It is believed that the key variables that influence truck safety include

- Truck type,
- Truck length,
- Truck trailer type,
- Truck weight,
- Driver type,
- Driver age, and
- Highway type.

These variables should dictate the experimental design and sampling requirements, and both accident and exposure data, as a minimum, have to be obtained for these variables.

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The contents of this paper reflect the views of the contractor, who is responsible for the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policy of the U.S. Department of Transportation.

Large-Truck Safety Research

M. D. FREITAS

ABSTRACT

Federal truck size and weight limits have existed since 1956. Their original intent was to protect the federal investment in the Interstate system. These limits were maximum allowable limits. Minor changes were made to these limits, but they remained maximum allowable. The Surface Transportation Assistance Act (STAA) of 1982 changed all that. Federal limits became mandatory minimums and for the first time vehicle length was included. The establishment of a national truck network was also included in the 1982 STAA. To clearly understand the problems facing the highway community in implementing the truck size and weight provisions of the 1982 STAA, it is necessary to fully understand those provisions. In this paper a history of the federal size and weight limits is presented, the new limits are explained, and some of the critical issues that have surfaced during implementation of these provisions are described.

In 1975 a new research project was initiated in the federally coordinated program of highway research and development in response to a need for better information concerning the safety implications of increased truck size and weight limits. This project was designated Project 1U, "Safety Aspects of Increased Size and Weight of Heavy Vehicles." In recent years this project has been expanded to include more general truck safety issues, and the title has been changed to "Large Truck Safety Research."

BACKGROUND

The trucking industry has experienced phenomenal growth in the last several decades; there have been significant increases in the volume of trucks, especially large combinations, on many highways. Trucks now represent more than 20 percent of the vehicles on some major arterials. The average size of trucks has gradually increased over the years, and twin-trailer combinations have become common in many western states. As the volumes of large commercial motor vehicles have grown, public concern about the safety of those vehicles has also increased. Not only are large combinations perceived as having poor handling and performance characteristics, but there is also concern about the increasing discrepancy between the size of trucks and passenger vehicles that are sharing the same roadway.

Federal and state regulations have generally limited increases in the sizes and weights of commercial motor vehicles. All states regulate truck dimensions such as length, width, axle weights, and gross vehicle weight, and federal statutes required states to restrict vehicle width, axle weight and spacing, and gross vehicle weight on the Interstate system.

The original federal limits were established in the Federal-Aid Highway Act of 1956, the same act that authorized the first expenditures for the Interstate highway system. Those limits were intended to protect the federal investment in the Interstate system. In 1974, in response to the oil embargo, Congress increased the federal weight limits, which permitted states to increase their weight limits on the Interstate system from 18,000 to 20,000 lb for single axles, from 32,000 to 34,000 lb for tandem axles, and from 73,280 to 80,000 lb gross vehicle weight.

Sweeping changes to truck size and weight limits were made in the Surface Transportation Assistance Act (STAA) of 1982. First, federal gross vehicle weight limits and axle weight limits on the Interstate system were made minimum as well as maximum to compel the three "barrier" states along the Mississippi River to raise their weight limits on the Interstate system to the federal maximum. Second, federal requirements concerning vehicle lengths were established for the first time. States were required to permit combinations with two 28-ft trailing units and combinations with a single 48-ft semitrailer on Interstate and designated Federal-Aid primary highways, and they were prohibited from imposing overall length limits for semitrailer and double-bottom combinations on those highways. Third, the 1982 STAA increased the maximum width of trucks that states were required to permit on Interstate and designated primary highways from 96 to 102 in.

Implementing the vehicle length and width provisions of the 1982 STAA was difficult. One reason for this difficulty was that the only criteria specified by Congress for designating highways were that segments be safe for the larger vehicles and that the segments form a "national network," which implies a considerable degree of route continuity. There was little firm evidence, however, to determine what highway design characteristics would be unsafe for vehicles with the dimensions specified by the STAA.

Sections 138 and 415 of the 1982 STAA, which called for a study of the feasibility of allowing truck combinations up to 110 ft in length to operate on a limited system of highways, also required an analysis of the safety of large trucks on various highway types. In the limited time available for that study, little original research could be conducted on access needs, equipment requirements, driver qualifications, operational restrictions, and other factors that affect the safe operation of longer combinations. Potential safety impacts could only be discussed in general terms on the basis of the limited research that had been done by states that currently permit the operation of turnpike doubles, Rocky-Mountain doubles, and triples. Much more research is required on the operational and safety problems of longer combinations.

FHWA, U.S. Department of Transportation, Turner Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, Va. 22101.

PROJECT DESCRIPTION

The original objective of Project 1U was to determine the impact of increases in truck size and weight limits on highway safety and to develop cost-effective solutions to identified safety problems. In recent years the objective has been expanded to include the identification of truck safety problems related to highway design or operation and the development of countermeasures.

The project can be divided into four basic areas of research, generally referred to as tasks. The first task, "Accident Investigation of Large Trucks," includes all the studies in the project involving accident and exposure data collection analysis. The second task, "Effect of Large Trucks on Safety of Traffic Operations," includes studies that involve the interaction of trucks with various highway design elements, such as grades, interchanges, two-lane roads, and intersections. The third task, "The Effect of Truck Size on the Interaction of Trucks with Other Vehicles," involves the investigation of the effect of trucks on other vehicles. Research on automobile driver behavior as well as splash and spray is included in this task. The fourth task, "Safety Impact of Large Truck Dynamics," focuses on vehicle handling. It includes the development and use of vehicle simulation models as well as full-scale testing of large trucks.

