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Contents

Transportation Safety Index Applicable to All Modes

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There are a number of basic classes of transportation safety indices, based on time, event, activity, and population. Each has special utility for certain modes, but they lack a common basis and general applicability. This paper presents a failure index that can be applied to all modes and can allow cross-modal and intramodal comparisons. By using available data, the failure index was calibrated for the case of passenger fatalities. Among the results were that air was found to be less safe than intercity bus or rail for trips of less than 2400 km (1500 miles) and that, for short trips, air is generally less safe than the automobile. The failure index was also used to show how two operators that have the same basic safety performance can appear to differ because of composition of routes (i.e., trip-length distribution).

During the years 1965 to 1975, almost 625 000 people were killed in accidents related to transportation. A breakdown of fatalities by mode for the period shows that about 93 percent of all transportation-related fatalities were highway related. A reasonable conclusion would then appear to be that the improvement of the overall transportation safety record requires that the highway mode receive priority treatment.

Nevertheless, such percentages have little meaning when presented without an accompanying measure of modal exposure (i.e., passenger or vehicle kilometers). For illustration purposes, the trend of fatality rates per passenger kilometer for the period 1955-1974 is shown in Figure 1.

From an analysis of this figure, one would deduce that the most dangerous transportation mode is not highway but general aviation. However, analyses based on various accident-exposure rates may lead to different results, posing a valid question regarding the basis for a comparative ranking of the safety performance of the various transportation modes.

The problem of the evaluation of relative safety is further complicated by the definition of accident severity. The frequency of various types of fatalities and injuries also differs for each mode. Thus, the average severity observed in accidents by each transportation mode is different: Simply counting the number of fatalities and injuries is not sufficient to analyze a transportation-system safety performance. However, to do this systematically on a common basis and to allow for inclusion of all severity levels is a beginning. This paper presents a failure index by which the safety of the various modes can be expressed on such a common basis and illustrates it for the fatalities-only case (due to lack of suitable data for other severities).

CLASSES OF TRANSPORTATION SAFETY INDICES

Safety performance of transportation systems is generally expressed as accident frequencies.

Establishing satisfactory numerical indices to measure the safety of transportation systems requires the use of proper exposure measures (denominators such as distance, time, and number of passengers).

There are four basic types of safety exposure measures (denominators) used in the transportation field:

1. Time-based denominators,
2. Event-based denominators,
3. Activity-based denominators, and
4. Population-based denominators.

Cheaney (1) has given a thorough discussion of safety indices based on these four exposure measures. He investigated modal authorities' motivation for using a particular index and points out that their choice is influenced by the desire to present their facility in the best possible light. Some of the exposure measures frequently used in the field of transportation are summarized below.

Base	Exposure Measure
Time	Vehicle hours, passenger hours, system year
Event	Number of operations, number of takeoffs and landings, number of entries to and exits from harbors
Activity	Vehicle kilometers, passenger kilometers
Population	Number of registered vehicles, number of licensed drivers, residential population, number of vehicles in operation, number of passengers

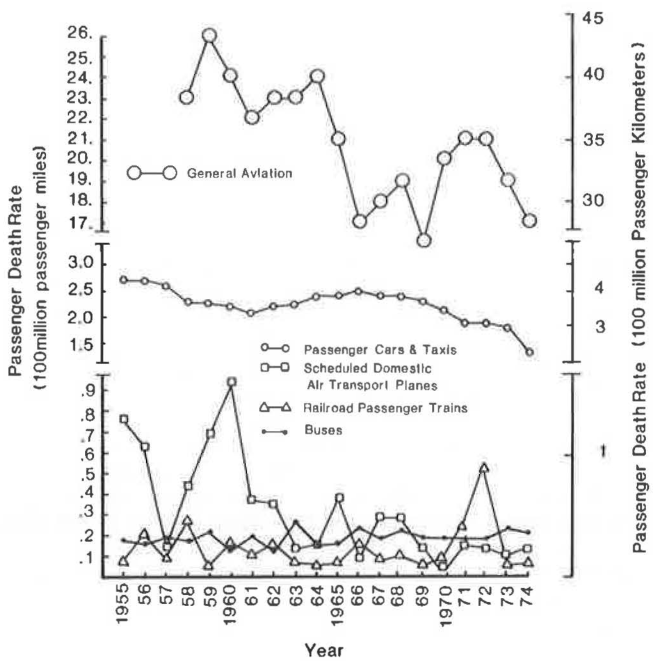
Time-Based Exposure Measures

These measures are important to both system operators and vehicle manufacturers for analyzing vehicle reliability. However, not all time spent in transportation-related activities is of equal risk, which leads to misleading conclusions regarding the safety of a system. Accident rates based on duration of operation or travel tend to neglect those accidents that occur during relatively short time periods, such as landings and takeoffs or entries to or exits from harbors. Aside from this, there are inconsistencies in comparing accident rates within a particular transportation mode. Annual changes in average speed are not directly accounted for and, thus, misinterpretations may arise in the meaning of accident rates. A faster vehicle may have an accident rate that is higher than that of a slower vehicle, without experiencing any increase in accidents, simply because the higher speeds of the faster vehicle will result in fewer hours of exposure, which in turn will reduce the accident rate.

Event-Based Exposure Measures

Not all of the time spent by an operator or a passenger in a given mode is of equal safety or danger. In the case of air transportation, the majority of accidents have more to do with the fact that a flight is made than with its duration or length. Thus, at least to a close approximation, the number of flights can be considered as a fair measure of exposure to accidents.

Figure 1. Comparison of fatalities by mode: death rates per passenger unit distance.



A rate such as "serious accidents per x million flights" is a reasonable technical indicator of safety achievement. This is only so, however, when the distribution of trip distance and landings and takeoffs per flight is invariant and, in actuality, this distribution has changed throughout the years.

Regarding the water mode, in his investigation of differences between U.S. and foreign vessel casualty rates, Tennenbaum (2) has pointed out that vessel years is not an appropriate exposure measure. Because most casualties occur in harbors and the approaches to harbors, he considers the number of harbor entries and exits to be a more satisfactory measure of exposure.

In rail transit, statistics (3) show that 78 percent of all accidents occur mainly in stations and in the immediate vicinity of car doors, indicating that the number of passengers who use the system, rather than the total travel time, may have the more significant impact on the total safety.

Activity-Based Exposure Measures

The most important activity-based exposure measure is annual kilometers of travel per vehicle. This type of measure is widely used for the analysis of both motor-vehicle and aircraft safety performance. Passenger kilometers as an exposure measure seems generally applicable to the evaluation of passenger safety.

This measure of safety becomes distorted when it is applied to aircraft or ships, where the probability of being involved in an accident is greater during takeoff and landing or when entering or exiting a harbor than during normal cruise.

Another major disadvantage of this exposure measure is its inability to account for differences in capacity and loading factors among the various modes. Also, in practice, problems exist in obtaining accurate data on the distances traveled by private automobiles and recreational boats.

Population-Based Exposure Measures

One exposure measure applicable to all modes of transportation is the number of deaths per million of population. Such statistics appear in demographic summaries and are used to compare the relative contributions of various factors (such as disease and accidents) to the population mortality rate.

Large variations in vehicle use among the different modes and on annual bases blur the real issue: human and property exposure to hazards. Modal comparisons are meaningless when there are significant differences in vehicle use and vehicle size. Even within the same mode, comparisons based on this measure may be misleading. Consider, for example, the taxi versus the private automobile. The taxi is exposed to hazards for longer time periods and experiences more accidents per number of licensed drivers or of registered vehicles than does the private automobile. This, however, does not mean that using a taxi is more dangerous than using a private automobile.

A NEWLY DEVELOPED MEASURE: THE FAILURE INDEX

The above analysis leads to the conclusion that only two of the four measures of exposure presented—time and distance—may be useful for the accurate evaluation of the safety of a system.

The most important question is, Is the probability of failure best expressed in terms of an exposure-hour or service-hour figure or in terms of distance traveled?

From the point of view of the transportation-service consumer, the issue is clear: The consumer must travel a distance X to arrive at a specified and desired destination. Therefore, the final issue is total risk or safety while traversing the specified distance.

However, this does not address the basic issues of device failure (e.g., motor or pilot) or exposure (hours in the air or time for conflicts) that are of interest to those doing risk modeling of a specific system. Indeed, the critical issue to some is, Is the risk more dependent on time or distance in this context?

If one turns to the macroscopic aspects and asks whether time or distance is more appropriate on that level, a review of the data renders the point moot. Rather than asking about basic engineering hazards on this level, we are simply asking whether time or distance has better explicatory value. The literature shows that time and distance are rather well correlated on a macroscopic level; however, the use of macroscopic data must be viewed with caution.

To account for the various problems noted to this point, this paper introduces and justifies a novel failure index (FI) that can be used as a measure of safety.

One of the central concepts in the development of such an FI is the identification of the functional differences during a trip throughout its various environments where an environment is defined and classified as a function of a specific segment of a trip. For instance, a flight can be divided into three functional environments (see Figure 2):

1. The entire takeoff phase, including terminal activities, taxiing, and actual takeoff to the point of being headed to the en-route controllers;
2. The normal cruise, from the vicinity of its origin to the vicinity of its destination; and
3. The landing phase, including any required holding patterns and related clearances.

Figure 2. Division of a flight into three functional environments.

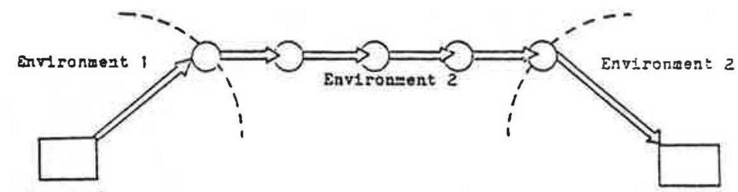
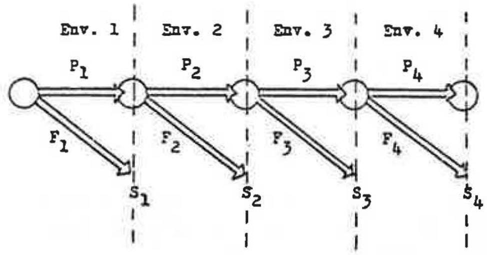


Figure 3. Generalized mathematical presentation of a trip.



It is important to recognize that each environment can be distinguished in the following ways:

1. Its physical description;
2. Its accident probability or risk, due to changes in its function or complexity, compared with that of the previous trip phase; and
3. Its severity, or the consequences of an accident, in terms of differing likelihoods of death, serious injury, and other effects on each of the environments.

For instance, 75 percent of all air-carrier accidents involving passenger fatalities occur during takeoff or landing. However, this does not tell the complete story: 90 percent of all passenger fatalities occur in those takeoff and landing accidents that involve fatalities. Therefore, one must distinguish between accidents involving fatalities (which are commonly called fatal accidents) and the actual distribution of the numbers of fatalities.

There is some value in standardizing the extent of those environments that could be rather open-ended or vary over wide-ranging values.

Among other effects, this definition of individual, consecutive environments that have the same time span allows for various probabilities of accident or severity due to such factors as weather or traveling over water.

It is also possible to aggregate environments when they can be paired logically, such as a takeoff and a landing.

Generalizing on this concept, Figure 3 illustrates a case of traveling through several environments, where the notation used is defined below:

Notation	Definition
P_i	Probability of successfully (i.e., safely) traversing environment i
F_i	Probability of unsuccessfully traversing environment i (i.e., an incident occurs)
S_i	Expected, average, or weighted severity given that an incident worthy of note occurs in environment i

Thus, $F_i + P_i = 1$. In addition, Figure 3 assumes that an incident occurring in any environment terminates the trip. The probability of successfully completing the trip is then

$$\prod_{i=1}^{N} P_i \tag{1}$$

where N = total number of environments.

Because different failure paths can occur, each with its own possible resulting severity, an equation that expresses the expected severity is not a simple one. In the illustration shown in Figure 3, the probability of an incident occurring in environment 3 is $(P_1 \times P_2 \times F_3)$ because environments 1 and 2 must be traversed successfully in order to reach environment 3.

Based on this approach, an FI can be formulated in the following way:

$$FI = \sum_{i=1}^{N} S_i F_i \left(\prod_{k=1}^{i-1} P_k \right) \tag{2}$$

That is, the FI is the expected severity (the weighted average of all the possible severities).

For the simple case in which the severity is equal across all environments ($S_i = S$ for any i), Equation 2 can be rewritten as

$$FI = S\left(1 - \prod_{i=1}^{N} P_i\right) \tag{3}$$

which is based on expressing the probability of failure as one minus the probability of success.

Table 1 defines the environments as they can currently be divided for the data commonly available.

The use of Equation 3 is justified only if the severity does not vary from environment to environment or if the only accidents being considered are those that have a certain specified severity. Thus, this equation can be used in only two cases: (a) where any incidents that naturally occur happen to have comparable severities and (b) when only a certain class of accidents or incidents are considered, such that they all have comparable severities. In the latter case, the probabilities used would be those specific to that class of incident.

In practice, there is no commonly accepted, well-defined severity scale. At present, it is generally not possible to identify S_is for different environments. Therefore, the illustrations of this concept are based on the severity value of fatality only.

For each environment, it is necessary to identify a P_i of successfully traversing the environment. It is useful to define this in the form of

$$P_i = \exp(-\alpha_i) \tag{4}$$

For the case of the air-transport illustration, one might define

$$P_1 = \exp(-\alpha) \tag{4a}$$

$$P_2 = \exp(-\beta) \tag{4b}$$

$$P_3 = \exp(-\gamma) \tag{4c}$$

Table 1. Definition of environments for various transportation modes.

Mode(s)	Environments	Factors Affecting Safety
Air carrier and general aviation	Takeoff and landing and normal cruise	Weather, sophistication of air traffic control system, character of area overflown, total number of operations, type of airport, and maintenance
Highway	Rural freeway, rural arterial, urban freeway, urban arterial, and local street	Weather, traffic volume, lighting, geometrics, speed, and maintenance
Rail	Yard switching, normal cruise, station operation, and railroad crossings	Weather, number of passengers, geometrics, speed, conflicts with other modes, and maintenance
Rail rapid transit	Normal cruise, station operation, and yard operation	Weather, number of passengers, speed, and maintenance
Vessel	Harbor operation, ocean cruise, lake cruise, river cruise, and docking operation	Weather, traffic volume, harbor location and configuration, speed, and maintenance
Recreational boat	Ocean cruise, lake cruise, river cruise, stream cruise, and docking operation	Weather, traffic volume, speed, and type of craft

where the environments 1, 2, and 3 are as defined in Figure 2 [i.e., takeoff, normal cruise of 80 km (50 miles) standard distance, and landing] and α stands for environment 1, β stands for environment 2, and γ stands for environment 3. In the course of normal flight, N_1 takeoffs are encountered; N_2 cruise segments, each 80 km long, are encountered; and N_3 landings are encountered. Of course, $N_3 = N_1$. Thus, by using Equation 3,

$$FI = S\left(1 - \prod_{k=1}^{N} P_k\right)$$
$$= S\,[1 - \exp(-N_1\alpha)\exp(-N_2\beta)\exp(-N_3\gamma)] \qquad (5)$$

where the product of the exponential terms gives

$$FI = S\left\{1 - \exp[-(N_1\alpha + N_2\beta + N_3\gamma)]\right\} \qquad (6)$$

The use of Equation 3 is justified only if the severity does not vary from environment to environment or if the only accidents being considered are those that have a certain specified severity. For example, we may wish to consider fatality as the only severity worthy of note. In that case, the probability terms α, β, and γ (or, more generally, α_i) would be those associated with a fatal event.

The exponential term in Equation 6 can be expressed as a power series and approximated for small θ as

$$\exp(-\theta) = \sum_{i=0}^{\infty} (-)^i\theta^i/i! \simeq 1 - \theta \qquad (7)$$

Because any accident is a relatively rare event, the values of θ are indeed small. Thus, Equation 6 can be approximated as

$$FI \simeq = S[N_1(\alpha + \gamma) + N_2\beta] \qquad (N_1 = N_3) \qquad (8)$$

which is a convenient linear form. A similar useful approximation can be derived for each mode; generally resulting in

$$FI \simeq A + B(\text{distance}) \qquad (9)$$

where A and B are constants related to the α_i.

CALIBRATION OF THE FAILURE INDEX

Because of the lack of a uniform severity ranking for the various environments and modes, attention in this section is restricted to fatalities. That is, the only severity considered explicitly will be a totality and probabilities will be developed for such events only.

This section describes

1. The development of the coefficients (α_is) involved in the probability P_i in the failure index and
2. The comparison of modal safety as a function of trip distance.

For each mode, the data that are typically available include (a) the number of fatalities, (b) descriptive statistics such as vehicle kilometers of travel and number of terminal operations, and (c) trip characteristics such as average distance traveled.

It is important to recognize that most statistics [which are expressed in terms of fatalities over some operating statistic (e.g., millions of vehicle kilometers)], are in fact fatalities per typical trip. This is shown graphically in Figure 4, as a single point where the two coordinates are "fatalities" and "typical trip". A typical trip by U.S. air carrier might be represented as one takeoff, one landing, and 960 km (600 miles) of normal cruise. For a local bus, it might typically be represented by one boarding, one alighting, and 16 km (10 miles) of onboard travel. The conventional statistics are in some sense a weighted average of the FI.

To the maximum extent possible, existing data were used to estimate the α_is for the various environments. The results are tabulated below (1 km = 0.6 mile).

Environment	$\alpha_1 + \alpha_3$	α_2 (fatalities/ 15 000 km)
U.S. air carrier	0.80×10^{-6}	0.0034×10^{-4}
Local roads and streets (rural)		2.31×10^{-4}
Federal-aid primary roads (rural)		1.47×10^{-4}
Local roads and streets (urban)		0.72×10^{-4}
Federal-aid primary roads (urban)		0.60×10^{-4}
Passenger automobiles and taxis		0.96×10^{-4}
Passenger automobiles on turnpikes		0.65×10^{-4}
Buses (city)		0.20×10^{-4}
Intercity buses		0.06×10^{-4}
Railroad passenger trains		0.07×10^{-4}

Where the data to obtain the α_is for the major phases were not available—takeoff and landing versus normal cruise—estimates were made according to the following logic:

1. Terminal activities (boarding and alighting) for local buses are assigned an A-value of 0.1 $\overline{FI}$; this is

somewhat arbitrary but necessary because of lack of data.

2. Personal transportation by automobile is assigned an A-value of 0.0; a driver and his or her automobile immediately enters the normal-cruise activity and has negligible landing-takeoff risk.

3. Rail rapid transit and passenger railroad are assigned A-values of 0.1 $\overline{FI}$ on a basis similar to local buses.

4. Intercity buses are assigned an A-value of 0.0, not because of negligible terminal activity, but rather because the nature of intercity buses is that they have some terminal risk, travel x kilometers on the road, encounter another terminal situation (e.g., a rest stop), continue for another x kilometers on the road, enter another terminal situation, and so on. The assumption of A = 0.0 is the approximation to this situation.

5. Airline data are available from which to estimate A and B independently, and this is done. Because A is so significant, the graphical presentation will show distinctly different results, depending on the number of stops during a journey.

Figure 4. Failure index for a typical trip.

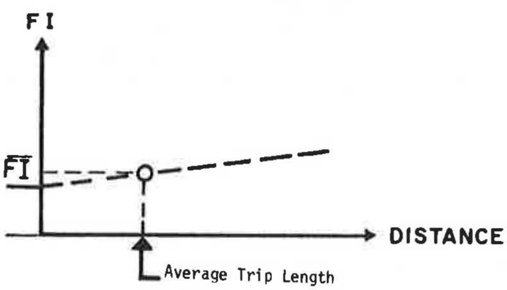


Figure 5. Detailed safety comparison among modes.

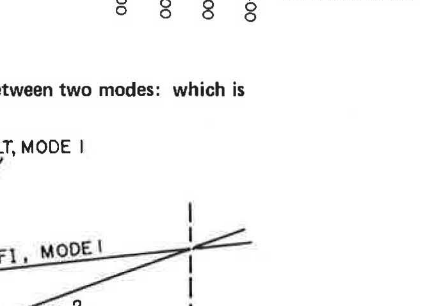
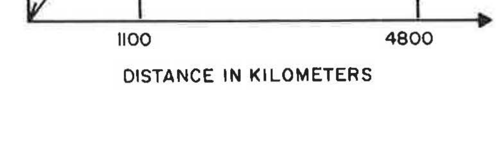


Figure 6. Comparison between two modes: which is safer?



KEY RESULTS OF FAILURE-INDEX FORMULATION

Figure 5 summarizes the key results for the failure index, given consideration of only fatal accidents from the point of view of the individual consumer—the passenger. With the exception of the air carrier estimate, these results are based on 1974 data; because of the inherent variability of that estimate (due to the rare-event nature of such accidents), the average for 1974-1976 is used. The use of only 1974 data would have led to an estimate almost twice that indicated.

The use of the failure index, as defined in this paper, shows that, as concerns the passenger interest and solely from the point of view of passenger fatality,

1. Travel by passenger automobile tends to be the worst choice of all modes, except when the only available modes are automobile or air—in this case, private automobile has a clear and distinct advantage for trips less than 240 km (150 miles) long.

2. For private automobile travel, as is commonly accepted, the preferred order of roadway systems with regard to safety is (a) federal-aid primary (urban), (b) turnpikes (toll roads), (c) local roads and streets (urban), (d) federal-aid primary (rural), and (e) local roads and streets (rural).

