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NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT **214**

DESIGN AND TRAFFIC CONTROL GUIDELINES FOR LOW-VOLUME RURAL ROADS

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT 214

DESIGN AND TRAFFIC CONTROL GUIDELINES FOR LOW-VOLUME RURAL ROADS

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:
- FACILITIES DESIGN
- PAVEMENT DESIGN AND PERFORMANCE
- TRANSPORTATION SAFETY
- OPERATIONS AND TRAFFIC CONTROL
- (HIGHWAY TRANSPORTATION)

TRANSPORTATION RESEARCH BOARD
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WASHINGTON, D.C. OCTOBER 1979

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
Transportation Research Board

Highway design engineers, safety specialists, and local road officials will find this report to be of particular interest. It contains suggested guidelines, based on a series of functional analyses relating safety performance to specific design and operational elements applicable to low-traffic-volume rural roads. With increasing highway construction costs and budget constraints, the suggested guidelines can be used to determine the best use of available funds during planning, design, and rehabilitation of low-volume roads.

Safety criteria for roads with low traffic volume are generally similar to or extrapolated from criteria established for other road systems. To ignore safety on low-volume roads could result in accidents and loss of life; on the other hand, to impose unrealistic safety criteria on such roads is wasteful of money and materials. The objectives of this study were to (1) evaluate existing safety guidelines, requirements, and criteria with regard to their applicability and relevancy for roads carrying fewer than 400 vehicles per day at normal and reduced speeds; and (2) prepare suggested guidelines for use of such features as guardrail, signs, lane markings, pavement width, shoulders, and clear roadsides suitable for design and rehabilitation of low-volume rural roads.

The study was conducted by John C. Glennon, Transportation Consulting Engineer, Overland Park, Kansas. Current safety performance of low-volume roads was evaluated by the collection and review of accident data. These data combined with probabilities of conflict and cost-effectiveness analyses indicate that only relatively low-cost types of improvements can be justified on most low-volume roads. Specific elements evaluated were speed signs, curve warning signs, stop signs, centerline markings, no-passing stripes, roadway width, shoulder width, and roadside safety design.

The suggested guidelines presented in this report propose revisions and clarification of portions of AASHTO design publications and the Manual on Uniform Traffic Control Devices and are intended to supplement information in those publications. They are based on the analysis of currently available information combined with engineering analysis. Thus, they are suitable for use on an interim basis in the design and rehabilitation of low-volume roads. The report also describes types of research studies that can be conducted to verify or modify the guidelines in the future.

CONTENTS

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The research reported herein was conducted under NCHRP Project 20-7, T13 by John C. Glennon, Transportation Consulting Engineer, Overland Park, Kansas. John C. Glennon was Principal Investigator.

Sincerest gratitude is extended to the various state and other highway departments who contributed valuable information and comments.

DESIGN AND TRAFFIC CONTROL GUIDELINES FOR LOW-VOLUME RURAL ROADS

SUMMARY

Low-volume rural roads, those carrying 400 vehicles per day or less, constitute two-thirds of the total U.S. highway system. Their key importance to the national transportation objective cannot be denied. Not only are they the largest single class of highway, but they are also the vital link of the nation's agricultural economy.

National guidelines for the design of low-volume rural roads are contained in the 1971 AASHTO publication "Geometric Design Guide for Local Roads and Streets." For traffic control devices, the basic guidelines are presented in the "Manual of Uniform Traffic Control Devices." But, because these national guidelines reflect the safety needs of primary highways, their application to the reconstruction of existing low-volume rural roads is continually being questioned in a time when local highway agencies must spend a majority of their limited funds for highway maintenance.

This research was undertaken to reevaluate the safety needs on low-volume rural roads. On the basis of a series of functional analyses relating safety performance to specific design and operational elements, a set of revised guidelines was developed. The revised guidelines apply to total roadway width, horizontal curvature, roadside design, speed signs, curve warning signs, centerline markings, and no-passing stripes. These guidelines are proposed to supplement the existing national policies, with each revised guidelines either replacing or clarifying the existing national guideline.

The widespread application of the revised guidelines should provide for more consistent design and traffic control of low-volume rural roads consonant with a rational balance between highway investment, highway safety, and traffic service.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

PROBLEM STATEMENT

Low-volume rural roads, those carrying 400 vehicles per day or less, are the backbone of the U. S. rural economy. State "farm-to-market" roads, county roads, and township roads provide the accessibility required by agricultural commerce. Forest roads and park roads are necessary for the operation, maintenance, and accessibility of national forests and parks.

Although low-volume rural (LVR) roads only carry about 8 percent of the total U. S. highway travel, their economic importance in the national highway program is recognized because they constitute 2 out of every 3 miles (mi) of public highway (*22*). Because they are the largest single class of highway, objective guidelines for their design and operation are imperative to achieve a reasonable balance between cost and safety effectiveness. The bulk of the present LVR road system has been built using design and operational practices that have evolved from subjective experience and judgment rather than from an objective evaluation of quantifiable performance.

In designing and operating highways for safety, LVR roads have one intrinsic advantage over higher volume highways because of a considerably lower probability of vehicle-to-vehicle collisions. The basic requirements for the minimization of single-vehicle accident consequences, however, are similar for all roads. In this area, maximum safety requires wide roadways and shoulders, clear and flat roadsides, gentle alignment, and high quality traffic controls and informational signing.

When considering safety on LVR roads, the highway agencies have been faced with a dilemma. On one hand, the agencies were inclined or required by funding sources to provide the same high-type design and operational features as on the primary highway system. On the other hand, the cost of providing such features often conflicted with the agency's philosophy of economic expediency. Because so few dollars had to be spent over so many miles, LVR roads have historically been designed and operated at minimal cost. Safety was seldom a primary consideration.

Now, the basic scenario of the highway program is changing from the massive road building campaign of the 1950's and 1960's toward a concerted effort to rehabilitate existing highways. As this new emphasis mounts, the tendency is for federal matching funds to require that highways, regardless of their functional classification, be redesigned to meet all current standards. And, current standards tend to reflect the needs of primary highways and, therefore, could require extensive and costly reconstruction of existing LVR roads. Highway agencies express increasing concern on this trend because it will force them to spend unreasonably large amounts of money for the rehabilitation of LVR roads. The alternative, which is more likely, however, is for the highway agencies to avoid these apparently unjustified costs by implementing only a few low-cost road improvements without using federal funds.

What this discussion points to is the need for objective safety guidelines that will strike a rational balance between maximum safety and minimum cost for LVR roads. With these guidelines, highway agencies could determine where and when to improve LVR roads within the broader framework of highway rehabilitation for their entire highway system.

RESEARCH OBJECTIVES

The objectives of this research were as follows:

1. Evaluate existing safety guidelines, requirements, and criteria with regard to their applicability and relevancy for roads carrying low traffic volumes (under 400 vpd) at normal and reduced speeds.
2. Identify design and traffic control elements for which modifications of safety requirements should be considered and recommend interim safety criteria for low-volume rural roads.
3. Develop a systematic approach for collecting additional information related to safety requirements for low-volume rural roads.