PAST AND ONGOING RESEARCH

The first study conducted in Project 1U was the most ambitious, the most controversial, and has drawn the most attention since its inception in 1975. The contract was titled, "The Effect of Truck Size and Weight on Accident Experience and Traffic Operations," but it has been commonly referred to as the BioTech study because it was conducted by BioTechnology, Inc. (1,2). This contract was, in reality, two separate studies that were awarded as one contract. It is the first portion of the contract, which deals with truck accidents, that has drawn all the attention.

Effects of Trucks on Accidents

This study attempted to address the basic question of how various truck characteristics (especially truck size and weight variables) influence truck accident experience. To accomplish this, an ambitious data collection scheme was developed. Six states were selected partly on the basis of the range of truck types common to those states. Individual roadway segments were then selected in each state to represent various roadway types and characteristics.

For 1 1/2 years, detailed accident data were collected for each large truck accident at each site. These data included detailed descriptions of the trucks, roadways, drivers, environmental conditions, and accidents. At the same time, detailed classification data were being collected using automated cameras and surveys at truck scales. These data were used to calculate vehicle miles of travel by truck and driver characteristics.

These data were then used to calculate accident rates that could be compared to determine the influence of various factors on truck accident experience. The two primary issues that were to be addressed in this study, vehicle weight and truck type, resulted in the two most interesting and controversial results of the study. The truck type analysis concluded that twin-trailer combinations, or doubles, experienced higher accident rates than did tractor-semitrailer combinations, or singles. The truck weight analysis concluded that increased truck weight did not increase truck accident rates. The analysis of truck weight data indicated that empty combinations experienced much higher accident rates than loaded trucks.

These two conclusions were quite controversial. Reviews of the report by representatives of the trucking industry pointed out many errors in the report and in the way the study was conducted. They generally dismissed the report's conclusions as inaccurate or unsubstantiated. A later, much more extensive review pointed out many problems with the way data were collected and analyzed. Although the impact of these various criticisms on the accuracy of the study conclusions has never been determined, they have severely reduced the credibility of this study.

Effects of Trucks on Traffic Operations

The second portion of this contract dealt with the effect of truck size and weight on traffic operations. A number of geometric conditions were selected under which it was hypothesized that larger or heavier trucks might influence traffic operations. The selected geometries were upgrades (short, long, slight, steep), downgrades (long, steep), curves (freeway, nonfreeway), grade-curve combinations, merge areas, ramps, and urban intersections. Matched weight and operational data were gathered at these sites on nearly 6,000 trucks ranging in gross weight from approximately 20,000 to 160,000 lb. Operational measures including volumes, speeds, and vehicle headways were obtained for many different traffic conditions and roadway geometries by using electronic roadway sensors. From these data, accident potential, vehicle delay, and passing behavior were analyzed for the various roadway conditions.

Three general issues were examined: (a) whether trucks with different loads or configurations have different impacts on traffic operations; (b) whether there are correlations among truck characteristics and traffic speed, headway, and other measures of traffic operations; and (c) whether the effect of truck weight on speed can be predicted. Among the findings of the study were that (a) loaded vehicles and doubles traveled slower, deviated more from the average speed of other traffic, and caused following vehicles to decelerate faster and leave shorter headways on upgrades than empty trucks and singles, respectively; (b) truck length does not have a significant effect on traffic operations; (c) the most serious safety problems caused by large trucks are on upgrades; and (d) statistically, only about one-third of the effects of trucks on traffic operations could be explained by differences in truck size and weight for the range of sizes and weights tested.

Information About Trucks and Automobile Drivers

Many motorists experience anxiety when driving near large commercial motor vehicles. The FHWA was concerned that such anxieties might influence the driving performance and behavior of some motorists and actually create unsafe driving situations. In other words, the truck would contribute to accidents just by being there; the automobile drivers, while passing, merging, or otherwise interacting with these large trucks, might behave in an unsafe manner because of concern about a big truck.

Two studies were initiated to examine this issue. One specifically addressed the influence of truck

size on driver behavior; the second examined driver attitudes toward larger trucks.

The first study, titled "The Effect of Truck Size on Driver Behavior," was conducted by the Institute for Research (3). As part of this study, several specific automobile-truck interaction situations were selected for study. Specifically, the influence of truck length and configuration was examined at a freeway entrance merge, a main-line lane change, and a narrow bridge. Truck width was studied in a rural two-lane, two-way passing situation. The influence of length and configuration was also examined in a two-lane, two-way passing situation, but problems were experienced with data collection during this experiment and no results were obtained. Field work involved the collection of microscopic traffic measures using the Traffic Evaluator System, photographic data, and observations of erratic maneuvers and vehicle types. In general, the study did not indicate any serious driver behavioral problems associated with the presence of large trucks.

One experiment consisted of operating a wide truck on a narrow two-lane road to determine the behavior of automobile drivers in passing situations. In general, automobile drivers did adjust their driving behavior as trucks got wider, but not in an unsafe manner. Lateral placement, gap acceptance, and following distance all changed but within reasonable levels.

Another experiment examined the effect of truck length on the merging and weaving behavior of automobile drivers. To quote the authors of the report (3,p.14):

> While there were more forced lane changes, gore encroachments, and braking near the triple trailer combinations in the merging situation and more lane changes in front of the triple in a weaving situation, it is possible that these effects were due to lower speeds of the triple, which was interjected into the traffic stream for study purposes; hence, the effect may have been due to vehicle speed rather than configuration.
>
> Thus, there was no basis for an indictment of increased truck length regarding the safety of nearby vehicles. On the other hand the weaknesses in the data precluded any strong conclusions regarding the absence of deleterious effects.

The companion study, entitled "Motorists' Attitudes Towards Large Trucks," was intended to survey driver attitudes to determine whether certain truck configurations create high levels of anxiety or concern in specific driving situations. Administrative problems, however, prevented the study from being completed.

Braking and Handling Characteristics of Heavy Trucks

One hypothesis concerning larger and heavier trucks was that increases in size and weight would adversely affect the braking and handling properties of these vehicles. Several studies were initiated simultaneously to begin to address this issue.