3. Intercity bus travel appears to have a slight advantage over passenger rail travel.

4. Both intercity bus and passenger rail travel have a distinct advantage over air carrier travel for all trips of less than 2400 km (1500 miles) and, if the flight makes even one stop en route, will have the advantage up to approximately 4800 km (3000 miles) [thus, air carrier travel has a safety advantage (as regards fatalities) within the continental United States only for nonstop trips of more than 2400 km, although indirectness of route on the intercity bus may somewhat offset this].

INSIGHTS INTO COMPARATIVE ANALYSIS OF MODES

Some of the above conclusions are quite different from what conventional statistics would lead one to believe. Safety statistics are sometimes used in public relations for a variety of modes, and they are sometimes abused when a mode is comparing its records with those of other modes, for public relations or other purposes. Figure 6 illustrates an interesting condition. Based on the failure index as described in this paper, mode 2 has a clear advantage over mode 1 up to a distance of approximately 4800 km. However, the average trip distance on the two modes may be distinctly different. Indeed, when one considers the arithmetic average of all trips by intercity bus versus the arithmetic average of all trips by air, this is not at all unexpected. Thus, conventional statistics would indicate that mode 1 is the safer of the two because its simple slope (through the origin) is lower (line "alt, mode 1" versus line "alt, mode 2"). That is, its number of fatalities per unit distance for the average trip distribution is smaller.

Even within a given mode, such comparisons can be made. Consider three airlines—A, B, and C—which have identical failure indices. However, for the purposes of simplicity, assume that airline A specializes in 800-km (500-mile) trips only, airline B specializes in 1600-km (1000-mile) trips only, and airline C specializes in 2400-km trips only (see Figure 7). Because all three have the same failure index, one could say that they have the same safety capability. However, one expects that, in terms of fatalities per unit distance, airline C will be shown to be the best.

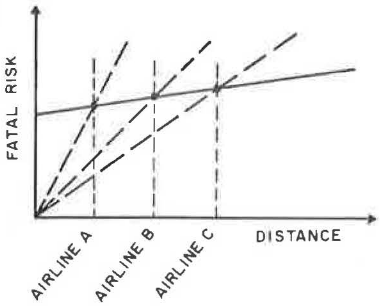


Figure 7. Comparisons among three airlines that have identical failure indices.

Clearly, then, when any two carriers within an individual mode are compared, one must consider not their basic or even conventional statistics, but rather their α_is as defined in the failure-index concept.

CONCLUSIONS

The failure index can be used to make clear and unambiguous modal cost comparisons. The actual case described in this paper was that of the individual passenger and fatalities only. If sufficient data were available, it would be possible to develop comparable graphs to illustrate fatality risks from the point of view of the operator and of society. These graphs would include deaths of crew members, bystanders, and others. If severity scales were assigned in some systematic way, one could then use future data to ascertain the relative merits of the several modes from the point of view of total severity and not just of fatality. One could then address the critical questions of allocating funds for improvement to the various modes on the basis of equalizing the risk of the individual passenger or assigning priorities to the modes on the basis of minimizing total severity. In actual fact, one would then deal not just with modal priorities but also with the priorities of treatment among all possibilities, irrespective of mode.

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**J. Byun was with the Transportation Training and Research Center, Polytechnic Institute of New York, at the time this research was performed.*

Changing Baseline in Transportation Safety: An Assessment of Some Key Factors

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Transportation accident experience depends on many factors, some very subtle. Although many countermeasures are introduced to enhance safety, it is also true that the accident experience can vary systematically over time even if no countermeasures are introduced. This variation in the baseline is investigated in this paper. How can the average condition vary if no new changes are introduced? Simply put, there are variations built into the total system—operators, roadway, and vehicles. Four major forces are considered in this paper: the changing age distribution of the automobile-driving population; the changing urban-rural balance; changes in modal trip lengths or vehicle types; and modal shifts induced by transportation system management actions. Each of these is found to have a significant effect (5-10 percent on the baseline), and other such forces can also exist. Clearly, it is not valid to explicitly or implicitly assume that the baseline does not change.

The development of a failure index (1) that can serve as a basis for comparing the relative safety of different modes is described in the previous paper in this Record. This paper uses that failure index and general indicators to study the apparent safety-record improvements that are induced by societal and other forces.

A number of forces are working in our society that will alter an apparent safety record or aggregate safety statistics or the true safety history. It is important to identify the possible factors that can change the safety experience over time in predetermined ways without the introduction of new forces. That is, factors that cause the reference level or baseline to change without other modification of the system should be identified.

In the situations of interest here, even if no new improvements are made after a given date, safety records will still change thereafter. If a number of ineffective safety measures are undertaken in the future, the baseline will still change. If one does not appreciate that the baseline is changing, one will erroneously conclude that the ineffective actions are useful accident countermeasures.

To consider one example, note that recent automobile models are safer than those of 10 and even 5 years ago. If there were no further improvements in new models from the present time, the future fleet would still become safer as new automobiles replaced those 5-10 years old. This would, of necessity, have an impact on the safety measure of actual safety; the baseline would change.

Because the overall safety of a system depends on a number of factors, the apparent safety level will vary, depending on how these factors themselves vary. Thus, it is entirely possible for safety performance to change over time, for no other reason than that underlying influences are themselves changing. That is, the baseline changes.

When assessing the effect of any projected transportation improvement, it is important to factor out expected changes due to other, identifiable effects. This paper discusses several case studies of such effects:

1. Overall societal forces and trends—population aging and urban-rural split,
2. Changes of mode or within-mode characteristics, and
3. Effects of a transportation system management action.

These case studies are not exhaustive, but they are representative of the effects that exist, some of which are subtle and some of which are clear.

FAILURE INDEX

A trip on any mode can be considered to be a set of trips through distinct, identifiable environments that are linked together as represented in Figure 3 of the previous paper in this Record. Each environment (say, environment 2) can be traversed successfully with probability P_i or unsuccessfully with probability ($F_i = 1 - P_i$). In the latter case, a fatality or injury of severity S_i occurs.

A failure index (FI) can be constructed (2) as

$$FI = \sum_{i=1}^{N} S_i F_i \left(\prod_{k=1}^{i=1} P_k \right) \qquad (1)$$

By considering the available data, the case of fatality-only was calibrated for the failure index as viewed by the individual passenger. Because the S_is are all equal to some common value S (which can be set at unity), Equation 1 can be rewritten as

$$FI = S \left\{ 1 - \prod_{k=1}^{N} P_k \right\} \qquad (1a)$$

The probability P_k can be expressed as $\exp(-\beta_k)$ and then approximated by $(1 - \beta_k)$. Further manipulation gives the form

$$FI \cong A + B(\text{distance}) \qquad (2)$$

for most modes, where the constant A is associated with the trip terminals and has units of probability of fatality per trip and the constant B is associated with the en-route portion of the trip and has units of probability of fatality per unit distance.

The A- and B-values estimated for the various modes are summarized below, where B has units of fatalities per 15 000 km (24 000 miles); the FIs plotted from these estimates are shown in Figure 2 of the previous paper in this Record.

Mode	A	B
U.S. air carrier	0.80×10^{-6}	0.0034×10^{-4}
Local roads and streets (rural)		2.31×10^{-4}
Federal-aid primary roads (rural)		1.47×10^{-4}
Local roads and streets (urban)		0.72×10^{-4}
Federal-aid primary roads (urban)		0.60×10^{-4}
Passenger automobiles and taxis		0.96×10^{-4}
Passenger automobiles on turnpikes		0.65×10^{-4}
Buses (city)		0.20×10^{-4}
Intercity buses		0.06×10^{-4}
Railroad passenger trains		0.07×10^{-4}

Clearly, for certain distances, some modes have distinct safety advantages. These results are sometimes not consistent with conventional understandings of relative safety advantage, particularly of air. For instance, air is less safe than rail or intercity bus for trips of less than 2400 km (1500 miles), whereas it is often said to be the safest mode.

CASE 1: POPULATION AGING

It is well known that the population is aging and that coming decades will witness a steady growth in the relative percentage of older persons. At the same time, there is a known difference in relative frequency of accidents by age group. Figure 1, which is based on National Safety Council data (2), shows this relative frequency. There was a significant stability in this pattern over the 7-year period investigated (1969-1975).

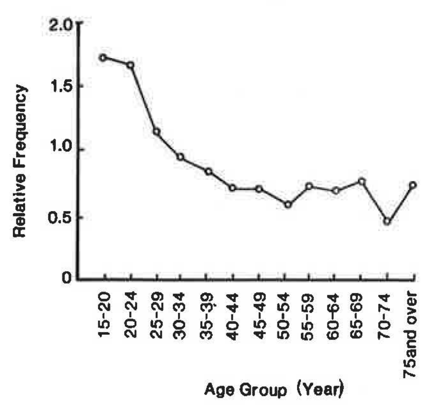
The ratio (R) between the relative motor-vehicle-accident frequency for year k, due only to population aging, and that for the base year 0, can be expressed as

$$R = AR^k / AR^0 \qquad (3)$$

where

AR^k = relative accident frequency for year k and
AR^0 = relative accident frequency for year 0.

Figure 1. Relative frequency of accidents in age group: involvement based on overall driving population.



The relative accident frequency for year k due to demographic change is

$$AR^k = \Sigma P_i^k Q_i^k a_i / \Sigma P_i^k Q_i^k \quad (4)$$

where

P_i^k = percentage of population in age group i in year k,
Q_i^k = percentage of population licensed in age group i in year k, and
a_i = relative motor-vehicle-accident frequency, due to driver age group i.

The percentage of population in each age group has been estimated by the U.S. Bureau of the Census for future years.

The percentage of the population within each age group that is licensed was estimated according to two scenarios: (a) the percentage in each group remains unchanged and (b) no licensed drivers give up their licenses as they age, so that the percentage in the older groups grows. The first scenario is conservative in that those who will be 60- to 65-year-old drivers in the late 1990s are now 40-45 years old and, to a greater extent than persons currently 60-65 years old, grew up with the automobile. It is more likely that they (and other age groups) will retain their licenses, thereby increasing the representation of older drivers in the licensed population.

Figure 2 compares the relative accident frequency AR^k with the 1975 base condition. Under either scenario, it is reasonable to expect a 10 percent decrease in accident frequency by the year 2000 simply because the population will be older.

CASE 2: URBAN-RURAL SPLIT

Two phenomena are worthy of note: (a) the traffic density in rural areas is increasing and (b) the ratio of urban vehicle travel to rural vehicle travel is increasing. Thus, more of the total vehicle travel is occurring in urban areas. At the same time, however, the amount of vehicle travel per kilometer of highway in rural areas is also increasing. These two forces will combine to affect the future accident history.

State-based data separated by urban and rural fatal-accident rates are available from the Federal Highway Administration (3) as are estimates of vehicle kilometers and kilometers of highway.

Traffic densities (vehicle kilometers per kilometer of highway) were computed and subjected to regression analysis. Figure 3 shows the results of the regression analysis—rural accident rates are statistically related to traffic density as defined herein, whereas no such relationship exists for urban accident rates.

$$f = 2.95 - 1.59\, d_k \quad \text{(rural)} \quad (5a)$$

and

$$f = 1.4 \quad \text{(urban)} \quad (5b)$$

where f = fatality rate (fatalities per 100 million vehicle kilometers) and d = density (million vehicle kilometers per highway kilometer). A level of significance of α = 0.05 was used.

Comparison of Figures 3a and 3b shows that the urban rate is significantly lower than the rural rate and that the two data clusters form a continuum; generally d ≤ 0.40 for rural traffic and generally d ≥ 0.40 for urban traffic.

Estimates for future urban and rural vehicle travel are given in Table 1 (4). The relative fatal risk due to traffic-density changes for the year k (R_k), based on the year 0, will be

$$R_k = \{[(M_k^r \times f_k^r) + (M_k^u \times f_k^u)]/(M_k^r + M_k^u)\} \div [(M_0^r \times f_0^r) + (M_0^u \times f_0^u)]/(M_0^r + M_0^u) \quad (6)$$

where

M_k^r and M_k^u = rural and urban, respectively, vehicle kilometers in year k;
M_0^r and M_0^u = rural and urban, respectively, vehicle kilometers in year 0;

Figure 2. Relative frequency of accidents in age group: involvement based on two scenarios of overall driving population.

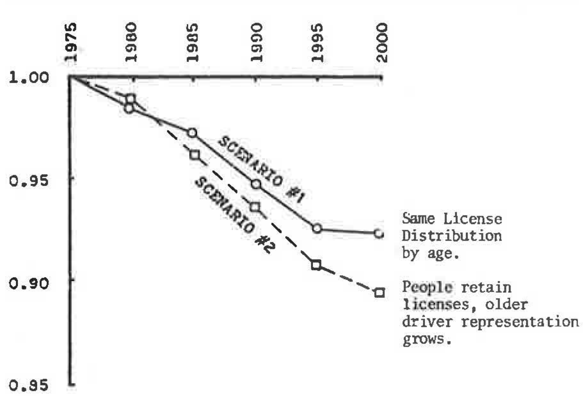


Figure 3. Relationship between fatal-accident rate and traffic density: (a) rural highways and (b) urban highways.

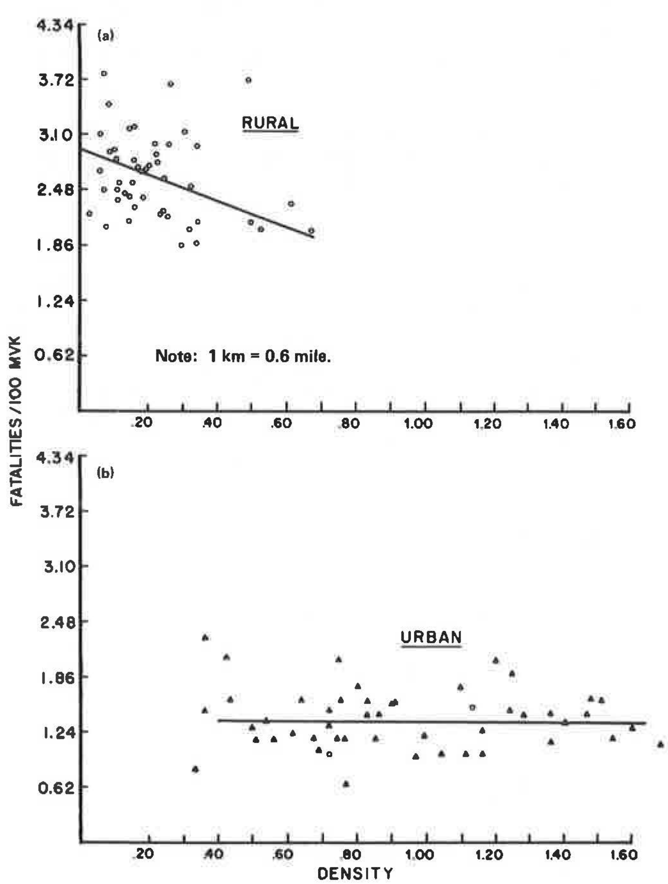


Table 1. Projections of highway travel.

Year	Automobile Travel (km billions) Rural Interstates	Main Rural Roads	Local Rural Roads	Urban Interstates	Urban Highways and Streets	Truck Travel (km billions)	Bus Travel (km billions)
1975	140.305	271.005	315.677	146.28	757.130	69.603	8.152
1976	146.129	267.413	314.125	154.602	773.452	71.568	8.278
1977	152.659	265.562	314.201	162.893	793.695	72.700	8.455
1978	161.105	267.168	318.232	175.424	826.592	77.320	8.744
1979	170.089	269.544	323.026	188.206	862.502	81.137	9.06
1980	179.274	272.057	327.853	201.519	899.601	85.653	9.401
1981	184.691	268.873	325.65	210.953	918.458	90.209	9.543
1982	191.492	267.842	325.900	222.279	944.887	94.634	9.762
1983	197.255	265.429	324.331	232.717	966.786	99.015	9.932
1984	201.443	263.160	322.798	243.49	989.416	103.342	10.108
1985	209.159	261.340	321.702	254.95	1014.034	107.644	10.305
1986	215.925	260.323	321.497	267.539	1042.251	111.939	10.537
1987	214.927	259.578	321.544	280.62	1072.18	116.222	10.785
1988	230.048	258.770	321.441	294.50	1102.549	120.657	11.036
1989	236.683	257.381	320.584	307.97	1131.078	125.039	11.268
1990	242.118	254.663	317.92	320.204	1154.15	129.307	11.445

Note: 1 km = 0.6 mile.

f_k^r and f_k^u = fatality rate on rural and urban highways, respectively, in year k; and
f_0^r and f_0^u = fatality rate on rural and urban highways, respectively, in year 0.

The M_i^j-values can be taken from Table 1, and the f_i^j-values can be determined by using Equation 5. It was assumed that rural highway kilometers will be approximately constant over the period of interest, so that rural d will increase in the same proportion as rural vehicle kilometers.

The relative change in fatal risk over approximately two decades is summarized below.

Year	Relative Fatal Risk	Year	Relative Fatal Risk
1974	1.000	1983	0.960
1975	0.996	1984	0.956
1976	0.991	1985	0.952
1977	0.987	1986	0.948
1978	0.982	1987	0.944
1979	0.976	1988	0.940
1980	0.971	1989	0.935
1981	0.967	1990	0.933
1982	0.964		

There is a decrease of 7 percent, due simply to the changing urban-rural mix and the growing rural traffic density.

CASE 3: CHANGES OF MODE OR WITHIN-MODE CHARACTERISTICS

Two subcases are worthy of special note: the increased flight distances of airlines and the changing vehicle mix on highways.

Commercial Flight Distances

The failure index for nonstop flights by commercial air carriers can be determined by using Equation 2a

$$FI = (0.80 + 0.34 \text{ distance}) \times 10^{-6} \tag{2a}$$

where distance is measured in units of 15 000 km (24 000 miles). The constant 0.80 increases by 0.80 for each additional stop, because of the increased risk associated with landings and takeoffs.

Consider an airline that has an increasing average distance and a constant passenger volume: The FI will increase, but by much less than the passenger kilometers of travel. Thus, such conventional statistics as fatalities per million passenger (or vehicle) kilometers will decrease; indeed, they will change almost directly inversely to average distance. Nonetheless, the true safety of any individual trip of a given distance (as properly measured by the A- and B-values) will not have changed at all.

It should be noted that this case relates to an apparent change in the underlying safety of the mode and is induced by the use of a limited measure (e.g., fatalities per million passenger kilometers). The cases above related to actual changes that would take place, the true causes of which might be overlooked. Both types, however, illustrate the possibility that some extraneous and irrelevant measure might accidently claim credit—real or imagined—for changes that are observed in a simple before-and-after study.

Changing Vehicle Mix

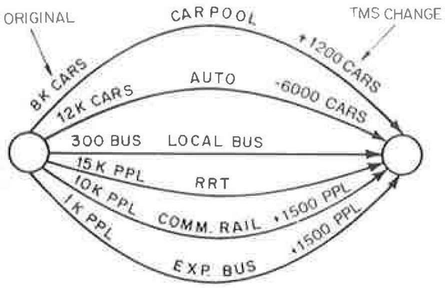
The current mix of heavy versus light vehicles is changing; the percentage of the light vehicles is increasing. The need for fuel efficiency is the most important reason for this change.

A change in the accident effect can be described in terms of the simple probability that two automobiles in a collision will be of significantly different weights. In the special case of a head-on collision at equal vehicle speeds, for example, the collision between two equally heavy vehicles would tend to result in equal severity of damage. If either vehicle were much lighter, it would suffer the greater severity of damage because it would be pushed backward. The heavier vehicle would have a lesser severity simply because it would not be forced to zero speed.

For this case, the total severity of damage could actually be maximized when the fleet is a 50:50 mixture of heavy and light vehicles. Thus, the accident history could worsen and then improve simply based on random conflicts, as the vehicle mix moves toward a preponderance of small vehicles. Preliminary (and somewhat crude) calculations show a possible increase of 5-10 percent, followed by a comparable decrease.

It is interesting that, if the problem is actually experienced (i.e., by an increase in severity), it would be logical for the appropriate authorities to attempt to solve it. However, even an unsuccessful solution would appear to be successful if the percentage of small automobiles significantly passed 50 percent simultaneous with the implementation of the solution.

Figure 4. Hypothesized distribution of trips.



CASE 4: EFFECTS OF A TRANSPORTATION SYSTEM MANAGEMENT ACTION

Because there is such a diversity of possible actions that can be taken in a transportation system management (TSM) effort, only a simple illustration is addressed here.

Consider the situation illustrated in Figure 4 and based on a hypothesized distribution of trips by several modes in the corridor of interest. The FI values for the subject modes can be used to compute a person-weighted total risk, defined as the summation over the modes of "people times the individual modal FI values", computed for the initial system.

Based on the forces that motivated the TSM action (e.g., increased utilization of capacity or energy conservation), it is hypothesized that the amounts shown on the right-hand side are put into effect. The total person-kilometers is not changed, but the total system risk has had a net decrease of 8.6 percent. This simply highlights the fact that a shift among modes, whether due to TSM actions or other causes, can itself induce changes in the total societal baseline. At the same time, the individual modal FI values for the individual traveler may not change.

CONCLUSIONS

In any systematic study, it is necessary to evaluate the effectiveness of any countermeasures taken. This is often done by a simple before-and-after study of relevant statistics. Unfortunately, many safety-related studies involve long time periods in the collection phases. This can be much more than an inconvenience. As illustrated in this paper, it is quite reasonable that major forces and trends in our society (or in any society) will cause the safety baseline to change during the analysis period. Without careful planning, historical data can be rendered meaningless and erroneous conclusions can easily be drawn. This paper illustrates some key forces that will cause future changes in the accident experience large enough to rival or exceed the effects of most rather successful accident countermeasures and make some rather meaningless ones look rather good.