RESEARCH APPROACH

The research was divided into three tasks that aligned with the research objectives. In task 1, a complete examination was made of the appropriate AASHTO geometric design policies (*1, 2*) and the Manual on Uniform Traffic Control Devices (*29*). The purpose of this exercise was both to identify the elements pertinent to LVR roads and to evaluate their suitability on the basis of their functional relationship to safety performance. Task 2 involved the majority of project effort. Here, the current safety performance of LVR roads was evaluated; the literature on the safety effectiveness of geometric design and traffic control elements was reviewed; and revised safety requirements for several elements were developed based on functional analyses, probabalistic conflict analyses, and cost-effectiveness analyses. In task 3, recommendations were developed for the collection of additional data to verify the revised safety requirements developed in task 2.

CHAPTER TWO

SUMMARY OF FINDINGS

This chapter summarizes the findings of the research. Discussed are: (1) an evaluation of the current safety performance on LVR roads; (2) the examination of national policies on geometric design and traffic control devices; (3) the development of revised requirements for several geometric design and traffic control elements; and (4) the recommendation of additional data collection activities to verify the revised safety requirements.

CURRENT SAFETY PERFORMANCE ON LVR ROADS

In analyzing the safety requirements for LVR roads, it is first important to dimension their current safety performance. Appendix A gives a complete evaluation of the current safety performance of LVR roads.

Table A-1 gives national statistics for "local-rural" roads, which are basically county and township roads with an average ADT of 105 vpd. Although these roads constitute 58 percent of the total U.S. public road mileage, they experience only 11 percent of the fatal accidents and 8.4 percent of the injury accidents. These statistics indicate that the average frequency of fatal plus injury accidents is 1 every 13.7 mi (22.1 km) per year on these LVR roads.

These national statistics, together with other empirical data found in the literature, were used to generate the best-fit curves of total accidents, injury plus fatal accidents, and single-vehicle accidents, as shown in Figure 1. The total accident rates range from 0.098 acc./mi/yr (0.061 acc./km/yr) at 50 vpd to 0.367 acc./mi/yr (0.228 acc./km/yr) at 400 vpd. In other words, the average road carrying 50 vpd will have one accident per year for every 10.2 mi (16.4 km), and the average road carrying 400 vpd will have one accident per year for every 2.7 mi (4.3 km).

The generated rates for injury plus fatal accidents are 47 percent of the total accident rates. The generated rates for single-vehicle accidents as a percentage of total accidents range from 52.9 percent at 400 vpd to 71.4 percent at 50 vpd.

Also considered in Appendix A is the proportion of total hazard by accident type. Defining hazard as the annual number of fatal and injury accidents per mile, and weighting the different kinds of accidents by their average severity (percent of fatal plus injury accidents), single-vehicle accidents were found to have the majority contribution to LVR road hazard. The percent of the total fatal and injury accidents contributed by single-vehicle accidents ranges from 61.1 percent for roadways with 400 vpd to 77.8 percent for roadways with 50 vpd.

Another way of looking at the current safety performance of LVR roads is to evaluate the impact of accident costs. By using the costs by accident severity class reported in *NCHRP Report 162* (*21*) and applying the percent of accidents by severity class reported in Appendix A, the average cost of an accident on LVR roads was estimated at $9,500. Applying this average cost to the generated accident rates yields an average cost of accidents per mile of LVR road ranging from $665/mi ($413/km) for a road carrying 50 vpd to $3,570/mi ($2,217/km) for a road carrying 400 vpd.

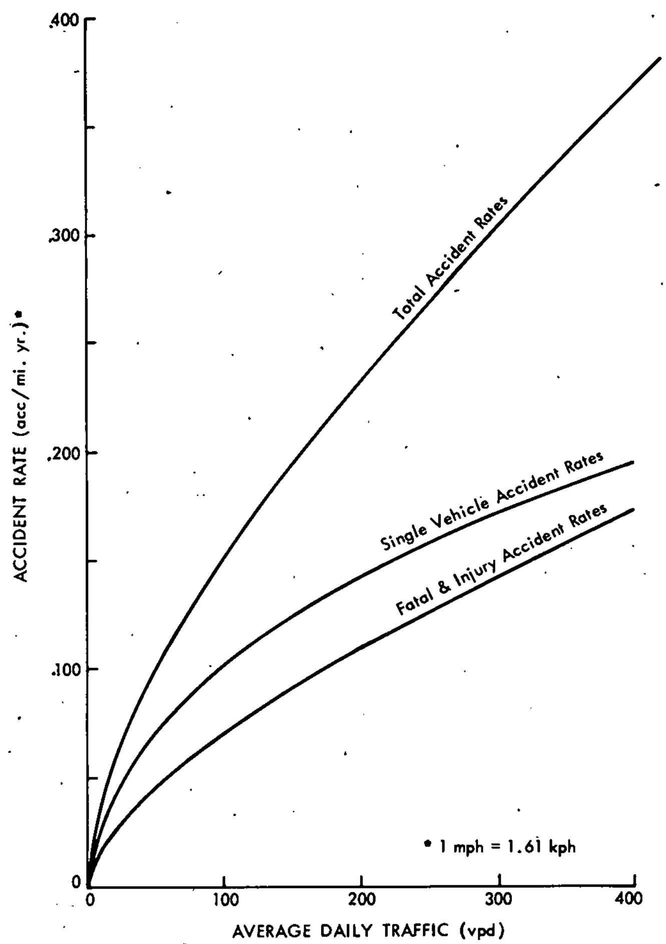

Figure 1. Estimated safety performance of existing low-volume rural roads.

These accident rates and costs begin to indicate the difficulty of making safety improvements that have any recognizable impact on the over-all safety performance of LVR roads. Given a goal of a 25 percent reduction in accidents, an average of only $300/yr could be justified per mile of LVR road if a cost-benefit balance is to be achieved. What this suggests is that, even with the safety-conservative (high) unit values used for the cost of accidents, only relatively low-cost kinds of improvements can be justified on most LVR roads.

EXAMINATION OF NATIONAL POLICIES

The examination of national policies on geometric design and traffic control elements was conducted early in the project period to identify the standards, criteria, and guidelines currently applicable to LVR roads and to evaluate their functional suitability to the safety performance of LVR roads. This section of the report summarizes a more complete discussion found in Appendix B.

The 1971 Manual on Uniform Traffic Control Devices (MUTCD) is the national policy on traffic control devices. If the MUTCD is interpreted literally, the only traffic control devices that are mandatory on LVR roads are crossbucks at railroad grade crossings. All other devices have generalized warrants or otherwise discretionary application. In evaluating the application of the MUTCD to LVR roads, five traffic control devices appeared to require further clarification regarding their safety requirements on LVR roads. These devices, which are discussed further in the next section of this chapter, are speed signs, stop signs, curve warning signs, centerline markings, and no-passing stripes. Although most of the other regulatory and warning devices might apply under certain circumstances on LVR roads, this application must remain discretionary because of their unclear relationship to safety performance.

The national policies on the geometric design of LVR roads are contained in two AASHTO publications, "A Policy on the Geometric Design of Rural Highways, 1965" (AASHTO Bluebook) and "Geometric Design Guide for Local Roads and Streets, 1970." The AASHTO Local Road Guide mainly summarizes the parts of the 1965 AASHTO Bluebook pertaining to LVR roads.

The major differences in the Local Road Guide relate to the specification of design speeds. In this change, minimum design speeds ranging from 20 mph (32.2 kph) to 50 mph (80.5 kph) are specified depending on the ADT and type of terrain on the LVR road. Lower ADT's and more severe terrain justify lower minimum design speeds, and higher ADT's and more level terrain justify higher minimum design speeds. These design speed specifications allow a balance between the objectives of safety, service, and economy consistent with roadway function and expected operating speeds.