The purpose of the first study was to develop or modify a vehicle simulation model to analyze the braking and handling ability of large trucks. In this study, the Highway Safety Research Institute (HSRI), now known as the University of Michigan Transportation Research Institute (UMTRI), modified their handling and performance model to allow analyses of multitrailer combinations and to meet other FHWA specifications. The second study was simply an effort to make several full-scale tests of large trucks to validate the model described. This study was conducted by the Texas Transportation Institute. The third study involved the development and fabrication of an instrumentation package to be used in later full-scale truck-handling tests. This contract was conducted by Systems Technology, Inc.

None of these studies resulted in any actual examinations of vehicle handling. All three studies were aimed at developing useful tools for vehicle-handling research. As such, they served as a lead-in to a major truck-handling study conducted by UMTRI as part of Project 1U. The final report of that study, titled "Influence of Size and Weight Variables on the Stability and Control Properties of Heavy Trucks," is being prepared for publication (4).

As the title indicates, this study examined how variations in truck size and weight influenced the braking and handling ability of large trucks. The study included both vehicle simulations and full-scale testing. Six general issues were addressed in the final report: axle weights, gross vehicle weight, length of individual units or overall vehicles, types of multitrailer combinations, vehicle width, and bridge formula constraints.

The conclusions are too extensive to list here but two conclusions are especially worthy of discussion because of their timeliness and the interest generated by them. The first involves various multitrailer combinations and a phenomenon known as "rearward amplification." This is a characteristic of multiunit vehicles in which the lateral acceleration experienced by the first unit in a quick evasive maneuver is amplified rearward to such an extent that the rearmost unit could be caused to roll over.

A comparison of various truck configurations determined that western twin-trailer combinations and triple-trailer combinations exhibited the highest amplification ratios (lateral acceleration of the rearmost trailer/lateral acceleration of the tractor) with the triple experiencing an amplification ratio of 3 to 1. Other research conducted by UMTRI for Canada indicates that improved coupling mechanisms could reduce these amplification ratios and reduce the opportunity for rear-trailer rollover accidents.

A second important conclusion of the study about vehicle width was that increasing truck widths from 96 to 102 in. might be one of the most important improvements to vehicle dynamics possible. Widening tractor-semitrailer combinations could result in improvements in rollover thresholds of up to 18 percent (if both the tractor and trailer are widened). To realize the full improvement in stability, however, a 102-in-wide trailing unit must be equipped with wider axles than are used on a 96-in-wide unit. The wider tire and spring spacings significantly improve the rollover threshold.

Cargo Shifting in Trucks

There was one contract let to study vehicle handling in Project 1U entitled "Simulation of Cargo Shifting Effects on Vehicle Handling," which was conducted by the Applied Physics Laboratory. This study (5) attempted to model the dynamics of trucks with shifting cargoes such as tankers and trucks with hanging sides of meat. The final model developed could predict the influences of liquid sloshing with some success but could not predict the influence of swinging meat. Fortunately, from a vehicle dynamics point of view, the meat industry appears to be reducing the amount of beef and pork that is shipped in sides. Most meat leaves slaughterhouses boxed in smaller portions, which eliminates this problem.

Truck Splash and Spray

Another study in Project 1U addressed an issue that has concerned the public for many years--truck splash and spray. There was some fear that longer trucks, especially multitrailer combinations, would create more splash and spray than conventional combinations.

The study, which was conducted by Systems Technology, Inc., examined the influence of truck size and weight on splash and spray and tested possible solutions to the splash and spray problem (6). The study involved wind tunnel tests and extensive controlled field tests. The study concluded that larger trucks did not create significantly greater spray patterns primarily because as trailers are added to a combination, the wheel paths of the last trailer are virtually dry. The major part of the spray for any combination is generated by the tractor so little water is left for the trailer wheels. The field tests of countermeasures resulted in several promising prototype solutions and one quite effective off-the-shelf device. That type of device, which might be generically called a fuzzy mud flap, is now being marketed by several companies and is the basis of the recent legislation requiring splash and spray suppression devices on trucks.

As stated earlier, the initial intent of Project 1U was to examine the safety consequences of increased truck size and weight and to develop possible countermeasures. As the project plan was being developed and input was being solicited from FHWA operating offices and the states, it became apparent that there were a number of general truck safety issues, not related to truck size and weight, that were of concern to the highway community. When these were determined to be related to the design or operation of the highway, they were included in the project. To understand the full scope of the project, it is important to be familiar with these studies.

Runaway Trucks on Steep Downgrades

A longstanding safety concern for both truckers and the highway community is the problem of runaway trucks on steep downgrades. Several years ago the American Trucking Association suggested that a method be developed to help drivers anticipate the severity of downgrades so that they could adjust their approach speeds accordingly. Systems Technology, Inc. (STI), conducted a study to determine the feasibility of a grade severity rating system (7).

This method originally consisted of a numerical rating scheme for downgrades that would be based on their overall severity measured by truck brake temperatures. This concept proved to be unfeasible for trucks of various weights. Furthermore, although it would provide better information to the truck driver concerning the relative severity of the grade, it also presumed that the driver would know what to do with that information.

To resolve these two problems, a second study was awarded to STI to develop a system that would provide maximum safe speeds of descent based on the severity of the grade and the weight of the truck (8). This proved to be a feasible concept. The result was a system of weight-specific speed signs that are currently being field tested in several states by the Transportation Research Corporation.