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**J. Byun was with the Transportation Training and Research Center, Polytechnic Institute of New York at the time this research was performed.*

Applicability of Behavior Theory to Transportation

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The applicability of two theories of behavior—the humanistic theory and the behavioristic theory—to two areas of transportation—safety and modal choice—was tested. The first experiment supported the use of the behavioristic theory to explain driver compliance to speed limits and found that public information campaigns were ineffective. The second experiment also supported the use of the behavioristic theory and showed that reinforced choices will be made more often in an environment where positive controls govern the consumers' modal choice.

At least since 1970, the federal and state governments have attempted to coordinate their concerns with transportation environmental safety, pollution, and energy conservation. Throughout the same period, and especially since the 1973-1974 energy crisis, the federal government has attempted to reduce the use of petroleum-based fuels. As a result, by one means or another, the public is being asked to make greater efforts toward energy conservation and the control of environmental pollutants. A major contributor to both the high level of air pollutants and the rate of energy consumption is the motor vehicle and the driving behavior of its operators.

The motor vehicle and its operators are part of an interface within the transportation system. As with all systems subject to behavior analysis, it is necessary to assume that a transportation system has its own well-defined environment. The transportation environment supports a specific transportation culture that, in turn, controls a set of appropriate vehicle-operator behaviors. These behaviors are controlled, to some degree, in the following manner.

When a driving behavior is appropriate, positive and negative cultural controls reinforce that behavior, thus increasing the probability that it will be repeated. When a behavior is inappropriate, it is either ignored or punished, thus decreasing the probability that it will be repeated.

The transportation environment in the United States has been so designed that the only controls over driving behavior are aversive in nature. Such controls include (but are not limited to) the issuance of traffic summonses to noncomplying drivers, the loss of driving privileges, increases in insurance premiums, the loss of life and property (or the threat of loss of these), and increases in maintenance costs.

Statistical analysis of accident rates has shown a yearly increase in such rates and that there is no positive correlation between increased levels of aversive control by enforcement agencies and decreased levels of accidents or violations (1).

One of the most important pieces of information relevant to the control of driving behavior may have been gained during the 1974 fuel crisis. Drivers drove more safely when controls so demanded and, in addition, it was found that the use of single-occupancy vehicles decreased and that of public transportation modes increased.

During the fuel crisis, most drivers became aware of the direct relationship between speed and the fuel consumption of their motor vehicles. Although it was also true that, during this period, speed limits were lowered via legislation, it seems rather unlikely that such legislation alone caused the observed significant reduction in the speeding behavior of drivers. Similarly, the correlated decrease in the use of single-occupancy vehicles and increase in the use of public transportation modes cannot be explained as simply a consequence of the legislated lower speed limits. The fact that obtaining fuel was such an annoying task, requiring driving around to find a service station that was open and waiting in lines for hours, thus, having to give up other more rewarding activities, coupled with the sudden and marked increase in the cost of fuel, provided the needed negative control over speeding behavior. The reinforcement contingency was quite clear: Driving at 88.5-km/h (55 mph) resulted in the removal (from the drivers's immediate environment) of a series of unpleasant and aversive stimuli to which he or she might have been exposed.

Driving at lower speeds resulted in the removal of unpleasant stimuli and in an increase in the frequency with which appropriate behavior was being reinforced. Driving at higher speeds resulted in the continued exposure to aversive transportation-related stimuli and in the occasional punishment of such inappropriate behavior by enforcement authorities.

Once the fuel crisis was over, however, and gasoline was plentiful again, speeding behavior showed spontaneous recovery; no reasonable amount or severity of law enforcement can control such behavior and depress its level of frequency to that observed during the fuel crisis.

Everett, Hayward, and Meyers (2) have described a design that tested bus ridership as a function of the reinforcement of such behavior. When positively reinforced, ridership on a campus bus increased markedly. When reinforcement was withdrawn, ridership on the bus decreased to the premanipulation levels.

Paine and others (3), in studies of consumer attitudes toward automobile versus public transportation modes conducted in Baltimore and Philadelphia, found these attitudes to be controlled by powerful social norms. These norms reinforce the choice of private automobile over public transportation. The choice of private automobile is reinforced despite the harmful consequences of increased air pollution and the continuing depletion of natural resources.

The major purpose of the studies presented in this paper was to test the divergent predictions of two theories of human behavior as they relate to the behavior of drivers in the transportation environment. The two theories are

1. The humanistic theory of behavior and
2. The behavioristic theory of behavior.

EXPERIMENT 1: 88.5-km/h SPEED LIMIT

The underlying hypothesis in this experiment was that there is a direct relationship between noncompliance with limits of vehicle operating speeds and the strategy of speed control used. Two strategies of speed control were considered.

The first strategy was based on the explanation of speeding behavior through the use of the humanistic theory of behavior. This theory hypothesizes that the variables that control vehicle operating speeds are

1. The depth of penetration of information relevant to the consequences of speed control,
2. The operators' motivation to become self-actualized by seeking a measure of self-control over the speed at which the vehicle is operated, and
3. The success of the reeducative strategy of driver education.

The second strategy of speed control considered was based on the explanation of speeding behavior through the use of the behavioristic theory. This theory hypothesizes that the variables that control vehicle operating speeds are the past histories of reinforcement and punishment relative to the speeding behavior of the operator.

There were 80 participants in this experiment (37 male and 43 female), all students at the State University of New York College at Fredonia. Participation was voluntary, and participants gave their informed consent before participating in the survey. All participants were licensed drivers.

A questionnaire devised to measure attitudes and behavioral practices regarding the 88.5-km/h speed limit was administered. On the average, participants had been licensed drivers for at least 3 years, and 45 percent owned their own automobiles. There were about 2.3 automobiles/driver's family. The fastest average speed ever driven on a major divided roadway was 133 km/h (83 mph). The slowest average speed reported was 54 km/h (34 mph), and the overall average operating speed reported was 94 km/h (59 mph). Forty-seven percent of the sample drivers had had at least one accident, and there was an overall average of 1.8 accidents/participant.

Ninety-one percent of all participants indicated that they believe that a national speed limit is necessary because people in general are not responsible enough to control their own speeding behavior. Eighty-four percent of all participants believed that compliance with the 88.5-km/h speed limit saves lives, and 79 percent asserted that it saves gasoline as well. Eighty-nine percent of all participants believed that automobiles are major contributors to air pollution and that driving at 88.5-km/h reduces overall pollutant emission.

Drivers' compliance with the speed limit was not positively correlated with the index of information penetration. A negative trend of correlation was observed but it did not reach significant levels ($r = 0.135$; $df = 78$).

With regard to choices of transportation mode, 55 percent indicated that they would use their own automobiles to commute from home to work. Thirty-four percent would carpool as an alternative, but only 6 percent indicated that they would consider a form of public transportation. Eighty-four percent of all participants indicated that they believed that public transportation (subway, bus, or train) is the transportation trend of the future, and 76 percent indicated that they believed public transportation to be more fuel efficient and less polluting than private automobile transportation.

The humanistic theory of driving behavior was not supported. The information tested had deep penetration as shown by the facts that 84 percent of the participants felt the 88.5-km/h speed limit did save lives, that a slightly smaller percentage believed that it saved gasoline, and that 89 percent believed that compliance with it will reduce air pollution. However, participants reported a low level of compliance with the speed limit, excessive speeding behaviors, and a high frequency of accidents. Ninety-one percent of the participants indicated that they favored law enforcement of speed limits and felt that people, on the whole, are not motivated by self-actualizing needs to control their own speeds.

Contrary to commonly held beliefs, the reeducative strategy of driver education has been largely ineffective in the actual control of speeding behavior. Government-sponsored programs aimed at increased dissemination of information relevant to the positive consequences of compliance with national speed limits were successful when information penetration was measured. These reeducative programs were responsible for the formulation of responsible attitudes with respect to transportation. However, these programs were not successful in controlling driver behavior.

Thus, the results of the experiment supported an explanation of driver speeding behavior through behavior analysis and a behavioristic theory of driving behavior. Furthermore, the results suggest that the control of driving behavior through the use of punishment and other aversive controls is ineffectual. The control of driving behavior through positive and negative reinforcement is the most effective means of behavioral control. In operation, such reinforcements could mean express lanes for buses and carpools and reduced tolls, insurance rates, and fuel costs for compliant drivers and for drivers opting to use public transportation rather than driving alone. The second experiment was designed to test these largely hypothetical assumptions.

EXPERIMENT 2: MODAL CHOICE

The major purpose of this experiment was to test the premise that the reinforced choices would be made more often in an environment where positive controls reinforce consumer choice of the public transit and carpool options and do not reinforce the private automobile choice.

Four experimental conditions were designed. It was hypothesized that pretest choices of private automobile would remain unchanged in the control and incentive conditions but would change significantly in the reinforcement and mixed conditions.

The participants in the experiment were 40 male and 40 female undergraduate students enrolled at the State University of New York College at Fredonia. All participants had a valid driver's license, gave informed consent before participating in the experiment, and did not take part in experiment 1. Participation in this experiment was voluntary.

The instrumentation used in the experimental design corresponded to two experimental phases:

1. Measurement of attitudes toward existing transportation modes and
2. Simulation of a commuting transportation environment.

The measurement of attitudes was accomplished by using a two-part (pretest and posttest) questionnaire to record participants' responses. A game that simulated the actions taken by a person living 80 km (50 miles) from his or her job was developed to reproduce a real commuting environment. Participants traveling to work were asked to develop transportation-related behaviors appropriate to one of four types of environment:

1. An environment that reinforces appropriate modal choice,
2. An environment that provides year-end incentives for appropriate modal choices,
3. An environment that provides both immediate reinforcement and year-end incentives for appropriate modal choices, and
4. An environment that does not control the modal choice.

In the last scenario, participants were, in effect, asked to use their past history of reinforcement to make a decision. Random assignments were made to the various conditions. In all conditions, participants traveled (twice each) to work by bus, as members of a carpool, or alone in their own automobiles. It was required that a formal economy that used simulated money and account sheets be maintained by all participants except those in the control and incentive conditions. All participants were required to follow instructions presented to them on cards placed at strategic locations.

These cards contained information relevant to keeping the formal economy. Thus, participants were informed about the gasoline efficiency of their automobiles, travel time, out-of-pocket costs, and travel routing. Under all conditions except the control condition, the participants were given information on tax incentives related to various transportation modes.

All participants completed the posttest questionnaire and were debriefed by the experimenter.

The general opinion of the participants was that the simulation resembled a real-world situation. Thus, 98 percent of all participants felt that the simulation game was very much like the real-world situation. One participant felt that the simulation was only somewhat like real-world transit conditions, and one participant did not answer the question during debriefing.

Of the participants in the immediate-reinforcement condition, 70 percent changed their choice of transportation mode in the desired direction. Of the participants in the combined condition (immediate reinforcement and year-end incentives), 60 percent changed their choice of transportation mode in the desired direction. Only 30 percent of the participants in the control and incentive conditions changed their choice of transportation mode in the desired direction.

To test for differences in frequency of change among the four different conditions, a χ^2 test was used. The control condition was used to supply the expected value, which was compared with each treatment condition.

The difference in frequency of change between the immediate-reinforcement condition and the control condition was significant at the $p < 0.01$ level. The difference in frequency of change between the combined condition and the control condition was significant at the $p < 0.02$ level.

There were no significant differences in frequency of change between the control and incentive conditions or between the combined and reinforcement conditions.

Thus, the results of this experiment conclusively support the experimental hypothesis. Participants overwhelmingly indicated their preference for the private automobile when asked to choose the mode by which they would commute from home to work. Those randomly assigned to the reinforcement and combined (reinforcement and incentive) conditions changed their opinion and chose, on a posttest, to commute by public transportation or by carpool. Incentives promising positive year-end consequences to appropriate behaviors were found to be ineffective in controlling driver behavior. The incentive and control conditions showed no differences between pretest and posttest choices.

It is becoming increasingly clear from experiments such as these that strategies of transportation behavior that use the explanatory powers of behavioristic theories promise to have real and lasting effects on the control of driving behavior and modal choice. On the other hand, predictions of greater control of driver behavior through the application of humanistic principles remain unsupported.

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Improving Traffic Safety in Rural Kansas

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The traffic engineer's goals are to provide safe, efficient, and convenient movement of persons and goods on streets and highways and to provide adequate modal transition. In larger urban areas and along primary roads, this purpose has been met to varying degrees. However, in rural areas where most cities have populations of less than 5000, there is a lack of proper traffic-control devices and of traffic engineering studies and help. In southwestern Kansas, the population density is less than 4 persons/km^2 (10 persons/mile^2), and there were no local traffic engineering personnel in the 41 150-km^2 (16 000-mile^2) area. The Greater Southwest Regional Planning Commission created a position of regional traffic engineer in late 1976, which was funded through the Kansas Department of Transportation and the Federal Highway Administration. During the first two years, the engineer has (a) involved 29 of the 45 cities in federally funded traffic-sign-improvement projects, (b) completed or initiated analysis at several high-hazard locations, (c) assisted local units of government to become aware of and obtain state and federal funds, and (d) worked with local government personnel in 18 of the 19 counties in the region to establish some local expertise in traffic safety. The primary benefit of the regional traffic engineer has been that traffic engineering has been brought to southwestern Kansas with a personal touch. The local units of government could not individually afford and, in fact, would not need a full-time traffic engineer. Under the commission assistance plan, the engineer is on call to all the local units, is governed by them, and is used by them. A regional traffic engineer is a means of providing expertise to rural areas.

"The manual presents traffic control device standards for all streets and highways regardless of type or class or the governmental agency having jurisdiction." This quotation (1, p. 3) from the Manual on Uniform Traffic Control Devices clearly states that control devices for all roadways should follow standards set in the manual. Typically, implementation of the manual standards is carried out by the traffic engineers of the applicable highway agency. The overall objective of the manual and the involved traffic engineer is the improvement of traffic safety throughout the country.

How is traffic engineering brought to rural America? Persons experienced in traffic engineering usually are located in larger urban areas and work for consulting firms, state governments, the federal government, or a few large cities. The need for a high concentration of traffic engineers in urban areas is obvious, because traffic problems usually relate directly to the size of the population. The result is improved roadway uniformity, safety, and efficiency, primarily where the higher concentration of people occurs. However, a continuing problem in rural areas, particularly those that have very low population densities, is the lack of traffic engineering studies and help. A major concern now is how to properly control the traffic in rural America. This paper describes how traffic engineering was more efficiently brought to southwestern Kansas. By using a grant from the federal highway safety fund, the Greater Southwest Regional Planning Commission (GSRPC) created the position of regional traffic engineer (RTE) to provide personal service to a part of rural Kansas.

TRAFFIC ENGINEERING IN SOUTHWESTERN KANSAS

Before 1977, there was no locally based traffic engineer in southwestern Kansas and, in that year, the membership directory of the Institute of Transportation Engineers listed 31 such professionals in the state. An RTE for the Chikaskia, Golden Belt, and Indian Hills (CGI) Regional Planning Commission moved to Pratt, which is about 208 km (130 miles) from the center of the Southwest Region, in December 1976. The next closest traffic engineer was in Wichita, another 120 km (75 miles) from Pratt. The Kansas Department of Transportation (KDOT) headquarters is in Topeka, about 480 km (300 miles) from the center of the region. Only half of the counties in southwestern Kansas have licensed county engineers.

In 1976, KDOT administrators were faced with the problem of providing better traffic engineering throughout Kansas. It was obvious that most of the communities had few if any traffic-control devices and that many of those were nonstandard. The options for improvement were

1. To continue to use the one KDOT field engineer and the consulting firms that had direct contacts with the local governments,
2. To retain one or more consulting firms to handle the state or regions of the state, or
3. To use highway safety funds available through the regional traffic engineering assistance program and hire an RTE.

With cooperation from the Federal Highway Administration (FHWA) division office, the third option was chosen. RTEs were hired by two of the seven regional planning commissions and were funded by KDOT and FHWA for a 3-year period. The concept of the RTE is similar to that of the circuit rider (2).

The specific objectives of the RTE project for the southwest region were

1. To upgrade the traffic-control devices in the region, preferably by compliance with the Manual on Uniform Traffic Control Devices;
2. To locate, analyze, and recommend improvements for highway locations having high hazard rates;
3. To ensure that each county and city is aware of the funding assistance available via the state highway safety program and to coordinate the use of these funds in the local areas of greatest need;
4. To increase the traffic engineering capabilities of personnel within the region by promoting the training program funded by the state highway safety program; and
5. To exhibit the benefits to be derived from traffic engineering expertise through retention of a traffic engineer on the GSRPC staff to serve the region.

At the state level, if all goes well, future state funding for RTEs will be applied to other regions and, eventually, there will be 12 RTEs. As stated in objective 5, the local units of government will ultimately finance the RTE positions.

The remainder of this paper presents the accomplishments of the RTE program through September 1978 (3), and briefly discusses the monetary benefits to the local units of government.

A LOOK AT SOUTHWESTERN KANSAS

The region (see Figure 1), has a population density of about 3.2 persons/km^2 (8.4 persons/mile^2) and a total area of about 41 150 km^2 (15 900 mile^2). The area represents 19.4 percent of the state, but the population (1977) of 133 341 represents only 5.7 percent of that of the state. The degree of ruralism is illustrated further if the populations of the three largest cities are omitted: Without the populations of Dodge City, Garden City, and Liberal, that of the region is 81 012 [a density of 2.0 persons/km^2 (5.1 persons/mile^2)]. As shown below, 39 of the 45 incorporated cities have populations of less than 3000 people. Most of the counties (12 of the 19) have fewer than 5000 people.

Category	Item	Population	Total Population for Category
A	Garden City	19 252	
	Dodge City	17 805	
	Liberal	15 272	
	Scott City	5 079	
	Ulysses	4 584	
	Hugoton	3 146	65 138
B	15 cities	1 000-2 999	23 624
C	7 cities	500-999	5 038
D	17 cities	0-499	488
E	Nonurban		39 053
Total for region			133 341

The present economy is primarily agricultural; the cities provide goods and services in support of the agricultural industries. Manufacturing is increasing, and many county seats are seeking light industry. Also, tourism in increasing. Personal income for many persons is dependent on farm output and, therefore, fluctuates frequently.

The terrain is relatively flat, and the lack of trees causes it to appear even flatter. The flat topography is interrupted by frequent dips into small river valleys. Sunshine is abundant, and there is little rainfall [about 46 cm/year (18 in/year)]. Winds are almost continuous and gusts frequently exceed 48 km/h (30 miles/h).

The basic mode of transportation is the motor vehicle, including a large number of trucks. However, general aviation is increasing and most county seats have landing

Figure 1. Area covered by Greater Southwest Regional Planning Commission.

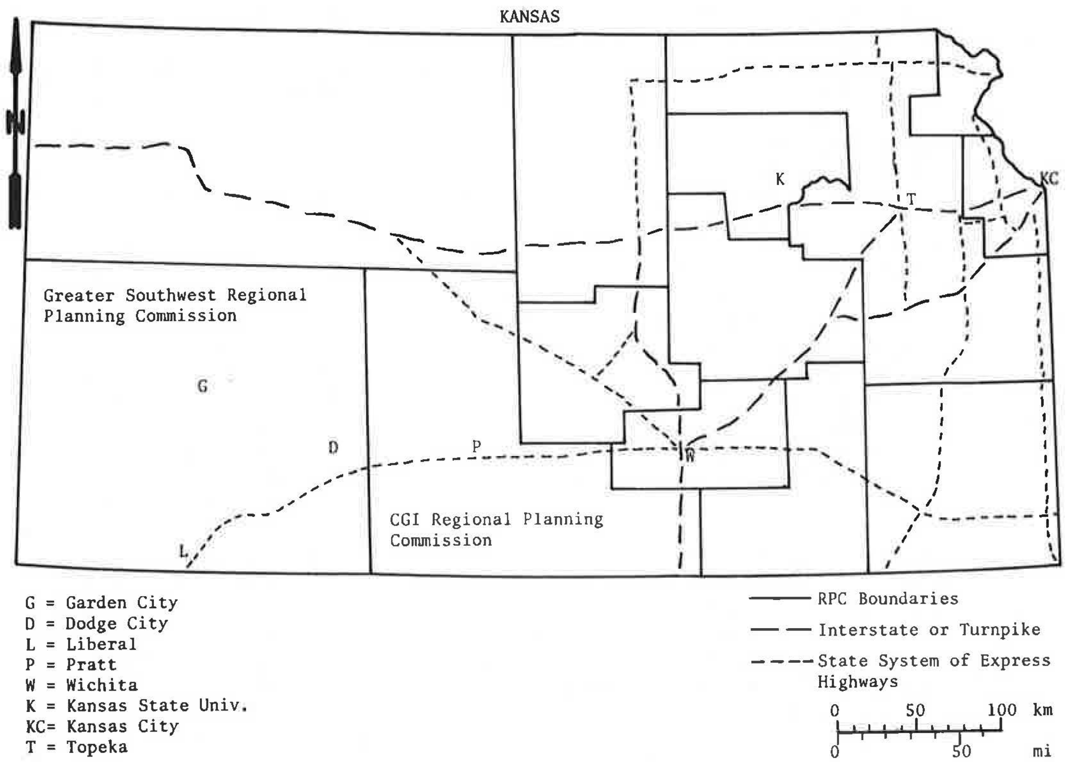


Table 1. Traffic accidents in southwestern Kansas: 1971-1977.