The design elements identified as pertinent to LVR roads and the general evaluation of the suitability of their AASHTO guidelines to the safety performance of LVR roads are as follows:

1. *Suitable safety requirements for LVR roads/requirements based on general analysis of trade-offs between safety, service, and economy:* highway grade, cross slope, shoulder cross slope, and structure width.

2. *Suitable safety requirements for LVR roads/requirements based on objective functional analysis of safety performance using design speed as basic criterion:* stopping sight distance, passing sight distance, corner sight distance, horizontal curvature, and vertical curvature.

3. *Questionable requirements for LVR roads/requirements not based on analysis of trade-offs between safety, service, and economy:* total road width (traveled way plus shoulders), shoulder width, and roadside design (guardrail, curbs, side slopes, clear zone, etc.).

The elements in the third category are discussed further in the next section of this chapter. For the most part, these design requirements call for dimensions that are much greater than those needed for acceptable safety at a reasonable cost.

DEVELOPMENT OF REVISED REQUIREMENTS

The development of revised safety requirements was undertaken for the eight traffic control and geometric design elements that were identified as having questionable national standards or guidelines as they apply to LVR roads. The elements evaluated were speed signs, curve warning signs, stop signs, centerline markings, no-passing stripes, roadway width, shoulder width, and roadside safety design.

Revised safety requirements were developed for most of these elements based on functional analyses, probability of conflict analyses, and cost-effectiveness analyses. The analyses were conducted using available data where possible and safety-conservative assumptions where data were not available. The term "safety-conservative" refers to assumptions that overtly favor safety in the analysis. By so doing, if errors are made in deciding appropriate requirements for design and operational elements, the errors will favor safety at the expense of highway investment, rather than the opposite. The following discussion summarizes these developments.

Speed Signs

For most highways, drivers tend to judge their appropriate safe speed according to the geometric design, traffic characteristics, and roadside development of the highway. This would suggest for LVR roads that, because of minimum roadside friction and relatively infrequent incounters with other vehicles, geometric design elements are the primary determinants of vehicle speed. Without the other controls, however, drivers might tend to overdrive LVR roads except where directly influenced by physical constraints such as horizontal curvature. For this reason, speed limit signs keyed to the design speed of the highway appear to be an important adjunct to the safe operation of LVR roads.

In keeping with the correspondence between design speed and operating speed previously discussed, all LVR should be speed zoned for their design speed. This practice will provide a uniform display and guide to drivers that indicate the maximum operating speed for LVR roads. For drivers who are good judges of geometric conditions for setting their maximum operating speeds, the speed-zone signs will reinforce their judgment. For drivers who normally overdrive the geometrics, the speed-zone signs will provide a consistent reminder of why they experience discomfort in their driving.

Curve Design and Warning Signs

Appendix E presents an analysis of vehicle tracking on highway curves. The results of this analysis show that maximum acceptable tracking corrections allow for curve design speeds that are slightly below the general highway design speed. This tolerance is 5 mph (8 kph) at a 30-mph (48.3-kph) highway design speed, 10 mph (16.1 kph) at a 40-mph (64.4-kph) highway design speed, and 15 mph (24.2 kph) at a 50-mph (80.5-kph) highway design speed.

Although the practice of designing curves for less than the highway design speed is not generally recommended, it may be the only practical alternative in mountainous terrain, for example. Also, an existing highway will occasionally have a highway curve that has a design speed below the general operating speed of the highway. To be consistent with the design speed-operating speed correspondence proposed, these curves should always display curve warning signs with advisory speed plates showing their design speed, providing they are within the acceptable speed tolerance. Highway curves on existing highways that have design speeds below the acceptable speed tolerance should be upgraded.

Stop Signs

Appendix H evaluates the cost-effectiveness of installing two-way stop control at the intersection of two LVR roads using (1) a conflict analysis to estimate the annual number of right-angle accidents for various combinations of intersecting volumes, (2) the average accident cost of $9,500 generated in Appendix A, (3) an annualized cost of $20 for stop-sign installation, and (4) an increased operating cost of $0.021 per stopped vehicle reported by Anderson et al. (*3*). This analysis indicates that the increased costs are greater than the safety benefit even for a 100 percent reduction in right-angle accidents. Therefore, stop signs are not generally justified at the intersection of two LVR roads.

Because this analysis was based on average expected values, it should be recognized that the present discretionary warrants for stop control in the MUTCD are appropriate for LVR roads. Special problems with sight restrictions or with the assignment of right-of-way, particularly when an LVR road intersects a higher volume through highway, should warrant consideration of stop control on LVR roads.

Centerline Markings

No empirical data are available to show the safety effectiveness of centerline stripes on two-lane highways. The primary function of the centerline stripe is to guide drivers in judging the proper clearance interval to opposing vehicles.

Appendix G analyzes the need for centerline markings to separate opposing vehicles on LVR roads. To visualize the nature of the problem, this analysis first predicts the number of head-on meetings for various LVR road traffic volumes. This yields the following expected rates.

ADT	Estimated Number of Head-on Meetings Per Day
50	1.5
100	6.2
200	25.0
300	56.0
400	100.0

With these rates, many vehicles will travel on LVR roads without meeting an opposing vehicle.

The need for centerline markings was also evaluated in Appendix G on a benefit-cost basis using the accident rates and costs generated in Appendix A and assuming a 5 percent reduction in total accidents as reported in *NCHRP Report 162* (*21*). Using a centerline cost of $200/mi, a 1.5-yr life of marking, and the $9,500 average cost of accidents, the benefit-cost balance was found at an ADT of 300 vpd. Thus, centerline markings are warranted on paved LVR roads when the ADT equals or exceeds 300 vpd.

No-Passing Stripes

No empirical data are available on the safety effectiveness of no-passing stripes. The primary function of no-passing stripes is to prevent passing maneuvers where limited sight distance would make passing unduly hazardous.

Appendix F analyzes the need for no-passing stripes on LVR roads. To visualize the nature of the problem, this analysis first uses a Poisson probability distribution to predict the expected number of head-on conflicts created by passing maneuvers. This yields the following expected rates for various LVR road traffic volumes.

ADT	Expected Annual Number of Passing Conflicts Per Mile
50	0.02
100	0.17
200	1.44
300	4.81
400	11.06

Although these low rates of conflict would seem to preclude the need for no-passing stripes, a benefit-cost analysis was also performed in Appendix F. This analysis, which was similar to that conducted for centerline markings, indicated that no-passing stripes are not cost-effective on LVR roads.

Shoulder Width

An evaluation of several studies in the literature indicates conflicting results regarding the general safety effectiveness of highway shoulders (*17*). Then too, further analyses of some of the studies (*8, 25, 27*), which show that accident rates decrease with increasing shoulder width, indicate that the studies lacked statistical control for traffic volume. Therefore, what was really found was the well-known relationship, which shows decreasing accident rates with increasing traffic volumes.

The primary functions of shoulders are to provide additional width for tracking corrections, head-on clearances, emergency stops, and leisure stops. The need for shoulders for tracking and head-on clearance is treated in Appendix D as part of the total roadway width requirement. This analysis, which is described more fully in the next section of this report, indicates the need for shoulders on LVR roads for design speeds above 45 mph (72.5 kph).