Truck Stopping Sight Distance

Another issue of less apparent importance, but certainly a nagging issue of concern to the highway community, is the subject of truck stopping sight distance requirements. Present crest vertical curves have been designed to provide adequate stopping distance for typical automobiles. Trucks have always been ignored, on the assumption that the superior sight distance that large trucks provide their drivers more than compensates for the inferior braking ability of these vehicles. This assumption had never been adequately addressed, however, so a contract was awarded to Automated Sciences Group, Inc., to examine truck stopping sight distance requirements and to quantify any apparent deficiencies (9). The study consisted of an analysis of typical truck configurations on various combinations of grades. The study concluded that truck stopping sight distance is not adequate on a large number of crest vertical curves but that, on the majority of these hills, deficiencies occur on only a small portion of the curve. Few grade combinations produce a situation in which truck sight distance is overly restricted for any significant portion of the grade.

Offtracking

One performance characteristic of large trucks that has concerned highway engineers over the years has been offtracking. Design engineers use turning templates to determine the ability of trucks to negotiate a specific turn. Unfortunately, the existing templates are limited in terms of curve radii and truck configuration, and cumbersome and time-consuming procedures have been required to generate new templates. To correct these problems, a user-friendly offtracking program has been developed that operates on an Apple II computer. With this program, the user may specify the path and configuration of a truck and the program will generate a turning template for use in highway design. Currently, work is under way to develop an IBM-PC version of this program with increased capability. The new version will also provide digital output for users without an x-y plotter.

Currently, there are a number of truck safety research studies under way and several are planned for the near future. These studies involve improved accident and exposure data, truck-related operational problems, and future truck issues.

CURRENT STUDIES

Impact of Specific Geometric Features on Truck Operations and Safety at Interchanges

The BioTech study examined, among other things, accident location. Freeway off-ramps surfaced as a high accident location for trucks. For this reason a study was initiated to examine the causes of truck accidents at interchange off-ramps and to develop possible countermeasures. This study, which is nearing completion, is being conducted by UMTRI. Off-ramps with a history of truck accidents have been identified using state accident records; and descriptive data on the sites have been obtained from the state or through site visits. A vehicle dynamics model was used to assess the design adequacy of the ramps in question and to identify possible solutions.

Truck Tractive Power Criteria

One long-recognized performance problem that trucks exhibit on highways is their inability to maintain acceptable operating speeds on long upgrades. Present day highway design procedures provide guidance for determining the performance of a typical truck on the grade in question. Unfortunately, this informa-

tion is based on data collected nearly 30 years ago. Since that time, several factors have influenced truck power and performance requirements. Among those factors have been the heavier average loads that truck tractors have been required to pull, improved highways that allow higher operating speeds, and concerns for improving fuel efficiency. These factors have led to many changes in vehicle design and equipment, especially in the last 10 years. Some changes, such as improved aerodynamic design, affect vehicle performance without changing the weight-to-horsepower ratio. Because the effect of these many changes on hill-climbing performance is uncertain, a study has been initiated to determine the present performance of trucks on grades and to develop an improved procedure for highway designers to use in predicting truck performance. The study, being conducted by UMTRI, will consist of extensive field data collection and model development. A simplified procedure using charts or graphs plus a predictive model with the capability of predicting the performance of any specific truck will be developed as end products.

Development of a Large-Truck Safety Data Needs Study Plan

Recent studies, both under Project 1U and under other sponsorship, have indicated that a comprehensive large-truck accident study is needed. A study was recently begun to identify and rank the most critical issues that must be addressed in a major truck accident study and to develop a data collection and analysis plan for such a study.

Techniques for Improving the Dynamic Ability of Multitrailer Combination Vehicles

As mentioned earlier, multitrailer combinations exhibit certain undesirable dynamic properties that could result in vehicle rollovers under some conditions. Improved dollies and coupling devices appear to enhance the dynamic properties of such combinations. This is important in light of the expected expansion of doubles operations nationwide and the proposal to allow longer combinations nationwide.

A study is under way to determine the desirable properties of trailer coupling mechanisms using vehicle simulation, develop a prototype dolly incorporating these features, and test the full-scale prototype dolly as well as various on-the-market dollies. These dollies will be tested for both dynamic improvement and operational ease.

Operation of Larger Trucks on Roads and Streets with Restrictive Geometry

Some states have expressed concerns about operational and safety problems of STAA vehicles on roadways with narrow lanes and shoulders, sharp curves, and other substandard design features. For instance, the offtracking of a tractor-48-ft-semitrailer combination might cause operational and safety problems on some tight mountain curves. A study is under way to identify the types of geometric deficiencies that cause the greatest problems for long, wide combinations. Different vehicle configurations will be run over test segments to examine the operational and safety problems of each type of vehicle for different traffic and roadway design situations. The study will attempt to determine the geometric and traffic conditions under which various vehicles become unsafe.

Safety Criteria for a Multitrailer Truck Highway Network

The limited information available on the operation of very long combination trucks on turnpikes and western highways indicates that these vehicles have a good safety record. It is generally assumed that this is due in large part to the restrictive controls and special permits under which these vehicles operate. In this study, the cost-effectiveness of various controls on the vehicle, driver, and general operation of longer combinations will be examined to determine which controls may be most desirable in different situations.

PLANNED RESEARCH

Effectiveness of Truck Roadway and Lane Restrictions

Background

An informal telephone survey conducted in 1980 revealed that at least eight state transportation agencies have enacted traffic regulations that restrict trucks from certain lanes on multilane highways. Some regulations absolutely prohibit trucks from the median lane or restrict them to only the rightmost two lanes. Others allow brief travel in the restricted lanes for passing. In several states, the truck restriction applies to all freeways statewide; in others, it is in effect only on specific facilities. Reasons for prohibiting trucks from one or more lanes include reducing congestion and delay, increasing capacity, and reducing stress and intimidation in passenger automobile drivers.