County	1971	1972	1973	1974	1975	1976	1977
Clark	65	50	64	39	62	46	54
Finney	714	622	624	684	893	918	927
Ford	422	834	911	717	825	813	887
Grant	134	122	109	115	147	165	191
Gray	99	106	96	96	83	82	114
Greeley	30	26	18	6	46	52	48
Hamilton	51	51	55	64	72	81	82
Haskell	79	83	85	77	74	106	95
Hodgeman	81	69	84	79	54	64	54
Kearny	83	66	78	52	87	73	107
Lane	59	62	52	57	68	74	70
Meade	99	90	114	102	116	80	108
Morton	40	34	34	47	71	68	90
Ness	131	135	122	113	108	117	110
Scott	136	142	135	137	169	163	180
Seward	569	588	591	618	659	701	741
Stanton	36	53	33	44	26	47	46
Stevens	89	90	100	108	99	127	141
Wichita	60	41	75	53	22	37	49
Total	2977	3264	3380	3208	3681	3814	4094

fields. The airports at Dodge City, Garden City, and Liberal have limited commercial service. Some commercial bus lines traverse the region, but cross connections are poor and nine of the counties have no commercial bus service at all. On Amtrak, eastbound and westbound trains stop at Garden City and Dodge City once each day, but the late night and early morning stops are inconvenient.

There are about 42 700 km (16 500 miles) of roads in the region; about 10 percent are federal-aid primary system roads and another 20 percent are federal-aid secondary. Except for a few sections in several cities, all roads are two lanes wide; they range from high-quality two-lane pavements to narrow dirt roadbeds. Even some sections of U.S. routes are narrow [6.7 m (22 ft)] and without shoulders. The region has truck climbing lanes in two locations and two grade-separation structures (one at a highway-highway crossing and one at a highway-railroad crossing). There is no Interstate roadway in the region.

The 1976 annual motor-vehicle travel in the region was estimated to be almost 1.6 billion vehicle-km (1 billion vehicle miles). Several urban roads have average daily traffic (ADT) counts of more than 10 000 and, on several rural primary roads, the counts exceed 5000. However, many primary roads have counts of less than 500. In many cases, more than 20 percent of the ADT is composed of heavy commercial trucks. Some secondary roads near urban areas have ADT counts of 500 but more have counts of 50-200.

In 1977, there were 4094 motor vehicle collisions (a 7.3 percent increase from 1976) involving 10 553 persons. Of these persons, 64 were killed and 1360 sustained some injury. Table 1 shows the number of accidents per county for 1971-1977; the totals for the region as a percentage of the state totals are shown below.

Year	Percentage
1971	5.5
1972	5.3
1973	5.7
1974	6.0
1975	5.9
1976	5.8
1977	5.7

APPROACH TO IMPROVING TRAFFIC SAFETY

How does one go about improving traffic safety in an area of 41 150 km^2? Obviously, KDOT has spent much time and money making physical improvements and implementing various control devices on the state highway system. Counties that have professional engineers, in general, have relatively safe secondary roads, but off-system roads generally lack the basic controls. The three largest cities in the southwestern region have city engineers; however, these persons have had little training in traffic engineering. In fact, the city police departments are responsible for most of the traffic-control devices. Most local units of government have no person assigned to utilization of traffic-control devices.

The obvious finding is that, in the southwestern region, no person (except for a few from the KDOT district office) is trained in any phase of traffic engineering. This is quite apparent when one drives through the region: Many signs are improper (e.g., yellow yield signs); signs are improperly installed [most are less than 1.5 m (5 ft) high]; most intersections (rural and urban) are uncontrolled; the few signalized intersections have out-dated equipment in locations difficult to see. There has been a lack of real communication between local officials and those concerned with traffic engineering. Most cities and counties have a copy of the manual, but few officials have used it.

Thus, communication is the key link for improving traffic safety in rural Kansas. Communication needs include

1. Highway and traffic-control-device standards and what they mean,
2. Recent changes in applicable standards,
3. Awareness of federal and state funding programs,
4. Short courses and educational opportunities, and
5. Better dialogue between local officials (nonengineers) and KDOT officials (engineers).

The first task was to have the RTE serve as a liaison between the local units of government and KDOT. Because many traffic improvement projects require much time and effort, the second task was to find effective traffic safety projects that were low cost and would require only short implementation times. This would allow rapid initiation of projects in local units and ensure that the public could soon see the improvements. Once the early projects were initiated, then efforts could be shifted to larger, more time-consuming projects.

Traffic-Sign Improvements

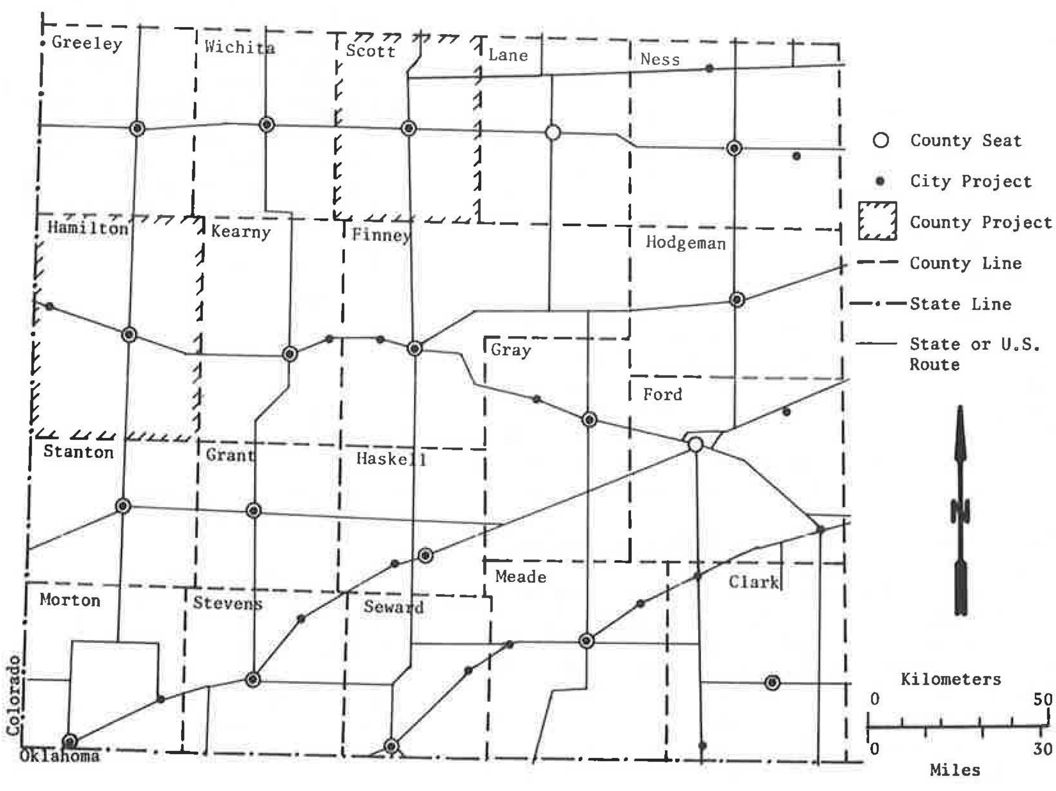
The first major effort at improving traffic safety was to up-date traffic signs. Because most on-system highway signs are provided through federal funds, the effort was directed toward off-system signs. The cities, for the most part, lacked any engineering capability and, because of their higher traffic volumes, were in greatest need of help.

Inspection of several cities in southwestern Kansas showed that existing traffic signs did not generally meet current standards and were not being properly maintained. Most signs were an improper color or nonreflectorized or both; many had been vandalized or were improperly located. Because sign maintenance in small cities has been carried out or supervised by the county engineers, mounting heights were frequently less than 1.5 m (5 ft). The major lack of specific signs included those reading STOP, YIELD, SPEED LIMIT, and DIP and those related to schools.

The federal-aid safer-roads demonstration program was established to improve safety on off-system roads and is funded with 90 percent federal and 10 percent local monies. The urban department of KDOT had made these funds readily available to all incorporated cities for off-system improvements, provided a proper engineering study was prepared. Before the RTE position was created, only four cities had applied and had been approved for these funds. But after the RTE contacted other city and county officials and encouraged them to request funding from KDOT, 29 cities and 2 counties applied and were approved for sign projects. A total of 33 out of 45 cities and 2 of 19 counties in the region now are involved or have completed sign projects (see Figure 2).

It has been the RTE's job to prepare the sign surveys, to assist the cities in submitting the projects, to coordinate installation, and to serve as a liaison between KDOT and the cities when needed. In preparing the surveys,

Figure 2. Off-system traffic-sign improvement projects.



the RTE was aware that local city budgets were very tight and, therefore, included only essential signs. Excess signing would probably create more hazardous conditions in the future because most cities could not afford high maintenance costs. Occasionally, it was necessary to convince local officials that standards were made for their safety and that the essential signs were really needed.

Seven of the 29 cities in which surveys were made have ordered or are now installing signs. Eleven cities, in addition to the original 4, have completed sign installation. The time required from initial request to beginning of implementation is less than a year and to completion is less than 18 months.

One indirect advantage of the sign projects has been that local officials are beginning to think about traffic safety and to know how to communicate with KDOT directly. Also, as signs are being installed, street superintendents are becoming familiar with the manual. New signs also help dress up a city and give more pride to the people.

High-Hazard Locations

High-hazard locations have been more difficult to determine and to analyze. Before creation of the RTE position, the cities hired a consulting firm or requested assistance from their county department of transportation or KDOT, who were usually slow in responding because of their backlog of work. In most cases, however, nothing was done beyond local complaints. The RTE's task was to first determine the locations and to then establish priorities for remedial studies. Hazardous locations, obviously, are locations that have high accident experiences; however, locations that have high accident potentials can also be classified as hazardous. Such include (a) bridge sites, (b) railroad-highway crossings, (c) signalized intersections, and (d) noncontrolled intersections that have high traffic volumes or reduced visibility or both. Certain crops, such as corn, frequently create serious problems on a seasonal basis.

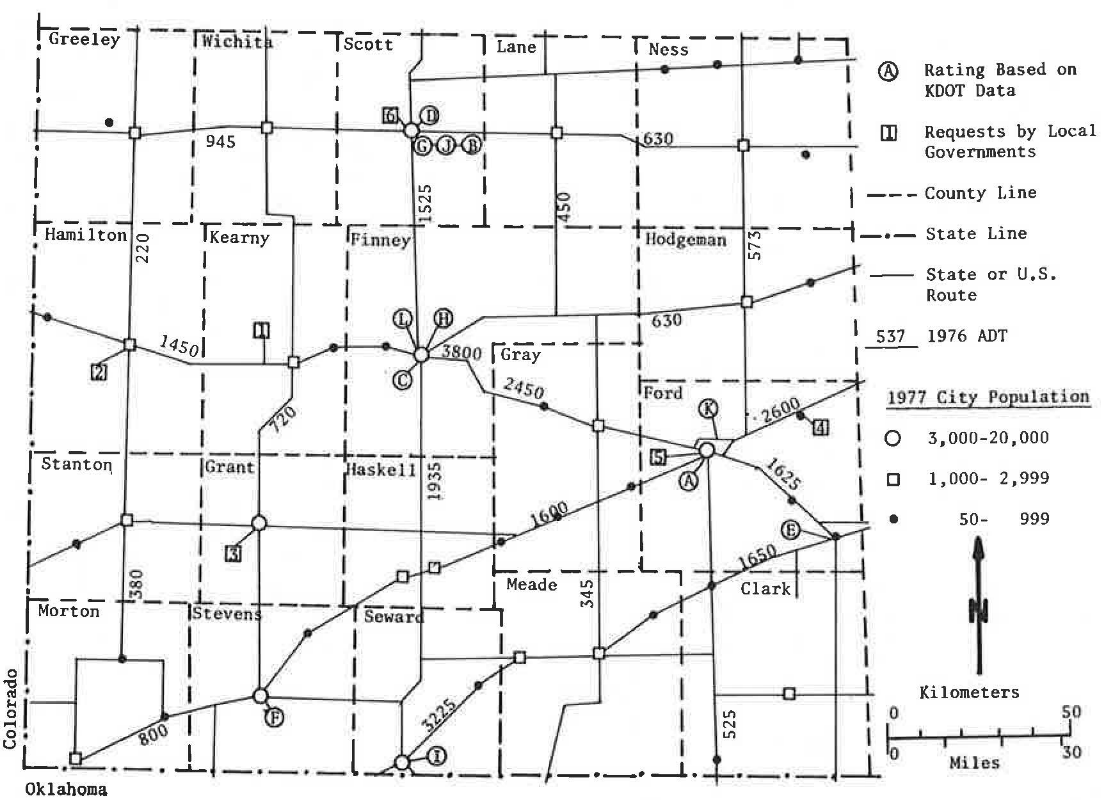
As a first source of information about high-hazard locations, a pin board that showed the locations of all fatal accidents from 1972 to 1977 was prepared. Unfortunately, this display did not show specific problem locations because most fatal accidents are isolated cases. The number of fatal accidents per year for the region ranged from 30 to 50. This means that, on the average, each year there is one fatal accident within an area of about 1050 km^2 (405 $mile^2$). Vehicle accidents (all types) are also infrequent and average one in an area of about 12 km^2 (5 $mile^2$).

Another source of data is the KDOT computerized accident-record file. By using the KS-HYSIS program (4), regional high-frequency accident locations for state highway sections were obtained for 1976 through May 1978. This gave a list of the 12 highest-frequency locations according to a rating based on the accident rate and a critical factor. The rating is determined as follows:

1. For each year, route sections are ranked by accident rate and by critical factor,
2. Each section is assigned points according to its rank by accident rate and its rank by critical factor (+3 if highest or second highest, +2 if third to fifth, +1 if sixth to tenth, 0 if eleventh to fifteenth, -1 if below fifteenth); therefore, a section could receive up to 6 points/year (if it were first or second in both accident rate and critical factor),
3. The points are totaled for the 3-year period, and
4. The rating is based on (first) the number of years the section did not have a negative value and (second) its total number of points.

All locations found were within city limits. Figure 3 shows the general locations; the "A" location had the highest priority, the "B" was second, and so on. Popu-

Figure 3. High-hazard locations in southwestern Kansas.



lation and traffic flow appear to be major factors in determination of the locations. Fourteen of the 19 counties do not have any high-hazard locations. However, it should be noted that this analysis compares lengths of state routes and does not consider point locations or off-system locations.

A third source of data about high-hazard locations was in the form of comments and requests from local units of government, especially law enforcement officials. Local governments have requested analysis of six locations. These locations tend to reflect recent accidents that have received public attention. It was apparent that local units of government had been aware of their needs and that some had requested state assistance. The problems of applying for remedial funds, apparently, had been twofold:

1. KDOT did not have sufficient staff to prepare the engineering studies or the local units did not have the technical ability to communicate their needs in engineering terms and
2. City populations and traffic movements were low; therefore, the question of state priority was raised.

The obvious role of the RTE was to serve as a translator for the local units of government—to express the local needs in engineering terms.

In establishing the priority order for preparing high-hazard studies, several factors were considered:

1. There had, in fact, been few accidents at the identified hazardous locations and, therefore, the highest-priority locations were not significantly higher than the lower-priority ones.
2. Studies requested by local units of government received higher priority because local traffic records (if they existed) were more accessible.
3. Locations that appeared to have minimal opportunity to receive state or federal funds and anticipated high remedial costs were given lower priority because few local governments could bear high traffic-safety costs.
4. Locations that had higher ADT counts received higher priority.
5. Because the services of the RTE are for the entire region, attempts were made to distribute the studies equitably throughout it.

Because the local requests were received before the KDOT data were analyzed, the six local requests shown in Figure 3 received attention first. Some of the requests received related to locations that did not have a recent accident history and had a low ADT count. These less important locations were filed for possible future action. Studies have been completed at sites 1, 2, and 4 and begun at sites 3 and 5. Analysis of site 6 will begin in early 1979.

At the intersection of US-50 and US-270 (site 2) in downtown Syracuse, out-dated traffic signals on vertical posts (at all four corners) provided poor traffic control because the signals were very difficult to see. Of the 19 accidents that occurred in the 3-year study period, only one involved an injury and there were no fatalities. However, the average accident rate was 21 accidents/10 million vehicles. Contracted costs for signal improvements are about $40 000, which would be difficult for a city of 1995 people to finance. Through proper engineering documentation, KDOT has agreed to provide 90 percent funding (federal) for the improvements (which should be completed by early 1979).

The Syracuse project made evident to local officials that KDOT is willing to partially fund traffic-safety projects in rural Kansas. Improvements along US-50 by KDOT have made the entrance to Loucks Park safer (site 1). It is anticipated that proposed signal improvements at sites 3 and 6 will also receive state and federal assistance.

Traffic analyses by the RTE have been performed not only to obtain funds from KDOT but also to justify local expenditures and improvements to local units of government. These studies would normally have been prepared by consulting engineers (at a cost to the local unit) or probably omitted and action taken without proper guidance. The special studies have included the evaluation of

1. The traffic flow and parking facilities around Dodge City High School (Ford County),
2. The need for a crossing guard at a Garden City elementary school (Finney County),
3. The traffic flow around the Scott City park (Scott County),
4. The need for positive control at a minor railroad-street crossing in Spearville (site 4),
5. The traffic flow on a federal-aid secondary route at a feed-lot entrance used by numerous trucks (Haskell County), and
6. The need for flashing school-speed-limit signs at a suburban elementary school (Finney County).

Awareness of Funding Assistance

One objective of the RTE was to assist local units of government in obtaining federal and state funds and to keep them aware of available funding sources. All city and county officials were advised concerning funds for traffic-sign improvements on off-system roads. As a result, more than 70 percent of the incorporated cities applied for such funds. County officials were also informed concerning funds for pavement striping and for off-system road improvements; however, requests for assistance were few. In general, the county officials were concerned with the condition and the maintenance of the road surface only, because of limited funds. The GSRPC mailed a monthly newsletter that included an article on traffic-safety projects and funds to all local units of government. In addition to informing the local units, the RTE has continued to assist local officials in completing forms and to serve as liaison between them and state and federal officials. An initial problem was the determination of the best contact in each local unit of government: Should it be the mayor, street superintendent, clerk, police, or someone else? Also, it soon became apparent that written correspondence was usually shelved and that telephone or (better yet) personal contact was necessary.

Training Programs

Development of local expertise in traffic safety was another objective. Through contact with local government personnel in projects such as the traffic-sign improvements, limited training in traffic safety was provided by the RTE. During the first year, there were projects in 16 of the 19 counties and, through working with these projects, the local officials received some personal training.

Efforts to have local personnel attend short courses in traffic engineering at Kansas State University have been unsuccessful. The problems appear to be that

1. The university is too far away [320-560 km (200-300 miles)],
2. The courses are too long—for example, street

superintendents are often responsible for all other city physical operations,

3. There is apathy toward east Kansas, and
4. The content of the courses is not rural-directed enough.

As a result, the GSRPC submitted an application for a highway safety grant entitled "Traffic Safety Training Program in Southwest Kansas" that would have offered 2-day training courses at one or more locations within the region and presented an overview in traffic safety to serve as a stepping stone for the short courses at the university. It was felt that such a course would encourage more participation from southwestern Kansas in the university courses; however, it appears that the application has been rejected.

Benefits to Local Units of Government

To be a member of the GSRPC, each local unit of government is charged $0.10/capita/year; this includes services for all areas of community development, not only traffic safety. Therefore, city dues per year ranged from $6 to $2000 and county dues ranged from $200 to $2500. Thirty-nine of the cities paid less than $300/year. The remainder of the commission staff finances come from state and federal grants, such as the RTE grant. Current plans are for the commission to absorb and finance the RTE's position without change in membership dues. The RTE grants have ranged from $36 800 to $41 400/year. Labor costs have been about 75 percent, and travel and engineering equipment have been about 8 and 4 percent, respectively. The first-year grant included about $5000 extra for special engineering equipment (counters, radar, and such) and office equipment.

The benefit/cost ratio for the local units is very high. Most traffic-sign projects have required at least 20 hours of the RTE's time, and the larger projects have required more than 50 hours. The costs (materials and labor) of the sign projects, excluding the RTE's time, have ranged from $1000 to $15 000. In addition to direct project help, the RTE is on call all year and keeps the local officials aware of potential funds.

The primary benefit, however, cannot be quantified. The RTE position has brought traffic engineering to rural Kansas. The number of cities and counties participating in traffic-improvement projects during the first two years has been 34 of 45 and 5 of 19, respectively. Although direct comments from local officials and citizens have been few in number, it is apparent that the RTE position is well received because of the participation of local units of government and the support by the GSRPC.

RESULTS

Rural areas of Kansas, such as the southwest, were not receiving proper traffic engineering improvements because of the lack of local traffic engineering expertise. To offset this need, KDOT established a traffic engineering position in the GSRPC. The RTE was to provide assistance to the local units of government.

Some specific results in the area of traffic safety include

1. Making local units of government aware of traffic safety and the need to meet established standards,
2. Making local units of government aware that KDOT is concerned about traffic safety in rural Kansas and that federal monies are available for assistance,
3. Identifying contact persons in most local units of government and assisting them in becoming involved with traffic safety,
4. Assisting 29 cities and two counties in traffic-sign-improvement projects for off-system roads,
5. Providing an engineering study to assist the city of Syracuse to obtain federal monies for signal improvements,
6. Preparing recommendations for school traffic-safety improvements for several units of government, and
7. Providing traffic counts and speed data for several units of government that would allow them to evaluate possible improvements.