Appendix C analyzes the need for highway shoulders to accommodate emergency and leisure stops on the roadway for LVR roads. This analysis looks at the expected number of head-on conflicts generated by a vehicle stopped in the roadway. The expected conflict rates range from 1 every 9 yr/mi of 50-vpd roadway to 54/yr/mi of 400-vpd roadway. An order of magnitude comparison shows that a road carrying 3,000 vpd is expected to have about 6,500 of these conflicts per mile per year. This would suggest that the hazard associated with stopped vehicles on LVR roads is relatively insignificant.

The conflict rate for the higher volume LVR roads might be considered as justifying shoulders to accommodate stopped vehicles. However, as discussed earlier, Appendix D already shows justification for shoulders for the higher (more critical) design speeds that generally correspond with the higher ADT categories. Therefore, no separate justification for shoulders is recommended.

Total Roadway Width

Total roadway width is defined here as the width of traveled way plus shoulders, if necessary. Appendix D presents a functional analysis of total roadway width requirements based on the safety of vehicular tracking and lateral clearances to opposing vehicles. Roadway width requirements for safe tracking were computed for various design speeds such that the tracking correction recovery was equivalent to that provided by a 36-ft roadway (two 12-ft lanes and 6-ft shoulders) at 60 mph (96.6 kph). Lateral clearances to opposing vehicles were related to design speed and traffic volume such that the total roadway width accommodated reasonable frequencies of head-on meetings (using App. G data) involving buses or large trucks. The minimum total roadway widths to satisfy both functions range from 18 ft (5.49 m) on a 50-vpd roadway designed for 20 mph (32.2 kph) to 30 ft (9.15 m) on a 400-vpd roadway designed for 50 mph (80.5 kph).

Roadside Safety

The AASHTO guidelines of 10- to 15-ft (0.31- to 0.61-m) roadside clear zones and 4:1 or flatter side slopes are related to acceptable safety performance and should be retained. They are, however, recognized as idealistic objectives when applied to existing LVR roads with limited rights-of-way. As such, these values serve as general guidelines in the continuum of "more is better."

A realistic treatment of roadside hazards on LVR roads depends on achieving a balance between the cost and safety effectiveness of the treatment. Appendix I of this report analyzes the cost-effectiveness of various roadside safety treatments on LVR roads using the developments of three research reports (*10, 12, 13*) dealing with the nature of roadside accidents. The results of this analysis, although they indicate very small safety contributions by individual roadside safety improvements, show that some low-cost improvements are cost-effective, especially on highway curves. On highway curves, tree removal and breakaway signposts, utility poles, and mailboxes are generally cost-effective for all LVR traffic volumes, and all reasonable unit costs of improvements. On highway tangents, these same improvements are only cost-effective for the higher LVR road volumes. Guardrail placement on steep slopes, the removal of unnecessary guardrail on flat slopes, and the flattening of steep but low embankments are also generally cost-effective on highway curves for the higher LVR road volumes. All other kinds of roadside improvements, including placing guardrails at fixed objects and moving fixed objects laterally, are not cost-effective.

RECOMMENDED RESEARCH

The intent of task 3 of this research was to recommend follow-on data collection activities leading to multivariate analysis relating highway design and traffic control elements to highway accidents on LVR roads. Review of several researches, however, demonstrates the futility of these kinds of studies, even for primary highways. And, of course, several of the probability analyses of this report clearly demonstrate that the frequencies of various critical events on LVR roads are very much smaller than on primary highways. Because of these factors, dependency on discrete empirical studies to isolate any functional relationships would be not only cost prohibitive but also potentially fruitless.

Although the multivariate analyses previously described do not appear feasible, some other kinds of studies might be helpful to either verify or modify the revised safety requirements developed in this project. For example, several of the developments used the safety-conservative assumption (either expressed or implied) that LVR roads have a 50-50 directional traffic split during all periods of the day. A study of continuous traffic counts on LVR roads with different ADT's would not only show just how conservative the 50-50 assumption is, but would also measure hourly volumes to verify the efficacy of the average hourly volumes and 18-hr traffic-flow periods assumed.

Two basic kinds of studies are recommended to verify, modify, or add further depth to the developments of this report. One study would collect accident data on LVR roads to draw a clearer picture of the current safety performance of LVR roads. The other study would collect on-site data of traffic characteristics on LVR roads for the purpose of verifying the revised safety requirements. A brief discussion of these studies is given as follows:

Accident Studies

Accident studies could be conducted at one of three levels of detail. The first level would compile accident data on LVR roads in general. The second level would attempt to further classify these data by several traffic volume categories for LVR roads. The third level would add to the second level by relating the accident data to some general quality measure (e.g., high-, medium-, or low-type design) of individual roads. In proceeding from each level to the next higher level, the difficulty and effort involved in collecting data become more demanding, and the feasibility of study becomes more uncertain because of limitations on existing data sources.

For the first level of study, accident data could be obtained from those states such as Missouri and North Carolina that have both many miles of LVR roads on the state highway system and accident records for those roads. Although the states may not be able to completely isolate LVR roads (400 vpd or less), some other classification may provide a sample that is mostly LVR roads.

The kinds of data desired for the first level of study include accident type, severity, and location. These kinds of data would provide general statistics on the proportions of the various accident types and would allow relative comparisons such as: (1) single versus multiple-vehicle accidents, (2) intersection versus mainline accidents, and (3) accidents on curves versus accidents on tangents. Adequate statistical reliability for this level of study would require a sample of about 10,000 mi-yr of accident data.

Although the second level of study would add considerable depth to the first level, its feasibility is not clear. Collecting traffic volume data for LVR roads is not a routine task in most jurisdictions. Therefore, some method, such as using personal estimates by local highway agency personnel, might have to be developed. If feasible, this level would allow classifying the comparative data of the first level into discrete traffic volume categories for LVR roads.

The third level of study, of course, is both the most desirable and the most difficult to accomplish. The goal here is to further classify the data of the second level to generally relate safety performance to some measure of design quality. Although most state highway agencies usually develop sufficiency ratings for their highway system, these ratings do not usually extend to LVR roads. Therefore, some form of either personal estimates by local highway agency personnel or on-site inspection by the project staff might be necessary.

One aspect of data collection that might ease the burden and make the second and possibly the third level of study more feasible, especially if secondary data sources are not available, is if several years of accident data are available. This would limit the number of miles of roadway for which some form of primary data would be necessary.

Another potential form of accident study would involve, say, 10 to 20 LVR road sections in a complete case-study analysis. Although this form of study would not be as statistically tractable as the three-level study previously described, it could provide some valuable insights on the safety performance of LVR roads.

Traffic Characteristics Studies

This study could be designed to measure several traffic characteristics to verify the adequacy of several assumptions used in the development of the revised safety requirements in this report. Highway sites could be instrumented with sensors and a multichannel recorder to simultaneously measure speed, speed profile, lateral placement, hourly volume, directional split, vehicle type, etc.

For complete statistical tractability, about 320 days of data collection would be necessary. This would include, for example, an average of 4 days of data at 4 sites each of four different design classifications within five categories of LVR traffic volumes. Although this is a very expensive kind of research, the fact that several kinds of data can be collected simultaneously makes the data collection very cost-effective, especially because these kinds of data are not presently available. Also, one possible modification to the general experimental plan described earlier is to eliminate one or more of the lowest volume categories. Based on the orders of magnitude of various probabilities calculated in this report, the data from the higher volume categories could probably be extrapolated to make reasonable estimates for these lower volume categories. Another expediency in the total study of LVR roads would be to conduct the accident studies and traffic characteristics studies together such that both the selection of study sites and the collection of primary and secondary data could be done simultaneously.