The present scarcity of funding for major transportation improvements and the persistently high cost of energy suggest a continued trend toward smaller, more fuel-efficient passenger vehicles and, at the same time, larger, higher capacity trucks. Both use existing transportation facilities that, of necessity, must accommodate all types of motor vehicle traffic for many years to come. Partial segregation of the largest and smallest vehicles through the application of restrictive traffic regulations may be effective in reducing congestion, delay, and driver stress on freeways. On the other hand, there may be significant operational problems associated with restrictions of this type: Are entrance and exit maneuvers hampered by the concentration of trucks in the rightmost lanes? Is the level of compliance unacceptable? Is adequate enforcement impractical?

Objective

The experiences of those states (or other transportation authorities) who have enacted truck restrictions have not been collectively studied and documented. A comprehensive investigation of the various regulations and their effects on highway operations and safety would provide a valuable reference for transportation officials considering such options.

Aspects of the truck restrictions that should be studied include

- The basis for each restriction;
- Definition of vehicles subject to restrictions;
- Methods of conveying restrictions to drivers;
- Methods and levels of enforcement;
- Levels of compliance;
- Effects on operations (speed, lane-volume distribution, interchange operation, etc.); and

• Effects on safety (accidents, driver stress, etc.).

Safety Implications of Future Configurations

Background

In the past changes in truck size and weight limits sometimes resulted in unexpected truck configurations with less than desirable performance characteristics or other problems. Changes to existing limits in the near future could create similar problems. For example, if the 80,000-lb weight limit is eliminated, trucks of the future may include unusual features such as spread tandem axles, tri-axles, dual steering axles, and self-steering belly axles. Before consideration can be given to removing the 80,000-lb cap, the safety implications of such an action must be determined. Issues to be studied include braking, stability, and steering control.

Objective

The objective would be to identify possible truck configurations that would be feasible under various possible changes in the size and weight limits and to evaluate the dynamic properties of such configurations using vehicle simulation.

Evaluation of Accidents Involving Larger Combination Trucks on Designated Federal-Aid Highways

Background

To determine whether the current system of designated highways for western twins and other federally mandated vehicles can be expanded in the future or should be reduced to eliminate unsafe segments, an accident study to evaluate the operation of larger trucks on the current system is necessary. As noted previously, a plan for the conduct of this study is currently under development.

Objective

The objective would be to determine the accident experience of large combination vehicles on the designated system of Federal-Aid Highways.

In this paper the history, goals, and status of the FHWA truck safety research program have been reviewed. Some fairly complex and important studies and issues have been briefly described.

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Findings of the Longer Combination Vehicle Study

JAMES W. MARCH

ABSTRACT

In this paper are presented findings contained in the U.S. Department of Transportation's report to Congress entitled "The Feasibility of a Nationwide Network for Longer Combination Vehicles" that was mandated by Sections 138 and 415 of the Surface Transportation Assistance Act of 1982. The purpose of this study was to examine the feasibility of establishing a network of highways for the operation of Rocky-Mountain doubles, turnpike doubles, and triple-trailer combinations. Among the factors that were considered in assessing the feasibility of a network were (a) safety, (b) vehicle performance and handling, (c) highway improvements needed to allow the safe operation of longer combinations, (d) regulations imposed by states that currently allow longer combinations, and (e) increases in productivity that might be achieved by longer combinations. Among the findings of the study were that (a) longer combinations are almost always operated under special permits issued by states or turnpike authorities; (b) longer combinations usually must meet certain performance standards, and many states require special driver certification; (c) most Interstate interchanges would have to be modified to safely accommodate turnpike doubles; (d) it is unclear where and under what conditions various longer combinations could be operated safely; and (e) pavement condition, interchange spacing and geometrics, the availability of services, bridge characteristics, lane widths, curves and grades, and traffic levels would all have to be considered when assessing the suitability of a particular highway route for longer combinations.

Sections 138 and 415 of the Surface Transportation Assistance Act (STAA) of 1982 required that the Secretary of Transportation conduct a study of the feasibility of a nationwide network for the operation of long combination vehicles (LCVs) up to 110 ft in length. For purposes of the study, it was to be assumed that the 80,000-lb weight cap would be lifted and that gross weights would be limited only by the bridge formula.

Conceivably many different vehicle configurations could have been analyzed in this study. Three general vehicle configurations that currently are used on a limited basis were chosen for analysis--the turnpike double, which consists of a tractor and two trailing units each up to 48 ft long; the Rocky-Mountain double, which consists of a tractor and two trailing units, one of which may be up to 48 ft long and the other of which is limited to about 28 ft in length; and the triple, which consists of a tractor and three trailing units each up to 28 ft in length.

Among the factors considered in assessing the overall feasibility of a network for these long combinations were

1. Safety and the importance of operating restrictions on the accident experience of existing LCV operations,
2. The geometric adequacy of various highways in rural and urban areas,
3. The costs of highway improvements necessary to accommodate LCVs,
4. The need to construct special staging areas where LCVs could assemble and disassemble adjacent to segments of a network,
5. The potential increases in productivity achievable if longer combinations were allowed to operate,
6. Damage to pavements and bridges if longer combinations were allowed to operate, and
7. The administrative constraints to establishing a national network for longer combinations.

The primary sources of information for this study were (a) reports from previous state studies of longer combinations; (b) a survey, sponsored by the Western Highway Institute, the American Trucking Associations, and the Private Truck Council, of shippers and carriers that potentially might use longer combinations; (c) a survey by the International Bridge, Tunnel and Turnpike Association of LCV operations on turnpikes; (d) a survey of the states, sponsored by AASHTO, to identify problems that states foresaw if various longer combinations were allowed on their highway systems; (e) comments to the docket established for the study; and (f) the Truck Inventory and Use Survey and the Commodity Transportation Survey conducted by the Census Bureau.