The RTE position has been supported for three years with federal funds. It is anticipated that the local units of government (through the GSRPC) will now be aware of the benefits of a traffic engineer and willing to continue such a position with local funds. The executive director of the GSRPC has been planning for such a step, and the RTE position now includes the total area of transportation rather than only traffic safety. This includes traffic safety, public transportation, airport planning, and general civil engineering and systems analysis. A special task for the RTE has been to serve as group coordinator of special projects—traffic safety, airports, public transportation, law enforcement services, and emergency medical services. The integration of traffic safety into a comprehensive public safety program has been initiated.

Because there is only one RTE, all local units of government do not receive personal assistance all the time. However, thus far, local units in 18 of 19 counties have received direct assistance and have had traffic-safety projects. Because no local unit of government could individually afford or, in fact, would need a full-time traffic (or transportation) engineer, the RTE concept has provided a feasible alternative that is locally controlled yet capable of communicating with the state and federal governments. In return, the state does not need to create a new position or spend extra money. A regional traffic (or transportation) engineer is an answer to providing more direct expertise to rural areas.

ACKNOWLEDGMENT

I wish to thank the GSRPC for the opportunity to serve as their regional traffic engineer and KDOT and FHWA for providing the grant. It has been a challenging and rewarding opportunity. The opinions, findings, and conclusions expressed in this report are mine and not necessarily those of the GSRPC, KDOT, or FHWA.

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Methods for Improving Analysis of Roadside Safety

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This paper reviews the state of the art of analysis of roadside-safety improvements. Particular attention is given to the roadside-hazard model developed by Glennon. The need for additional research to validate or add precision to the Glennon model is demonstrated. An empirical accident study is suggested that relates the hazard of specific roadside obstacles to design, operating, and environmental variables. The proposed analysis method assumes a Poisson probability distribution and uses Bayes' theorem for the construction of a discrete model for precisely predicting the hazard of any roadside location.

In the 1960s, highway accidents increased at an alarming rate. By 1972, the annual toll was 56 600 persons killed and more than 2.1 million injured (disabled beyond the day of injury). Of these, 19 900 persons died in single-vehicle accidents (e.g., ran off road, hit fixed object, or overturned)—a significant 34 percent of all highway fatalities (1). In addition, the chance of death is three times greater in single-vehicle accidents than it is in all other highway accidents (2). These human and economic losses can no longer be tolerated. Clearly, positive action is required to change this record of highway death and injury.

Another reason for giving special attention to this kind of accident is that it represents the most visible form of failure in the driver-vehicle-roadway system. Circumstances caused by other drivers and other vehicles are less important in single-vehicle accidents than in multivehicle accidents (2). Therefore, reducing the frequency and severity of single-vehicle accidents by implementing roadway improvements offers one of the most clear-cut opportunities for improving highway safety.

The hazards associated with roadsides are obvious to anyone who drives. Despite attempts to adequately design roadways, there is great danger to a vehicle that leaves the traveled way. Roadside obstacles do not always give the driver who has inadvertently left the pavement a chance to avoid, or at least survive, an accident. The elements of the roadside that contribute heavily to the consequences of single-vehicle accidents are obstacles such as bridge abutments and piers, bridge rails, rigid signposts, rigid luminaire supports, utility poles, trees, drainage structures, steep side slopes, and guardrails.

Quantitatively, the degree of accident hazard associated with a roadside obstacle can be defined in several ways. The accident hazard is a measure of the potential for a particular obstacle to experience a given time rate of accidents that have some average consequence (such as average cost, number of fatalities, or ratio of number of injury-plus-fatal accidents to total number of accidents). At any roadside location, then, the degree of accident hazard is a function of two variables: accident frequency and accident severity. When two locations are compared, if both have the same accident frequency, the location that has the lower accident severity is less hazardous. If both locations have the same accident severity, the location that has the lower accident frequency is less hazardous.

EARLY EVOLUTION OF ROADSIDE-SAFETY IMPROVEMENT

In the past, design efforts to improve roadside safety were limited. Many highway designers felt that a driver who ran off the roadway deserved the consequences of his or her imprudence. As the evidence mounted of the increase in single-vehicle accidents, however, this philosophy gradually changed and the ran-off-road driver became an important consideration in highway design. Consequently, many state highway departments began an active attempt to improve roadside safety.

In an overreaction to the recognition of the need for improved roadside safety, highway guardrails were at first thought to be the panacea. Large quantities of guardrail were placed where none was needed or where a minimum of slope grading could have eliminated the need. The tragic side of what was otherwise a promising improvement in roadside safety was the increase in the frequency of spectacular accidents involving guardrails.

This era of new highway-guardrail installations was marked by a lack of objective criteria for evaluating the consequences of alternative roadside designs. Warrants for guardrail installation were subjectively based on relative judgment. This judgment varied greatly from one state to another, precluding the possibility of minimizing the consequences of running off the roadway.

In 1966, Glennon and Tamburri (3) developed the first, now widely used, objective criteria for guardrail installation. A mathematical model was introduced that compared the relative safety of protective guardrails with various combinations of embankment parameters. The relationship was based on comparative severity indices for samples of ran-off-embankment and guardrail accidents.

Another study in 1966, by Hutchinson and Kennedy (4), was a major breakthrough in terms of understanding and predicting the nature of single-vehicle accidents. This study provided empirical data on the nature of roadside encroachments on freeways and included the following relationships:

1. The frequency of roadside encroachments as a function of traffic volume,
2. The exceedance distribution of encroachment angles, and
3. The exceedance distribution of lateral displacements of encroaching vehicles.

This study led to the eventual adoption of the 9.15-m (30-ft) clear-zone concept because it documented that very few vehicles encroached more than that distance from the edge of the traveled way.

Also during this period, other improvements in roadside design evolved. Embankments and cut slopes were flattened to increase the chance of recovery for errant vehicles. Full-shoulder widths and clear-safety zones were provided along the roadway. Better guardrail and median-barrier designs were developed, innovative breakaway sign and luminaire supports were discovered, and effective impact-attenuation devices were invented.

As a result of this evolution in the technology of roadside-safety design and because of the emphasis created by the AASHO publication, Highway Design and Operational Practices Related to Highway Safety (5), many states began a more comprehensive attack on the roadside-hazard problem. Some states even funded specific programs to reduce roadside hazards on existing highways. These programs all followed the same general strategy, which simply says

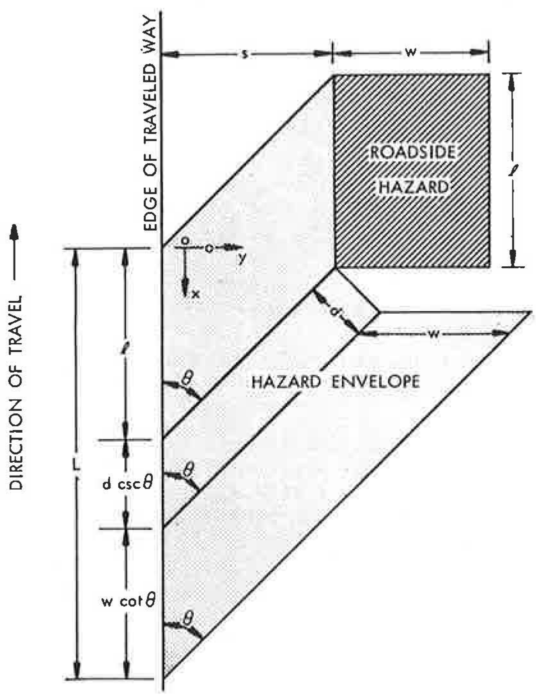
1. Remove unnecessary objects,
2. Move those objects that cannot be removed (this includes moving to a protected location or moving laterally),
3. Reduce the impact severity of those obstacles that cannot be moved (this includes flattening side slopes and installing breakaway devices), and
4. Protect the driver from those obstacles that cannot otherwise be improved by using attenuation or deflection devices.

This approach is ideal if sufficient funds are available to do everything needed. Under ever-present economic constraints, however, trade-offs must be made, even in each of the four basic concepts. The highway administrator is constantly faced with the problem of evaluating many alternatives. Therefore, a definite need exists for methods by which administrators can compare alternative safety improvements and thereby achieve the greatest return within the constraints of available funds.

RECENT ROADSIDE-HAZARD MODELING

The cost-effectiveness model developed by Glennon (6) provides a basic analysis technique for comparison of roadside improvements. This model, in a slightly less objective form, is also given in the AASHTO barrier guide (7).

Figure 1. Schematic illustration of roadside obstacle and its relationship to an encroaching vehicle.



The model depends on the concept that an injury-producing roadside impact is a result of a sequence of four conditional events. First, the vehicle must be within the discrete increment of roadway associated with a potential collision with the roadside obstacle. Then, a roadside encroachment must occur. Next, the lateral displacment of the vehicle must be great enough for collision with the roadside obstacle. And finally, the collision must be of sufficient magnitude to produce an injury.

This sequence of events suggests a conceptual approach for evaluating the degree of hazard for roadside situations. Such an approach considers the vehicular exposure; the expected vehicular-encroachment rate; the expected distribution of encroachment angles; the expected distribution of lateral displacements of encroaching vehicles; and the severity, size, and lateral placement of the roadside obstacle. Figure 1 is a schematic illustration of the increment (L) of highway length associated with a particular roadside obstacle. The hazard envelope is defined by the locus of the right-front corner of the colliding vehicle.

The description of variables suggests that a mathematical relationship is required to truly evaluate the hazard index of a particular roadside situation. For a given angle of encroachment (θ) the explicit hazard equation (6) is as follows:

$$H = (E_f S/5280)\left[l\int_{s}^{\infty} f(y)dy + \int_{l}^{l+d\csc\theta}\int_{s+(x-l)\cos\theta\sin\theta}^{\infty} f(y)dydx + \int_{l+d\csc\theta}^{l+d\csc\theta+w\cot\theta}\int_{s+d\cos\theta+(x-l-d\sec\theta)\tan\theta}^{\infty} f(y)dydx \right] \quad (1)$$

where

- H = hazard index (number of fatal and non-fatal-injury accidents per year),
- E_f = encroachment frequency (number of encroachments per mile per year),
- S = severity index (number of fatal and non-fatal-injury accidents per total number of accidents),
- l = longitudinal length of obstacle (ft),
- w = lateral width of obstacle (ft),
- s = lateral placement of obstacle (ft),
- d = width of vehicle (ft),
- θ = angle of encroachment (°),
- x = longitudinal distance from furthest downstream encroachment point to encroachment point of reference (ft), and
- $f(y)$ = probability density function of lateral displacements of encroaching vehicles.

Each integral expression given in the brackets in the hazard equation multiplied by $E_f/5280$ gives the number of fatal and non-fatal-injury accidents per year expected for each subdivision of the hazard envelope. Thus, the first expression in the brackets represents the contribution of the exposure length l and considers the probability of a vehicle lateral displacement greater than s. The second expression is the contribution of the exposure length $d\csc\theta$ to the hazard index. The third expression is the contribution of the exposure length $w\cot\theta$. The double integrals account for the varying lateral displacements of a vehicle required for collision.

Because of the limitations of the Hutchinson and Kennedy data, the model given by Equation 1 applies only to freeway situations. If a highway department wishes to mount a total roadside-safety program, one in which improvements are implemented in a cost-effective priority order, then the inputs of the model must also account for roadside hazards on highways other than freeways.

More recently, Glennon and Wilton (8) have successfully expanded the applicability of the hazard model. In its final form, the model can be used by a highway department in a cost-effectiveness methodology to (a) determine priorities for implementing roadside safety improvements on all classes of highways and (b) determine priorities for inventorying roadside hazards based on class of highway and average daily traffic. Although this research was limited to developing estimates of the necessary inputs and modifications to the original model, it can be used to predict the effectiveness of roadside-safety improvements on all classes of highways. The new formulation contributes additional information about the nature of vehicle encroachments and the severity indices of roadside hazards for all classes of highways other than freeways—urban arterial streets, rural two-lane highways, and rural multilane highways.

APPLICATIONAL PROBLEMS OF THE ROADSIDE-HAZARD MODEL

Since the publication of the original Glennon model (6), several state highway agencies have attempted to use it. Although the model is conceptually attractive and represents the most advanced analysis technology, and although some states are using some form of the model in their safety programming, other states have been skeptical and have proceeded as usual with some form of administrative wisdom in making decisions on roadside safety. The reasons why some states have rejected the model are as follows:

1. Many practicing highway engineers have not used advanced mathematics since leaving college and are accustomed to analysis and decision-making procedures that use good engineering judgment (whatever that is). Although the Glennon model is a straightforward mathematical formulation, and can also be approximated by a simpler algebraic form that uses probability statements and summations, these engineers have been perplexed by the mathematics.
2. In its most precise form of application, the model is somewhat more complex to use because it requires consideration of contiguous hazards (e.g., a steep embankment immediately behind a breakaway sign support).
3. The application of the model in roadside-safety programming requires a roadside-hazard inventory. Although this has been regarded as a formidable task, the early phases of implementation would require only an inventory of the more severe obstacles on the higher-traffic-volume highways.
4. The model is simplistic because of the nature of available input relationships. For example, the input relationships do not account for variances in the number of fatal and injury accidents based on the dimensions of highway geometric features (such as curvature and grade). Also, although the suggested severity indices vary according to the type of highway (number of lanes, divided or undivided, and urban or rural), they do not directly account for the specific operating speeds (or speed limits) of particular sections of highway. However, this lack of sophistication does not invalidate the model application, although it does make the results less than optimal.
5. Many practicing engineers are skeptical because the model does not demonstrate hazard through direct empirical results.
6. The input relationships developed by Hutchinson and Kennedy (4) and by Glennon and Wilton (8) have not been validated.

Both the Michigan and Maryland Departments of Transportation have sponsored further research in an attempt to improve on the Glennon model. The Michigan study (9) used a multivariate analysis that, with one exception, was unsuccessful in improving the technology but did find that, in general, both the frequency and severity of roadside accidents were higher on highway curves than on tangent sections. The Maryland study (10) generally accounted for the contributions of highway geometrics and operating speeds to roadside hazard, but not in a way that could be incorporated into an objective hazard formulation.

POSSIBLE APPROACHES TO IMPROVING ROADSIDE-SAFETY ANALYSIS

The discussion above illustrates that there are recognized impediments to the application of the Glennon model. To move ahead in the cost-effective implementation of roadside-safety improvements, therefore, will require additional empirical research to either validate and improve the precision of the Glennon formulation or to completely replace it with a more explicit method.

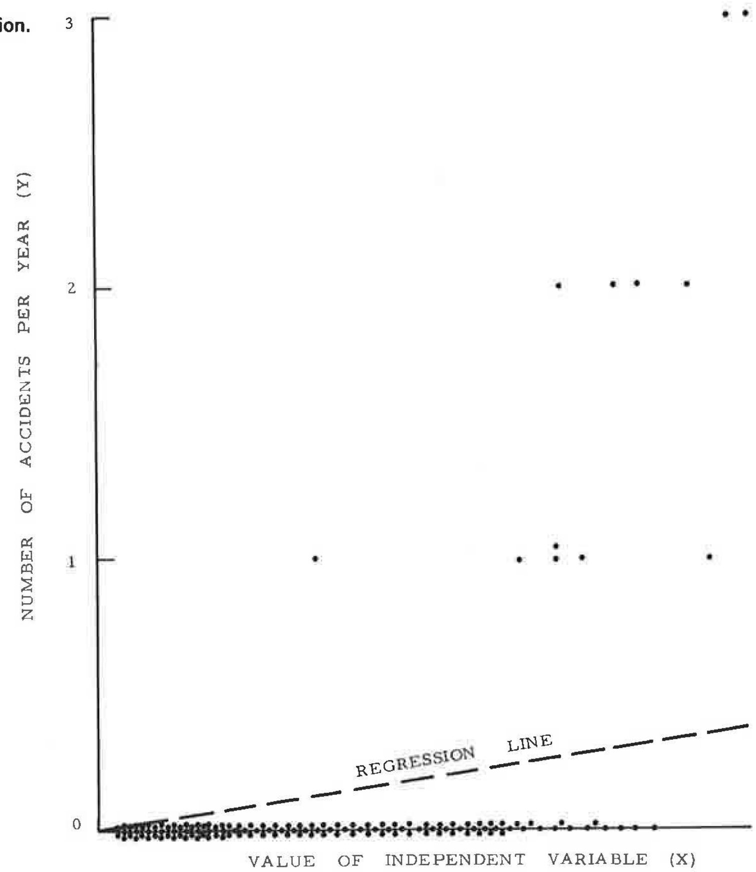
The prediction of the hazard associated with any particular roadside obstacle by using empirical studies is difficult because of both the large number of variables involved and the extremely low probability of a collision with that obstacle. For example, the Glennon model predicts that a concrete bridge column 3.05 m (10 ft) from the edge of the traveled way of a freeway that carries 100 000 vehicles/day will average one fatal or non-fatal-injury accident about every 10 years. Similarly, a rigid light pole 6.1 m (20 ft) from the edge of the traveled way on a freeway that carries 10 000 vehicles/day is expected to average only one fatal or non-fatal-injury accident every 167 years. However, although these probabilities may seem too low to be concerned about, the cost-effectiveness of roadside-safety improvements for the larger, more hazardous obstacles that are placed relatively close to relatively high-volume highways has been clearly demonstrated (6).

Consideration of Standard Multivariate Analysis

Whenever a response variable such as accident rate or number is believed to be a function of several independent variables, the most common study approach is to collect large amounts of data and use a multivariate analysis technique such as multiple regression. A review of several studies in the highway-safety area, however, indicates the futility of these types of studies, even for relatively high-probability situations. And, of course, the indications are that the frequencies of accidents at particular roadside locations are very much smaller than are those for many other situations (such as intersection accidents).

Because roadside accidents are very low-probability events, an attempt to use a standard multiple-regression technique would have problems because of the discrete nature of the dependent variable, i.e., number of accidents per year. This can be illustrated by a simple example, as shown in Figure 2. In this figure, it is assumed that there is one independent variable

Figure 2. Illustrative frequency distribution.



(X) and that the typical 1-year accident response (Y) will be one in which most of the locations sampled have had zero accidents and only a few have had one or more accidents. This type of frequency distribution will produce a regression line such as the one shown. From this simple graphical example, it can be seen that all probability statements about a bad location (one that has a large Y) will be of questionable validity because the actual Y is very remote from the regression line.

Another problem in using regression analysis in this context is that this type of analysis is a continuous representation, whereas many of the candidate independent variables are discrete (e.g., type of roadway, type of object, urban or rural). Thus, the discrete nature of both the dependent and independent variables suggests that another way to model the problem is to create categories for the continuous Xs and build a discrete prediction model rather than a continuous one.

Consideration of Bayes Theorem

The Reverend Thomas Bayes (1702-1761) considered how one might make inferences from observed sample data about the larger groups from which the data were drawn. His motivation was his desire to prove the existence of God by examining the world around him. Mathematicians had previously concentrated on the problem of deducing the consequences of specified hypotheses. Bayes was interested in the inverse problem of drawing conclusions about hypotheses from observations of consequences. He derived a theorem that calculated probabilities of causes based on observed effects.

Bayes' theorem is really nothing more than a statement of conditional probabilities:

$$P(B_r|A) = P(B_r) \times P(A|B_r)/P(A) \qquad (2)$$

where

B_i = $B_1, B_2, \ldots B_r, \ldots B_n$ constitute any partition of the sample space,
$P(B_i) \neq 0$ for every i, and
$P(A) \neq 0$.

In other words, the conditional probability of B_r given A is known if all the reverse conditional probabilities and all the unconditional probabilities are known.

This theorem appears to be a more promising approach to the solution of the problem of better predicting the hazard of a roadside obstacle.

Suggested Study of Roadside Hazards

The study suggested here is intended to validate and add precision to the Glennon model for roadside-hazard prediction. It would include an experimental design, a large-scale inventory of roadside obstacles, collection of accident records to match the obstacle inventory, and an analysis that used Bayes' theorem. (Rather than a full-scale study, an alternative plan might be to conduct a pilot study for one kind of roadway or one kind of roadside obstacle. The remainder of this discussion, however, assumes a full-scale study.)

To be statistically tractable, the roadside inventory would require a fairly massive effort. This effort might be facilitated, however, by using the fixed-object inventory recently (1974) mandated by the U.S. Department of Transportation or state photo-logging records. It would also be necessary to ensure that the candidate independent variables included samples across their entire dimensional ranges. The candidate independent variables are listed below.

General Highway Variables	Design Feature Variables	Roadside Obstacle Variables
Traffic volume	Number of Lanes	Type of obstacle
Environment (urban or rural)	Median width	Lateral placement of obstacle
Type of access control (full, partial, or none)	Shoulder width	Side of placement (median or right side)
Divided or undivided	Degree of curvature	Length of obstacle
	Percentage grade	Width of obstacle
	Presence of curbs	
	Special design features (intersections, gores, and such)	

Accident data could be gathered from statewide computer files. This compilation should be limited to a 1- to 3-year sample to avoid errors caused by highway-design changes. Single-vehicle injury (fatal plus nonfatal) accident records would be matched to inventory records by milepost location and type of obstacle. Although these kinds of accident studies have been known to be unreliable, the major source of error, accident-reporting level, is not a particular problem here because only the more severe accidents (which have higher reporting reliability) are of interest.

In the application of Bayes' theorem to the analysis of the study data, each roadside location is considered to belong to one of two accident classes: good ($Y = 0$) or bad ($Y > 0$). Although there are no formal reasons to not consider all integer values of Y, the number of locations that would have more than one accident is expected to be so small that the estimates would be unreliable.