CHAPTER THREE

DESIGN AND TRAFFIC CONTROL GUIDELINES

These guidelines present some proposed revisions to the AASHTO design policies and to the Manual on Uniform Traffic Control Devices. These interim guidelines are related to the safety performance of low-volume rural

(LVR) roads, those carrying less than 400 vpd, and are intended to supplement the national policies. Each individual guideline presented here is intended to either replace or clarify an existing guideline as it applies to LVR roads. Table 1 gives the major geometric design and traffic control elements for LVR roads and shows how these proposed guidelines might supplement the national policies.

REVISED GUIDELINES FOR GEOMETRIC DESIGN OF LVR ROADS

The basic source of national standards, criteria, and guidelines for rural highway design is the 1965 AASHTO Bluebook, "A Policy on Geometric Design of Rural Highways." With a few changes, the major geometric design aspects as they apply to LVR roads are summarized in the 1970 AASHTO publication, "Geometric Design Guide for Local Roads and Streets." The revised geometric design guidelines presented here supplement this AASHTO Local Road Guide as described in the following.

Total Roadway Width

Total roadway width is defined here as the width of the traveled way plus shoulders (if needed). The primary functions of shoulders, in general, as based on safety performance related to providing additional roadway width for tracking corrections, head-on clearances, emergency stops, and leisure stops. Because of the very low frequencies of stopped vehicles on LVR roads, the frequencies of traffic conflicts associated with stop vehicles do not justify shoulders for shadowing stopped vehicles.

The proposed revisions for total roadway width on LVR roads are given in Table 2. Where values exceed 24 ft (7.3 m), shoulders should be provided as part of the total width. When compared to the sum of pavement width plus shoulder width specified in the AASHTO Local Road Guide, the revised values are smaller for the lower design speeds and larger for the higher design speeds.

The dimensions given in Table 2 are rationally based on satisfying the safety requirements both for vehicle tracking and for lateral clearances to opposing vehicles. Roadway width requirements for safe tracking for the various design speeds provide equal or greater tracking error recovery than that provided by a 36-ft (11-m) total roadway width at 60 mph (96.6 kph). Roadway width requirements for safe lateral clearances to opposing vehicles for the various design speeds are related to traffic volume such that the total roadway width accommodates reasonable frequencies of head-on meetings where one or both vehicles are buses or large trucks.

Four values of total roadway width are given for each design speed in Table 2. These four values derive from four different combinations of design vehicle widths used in the head-on clearance determination. With the design speed established, selecting the appropriate road width value depends, first, on the percent that buses and large trucks are of the highway ADT, and, second, whether the movement of large farm machinery is frequent enough to justify a wider roadway. The deciding values for the percentage of buses and large trucks are different for each ADT range as shown at the top of the table. In considering whether to design for the movement of large farm machinery, the definitions of "frequent" and "infrequent" are left to the discretion of the designer.

Horizontal Curvature

The modification to the general design of highway curves on LVR roads relates to curves with design speeds that are lower than the general highway design speed. Although this practice is not generally recommended, it may be the only practical alternative in mountainous terrain, for example. Then too, many older existing highways have a highway curve with a design speed lower than the general highway design speed. These are the curves that are usually marked with curve warning signs and advisory speed plates.

A tracking analysis similar to that used for total road width demonstrates, for the total roadway widths proposed in Table 2, that certain highway curves with design speeds below the general highway design speed can satisfactorily accommodate recovery from tracking errors by vehicles traveling at the highway design speed. Table 3 presents this correspondence as an allowable but not generally recommended practice. A highway curve with this allowable tolerance should always be marked with curve warning signs and with advisory speed plates displaying the design speed of the highway curve.

Roadside Design

The AASHTO Local Road Guide presents general guidelines for roadside design for safety. Its suggestions of 10- to 15-ft (0.31- to 0.61-m) roadside clear zones and 4:1 or flatter side slopes are related to acceptable safety performance and should be retained. These suggested values, however, are recognized as idealistic objectives in a "more is better" continuum as applied to existing LVR roads with limited rights-of-way.

A more realistic approach to roadside safety design on LVR roads depends on achieving a balance between the cost and safety effectiveness of the design treatment. For this purpose, these guidelines recommend: (1) the use of the roadside hazard model presented in a report by Glennon (*12*) to compare the relative hazard reduction of various roadside safety treatments; (2) the use of a multiplier of 4 to modify the referenced model for highway curves; (3) the use of the accident cost values by severity type presented in *NCHRP Report 162* (*21*) to compute the benefits of the various hazard reductions; and (4) the application of local values for the cost of roadside safety treatments to compute the benefit-cost balance for the various roadside treatments.

Although the application of this procedure to LVR roads (using typical cost ranges for various treatments) indicates that individual roadside safety treatments yield very small safety contributions, some low-cost improvements do appear to be cost-effective, especially on highway curves. For example, on highway curves, tree removal

TABLE 1

RECOMMENDED SPECIFICATIONS FOR DESIGN AND TRAFFIC CONTROL ELEMENTS ON LVR ROADS

Element	Use AASHTO Bluebook	Use AASHTO Local Road Guide	Modify AASHTO Local Road Guide	Replace AASHTO Local Road Guide Specification	Use MUTCD	Modify MUTCD
		DESIGN ELEMENTS				
Design Speed		X				
Stopping Sight Distance		X				
Passing Sight Distance		X				
Vertical Curvature		X				
Highway Grade		X				
Horizontal Alignment			X			
Cross Slope		X				
Shoulder Cross Slope	X					
Superelevation Runoff		X				
Total Roadway Width				X		
Shoulder Width				X		
Structure Width		X				
Roadside Safety			X			
Corner Sight Distance		X				
Other Design Aspects	X					
		TRAFFIC CONTROL ELEMENTS				
Speed Signs						X
Stop Signs					X	
Curve Warning Signs						X
Centerline Markings						X
No-Passing Stripes						X
Other Regulatory and Warning Devices					X	

and breakaway signposts, utility poles, and mailboxes appear to be cost-effective for all LVR road traffic volumes and all reasonable unit costs of treatment. On highway tangents, these same improvements do not appear as cost-effective except for the higher (say, greater than 200 vpd) LVR road traffic volumes.

Guardrail placement on steep slopes, the removal of unnecessary guardrail on flat slopes, and the flattening of steep but low embankments also appear to be cost-effective on highway curves for the higher LVR road traffic volumes. All other kinds of roadside safety treatments, including placing guardrail at fixed objects and moving fixed objects laterally, do not appear to be cost-effective.

REVISED GUIDELINES FOR TRAFFIC CONTROL DEVICES ON LVR ROADS

The basic source of national guidelines for the application of traffic control guidelines is the "Manual on Uniform Traffic Control Devices" (MUTCD). If this manual is interpreted literally, the only traffic control devices that are mandatory on LVR roads are crossbucks at railroad grade crossings. All other devices have generalized warrants or otherwise discretionary application.

In evaluating the application of the MUTCD to the safe operations of LVR roads, four devices require further clarification. These are: speed signs, curve warning signs, centerline markings, and no-passing stripes. The revised guidelines for these devices are presented next.

Speed Signs

On many highways, drivers tend to judge their safe speed by the geometric design, traffic characteristics, and roadside development of the highway. This suggests for LVR roads that, because of little roadside friction and infrequent encounters with other vehicles, geometric design is the primary determinant of vehicle speeds. Without the other controls, however, drivers might tend to overdrive LVR roads except where directly influenced by physical constraints such as horizontal curvature. Therefore, speed limit signs keyed to the design speed appear to be an important adjunct to the safe operation of LVR roads.