Table 1 gives the states that currently allow longer combinations to operate on part or all of the state highway system. Maximum lengths and weights and the number of miles of state highways open to each combination are also given. Rocky-Mountain doubles are currently permitted in 11 states, triples in 6 states, and turnpike doubles in 7 states. Allowable weights for these operations range from 80,000 lb in Colorado to 129,000 lb in Utah and South Dakota (turnpike doubles only). In most states the longer combinations are allowed to operate on only certain state highways, and not all configurations may be allowed to use the same highways. With the exception of California, which does not allow longer combinations, and Arizona, which allows them on only 29 mi, there is a solid block of western states that allow various longer combinations to operate on an extensive network of highways. Rocky-Mountain doubles can travel on a total of more than 60,000 mi in those states.

In addition to the states that allow longer com-

Office of Program and Policy Planning, FHWA, U.S. Department of Transportation, Nassif Building, 400 7th Street, S.W., Washington, D.C. 20590.

TABLE 1 Current Length, Weight, and Route Miles for Longer Combination Vehicles Operating on State Highways

State	Rocky-Mountain Doubles [length (ft) weight (lb) miles]	Triples [length (ft) weight (lb) miles]	Turnpike Doubles [length (ft) weight (lb) miles]
Alaska			105 109,000 475
Arizona	90 111,000 29	105 111,000 29	105 111,000 29
Colorado	95 80,000 9,218	105 80,000 9,218	105 80,000 9,218
Idaho	105 105,500 2,150	105 105,500 2,150	
Montana	95 105,000 11,405		
Nevada	105 129,000 4,872	105 129,000 4,872	105 129,000 4,872
North Dakota	110 105,500 2,170	110 105,500 2,170	110 105,500 2,170
Oregon	75 105,500 4,065	105 105,500 3,525	
South Dakota	90 105,000 7,875		110 129,000 679
Utah	90 129,000 5,000	105 129,000 690	105 129,000 690
Washington	75 105,500 6,917		
Wyoming	85 117,000 6,378		

TABLE 2 Current Length, Weight, and Route Miles for Longer Combination Vehicles Operating on Turnpikes

State	Rocky-Mountain Doubles [length (ft) weight (lb) miles]	Triples [length (ft) weight (lb) miles]	Turnpike Doubles [length (ft) weight (lb) miles]
Florida			110 138,000 272
Indiana	NA 127,400 157	NA 127,400 157	NA 127,400 157
Kansas	119 120,000 231	119 120,000 231	119 120,000 231
Massachusetts	108 127,000 132		108 127,000 132
New York	114 143,000 531		114 143,000 531
Ohio	108 127,000 241		108 127,000 241

binations to travel on state highways, there are several states in which longer combinations are allowed to travel on turnpikes. Table 2 gives the lengths and weights of longer combinations that are allowed on turnpikes as well as the number of miles on which they can travel in each state.

Whether they operate on state highways or on turnpikes, longer combinations are subject to restrictions that are not generally applied to conventional vehicles. There are three main areas of regulation--vehicle equipment, operations, and driver qualifications. The items of equipment most often subject to regulations are brakes, pintle hooks, and draw bars. Operating restrictions imposed by various states may require that LCVs (a) maintain a minimum speed, (b) maintain minimum following distances, (c) travel only in good weather, (d) travel only during off-peak periods, and (e) travel only on certain specified highways. More than half of the states have special driver requirements that may cover age, experience, training, or safety record.

In reports on the safety of longer combination vehicles, there appears to be a consensus among both researchers and highway agency officials that the various restrictions imposed on LCV operations have contributed significantly to the relatively good safety record of LCVs. Perhaps even more important than operating restrictions are the permits that carriers must have to operate longer combinations. The knowledge that permits will be revoked if carriers do not comply with operating restrictions or if they have poor safety records is a strong incentive for them to follow the strictest of safety standards. Although the relative contribution of specific restrictions cannot be determined, permits and restrictions almost certainly have improved the safety records of longer combinations currently in use.

AASHTO SURVEY

An important aspect of the longer combination vehicle study was assessing the operational characteristics of LCVs and analyzing how those characteristics would affect the safe and efficient operation of an LCV network. Officials of AASHTO were particularly concerned about the potential costs of highway improvements that might be necessary to allow LCVs to operate. In July 1984 AASHTO sent a questionnaire to members of its Subcommittee on Design requesting information on the nature and extent of potential highway problems in each state and the cost of improvements needed to safely accommodate LCVs.

Forty-six states responded to the AASHTO survey, and responses to the survey were made available to the FHWA so that relevant findings could be summarized in the report to Congress on the longer combination vehicle study. In this paper, survey responses are discussed in greater detail than was possible in the report to Congress.

SUMMARY OF SURVEY RESPONSES

One question concerned Interstate highway system interchanges that could not accommodate various types of longer combinations. Part A of that question requested information on the percentage of interchanges in rural and urban areas that could accommodate the various longer combinations. Part B requested an estimate of the percentage of deficient interchanges that could not be reconstructed for various reasons, and Parts C and D concerned the average cost of improving interchanges to safely accommodate LCVs.

The average percentages of rural and urban Interstate system interchanges that states estimated could accommodate the several longer combination vehicles are as follows:

	Rural	Urban
Turnpike double	27.5	27.2
Triple	42.1	43.7
Rocky-Mountain double	33.6	34.1

More than 40 percent of the interchanges nationwide were judged by the states to be adequate for triples, but only about one-quarter of Interstate interchanges were deemed adequate for turnpike doubles. Among the states there were substantial differences reported in the adequacy of Interstate interchanges. Many states responded that fewer than 10 percent of their interchanges were adequate for LCVs, but many others indicated that 75 percent or more of their interchanges could accommodate longer combinations without improvements. Most of these latter states are in the West where longer combinations already operate on a limited basis.

There were large variations in state estimates of required interchange improvement costs. Many states estimated costs of less than $100,000 to improve typical interchanges to accommodate LCVs, but in several states improvements were estimated to cost more than $2 million per interchange. Costs were typically at least 50 percent greater in urban areas than in rural areas. Cost variations reflect differences in the amount of additional right-of-way required, whether complete or only partial reconstruction would be necessary, whether structures would have to be reconstructed, and many other factors.