To see how Bayes' theorem fits the problem at hand, assume temporarily that only one independent variable—X, which has categories $j = 1, 2, \ldots, k$—is necessary to predict the number of accidents. The required estimate is the probability that a location will have one or more accidents in the specified time period given $X = j$. According to Bayes' theorem, this probability is

$$P(Y>0 | X=j) = P(Y>0) \times P(X=j | Y>0)/P(X=j) \quad (3)$$

A similar expression exists for $P(Y=0 | X=j)$.

Once the categories for an independent variable are selected, it is easy to estimate $P(Y>0|X=j)$ for that variable. The probability, $P(X=j|Y>0)$, is the fraction of locations that are in category j out of the bad-location ($Y > 0$) class. The unconditional probability, $P(Y > 0)$, is simply the fraction of total locations in the bad-accident class. And the unconditional probability, $P(X=j)$, is simply the fraction of total locations exhibiting $X = j$.

The next step in the development of the prediction model is to provide for estimating the probability of more than zero accidents given the set of independent-variable values for each roadside location. This is accomplished by assuming independence of the X_is. For example, for three independent variables,

$$P(X_1=2, X_2=1, X_3=4 | Y>0) = P(X_1=2 | Y>0) \times P(X_2=1 | Y>0) \times P(X_3=4 | Y>0) \quad (4)$$

Of course, it would be ideal to avoid the assumption of independence, but to do so would require an analytical description of all the dependencies among the Xs. This is certainly not available from anything less than a very large data set, if it is available at all. Although the independence assumption is exactly equivalent to the additivity assumption used in standard multiple-regression analyses, rather than trying to account for these dependencies, this study could select the variables in such a way as to avoid any logical dependencies. Given the independence assumption then, the model would take the general form,

$$P(Y>0 | X_1=j, \ldots X_n=k) = P(Y>0) \times P(X_1=j, \ldots X_n=k | Y>0) \div P(X_i=j, \ldots X_n=k) \quad (5)$$

This model, which simply estimates whether a roadside location is in the bad population, can be transformed into a more explicit model that estimates the unexpected number of accidents by using the assumption that accidents follow a Poisson distribution. By using the previously derived relationships in the Poisson equation, one obtains the following derivation of the expected number of fatal-plus-nonfatal accidents (λ):

$$P(Y=0 | X_1=j, \ldots X_n=k) = e^{-\lambda}(\lambda)/0! = e^{-\lambda}$$

$$1 - P(Y>0 | X_1=j, \ldots X_n=k) = e^{-\lambda}$$

$$\lambda = -\ln\{1 - [P(Y>0) \times P(X_1=j, \ldots X_n=k | Y>0)/P(X_1=j, \ldots X_n=k)]\} \quad (6)$$

Conceptually, then, this model gives a value equivalent to that of the Glennon model. To validate the Glennon model (or conversely to validate this model by using the Glennon model) requires that the new model be estimated by using only those variables expressed in the Glennon model or its available inputs (e.g., type of highway, average daily traffic, type of obstacle, length of obstacle, width of obstacle, and lateral placement of obstacle). If a reasonable level of correspondence is found, then the more explicit form of the new model can be judged to be the best available representation of roadside hazard.

CONCLUSION

The proposed Bayesian approach to the analysis of roadside hazard merits further investigation. It offers a potential model that could be directly supported by empirical data, rather than a nonvalidated conceptual model of how component events of a roadside accident are conditionally related. It also has the potential for a more precise formulation because the developmental data collection could also consider the relationships of roadside accidents to geometric and traffic-operating variables.

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Procedure for the Evaluation of Completed Highway-Safety Projects

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The highway-safety engineer must constantly make crucial decisions involving the selection and implementation of safety-improvement countermeasures. To facilitate decisions regarding the continuation, addition to, or deletion of various types of highway-safety programs, valid effectiveness evaluations of completed safety projects should be conducted and made available to other engineers. Critical to the decision-making process are quantitative answers as to whether or not the project is accomplishing its intended purposes, how the purposes are being accomplished, and whether the project is producing unexpected or contrary results. Without the evaluation of individual projects, the effectiveness of highway-safety programs cannot be determined and limited safety funds cannot be allocated to those programs that are most effective in saving lives and reducing injuries and property damage. Too often, effectiveness-evaluation efforts are deemphasized because of monetary and staff constraints and the absence of a single, comprehensive procedure, designed specifically for the evaluation of deployed highway-safety countermeasures. In this study, the literature and current practices relative to effectiveness evaluations were examined to determine whether or not existing techniques and methods are appropriate for use in a single methodology for the evaluation of various roadway- or roadside-improvement projects. It was concluded that existing techniques are appropriate but that they should be organized into a structured procedure that would be practical for use by engineers and highway-safety personnel. This paper describes the procedure developed from state-of-the-art techniques for performing effectiveness evaluations of various types of completed highway-safety projects.

National highway-accident statistics (1) indicate that the annual number and rate of traffic-accident deaths have declined to their lowest levels since the early 1960s. This, together with the fact that annual vehicle kilometers of travel have generally increased throughout the same period, indicates that positive gains are being achieved from recent highway-safety efforts. In general, programs aimed at improving highway conditions, vehicle designs and driver awareness are responsible for the improvement in highway safety.

Transportation programs administered by the Federal Highway Administration (FHWA) are directed toward reducing traffic-accident fatalities, injuries, and property damages attributable to highway-system failures (as opposed to vehicle or driver failures). To create a hazard-free highway system, FHWA has developed a collection of highway-safety programs that consists of a full range of projects and types of improvements. These projects include improvements at railroad-highway crossings, installation of pavement markings, improvements at high-hazard locations, and elimination of roadside obstacles. On an aggregate basis, these projects have definitely affected the number and severity of traffic accidents. However, the extent to which individual projects and types of improvements have affected the accident experience at specific locations is not fully known. Thus, the effectiveness of individual projects and improvements needs to be determined. This could be accomplished by conducting effectiveness evaluations of existing highway-safety treatments.

The need to conduct effectiveness evaluations is generally recognized by the highway-safety profession. In fact, evaluation data on project effectiveness is required for all federal-aid safety projects. All too often, however, effectiveness-evaluation efforts are deemphasized because of monetary and staff constraints and the absence of a single, comprehensive procedure capable of evaluating the full range of possible highway-safety treatments.

This paper describes a procedure that was developed specifically for evaluating highway-safety projects. It is based on existing state-of-the-art techniques and procedures and is intended for use by practicing state and local highway-safety engineers for conducting intensive effectiveness-evaluation studies of completed highway-safety projects. The development of the procedure included the development of a guide (2) and a 3-day training session and workshop for practicing highway-safety engineers.

EVALUATION PROCEDURE

A highway-safety project, in the context of the evaluation procedure, is defined as a roadway- or roadside-safety improvement that has been implemented to affect the frequency, rate, or severity (or a combination thereof) of traffic accidents. For a project to be considered a safety improvement, traffic-accident reduction must be its primary raison d'être, although the improvement of traffic operations is allowable as a secondary effect. A project can be composed of one or more countermeasures, implemented at an intersection or on an extended roadway section. A project can also consist of several locations, each of which are treated by a similar countermeasure or set of countermeasures.

The initial step in the development of the procedure consisted of a literature review and a current-practices survey. Existing valid evaluation methodologies were identified, and deficiencies in current and past evaluation procedures were defined. It was found that the evaluation studies conducted thus far often suffer from deficiencies in traffic-accident-recording systems, inappropriate experimental plans, lack of statistical testing procedures, misinterpretation of evaluation results, or the absence of proper documentation and dissemination of results. It was, however, concluded from the literature review that the current state of the art has the sophistication to allow the development of an evaluation methodology that can overcome the deficiencies of past evaluation efforts.

The evaluation procedure was designed to provide a logical structure for assessing the effectiveness of a highway-safety project. It consists of six functions, each formulated into a series of systematic steps that guide an evaluator through the activities and decision-making processes of a properly designed evaluation study. Worksheets and data forms were developed to aid the evaluator in organizing the data. The functions are

Function	Description
A	Develop evaluation plan
B	Collect and reduce data
C	Compare measures of effectiveness
D	Perform tests of significance
E	Perform economic analysis
F	Prepare evaluation documentation

Function A: Develop Evaluation Plan

Function A addresses fundamental planning activities that should be considered before the evaluation study is performed. The project purposes, evaluation objectives, measures of effectiveness (MOEs), experimental plans, and data-collection schemes are examined in this function. This function is designed as a guide for the establishment of future evaluation activities for programmed projects as well as for the organization of a plan for evaluating completed projects.

The project purposes are based on the identified safety deficiency at the project site and the types of accidents that the countermeasures are expected to reduce. They may include the reduction of total accidents, of accidents of a specific type, or of specific accident-severity categories, or combinations thereof.

The project purposes serve as the basis for the selection of the evaluation objectives and the MOEs. Evaluation objectives are statements that reflect the specific type of accident or category of severity to be evaluated in the study. An objective can correspond to a project purpose or any other accident-severity or traffic-performance measure. However, as a minimum, the evaluation objectives should include the effects of the project on total, fatal, injury-producing, and property-damage accidents. The evaluation of these fundamental objectives allows for the determination of the effect of the project on the overall accident picture. MOEs are expressed as the percentage change in rate of occurrence of each evaluation objective. For example, objectives for a project in which a traffic signal was installed might be to test the effect of the new signal installation on total, angular, rear-end, fatal, and injury-producing accidents by using MOEs that relate to the percentage change in accident rates for each of the accident categories cited.

Four experimental plans were selected for inclusion in the procedure. These plans provide the evaluator with an analytical framework for the evaluation. The experimental plans are

1. A before-and-after study that includes control sites (B-A and C),
2. A before-and-after study (B-A),
3. A comparative parallel study (CP), and
4. A before-, during-, and-after study (B-D-A).

The B-A and C plan can be used when there are available several unimproved sites (control sites) that are similar to the improved site before the project was implemented. This plan allows the evaluator to control for such factors as weather, road conditions, and enforcement. The B-A plan can be used when control sites are not available. When the quality of accident data is suspect, the CP plan can be used. In this plan, before accident data are not required; however, control sites must be available. The B-D-A plan can be used for temporary or experimental improvements that will be removed or discontinued. To aid the evaluator in the selection of an appropriate experimental plan, the function includes a general set of criteria based on control-site availability, accident-data quality, and staff resources. In addition, it includes a discussion of pro-and-con aspects and the assumptions and applicability of each plan.

Finally, the function describes the data-collection scheme including identification of required data variables, sample sizes, and data-collection techniques needed.

Function B: Collect and Reduce Data

Function B provides guidance in collecting and reducing the data according to the data-collection scheme developed in function A. The use of three years of accident data, both before and after project implementation, is recommended. Guidance in identifying and selecting control sites is also provided in this function.

Function C: Compare Measures of Effectiveness

Function C presents the analytical techniques for determining the changes in the MOEs of the study. These techniques vary, depending on the experimental plan selected for the evaluation. Components of the MOE comparison process that are determined in this function are (a) the expected MOE values if the project had not been implemented, (b) the percentage change in each MOE, and (c) the expected accident frequency if the improvement had not been made. The latter two components are used directly in the statistical procedure for testing the significance of the changes in the accident-related MOEs.

Function D: Perform Tests of Significance

Function D provides four statistical procedures for testing the significance of the changes observed in the MOEs. The Poisson test was selected for testing changes in accident-related MOEs. This technique requires the percentage changes and expected accident frequencies for the do-nothing alternative determined in function C. A set of criteria based on project cost and the cost of a type 1 error versus that of a type 2 error is provided to assist in the selection of an appropriate level of statistical confidence. Three additional statistical techniques are provided for testing MOEs related to traffic-performance variables. These include tests for assessing changes in (a) discrete-variable MOEs such as the change in the proportion of vehicles exceeding the speed limit—i. e., a test of proportions, (b) continuous-variable MOEs such as the change in average intersection delay—i. e., a t-test, and (c) variance-related MOEs such as the change in speed variance—i. e., an F-test.

Function E: Perform Economic Analysis

In function E, two economic-analysis techniques are provided: the benefit-cost ratio and the cost-effectiveness. The procedure recommends that one of them be applied to all MOEs found to be significantly changed at the level of confidence selected for the evaluation. The use of these techniques enables the evaluator to perform a fiscal evaluation of project effectiveness. A set of criteria based on the types of projects and the evaluation objectives to be considered is provided to assist in the selection of an economic-analysis technique. Each technique is then described by a step-by-step procedure. Current accident-cost figures developed by the National Safety Council and the National Highway Traffic Safety Administration are provided for use in the procedure. However, agencies that have adopted a specific set of cost figures are encouraged to use them. Also, recommendations on service lives, interest rates, salvage values, and other economic components are provided.

Function F: Prepare Evaluation Documentation

Because the full benefit of conducting evaluation studies depends on disseminating the results to other safety engineers, function F is included in the procedure to provide guidance in interpreting results and documenting both successful and unsuccessful highway-safety projects. Guidance is also provided for the development of data bases of effectiveness results of various types of projects, stratified by surrounding land use, type of roadway, type of location, and ranges of traffic volumes and other variables that may relate to the project site and its environs.

CASE STUDY

As part of the project in which the procedure was developed, five case studies were prepared to illustrate the use of the evaluation methodology and to serve as instructional aids in the training session and workshop. One abbreviated case study, based on a skid-resistance treatment to a section of an Interstate freeway, is discussed here to demonstrate the evaluation procedure.

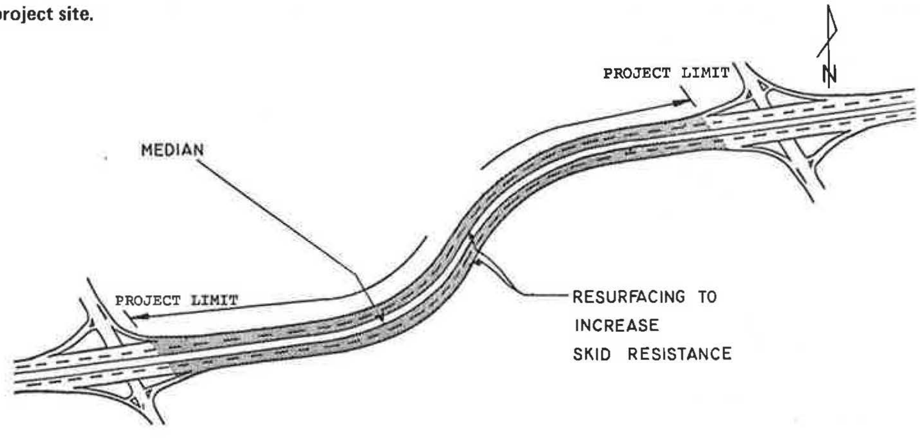
The project site consisted of a 1.4-km (0.9-mile) section of a four-lane, divided, rural Interstate highway that was resurfaced to increase its skid resistance and reduce the high number of total and wet-surface accidents. The project consisted of applying a 2.54-cm (1-in) open-graded asphalt-friction-course overlay to the existing pavement surface. The project site (see Figure 1) was treated in June 1973 at a cost of \$80 000. The cost of maintaining the section after project implementation averaged \$200/year for the first 3 years. Accident records were obtained for an analysis period that included 3 years before and 3 years after project implementation.

Function A: Develop Evaluation Plan

A review of the traffic-accident summaries for the 3-year before-project period indicated that 59 of the 123 accidents (48 percent) that had occurred were wet-surface accidents. This percentage was high in comparison with other sections of the Interstate system in the area, which averaged only 10 percent. Also, the percentage of the total of wet-surface accidents at the study site that involved injuries or fatalities was 56 and the area average was 30. Based on these findings and the nature of the implemented project, the purposes of the project were determined by the evaluator to be

1. To reduce total accidents,
2. To reduce wet-surface accidents, and
3. To reduce wet-surface accidents that involved injuries or fatalities.

Figure 1. Case-study project site.



The project purposes were recorded on a project-purpose listing form, and the objectives and MOEs of the evaluation were identified. The objectives included the determination of the effect of the skid-treatment project on

1. Total accidents,
2. Fatal accidents,
3. Injury-producing accidents,
4. Property-damage accidents,
5. Wet-surface accidents that involved injuries and fatalities, and
6. Wet-surface accidents.

(Objectives 1-4 are selected for all evaluation studies. Additional objectives can be based on project purposes or other accident types of interest to the evaluator.)

Rate-related MOEs were chosen because average annual daily traffic volumes were available for 3 years before and 3 years after project implementation. Because the project site is an extended roadway section, the exposure factor selected was vehicle travel for both directions of travel combined. The MOEs for each listed objective include the percentage changes in

1. Total accidents per unit of vehicle travel,
2. Fatal accidents per unit of vehicle travel,
3. Injury-producing accidents per unit of vehicle travel,
4. Property-damage accidents per unit of vehicle travel,
5. Wet-surface accidents that involved injuries and fatalities per unit of vehicle travel, and
6. Wet surface accidents per unit of vehicle travel.

The evaluation objectives and the corresponding MOEs were recorded in an objective-and-MOE listing form for future reference.

In selecting the experimental plan, it was recognized that there might be differences in the number of inclement days occurring during the before and the after periods and that this factor would affect the wet-surface accident experience at the project site, regardless of the skid treatment. Thus, a B-A-and-C plan was selected to compensate for climatic variations. The use of this type of study was considered feasible in terms of the available time and staff of the evaluating agency and the availability of similar sites.

Data needs were next established to facilitate the data-collection process. Accident data at the project and control sites (to be selected in function B) for each MOE were specified for the analysis periods of 3 years before (July 1970 to June 1973) and 3 years after (July 1974 to June 1977) project implementation. A construction and adjustment period of 1 year (July 1973 to June 1974) was used. Annual traffic-volume data were also specified for the two 3-year analysis periods, along with environmental and highway-features data for the project and control sites. All data to be used in the evaluation were recorded in a data-requirements form.

Function B: Collect and Reduce Data

Control sites were selected that were similar to the project site in terms of the following key variables:

1. Percentage of wet-surface accidents,
2. Type of pavement surface,
3. Number of lanes,
4. Posted speed limit,
5. Horizontal alignment,
6. Grade, and
7. Skid number.

Volume data for all sites were obtained from state traffic-volume data files and tabulated on traffic-volume summary tables.

Wet-surface-accident data for the control sites were checked for completeness and accuracy and tabulated on accident-summary tables.

An investigation was made for each project and control site to determine whether environmental or highway-feature changes had taken place during the analysis periods. It was recognized that the 88.5-km/h (55-mph) speed limit was imposed in October 1973, but this was considered to be of no consequence because this variable was common to both test and control sites. No other major changes at the sites were identified that would affect accident experience.

Function C: Compare Measures of Effectiveness

An MOE data-summary form was prepared for the experimental plan of the evaluation to show the MOE data to be evaluated. Based on the data contained in the MOE data-summary table, expected rate-related MOEs and percentage changes were calculated for all MOE variables and recorded in the MOE data-summary form. The following equation was used to determine the expected rate-related MOEs.

$$E_R = B_{PR}(A_{CR}/B_{CR}) \quad (1)$$

where

E_R = expected MOE rate at the project site if the improvement had not been made,
B_{PR} = MOE rate at project site during before-project period,
A_{CR} = MOE rate at control sites during after-project period, and
B_{CR} = MOE rate at control sites during before-project period.

For illustration purposes, the expected MOE rate for total accidents is calculated below. The MOE rate at the project site for the before-project period was 3.05 accidents/million vehicle-km (4.88 accidents/million vehicle miles) and the before-project and after-project rates at the control sites were 2.63 and 2.54 accidents/million vehicle-km (4.20 and 4.06 accidents/million vehicle miles), respectively.

E_R = 3.05(2.54/2.63) = 2.95 total accidents/million vehicle-km (4.72 total accidents/million vehicle miles).

The percentage changes were determined by using the following equation:

$$\text{Percentage change} = [(E_R - A_{PR})/E_R] \times 100 \quad (2)$$

where A_{PR} = MOE rate at project site during after-project period.

The percentage change in the total-accident MOE rate was determined by using the expected MOE rate found above and an after-project-period total-accident rate of 2.25 accidents/million vehicle-km (3.60 accidents/million vehicle miles) at the project site as shown below:

Percentage change = $[(2.95 - 2.25)/2.95] \times 100 = 23.7$.

Percentage reductions were calculated for the remaining MOEs and tabulated in an MOE comparison table.

Next, the expected before-project-period accident frequencies were determined for statistical testing purposes. The following equation was used to transform the expected rate-related MOE to the expected before-project-period accident frequency.

$$E_F = E_R \text{ (after-project-period average daily traffic} \times 365 \times T_A \times L_p)/10^6 \quad (3)$$

where

E_F = expected before-project-period accident frequency,
T_A = length of time of after-project period, and
L_p = section length for the project site.

The expected 3-year accident frequency for total accidents was then calculated as illustrated below. In this case, the 3-year average daily traffic was 14 230 vehicles for the 2.9-km (1.8-mile) project site.

$E_F = 2.95(14\,230 \times 365 \times 3 \times 2.9)/10^6 = 133.3$ total accidents for 3 years.

Expected before-project-period accident frequencies were calculated for each MOE.

Function D: Perform Tests of Significance

The implementation cost of the skid-resistant-overlay project was \$80 000; this was a moderately cost-intensive project in relation to other highway-safety projects. Therefore, neither an extremely high nor an extremely low level of confidence is required. It was decided that the 90 percent level of confidence was appropriate for the evaluation.

Inputs to the statistical analysis from function C (percentage changes and expected 3-year before-project-period frequencies) were tabulated and recorded in a statistical-test summary table. The Poisson test was used to determine the significance of change in each MOE. Most of the MOEs were found to be significantly reduced at the 90 percent level; the injury-producing-accident rate was not, and the fatal-accident rate was found to be too small to test.