These guidelines recommend that all LVR roads have regulatory speed limit signs displaying their design speed. Signs should be placed at frequent enough intervals so that drivers will see them for most trips. Also, the speed limit should have zoned values that change as often as needed to maintain correspondence with localized general design speeds.

This practice will provide a consistent display and guide to drivers that indicate the maximum operating speed for LVR roads. For drivers who are good judges of geometric design conditions for setting their maximum operating speed, the speed limit signs will reinforce their judgment. For drivers who normally overdrive the geometrics, the speed limit signs will provide a persistent reminder of why they continually experience discomfort.

Curve Warning Signs

In conjunction with the revised guideline given previously for highway curve design, curve warning signs and advisory speed plates are recommended on all highway curves with design speeds below the general highway design speed and within the tolerance specified previously in Table 3. The MUTCD give guidelines for the location of these signs.

TABLE 2

MINIMUM ROAD WIDTH REQUIREMENTS

Design Speed (mph)[c]	Total Road Width Requirements (ft) [a] [b]			
	Lower % Busses & Trucks (as specified below): < 28% for 0- 50 ADT; < 12% for 51-100 ADT; < 7% for 101-200 ADT; NA for 201-400 ADT		Higher % Busses & Trucks (as specified below): ≥ 28% for 0- 50 ADT; ≥ 12% for 51-100 ADT; ≥ 7% for 101-200 ADT; All% for 201-400 ADT	
	Infrequent Trips by Farm Machinery[d]	Frequent Trips by Farm Machinery[d]	Infrequent Trips by Farm Machinery	Frequent Trips by Farm Machinery
20 mph	18 ft.	22 ft.	20 ft.	24 ft.
25	20	24	22	26
30	20	24	22	26
35	22	24	24	26
40	22	26	24	28
45	26	26	26	28
50	30	30	30	30

[a] 1 ft = .305 m

[b] Widths above 24 ft. (7.3m) include appropriate shoulder widths.

[c] 1 mph = 1.61 kph

[d] The determination of "frequent" and "infrequent" are at the discretion of the designer.

TABLE 3

MINIMUM DESIGN SPEEDS FOR HORIZONTAL CURVES THAT DEVIATE FROM THE GENERAL DESIGN SPEED OF THE HIGHWAY BUT DISPLAY CURVE WARNING SIGNS AND ADVISORY SPEED PLATES

Highway Design Speed (mph)	Minimum Design Speed of Deviant Curve (mph)
20	20
30	25
40	30
50	35

1 mph = 1.61 kph

Centerline Markings

This guideline clarifies the general warrants given by the MUTCD by providing a specific traffic volume warrant of 300 vpd or more. This warrant is based on: (1) an analysis of the frequency of head-on meetings, which is very low below this level; and (2) a benefit-cost analysis, which shows this level to be the balance point between accident reduction benefits and centerline costs.

No-passing Stripes

The MUTCD requires no-passing stripes where passing sight distance is restricted, but only when centerline markings are present. This revised guideline clarifies the MUTCD as it applies to LVR roads by saying no-passing stripes are *not* necessary for the safe operation of LVR roads. This guideline is based on: (1) an analysis of the frequency of head-on passing conflicts, which shows that this frequency is less than 14 per year for an ADT of 400 vpd and considerably less for lower ADT's; and (2) a benefit-cost analysis, which shows that the balance point between accident reduction benefits and striping costs applies to ADT's higher than 400 vpd.

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APPENDIX A

CURRENT SAFETY PERFORMANCE ON LVR ROADS

In analyzing the safety requirements for low-volume roads, it is first important to dimension their current safety performance. Table A-1, taken from "Fatal and Injury Accidents Rates for 1975," U.S. Department of Transportation (*28*), gives some general statistics. This table shows that, although local rural (county and township) roads constitute 58 percent of the total U.S. highway mileage, they experience only 11 percent of the fatal accidents and 8.4 percent of the injury accidents. Also, whereas local-rural roads have an average ADT of 105 vpd, the computation for all other roads shows an average ADT of 2,124 vpd or 20 times higher.

PER MILE ACCIDENT RATES RELATED TO TRAFFIC VOLUME

Per mile accident rates for the range of 0 to 400 vpd are not clearly documented anywhere. However, data from four sources (*11, 22, 26, 28*) can be used to generate reasonable estimates of the average accident experience.

Table A-2 gives component accident rates and proportions for low-volume rural roads in three states taken from three of the referenced sources. Although the numbers indicate a wide divergence for some states, these variances are probably explained by: (1) differences in reporting levels (which are very sensitive at these levels of accident rate); (2) differences in the distributions of ADT, road types, and terrain; (3) common errors in retrieval experienced to different degrees in most state accident data systems.

Considering Table A-2, the Louisiana data are discounted because of obvious discrepancies, and the Missouri rates are judged as slightly high because they come from a straightline regression that does not pass through the origin and was generated from data for ADT's greater than 700 vpd. Figure A-1 is a best fit plot of the data in Tables A-1 and A-2. This figure was generated as follows:

1. The percent of fatal plus injury accidents was estimated for Table A-2 as a constant 47 percent of total accidents.
2. Using the 47 percent, the total accident rate at 105 vpd (from Table A-1) was computed as 0.155 acc./mi/yr.
3. The total accident curve was drawn as a best fit through the origin, through 0.155 acc./mi/yr at 105 ADT, and dividing the three points for Ohio, California, and Idaho in the 325 to 334 ADT range.
4. The fatal plus injury accident curve was drawn as 47 percent of the total accident curve.
5. The single-vehicle accident rate was estimated from Table A-2 as 55 percent at an ADT of 330 vpd. This yields a single-vehicle accident rate of 0.178 acc./mi/yr at 330 vpd.
6. Because the single-vehicle accident curve must pass through the origin, and the proportion of single-vehicle accidents should be close to 100 percent at an ADT of 1 vpd, the single-vehicle accident curve was drawn as a descending percentage of the total accident curve. The following adjustments were made to the Missouri regression to estimate this curve that passes through 0.178 acc./mi/yr at 330 ADT.

ADT	Missouri Regression Point	Adj. Percent	Adj. Rate
0	0.159	0	0.000
100	0.173	50	0.087
200	0.187	72	0.134
300	0.202	83	0.168
400	0.216	93	0.201

Summarizing (Fig. A-1), total accident rates range from 0.098 acc./mi/yr at 50 vpd to 0.367 acc./mi/yr at 400 vpd. In other words, the average road with 50 vpd will have 1 acc./mi/yr for every 10.2 mi. And, the rate for a road with 400 vpd is 1 acc./yr for every 2.7 mi. The reader should recognize that these rates are estimated averages and that particular roads, or sections thereof, might produce average annual accident rates that are substantially higher or lower than these rates. Nevertheless, these estimates do provide reasonable order-of-magnitude comparisons for the evaluation of safety requirements.