The average interchange improvement costs to accommodate each of the LCV types in rural and urban areas were

	Rural ($)	Urban ($)
Turnpike doubles	500,452	877,031
Triples	320,375	505,748
Rocky-Mountain doubles	386,759	625,797

Costs generally varied directly with the relative turning radius of each vehicle.

On the basis of estimates of the number and average cost of interchanges needing improvements, the cost of improving all inadequate interchanges was calculated. Total estimated needs in many states would be less than $5 million, but, in several others, total improvement needs would be more than $250 million. The average costs in each state to make all necessary interchange improvements to accommodate various longer combinations in rural and urban areas were estimated to be

	Rural ($ millions)	Urban ($ millions)
Turnpike doubles	50	89
Triples	32	48
Rocky-Mountain doubles	37	57

In practice, not all Interstate interchanges would have to be improved before a network for LCVs could be established; needs in each state would depend on many local factors.

Costs for states to improve every inadequate interchange that could feasibly be improved are given in Table 3. Estimated costs vary widely; costs in many states would be less than $10 million, but in several states costs would be more than $300 million. The average cost for each state to make all necessary and feasible improvements to accommodate turnpike doubles would be almost $50 million.

Although potential problems at interchange areas were of particular concern to AASHTO in its survey, information on several other topics related to the operation of LCVs was also requested in the survey. Those topics were (a) the spacing between interchanges with nearby truckstops, (b) problems on through portions of the Interstate system, and (c) the cost of improving typical at-grade intersections to accommodate LCVs.

Figure 1 shows the distance between Interstate interchanges that have truckstops and other service facilities within a mile of the interchange in various states. The average distance between interchanges with nearby service facilities is 24 mi. Only six states indicated that service facilities were spaced farther than 50 mi apart. Although the survey question stipulated that the facilities had to be capable of accommodating LCVs, interchanges and access roads might have to be improved in many instances to allow longer combinations to safely get to the service facilities.

TABLE 3 Number of States with Various Costs for All Feasible Interchange Improvements To Accommodate LCVs

Cost ($ millions)	Turnpike Doubles	Triples	Rocky-Mountain Doubles
0-9	14	20	18
10-19	7	8	8
20-49	5	6	4
50-99	4	5	7
100-199	7	1	4
200-299	5	3	2
> 300	3	2	4

Most states indicated that through portions of the Interstate system were generally safe for LCVs. Several specific problems associated with LCV operations were mentioned, however, including (a) poor performance on steep grades, (b) safety and operational difficulties on congested urban segments, (c) rest areas and weigh stations that could not accommodate LCVs, and (d) safety and operational difficulties during adverse weather. The number of states that mentioned each of these problems is

Problem	States
Steep grades	18
Weigh stations and rest areas	14
Urban congestion	14
Poor weather	4

The questionnaire did not suggest these or other potential problems to the states; the states identified the problems on their own. Other states might also have identified these problems if they had been suggested to them.

In mentioning problems that LCVs would have on steep grades, states implicitly assumed that LCVs would not be pulled by more powerful tractors than are used with conventional combinations. Without more powerful tractors, LCVs could not accelerate or climb hills as well as conventional combinations. To reduce operational problems caused by speed and performance differentials, states indicated they might have to construct additional climbing lanes and extend acceleration lanes leading on to some Interstate highways. Most states that currently allow LCVs require that they be able to maintain a minimum speed of about 20 mph. Such regulations reduce performance differentials between heavy LCVs and conventional combinations and eliminate the need for many costly improvements.

The problem of turnpike doubles and perhaps Rocky-Mountain doubles not being able to get into weigh stations and rest areas because of their large turning radii was mentioned by 14 states but would probably apply to many others as well. Reconstructing every rest area and weigh station on the Interstate system to accommodate turnpike doubles would require a significant investment and would be difficult to justify in many states if an LCV network were established. On the other hand, weighing heavy vehicles and providing drivers ample opportunities to stop for rest contribute to safe and efficient highway

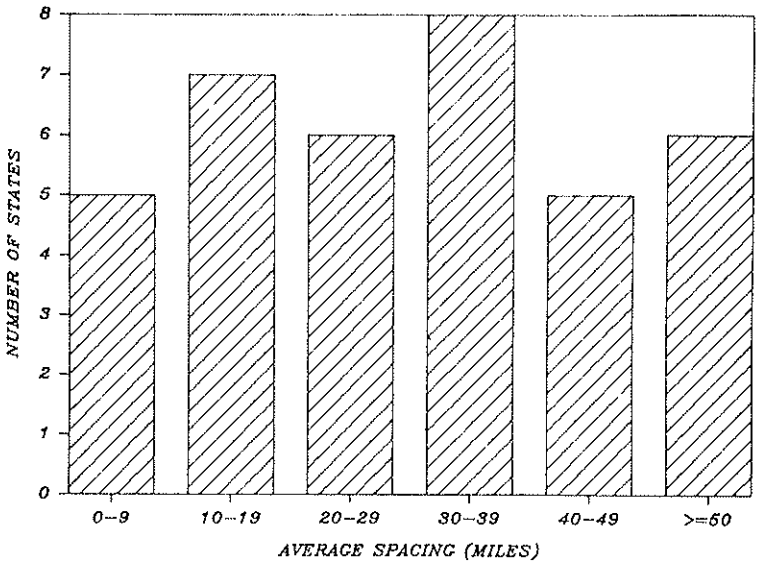

FIGURE 1 Truckstop spacing in various states.

operations. Each state would have to develop a plan for dealing with problems of access to weigh stations and rest areas.