Function E: Perform Economic Analysis

The project was subjected to an economic analysis because statistically significant reductions were observed in some of the MOEs. The cost-effectiveness (C/E) approach was selected by the evaluator because its results indicate the cost incurred by the agency to reduce the total number of accidents by one. This result was considered more useful for this particular study than a benefit/cost ratio (which shows the effectiveness in reducing the accident severity at the site). The analysis used equivalent uniform annual costs and benefits. A C/E analysis worksheet was used to perform the analysis.

The economic analysis indicated an average cost of \$1260/accident reduced/year.

Function F: Prepare Evaluation Documentation

All evaluation materials—listings, raw data, reduced data, and analysis results—were collected together for the purpose of interpreting the results of the evaluation and writing the evaluation report.

The results of the comparison of MOEs (function C) were reviewed to determine whether the project purposes had been satisfied. Based on the accident rates found, the purposes of reducing the total and the wet-surface accidents had been satisfied. Furthermore, the MOEs of the evaluation objectives were found to be statistically significant at the selected level of confidence (90 percent) except for the fatal- and injury-producing accident rates, which were not significantly reduced. The economic analysis indicated a cost to the agency of approximately \$1260/accident reduced/year.

The evaluation process was completely reviewed for appropriateness in testing each of the study objectives. It was concluded that the evaluation results were valid and appropriate for inclusion in an aggregate data base on the effectiveness of skid-proofing-overlay projects.

The final evaluation report was prepared by completing the final report form.

CONCLUSION

Although the current state of the art of highway-safety evaluation was found to contain adequate techniques for evaluating highway-safety projects, there was no single procedure that provided a step-by-step guide for evaluating completed projects. The absence of such a procedure was found to be a major factor in the current deficiencies associated with evaluation.

It is believed that the evaluation procedure presented in this paper will provide the highway-safety profession with a valuable tool that, when widely used, will significantly advance the highway-safety improvement process. For the procedure to obtain its widest use, practicing engineers must be exposed to and trained in its application.

ACKNOWLEDGMENT

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Use of Traffic-Conflicts Technique to Assess Hazards of Transporting Oversize Loads

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The traffic-conflicts technique was used to assess the hazards associated with transporting oversize loads over highways. The approach was based on the assumption that a driver applies brakes in response to a perceived danger when following, passing, or meeting other vehicles. Field tests were conducted to determine whether there are any differences in the hazards involved in moving 3.7-m (12-ft) wide housing units as compared with those of moving 4.3-m (14-ft) wide units. An analysis of the conflicts indicated that there were no major differences; however, the sample size was too small for the results to be accepted with a high degree of confidence. Although the conflicts data indicated that the large sample sizes needed to establish statistically reliable results may not be practical, the technique was useful in determining the types and relative frequencies of hazards associated with the movement of wide loads over a variety of highway systems. As a measure for assessing the hazards of moving wide loads, the conflicts technique provided more detailed information in a short period of time than could have been obtained from a conventional accident analysis.

This assessment of the hazards involved in moving oversize loads over highways was initiated when the Virginia legislature requested an evaluation of the potential effects of moving 4.3-m (14-ft) wide housing units over the state's roads. In Virginia, and in most states, the movement of oversize mobile and modular housing units is regulated to protect the motoring public from unnecessary hazards and inconveniences. However, there has been little study of the effects of oversize loads on other traffic. The only comprehensive study of the movement of 3.7- and 4.3-m (12- and 14-ft) wide housing units was conducted in 1973 by the Midwest Research Institute. The results of the study (1) suggested that "the question is not a simple one and, unfortunately, the data obtained in this study do not clearly show that states should or should not allow [4.3 m] 14-foot wide loads."

One of the primary concerns about the movement of wide loads is their effect on the safety of the traveling public. Although accident data seem to indicate that 3.7-m-wide loads are seldom involved in reportable accidents, it has been suggested that wide loads create causal factors that lead to accidents in which they are not directly involved (1, 2). Because of the rarity of reported accidents and the difficulty of obtaining accurate exposure (vehicle kilometers of travel) and accident information, the question of safety in moving wide loads is unresolved.

Although the use of accident data provided a poor measure for the evaluation of the safety of moving 3.7-m-wide loads, no data were available about 4.3-m-wide units because these units were prohibited in Virginia. Accident reports from states that allow travel of the 4.3-m-wide units provided little information. Because the Virginia legislature's resolution required that the evaluation be conducted within a five-month period, it was necessary to use a measure of safety other than accident reports.

The purpose of this paper is to present the methodology and results of field tests conducted to identify the type and frequency of hazards that occur during the transportation of oversize housing units. The specific objectives of the study were to examine the hazards that affect the traveling public, the wide load itself, and the highway system and to determine whether significant differences exist between hazards presented by 3.7-m-wide units and those presented by 4.3-m-wide units.

ALTERNATIVE METHODS OF MEASURING HIGHWAY SAFETY

Conventional methods of measuring highway safety rely on the use of accident records. Although accident data are widely accepted measures, there are a number of disadvantages in using them to measure safety. These disadvantages include (a) incomplete or inaccurate reports, (b) the high percentage of accidents not reported, (c) year-to-year fluctuations in numbers of accidents, (d) the failure of the records to identify specific problems and causal factors, and (e) usually of greatest importance, the long time interval required to accumulate a sufficient amount of data after any change is made in the environment-vehicle-driver system (3). Because of these deficiencies, the need for additional measures of safety that can be used to supplement accident data has long been recognized.

In recent years, several non-accident-based methods of measuring highway safety have been developed. Because the legislature required that the hazards of moving oversize loads be evaluated within a short period of time, the number of possible safety indicators that could be used was limited. Based on a review of the literature, the following techniques were considered: (a) acceleration noise, (b) erratic maneuvers, (c) near-miss events, and (d) traffic conflicts. Although all of these measures have been used in other studies, they have not been used to assess the type and frequency of hazards that occur during the movement of a load along a highway.

Acceleration noise has been used as a traffic parameter that describes the hazard of driving on a particular highway (4). Because of the instrumentation problems imposed by using a variety of mobile and modular units, the method was not used for the present study; another disadvantage of the method is that the technique is not sensitive to the hazards posed by a moving load to other traffic and the highway system.

To examine the applicability of either erratic maneuvers, near-miss events, or traffic conflicts, several test runs that used 3.7-m-wide loads were conducted. The results of the pilot study are summarized below.

Broadly defined, an erratic maneuver is an unusual single-vehicle movement. Current application of the technique has been limited to evaluating the movements of vehicles at exit gore areas (5). Because very few unusual single-vehicle maneuvers were encountered during the test runs, the use of erratic maneuvers as an evaluative tool was eliminated from consideration.

Because the primary hazards observed during the test runs were vehicle-load interactions, the possibility of modifying either the near-miss criterion or the traffic-conflicts technique was explored.

A near-miss event is defined as a serious traffic conflict in which the measured minimum time to collision of two vehicles is equal to or less than 1 s (6). Although near-miss events have thus far been used only for the assessment of the hazards of vehicle interactions at intersections, the concept of developing a minimum time to collision for vehicles in motion along a highway appeared to be possible. However, in nearly 644 km (400 miles) of travel during the test runs, no critical near-miss situations were observed. Consequently, the technique, as well as the possibility of developing a minimum time to collision for moving loads, was eliminated from consideration.

TRAFFIC-CONFLICTS TECHNIQUE

The traffic-conflicts technique was developed at the General Motors Research Laboratories as a method of observing and recording potential accident maneuvers at intersections. A traffic conflict is an evasive maneuver by a driver who either brakes, as indicated by a brake-light signal; changes lanes to avoid a collision; or commits a violation of the uniform traffic code. Specific conflict criteria and study procedures for intersections are described in the literature (7).

The theoretical basis for using conflicts data as a measure of safety is that conflicts describe hazards that can lead to accidents. Several workers have discovered significant relationships between conflicts and accidents at intersections (8-10). Also, conflicts have been found to be heavily dependent on traffic volume (11).

Unlike accident data, which may take several years to collect, conflicts data can be collected in a short time. However, a major disadvantage is that the definition of a conflict is subjective and conflict counts taken simultaneously by different observers at the same location can vary. Also, a recent evaluation of the technique showed that the large sample sizes required to establish reliability and utility of the analysis may not be practical (12).

In addition to its use at intersections, the traffic-conflicts technique has been applied with some modifications to the diagnosis of problems and the evaluation of the effects of various treatments at gore areas on freeways, long upgrades in mountainous terrain, and driveways (10, 12).

APPLICATION TO MOVEMENTS OF OVERSIZE LOADS

Although the traffic-conflicts technique had not previously been applied to the examination of the hazards associated with a moving oversize load, the results of the pilot study indicated that it might be applied successfully. Thus, on the assumption that a driver applies brakes in response to a perceived danger when following, passing, or meeting a wide load, the technique was used to evaluate the hazards in moving oversize loads.

Definition

One of the first requirements of the study was the development of a suitable definition of a conflict that could be used for all moving loads. Based on the preliminary tests, a traffic conflict was defined as an evasive maneuver, as evidenced by a brake-light indication, taken by a driver operating a vehicle in the vicinity of a wide load. The definition also was taken to include evasive maneuvers by a driver pulling a wide load in the vicinity of other traffic or narrow roadside obstructions (fixed objects). It did not include braking because of traffic-control devices (such as traffic signals and stop signs) or conflicts between wide loads and their escort vehicles (because escorts were considered to be integral components of the load). In addition, violations of the traffic code, e.g., driving to the left of a double solid centerline, were not taken as constituting conflicts. No attempt was made to define the severity of conflicts because the objective of the study was to identify all hazards.

Method

To provide a basis for evaluation, conflicts data were taken for both 3.7-m-wide units—the standard product—and 4.3-m-wide units—the product being evaluated. The housing units evaluated consisted of mobile, double-wide, and modular sections transported under permit in accordance with Virginia regulations for oversize loads. Housing manufacturers throughout the state provided the units, drivers, and escorts. Most of the units were driven from the plant to their final destination; however, occasionally, one was specially routed to include a variety of roadway geometric and traffic conditions. The study routes consisted of sections of Interstate, multilane and two-lane primary, and secondary routes. The sections were selected from routes throughout the state and represented a broad range of traffic, geometric, land-use, and environmental conditions. Both sizes of units were transported over each study route.

Data Collection and Reduction

Traffic-conflicts data were collected by a four-person crew that used two 16-mm cameras. A driver and a photographer rode in each of two unmarked research vehicles, one driven approximately 0.4 km (0.25 mile) ahead of the wide load and the other the same distance behind the load. As a vehicle approached or passed the load, the cameras were activated and the entire interaction was filmed. The purposes of filming from two directions were to maximize the number of observations and to provide as full as possible coverage of the vehicle-load interactions. Other data recorded on film included vehicle and load lateral placements and load encroachments at intersections and roadside obstacles. In addition to the film data, manual counts were made of the vehicles that formed queues behind the loads or passed or approached from the opposing direction of travel. Other data collected manually included load and roadway dimensions, load speeds, queue lengths and impedance times, vehicle passing times, and violations of permit regulations. The data were collected from August 16 to October 7, 1967, between 9:00 a.m. and 4:00 p.m. on Mondays through Thursdays.

During the planning of the research, there was concern that the presence of the vehicles and cameras used to collect the data would influence the behavior of the wide-load operator, the escort-vehicle operators, and the traveling public. Therefore, several practice trips were made with the research vehicles at various distances from the wide load and citizens' band radio transmissions were monitored to determine the relative interest of the public in the research vehicles and their operations. Only a few persons with radios noticed the study team and relayed their findings to other motorists. Obviously, the team and cameras did have some influence on passing vehicles, but it appeared that the influence was minimal, except on two-lane facilities. On these routes, the rear research vehicle usually had to be maneuvered to within 152 m (500 ft) of the wide load to maintain a view of it. The presence of the research vehicle caused traffic approaching the rear escort vehicle to decelerate and form a queue behind the research vehicle.

Thus, it was impossible to determine all of the rear-end traffic conflicts that would normally be attributed to the wide load. Also, when both brake lights of a vehicle were inoperative, it was not possible to determine whether or not the driver used the brakes.

Another concern expressed during the developmental stage of the project was that the lead research vehicle might retard the normal speed of the wide-load driver and influence the study results. Extensive practice sessions were held before actual data collection to determine whether it was possible to maintain a headway that would not influence the driver of the wide load. The results of the experiments indicated that the lead driver could accurately judge the speed of the load through various roadway geometrics and continuously maintain sufficient distance ahead of the load. The results of the speed data collected during the study also indicated that the lead vehicle did not influence the speed of the wide load (13).

The film data were reduced by using two photooptical analyzers, each equipped with a variable speed advance and stop-action capability. To give full coverage of the vehicle-load interactions, the films from the front and rear cameras were simultaneously projected on the same screen. The conflicts data were recorded from the film by two observers. To reduce observer bias and ensure reliability, extensive training sessions were held. After one week of training, the observers were able to consistently recognize and record the same type and frequency of conflict. During reduction of the film data, conflict counts by each observer were compared for each section of roadway and differences in the data were corrected by reviewing the film. Thus, the variability of the counts due to subjectivity on the part of the observers was minimal.

ANALYSIS

Although the time available for data collection was limited to eight weeks, 6087 km (3782 miles) of wide-load movement were filmed. The distribution of the data-collection effort by type of highway and load size is shown in Table 1. Because of rain, mechanical breakdowns, and other problems, comparison data for the two sizes of units were not available for some routes. Consequently, data for 507 km (315 miles) of travel were eliminated before the statistical comparisons were made.

Before the traffic-conflicts data were analyzed, the speeds of the two sizes of units and the interaction data for each study section were compared. Statistical analyses of these data, given in the study report (13), showed that there were no significant differences. Because the speed and volume data for the two sizes of units were not significantly different for the highway systems studied, it was assumed that any difference in traffic conflicts could be attributed to the width of the load.

The manner in which the traffic conflicts were identified and collected for this study is unique. Consequently, there were no previous data that could have been used to estimate sample size or to serve as a basis for comparing the results obtained in this study. Because there were no previous data, there was no documented mathematical basis for choosing a significance level for testing the differences in conflict counts. For the purpose of this study, a 99 percent confidence level ($\alpha = 0.01$) was used unless otherwise noted. This high confidence level implies a reluctance to reject the null hypothesis unjustly; i.e., large differences in the characteristics of the units were required to reject the hypothesis that there were no differences. A consequence of this approach is that the probability of not rejecting the null hypothesis when it is really false is large unless the deviation from the null hypothesis is large (commonly called a type 2 error). In other words, it was assumed that 3.7-m-wide housing units would continue to be used on Virginia highways and that 4.3-m-wide units would be permitted unless a substantial difference in safety characteristics were found. The only way any error in judgment, if in fact an error occurred, could be reduced would be to increase the sample size. Because of time constraints, it was not possible to extend the data-collection period.

Results of Traffic-Conflicts Analysis

After reduction, the data included 737 conflicts for 3.7-m-wide units and 832 for 4.3-m-wide units. These conflicts were defined for the 13 specific maneuvers shown in Figures 1 through 13. Wherever possible, the conflicts terminology given in the General Motors procedures manual (7) was used. Conflicts were classified as either vehicle or load conflicts. Vehicle conflicts were further classified as being either direct or indirect. A direct vehicle conflict is one that occurs when the driver applies brakes to avoid a collision with the wide load; an indirect vehicle conflict is one that occurs when two or more drivers in the vicinity of the load apply

Table 1. Data-collection summary.

Type of System	3.7-m-Wide Load		4.3-m-Wide Load		Travel Total Filmed (km)	Percentage of Travel Filmed
	No. of Trips	Travel (km)	No. of Trips	Travel (km)		
Interstate	12	816	16	1059	1875	30.8
Primary						
Four-lane divided	30	1279	34	1337	2616	43.0
Four-lane undivided	7	124	9	165	289	4.7
Two-lane	27	503	33	614	1117	18.4
Secondary	12	95	12	95	190	3.1
Total	88	2817	104	3270	6087	100.0

Note: 1 m = 3.3 ft; 1 km = 0.6 mile.

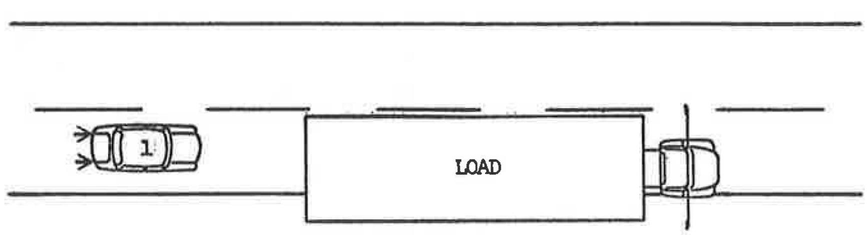

Figure 1. Direct conflict of rear-end vehicle: Vehicle no. 1, which is following the wide load, brakes to avoid collision with the load.

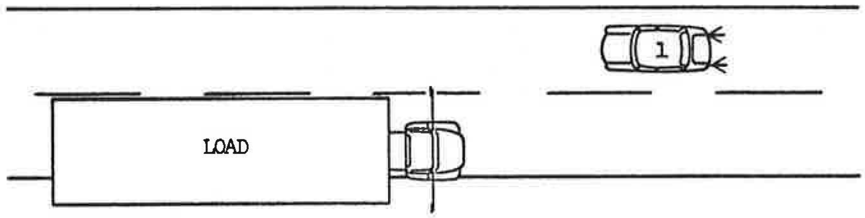

Figure 2. Direct conflict of opposing vehicle: Vehicle no. 1, which is approaching the wide load, brakes to avoid a collision with the load or an adjacent roadside obstacle.

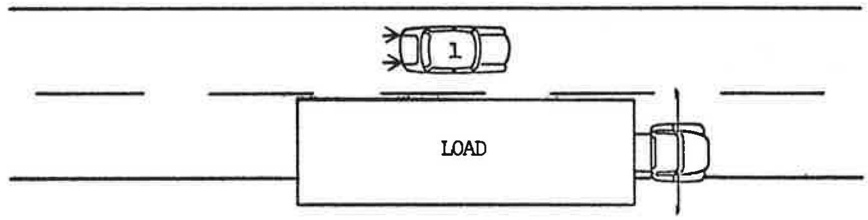

Figure 3. Direct conflict of passing vehicle: Vehicle no. 1, which is passing the wide load, brakes to avoid collision with the load, approaching traffic, or a roadside obstacle.

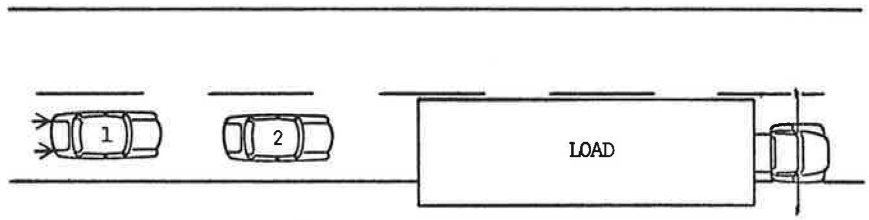

Figure 4. Indirect conflict of nonprevious rear-end vehicle: Vehicle no. 1 brakes to avoid a collision with vehicle no. 2, which is following the wide load.

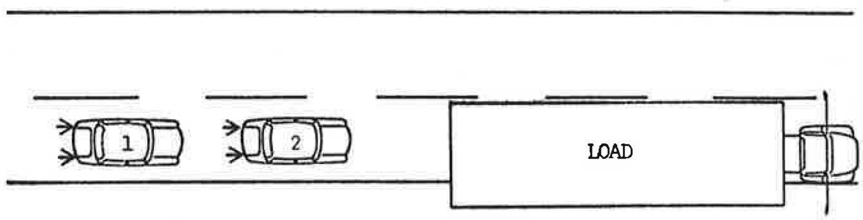

Figure 5. Indirect conflict of previous rear-end vehicle: Vehicle no. 1 brakes in response to vehicle no. 2, which is braking to avoid a collision with the wide load.

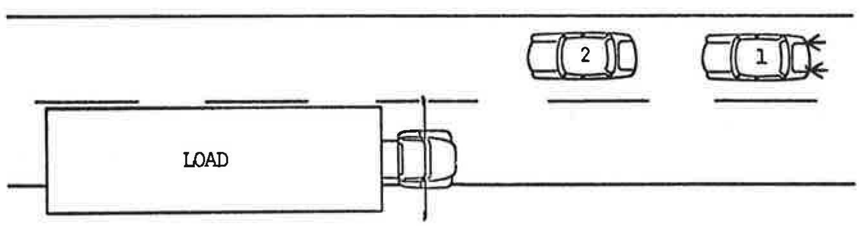

Figure 6. Indirect conflict of nonprevious opposing vehicle: Vehicle no. 1 brakes to avoid a collision with vehicle no. 2, which is approaching the wide load.

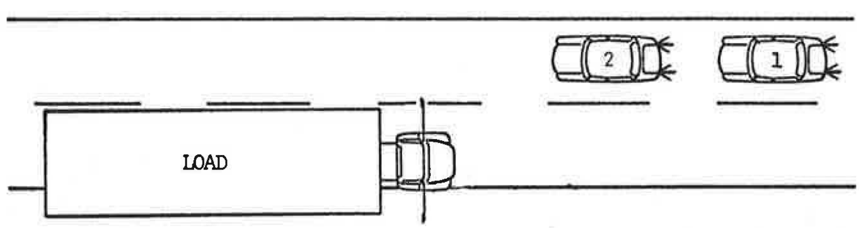

Figure 7. Indirect conflict of previous opposing vehicle: Vehicle no. 1 brakes in response to vehicle no. 2, which is braking to avoid a collision with the wide load or a roadside obstacle.