VERIFICATION OF ACCIDENT RATE RELATIONSHIPS

The fatal plus injury accident curve can be verified from Table A-1, which shows that 2,208,607 mi of road with an average ADT of 105 vpd produce 160,827 fatal plus injury accidents per year. The verification procedure is as follows: (1) estimate the traffic volume distribution of LVR

TABLE A-1

GENERAL ACCIDENT STATISTICS (*28*)

	Local Rural Roads	Total Roads	Percent Local of Total
Mileage[a]	2,208,607	3,814,499	58.0
Million vehicle mile/year[a]	84,626	1,329,359	6.4
Average ADT	105	955	9.1
Fatal accidents/year	4,299	39,993	11.0
Injury accidents/year	156,528	1,861,131	8.4
Fatal accidents/100 million vehicle miles	5.08	3.01	169.0
Fatal and injury accidents/ million vehicle miles	1.90	1.44	132.0
Fatal and injury accidents/ mile year[a]	0.0728	0.501	14.6

a/ One mile = 1.61 kilometers.

TABLE A-2

EMPIRICAL ACCIDENT DATA FOR LOW-VOLUME ROADS

	1962 Ohio (22)	1961-62 California (22)	1962 Louisiana (22)	1976 Idaho (28)		1974 Missouri (11)			
Miles of Road	3,995	5,014	5,534	230	310	-	-	-	-
ADT Range	0-600	0-600	0-600	0-250	250-400	-	-	-	-
Average ADT	332	334	296	150	325	100	200	300	400
Accident/Mile/Year	0.300	0.340	0.080	0.169	0.328	-	-	-	-
Fatal Accident/Mile/Year	0.0058	0.0122	0.0029	-	-	-	-	-	-
Fatal and Injury Accident/Mile/Year	0.141	0.170	0.038	0.073	0.140	-	-	-	-
PDO Accidents/Mile/Year	0.159	0.170	0.042	0.188	-	-	-	-	-
Single Vehicle Accidents/Mile/Year	0.166	0.219	0.036	-	-	0.173	0.187	0.202	0.216
Multi Vehicle Accidents/Mile/Year	0.134	0.121	0.044	-	-	-	-	-	-
Percent Fatal and Injury	47.0	50.0	47.5	43.0	42.8	-	-	-	-
Percent Single Vehicle	55.5	64.4	45.0	-	-	-	-	-	-

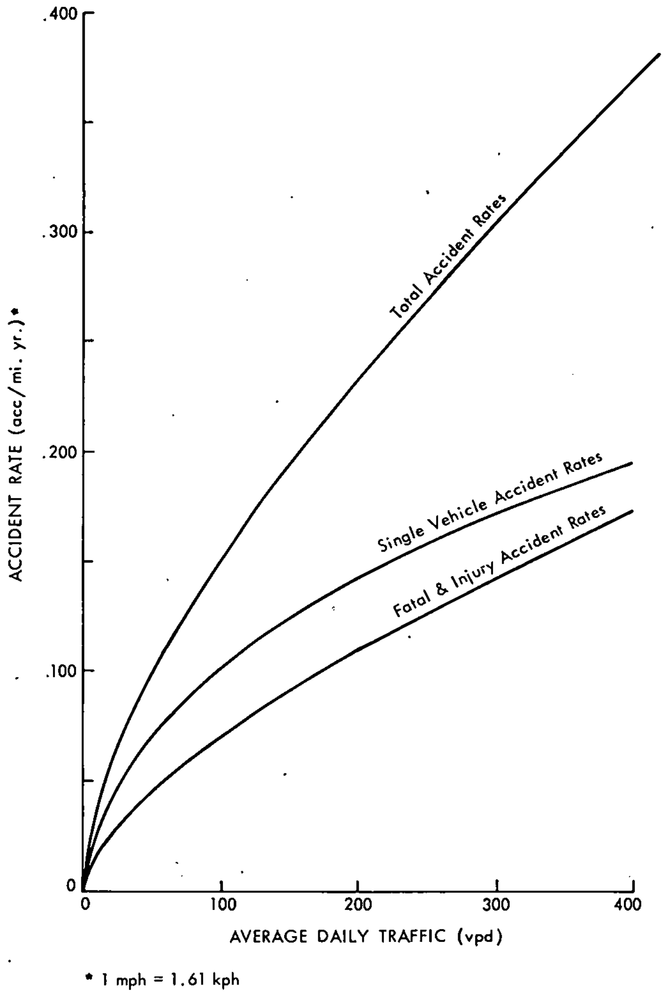

Figure A-1. Estimated safety performance of existing low-volume rural roads.

roadways with the average ADT of 105 vpd; (2) compute the mileage of each ADT class within this distribution; (3) multiply the mileage of each ADT class by the fatal plus injury accident rate for the midpoint of that class as taken from Figure A-1; (4) add the annual number of fatal plus injury accidents for each class to obtain the estimated total annual number of fatal plus injury accidents; and (5) compare the estimated number with the actual number (160,827).

Table A-3 shows the estimated distribution of mileage within given ADT classes that will yield an average ADT of 105 vpd. Although this distribution could vary slightly, it cannot be too much different and still yield an average ADT of 105 vpd.

Table A-4 shows the additional calculations to derive the estimated annual number of fatal plus injury accidents for local-rural roads. The estimated number of 151,114 fatal plus injury accidents is within 6 percent of the actual number, which is a reasonable tolerance for verification of Figure A-1.

PROPORTION OF HAZARD BY ACCIDENT TYPE

Another interesting aspect of the safety performance of LVR roads is the proportion of total hazard contributed by the various types of accidents. Hazard is defined here as the annual number of fatal and injury accidents per mile.

The proportion of the total hazard contributed by the various accident types can be estimated by combining the accident rates shown in Figure A-1 with the proportion of accidents by type reported for two-lane rural highways by Agent and Deen (*4*). Tables A-5 through A-9 show these calculations for LVR roads ranging in ADT from 50 to 400 vpd. To arrive at the percent of total hazard, estimates first required an adjustment of the Agent and Deen averages to reflect the percent of single-vehicle accidents by ADT shown in Figure A-1. After changing the single-vehicle category to the appropriate percentage, the remaining categories were adjusted proportionally to account for the remaining percentage. Next, using Agent and Deen's severity rankings, and considering the average percent of fatal plus injury accidents of 47 percent used in Figure A-1, the percent of fatal plus injury accidents by accident type was estimated. These percentages were estimated such that the summation of the second column numbers multiplied by the fourth column numbers approximates 47 percent. The fatal plus injury accident rate for each accident type was then calculated by multiplying the total accident rate by both the Column 2 number and by the Column 4 number for that type. The percent of total hazard, then, is simply the percent of total fatal and injury accidents contributed by each type of accident.

Tables A-5 through A-9 demonstrate very clearly that single-vehicle accidents contribute a significant majority of the total accident hazard on LVR roads—ranging from 61 percent at 400 vpd to 84.9 percent at 50 vpd. Therefore, any consideration of safety requirements ought to consider, first, the reduction of off-road vehicular encroachments and, second, the minimization of accident consequences given an off-road encroachment. For LVR roads, the total consequences of all accident types other than the single-vehicle type appear to be fairly insignificant and therefore difficult to deal with in any meaningful way.

PER MILE COST OF ACCIDENTS

Another way of looking at the current safety performance of LVR roads is to evaluate the impact of accident costs. *NCHRP Report 162* (*21*) reports the following accident costs by severity type: fatal accident = \$235,000, injury accident = \$11,200; and property damage only (PDO) accident = \$500.

Using these cost figures and deriving from Tables A-1 and A-2 a safety conservative average of 1 fatal accident for every 25 injury accidents, the average cost of an accident in the fatal plus injury category is estimated at \$19,600. Using this estimated cost, the \$500 figure for PDO accidents, and the previously developed 47 percent for the proportion of fatal plus injury accidents, the average cost of all accidents is calculated at \$9,500.