Safety and operational problems that LCVs would have in congested urban areas were mentioned by only 14 states but could be expected in most metropolitan areas. Potential remedies would be to either prohibit some or all LCVs entirely from certain segments or to restrict their operations to hours when congestion is not severe. If LCVs were banned during peak periods, productivity would be reduced far less than if they were completely banned from a segment, and the most severe safety and operational problems would be eliminated.

Weather-related problems were mentioned by only four states but could be anticipated wherever LCVs operate. Many states that currently allow LCVs restrict their operations during adverse weather. Although there is no solid research evidence that LCVs are significantly less safe than other large combinations in adverse weather, their length, weight, and number of articulation points suggest that LCVs could have greater safety and operational problems than conventional tractor-semitrailer combinations when visibility is reduced or when pavements are slick.

Another question on the AASHTO survey concerned the costs of staging areas adjacent to the Interstate system where longer combinations could assemble and disassemble. Such staging areas are used by many turnpikes that permit LCVs because the longer vehicles are generally not allowed on state highways connecting with the turnpikes.

Forty-two states estimated costs to construct staging areas on the fringe of urban areas. The survey asked for the cost of a 2-acre staging area plus all ramps that would be necessary to operate the break-up area. The average cost estimated by the states was $717,000 and ranged from $52,000 to $3 million. As shown in Figure 2, almost half the states estimated that each staging area would cost between $500,000 and $1 million. Of the states with estimates falling outside this range, many more estimated costs of less than $500,000 than estimated costs greater

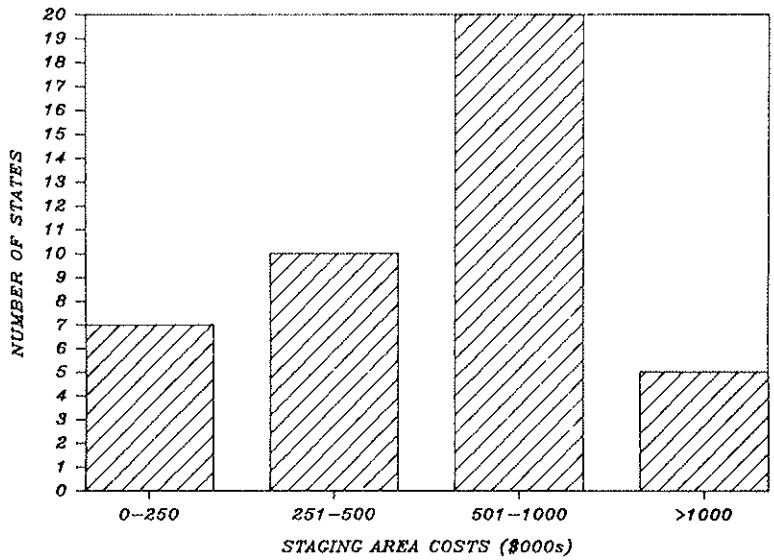

FIGURE 2 Estimated staging area costs.

than $1 million. Several states suggested that 2 acres would not be enough space for staging areas adjacent to large urban areas.

Several other questions that were included on the AASHTO survey will not be discussed in this paper. Those questions for the most part required narrative answers or detail that cannot be condensed in an overview of the survey.

In examining the results of this survey it is important to remember that the purpose was not to get precise estimates of improvement needs but rather to estimate the order of magnitude of the needs and to determine the factors that would influence costs for states in various regions of the country. Basic assumptions used in developing cost estimates varied considerably among the states, and these variations led to large differences in estimated improvement needs. The many views expressed by the states on access, staging areas, and other policy issues were perhaps just as important as their estimates of highway improvement costs.

SUMMARY OF FINDINGS

The DOT's longer combination vehicle study had several specific findings related to the operation of LCVs and the geometric design problems associated with longer combinations. Among those findings were

- Few nonfreeway street intersections could realistically be modified to accommodate turnpike doubles and, although modification could be considered for the Interstate system, most interchanges would have to be upgraded to accommodate them.
- LCVs operating at heavy weights need high-power engines to maintain speed on grades and thus avoid creating traffic operation problems or safety hazards.
- Performance and handling limitations of LCVs, as well as their higher gross weights, could create significant safety problems if LCVs are used more generally under a greater variety of road, environmental, and traffic conditions.
- Each potential LCV route should be analyzed segment by segment to determine whether LCVs could be safely operated.
- Mountainous terrain and urban areas are primary locations of geometric or capacity deficiencies on the Interstate system.
- Pavement condition, interchange spacing and geometrics, availability of services, bridge characteristics, lane widths, curves and grades, and traffic levels all must be considered when assessing the suitability of a particular highway route for inclusion in an LCV network.
- Costs of providing staging areas or, alternatively, of rebuilding interchanges to allow partial access to points off the network could be substantial. These costs are highly dependent on the access policies that are adopted.

Many issues concerning the administration and operation of a network for longer combinations could not be resolved during the course of the longer combination vehicle study. Among those unresolved issues were

- How could the federal government administer a network and ensure the enforcement of weight and operating restrictions?
- Which vehicles should be allowed on the network? The three vehicle types in use today have different operating characteristics that affect not only productivity and safety but also the improvements that would be required to accommodate those vehicles on a national network.
- What operating restrictions and permit practices, at a minimum, should be required for longer combinations nationwide?
- How extensive a network for longer combinations should be designated? Potential productivity gains would suggest a large network, but the investment required to afford longer combinations access to and from the network might prohibit a large network, especially because the necessary investment would be a front-end cost that would be incurred before any productivity gains were realized.
- How can a reasonable level of local access be assured, and will the local access policies result in large inequities among potential users of longer combinations and those who must pay the cost of special facilities for those vehicles?

Many factors other than geometric design were considered in the DOT longer combination vehicle study, but geometric design problems are clearly among the most important considerations in decisions regarding the operation of LCVs on the nation's highways.