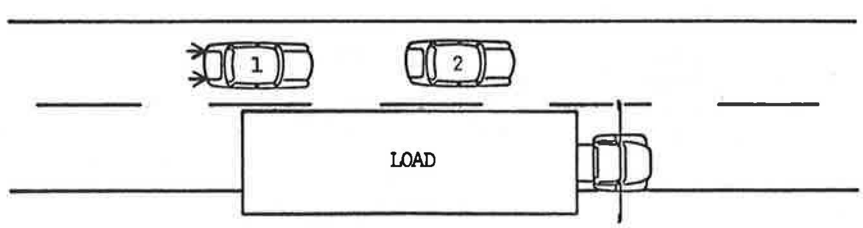

Figure 8. Indirect conflict of nonprevious passing vehicle: Vehicle no. 1 brakes to avoid a collision with vehicle no. 2, which is passing the wide load.

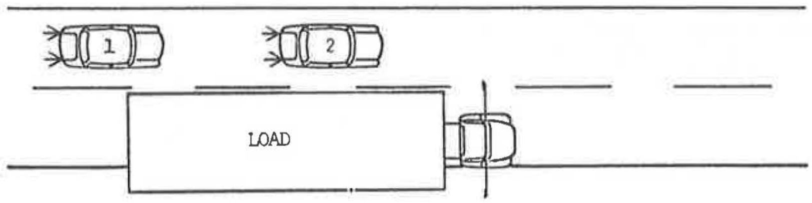

Figure 9. Indirect conflict of previous passing vehicle: Vehicle no. 1 brakes in response to vehicle no. 2, which is braking to avoid a collision with the wide load, opposing traffic, or a roadside obstacle.

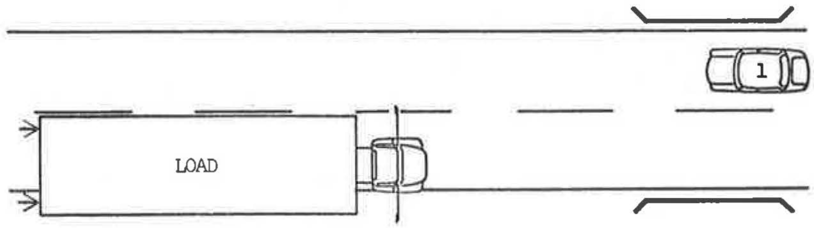

Figure 10. Conflict of load and opposing traffic and narrow structure: Load brakes to avoid collision with a narrow structure and vehicle no. 1.

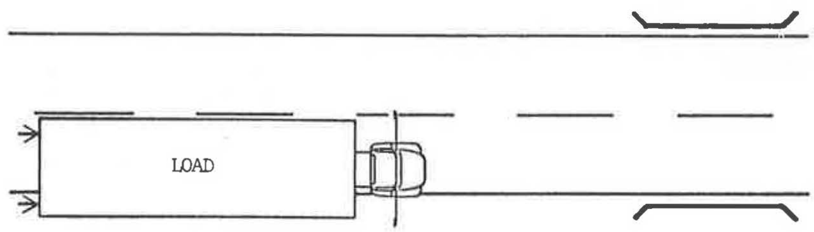

Figure 11. Conflict of load and narrow structure: Load brakes to avoid collision with a narrow structure.

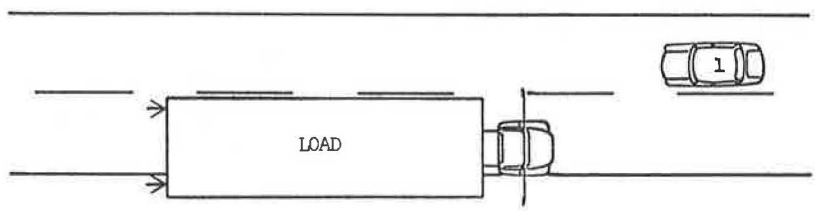

Figure 12. Conflict of load and opposing traffic: Load brakes to avoid collision with vehicle no. 1, which is approaching in the opposing traffic lane.

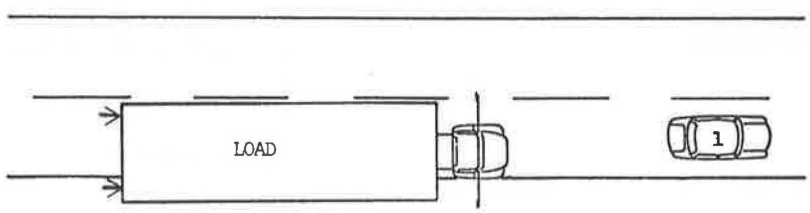

Figure 13. Conflict of load and rear end: Load brakes to avoid collision with vehicle no. 1, which is traveling in the same direction.

brakes to avoid a collision with another vehicle or a roadside obstruction. Conflicts in this latter category were considered to be indirectly caused by or related to the movement of the load.

The types and frequencies of traffic conflicts observed are given in Tables 2 and 3 for the 3.7- and the 4.3-m-wide units, respectively. Although there were frequent vehicle conflicts with the load, there were few load conflicts with other traffic or features of the highway system, e.g., roadside obstructions.

The conflict data in Tables 2 and 3 indicate some interesting relationships among the types of highway systems and the numbers of conflicts. For example, the Interstate system had the smallest number of conflicts for both load widths. The greatest number of conflicts occurred on two-lane primary facilities. The most important types of conflicts on these roads were direct and indirect conflicts of opposing vehicles (Figures 2, 6, and 7). The most frequent type of conflict on four-lane divided routes was the direct conflict of a passing vehicle (Figure 3). This observation can be explained by the fact that the most common vehicle-load interaction on four-lane divided highways is the passing maneuver. The indirect conflicts given in Tables 2 and 3 indicate that wide loads can create hazards for other vehicles without themselves being directly involved. After some cells were combined to obtain samples of sufficient size for each type of highway system, the χ^2 statistic was used to test for differences in the distributions of conflicts between 3.7- and 4.3-m-wide loads. As noted below, the distributions were not disproportionate; i.e., the type and frequency of occurrence of conflicts were not different for the two load widths.

Type of System	χ^2	Significant at $\alpha = 0.01$	df
Interstate	1.20	No	1
Primary			
Four-lane divided	3.77	No	2
Four-lane undivided	6.7×10^{-5}	No	1
Two-lane	8.89	No	3
Secondary	0.49	No	1

The mean number of traffic conflicts recorded for the two load widths by type of highway are compared in Table 4. Although no significant differences were found for each category, there were dramatic differences among categories, e.g., Interstate compared with two-lane primary. However, care should be exercised when making comparisons among groups because these are raw data that should be normalized to account for lengths of roadway sections and traffic volumes.

The effect of section length on number of conflicts was examined by dividing the number of observed conflicts for each test run by the length of the test section to determine the number of conflicts per kilometer, which was termed the traffic-conflicts index. In addition, the effect of traffic volume on number of conflicts was examined by dividing the conflicts index by the number of vehicle interactions. This result was termed the traffic-conflicts rate. The conflicts indices and rates for each type of highway system were then summarized; the results of tests for these measures are given in Tables 5 and 6. As noted, no significant differences were found. The conflicts indices and rates are of practical significance because they permit direct comparison of the hazards of the movement of oversize loads for any given route. For example, the data clearly indicate that such movement is more hazardous on two-lane primary and secondary systems than on Interstate and four-lane highways. Furthermore, because these conflicts indicators were developed for each section of road included in the study, it was easy to identify sections that deviated substantially from the mean. As a general rule, roadways

Table 2. Traffic-conflicts summary: 3.7-m-wide load.

	Vehicle Conflict										Load Conflict				
				Indirect											
	Direct			Rear End		Opposing		Passing							
Type of System	Rear End	Opposing Traffic	Passing	Non-previous	Previous	Non-previous	Previous	Non-previous	Previous	Other	Opposing Traffic and Narrow Structure	Narrow Structure	Opposing Traffic	Rear End	Other
Interstate	2	0	4	0	0	0	0	1	0	0	0	0	0	0	0
Primary															
Four-lane divided	20	0	58	1	0	0	0	7	8	0	0	0	0	2	2
Four-lane undivided	5	3	13	1	0	0	0	2	3	0	0	0	0	3	1
Two-lane	71	309	2	8	2	47	116	0	0	0	1	1	4	1	2
Secondary	0	28	0	0	0	0	0	0	0	1	0	0	8	0	0

Note: 1 m = 3.3 ft.

Table 3. Traffic-conflicts summary: 4.3-m-wide load.

	Vehicle Conflict										Load Conflict				
				Indirect											
Direct				Rear End		Opposing		Passing							
Type of System	Rear End	Opposing Traffic	Passing	Non-previous	Previous	Non-previous	Previous	Non-previous	Previous	Other	Opposing Traffic and Narrow Structure	Narrow Structure	Opposing Traffic	Rear End	Other
Interstate	6	0	10	1	0	0	0	4	3	0	0	0	0	0	1
Primary															
Four-lane divided	24	0	109	6	3	1	4	21	16	0	0	1	0	0	0
Four-lane undivided	4	0	19	1	0	1	0	3	5	0	0	0	0	0	1
Two-lane	44	297	5	7	1	49	145	0	0	0	1	3	3	0	4
Secondary	0	24	0	0	0	0	4	0	0	1	0	0	0	0	0

Note: 1 m = 3.3 ft.

Table 4. Number of traffic conflicts.

	3.7-m-Wide Load			4.3-m-Wide Load					
Type of System	No.	Mean	Variance	No.	Mean	Variance	t-Value	Significant[a]	df
Interstate	10	0.70	1.57	14	1.79	3.19	-1.74	No	24
Primary									
Four-lane divided	23	4.41	19.78	25	6.80	34.79	-1.67	No	46
Four-lane undivided	7	4.43	15.62	9	3.78	21.44	0.30	No	16
Two-lane	27	20.89	992.33	26	21.50	1093.38	-0.07	No	53
Secondary	12	3.08	9.90	12	2.42	4.27	0.61	No	20

Note: 1 m = 3.3 ft.
[a] $\alpha = 0.01$.

Table 5. Traffic-conflicts index.

Type of System	3.7-m-Wide Load			4.3-m-Wide Load					
	No.	Mean	Variance	No.	Mean	Variance	t-Value	Significant[a]	df
Interstate	10	0.23	0.13	14	0.72	0.54	-2.14	No	22
Primary									
Four-lane divided	23	2.88	8.67	25	4.89	14.68	-2.04	No	46
Four-lane undivided	7	6.18	25.45	9	3.22	5.75	1.43	No	9
Two-lane	27	18.31	346.52	26	18.26	407.31	0.01	No	52
Secondary	12	10.84	74.15	12	11.05	178.56	-0.05	No	20

Notes: 1 m = 3.3 ft.
Conflicts index is expressed in conflicts per kilometer × 10.
[a] $\alpha = 0.01$.

Table 6. Traffic-conflicts rate.

Type of System	3.7-m-Wide Load			4.3-m-Wide Load					
	No.	Mean	Variance	No.	Mean	Variance	t-Value	Significant[a]	df
Interstate	10	0.53	0.80	14	1.09	1.86	-1.21	No	24
Primary									
Four-lane divided	23	7.47	66.92	25	12.12	165.84	-1.51	No	43
Four-lane undivided	7	2.53	5.26	9	1.14	0.44	1.56	No	7
Two-lane	27	92.61	46 900.38	26	37.41	2 019.46	1.30	No	28
Secondary	12	987.61	225 108.02	12	288.04	112 579.04	1.14	No	21

Notes: 1 m = 3.3 ft.
Conflicts rate is expressed in conflicts per vehicle kilometer × 1000.
[a] $\alpha = 0.01$.

Table 7. Linear correlation coefficients: volume and number of conflicts.

Type of System	3.7-m-Wide Load				4.3-m-Wide Load			
	N	r^2	r	Critical r^a	N	r^2	r	Critical r^a
Interstate	10	0.07	0.27	0.63	14	0.34	0.58	0.53
Primary								
Four-lane divided	23	0.35	0.59	0.42	25	0.49	0.70	0.40
Four-lane undivided	7	0.13	0.37	0.75	9	0.04	0.19	0.67
Two-lane	26	0.94	0.97	0.39	26	0.87	0.93	0.39
Secondary	12	0.34	0.59	0.58	12	0.84	0.92	0.58

Note: 1 m = 3.3 ft.
[a] $\alpha = 0.05$.

that had narrow pavements had high conflict rates. The obvious importance of this finding to the highway manager would be to minimize the movement of wide loads on these routes.

Volume and Conflicts Relationship

Traffic-conflicts data recorded for intersections have been shown to be highly volume-dependent (11, 12). To examine the dependence of the number of conflicts on the number of vehicles encountered during the movement of a wide load, linear correlation coefficients r's and coefficients of determination r^2s were computed; the results are given in Table 7. For the six cases on Interstate and four-lane facilities examined, only three coefficients were statistically significant. However, these coefficients indicate that the conflict-volume relationship is not strong. On the contrary, however, the relationship is highly positive for the two-lane primary system. These results confirm the observations of the data-collection crew, who reported that most vehicles on two-lane roads were involved in a traffic conflict. On divided highways, few vehicles were involved in a traffic conflict with a wide load.

Sample-Size Requirements

The sample size is usually controlled by either time or budget constraints; in this study, it was limited by time. In many studies, the mean and variance can be estimated from previous results and the sample size can be determined before the tests are made. However, for this experiment, there were no previous data or guidelines and there was concern that time constraints would limit the data collected to an amount that would be insufficient for statistical tests. In an attempt to secure as much data as possible within the eight-week data-collection period, two cameras were used on each trip. Thus, after the data were collected and summarized, the adequacy of the sample size for each load width and type of highway was determined by the following procedure.

For the data collected, the size of the sample (number of test runs) was known. A confidence level of 90 percent ($\alpha = 0.10$) was chosen, and the task was to determine the sample error. The procedure is illustrated in the example below taken from the 10 samples of 3.7-m-wide load movement on the Interstate system shown in Table 4.

The sampling error is obtained from Equation 1,

$$E = tv/\sqrt{N} \tag{1}$$

where

E = sampling error (percent),
t = sample risk (for $\alpha = 0.10$ with 9 degrees of freedom, $t_{0.95} = 1.83$),
v = variation coefficient (percent) = 100 (standard deviation of sample/sample mean), and
N = sample size.

For the 3.7-m-wide load on the Interstate system,

$$E = 1.83\ [100 \times (1.253/0.70)]/\sqrt{10} = 104 \text{ percent}$$

Therefore, it can be concluded with 90 percent confidence that the mean number of conflicts per test run for a 3.7-m-wide load on the Interstate system is included in the interval between 0 and 1.4.

This procedure was used to determine sample-size errors for the numbers of conflicts and the conflicts indices and rates. The results are given in Table 8. The magnitude of the errors generally indicates that it

Table 8. Computed errors in sample sizes.

Type of System	3.7-m-Wide Load No. of Conflicts	Conflicts Index	Conflicts Rate	4.3-m-Wide Load No. of Conflicts	Conflicts Index	Conflicts Rate
Interstate	104	95	97	48	48	59
Primary						
Four-lane divided	36	37	39	30	27	36
Four-lane undivided	66	60	67	76	46	36
Two-lane	50	33	77	52	37	40
Secondary	53	41	51	44	63	60

Notes: 1 m = 3.3 ft.
Errors are expressed as percentages.

Table 9. Comparison of accident and traffic-conflicts data: 3.7-m-wide loads.

Type of System	1969 Accidents No.	Percentage of Total	1976 Conflicts Data No.	Percentage of Total No.	Travel (km)	Percentage of Total Travel
Interstate	3	13	7	1	673	26
Primary						
Four-lane divided	7	29	98	13	1152	45
Four-lane undivided	2	8	31	4	124	5
Two-lane	9	37	564	77	531	20
Secondary	3	13	37	5	95	4
Total	24	100	737	100	2575	100

Note: 1 m = 3.3 ft; 1 km = 0.6 mile.

would be desirable to collect a larger number of samples. One of the recent criticisms of the traffic-conflicts technique, as applied to intersections, was that a large number of observations are needed to detect changes in numbers of conflicts at a high confidence level (12). The data given in Table 8 tend to support that comment.

For most traffic experiments, large sample sizes are not practical. The data collected for the wide-load study are probably as comprehensive as can be obtained by most state highway agencies. However, a high statistical confidence level can be achieved only by taking considerably more data. For example, for the case of the conflicts recorded for the 3.7-m-wide load on the Interstate system, 1073 test runs would have been required to reduce the sample error to 10 percent (assuming a constant variance-to-mean ratio of 2.2 and a confidence level of 90 percent). A sample size of this magnitude would seldom, if ever, be economically feasible for any traffic study. Furthermore, a reduction of the confidence level does not substantially reduce the number of samples required.

In most cases, the need for large sample sizes can be attributed to the relatively small number of conflicts and the large sample variance. Because of the large sample-size errors computed for the conflicts data, the results of any statistical comparisons of the data are questionable. Furthermore, because the increases in sample sizes necessary to increase the confidence in the results are extraordinarily large, the use of traffic conflicts for the assessment of the hazards of the movement of wide loads is questionable. However, the results of the statistical tests must be interpreted in light of the practical value of the information obtained. The use of accident data as an alternative was not possible because only a few accidents involving 3.7-m-wide units have been reported and 4.3-m-wide units were not allowed in Virginia. The conflicts data were useful for identifying the nature and frequency of hazards imposed on motorists by the movement of wide loads. The data also clearly indicated that the hazards were much greater on two-lane roads than on four-lane divided highways. Even if the sample size could be increased, it is doubtful that any new or different hazards would be found. Thus, the conflicts technique provided more detailed and useful information than conventional accident records could supply. For these reasons, the technique is a useful measure for evaluating highway safety.

The decision to permit or prohibit 4.3-m-wide units was based not only on the conflicts data but also on analyses of other data recorded in conjunction with the conflicts. These data included length and frequency of queues, vehicle passing times, encroachments, lateral placements, and other traffic and safety measures.

Conflicts and Accident Relationship

Because of the scarcity of accident data for oversize units, it is not possible to determine whether there is a relationship between traffic conflicts and accidents. No accidents involving the oversize loads occurred during the study. As the existing traffic-records system does not permit computer identification of wide-load accidents, little data were available. However, an extensive manual search of the Virginia records indicated that in 1969 mobile homes were involved in 24 accidents (2). In these accidents, there were no fatalities; six persons were injured and property damage amounted to \$17 500. Although exposure (vehicle kilometers of travel) information for the accident data is not available, a rough comparison of the 1969 accident data and the 1976 conflicts for 3.7-m-wide loads is given in Table 9. Although definite conclusions cannot be formulated, it is interesting that the majority of accidents occurred on two-lane primary and secondary systems, where, according to the conflicts data, the hazards associated with the movement of wide loads are greatest.

Future Applications

Any measure for the evaluation of traffic or safety factors should be judged in terms of its advantages and disadvantages. The large sample size required and the cost of collecting and reducing film data indicate that the traffic-conflicts technique, as applied to the assessment of the hazards of vehicles in motion along the highway, may not be an efficient tool for widespread use by most highway agencies. However, in the absence of accident reports, the technique is useful for the identification of the type and frequency of hazards encountered by vehicles in motion. Its primary advantages are (a) the systematic manner it provides for observing and recording hazards; (b) the more frequent occurrence of conflicts than of other events for measuring safety, e.g., accidents and near misses; (c) the short period of time required to collect the data; and (d) the fact that environment-vehicle-driver data recorded during a conflicts study provide a more exact relationship among these variables than that which can be obtained from a conventional accident analysis.

There are numerous highway-safety problems in which the conflicts technique can be used to provide a better measure of safety than can accident data. For example, the hazards associated with the use of such items as recreational vehicles, trucks, and mowing and centerline-painting machines could be identified by using the conflicts technique.

The following guidelines are offered to potential users of the conflicts technique.

1. A pilot study should be conducted to determine the relative types and frequencies of the hazards associated with the problem under consideration. A definition

should be formulated, and the conflicts should be classified in accordance with the observed hazards. The data-collection procedure, as well as other measures applicable for use in evaluating traffic and safety factors, should be developed.

2. A standard should be chosen as a basis for comparison with the results of the experimental condition, e.g., 3.7-m-wide loads compared with 4.3-m-wide loads.

3. An estimation of the errors in sample size should be made based on the data collected and the confidence level required, and the results should be interpreted in recognition of factors such as staff limitations and time constraints.

CONCLUSIONS

Based on an evaluation of the traffic-conflicts technique as applied to the examination of the hazards of oversize loads in motion along the roadway, the following conclusions are offered.

1. The traffic-conflicts technique can be adapted to a wide range of uses.
2. Hazards associated with moving oversize loads can be identified in a short period of time by using the conflicts technique. (Accident data, which require a long time to develop, could not be used to assess the hazards imposed by wide loads on other traffic.)
3. There are no apparent differences between the safety aspects of 3.7-m-wide loads and those of 4.3-m-wide loads in terms of the types or frequencies of conflicts observed.
4. The movement of wide loads is significantly more hazardous on two-lane facilities than on four-lane divided highways.
5. Traffic conflicts on two-lane highways are extremely volume dependent; however, on divided highways, the relationship is not strong.
6. It may not be practical to collect the large amount of conflicts data necessary to establish a high degree of confidence in the results.
7. Despite the problems of sample size, the conflicts technique is useful in assessing the relative hazards associated with the movement of wide loads on a variety of highway systems.
8. Because of the scarcity of accident data, no relationship between traffic conflicts and accidents for wide loads can be determined.

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