TABLE A-3

ESTIMATED DISTRIBUTION OF MILEAGE WITHIN GIVEN ADT CLASSES THAT WILL YIELD AVERAGE ADT OF 105 VPD

ADT Range (vpd)	Midpoint of Range (vpd)	Estimated Percent of Total Mileage	Contribution to Average ADT (vpd)
0 - 50	25	20	5.0
51 - 100	75	45	33.8
101 - 200	150	25	37.5
201 - 300	250	6	15.0
301 - 400	350	4	14.0
Average ADT			105.3

TABLE A-4

ESTIMATED ANNUAL NUMBER OF FATAL PLUS INJURY ACCIDENTS FOR LOCAL-RURAL ROADS

ADT Range (vpd)	Mileage in ADT Range	Fatal/Injury Rate for Midpoint of Range (accident/mile/year)	Annual Number of Fatal/Injury Accidents
0 - 50	441,722	0.030	13,252
51 - 100	993,873	0.058	57,645
101 - 200	552,152	0.090	49,694
201 - 300	132,516	0.125	16,565
301 - 400	88,344	0.158	13,958
	2,208,607		151,114

TABLE A-5

CONTRIBUTION TO TOTAL ACCIDENT HAZARD BY ACCIDENT TYPE FOR ADT = 50

Accident Type	Percent by Type (Agent and Deen)	Adjusted Percent by Type for 71.4% Single Vehicle	Severity Rank (Agent and Deen)	Estimated Percent Fatal + Injury	Fatal and Injury Rate (Accident/Mile/Year)	Percent of Total Hazard
One Vehicle	33.9	71.4	2	53	0.0371	77.8
Rear-End (plus same direction sideswipe)	30.5	13.2	5	30	0.0039	8.2
Head-On (plus opposite direction sideswipe)	19.2	8.3	3	52	0.0042	8.8
Miscellaneous	8.8	3.8	6	25	0.0009	1.9
Angle	6.3	2.7	4	45	0.0012	2.5
Pedestrian	1.3	0.6	1	70	0.0004	0.8
	100.0	100.0			0.0477	100.0

TABLE A-6

CONTRIBUTION TO TOTAL ACCIDENT HAZARD BY ACCIDENT TYPE FOR ADT = 100

Accident Type	Percent by Type (Agent and Deen)	Adjusted Percent by Type for 67.6% Single Vehicle	Estimated Percent Fatal + Injury	Fatal and Injury Rate (Accident/Mile/Year)	Percent of Total Hazard
One Vehicle	33.9	67.6	53	0.0534	74.5
Rear-End	30.5	15.0	30	0.0067	9.3
Head-On	19.2	9.4	52	0.0073	10.2
Miscellaneous	8.8	4.3	25	0.0016	2.2
Angle	6.3	3.1	45	0.0021	2.9
Pedestrian	1.3	0.6	70	0.006	0.8
	100.0	100.0		0.0717	100.0

TABLE A-7

CONTRIBUTION TO TOTAL ACCIDENT HAZARD BY ACCIDENT TYPE FOR ADT = 200

Accident Type	Percent by Type (Agent and Deen)	Adjusted Percent by Type for 61.3% Single Vehicle	Estimated Percent Fatal + Injury	Fatal and Injury Rate (Accident/Mile/Year)	Percent of Total Hazard
One Vehicle	33.9	61.3	53	0.0747	68.9
Rear-End	30.5	17.9	30	0.0124	11.4
Head-On	19.2	11.3	52	0.0135	12.4
Miscellaneous	8.8	5.1	25	0.0029	2.7
Angle	6.3	3.6	45	0.0037	3.4
Pedestrian	1.3	0.8	70	0.0013	1.2
	100.0	100.0		0.1085	100.0

TABLE A-8

CONTRIBUTION TO TOTAL ACCIDENT HAZARD BY ACCIDENT TYPE FOR ADT = 300

Accident Type	Percent by Type (Agent and Deen)	Adjusted Percent by Type for 56.5% Single Vehicle	Estimated Percent Fatal + Injury	Fatal and Injury Rate (Accident/Mile/Year)	Percent of Total Hazard
One Vehicle	33.9	56.5	53	0.0904	64.0
Rear-End	30.5	20.5	30	0.0186	13.2
Head-On	19.2	13.0	52	0.0204	14.4
Miscellaneous	8.8	6.0	25	0.0045	3.2
Angle	6.3	4.2	45	0.0057	4.0
Pedestrian	1.3	0.8	70	0.0017	1.2
	100.0	100.0		0.1413	100.0

TABLE A-9

CONTRIBUTION TO TOTAL ACCIDENT HAZARD BY ACCIDENT TYPE FOR ADT = 400

Accident Type	Percent by Type (Agent and Deen)	Adjusted Percent by Type for 52.9% Single Vehicle	Estimated Percent Fatal + Injury	Fatal and Injury Rate (Accident/Mile/Year)	Percent of Total Hazard
One Vehicle	33.9	52.9	53	0.1029	61.1
Rear-End	30.5	21.7	30	0.0239	14.2
Head-On	19.2	13.7	52	0.0261	15.5
Miscellaneous	8.8	6.3	25	0.0058	3.4
Angle	6.3	4.5	45	0.0074	4.4
Pedestrian	1.3	0.9	70	0.0023	1.4
	100.0	100.0		0.1684	100.0

TABLE A-10

COST IMPACT OF ACCIDENTS AND POTENTIAL COUNTERMEASURES ON LVR ROADS

ADT	Accident Mileage Rate (Accident/Mile/Year)	Average Accident Cost per Mile per Year ($)	Maximum Annual Cost per Mile Justified for Accident Countermeasure to Achieve Various Percentage Accident Reductions ($/Mile/Year)		
			10%	25%	50%
50	0.070	665	67	166	332
100	0.124	1,180	118	295	590
200	0.216	2,050	205	512	1,025
300	0.300	2,850	285	712	1,425
400	0.376	3,570	357	892	1,785

Table A-10 presents the current cost impact of accidents per mile of LVR road using the $9,500 average. Also given are the break-even average annual costs for countermeasures that achieve various percentage accident reductions. What these figures show is that, even with safety-conservative unit values for the cost of accidents, only relatively low-cost kinds of improvements can be justified in a benefit-cost break-even analysis.

APPENDIX B

EXAMINATION OF NATIONAL POLICIES RELATIVE TO THE SAFETY OF DESIGN AND OPERATIONAL ELEMENTS ON LVR ROADS

This appendix summarizes portions of the AASHTO geometric design policies and the Manual on Uniform Traffic Control Devices. This exercise was conducted early in the project period for the purpose of defining and evaluating the elements of interest on LVR roads.

No attempt is made here to reproduce verbatim all of the pertinent criteria and standards of the national policies. Rather, this appendix summarizes the policies with reference to page numbers in the policy books where appropriate. Also given is an evaluation of the functional relationship, if any, of the standards and criteria to the safety performance of LVR roads.

EXAMINATION OF THE MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES

If the 1971 Manual on Uniform Traffic Control Devices (MUTCD) is interpreted literally, the only traffic control devices that are mandatory on LVR roads are crossbucks at railroad crossings. All other devices have generalized warrants or otherwise discretionary application.

Although most regulatory and warning signs probably apply under certain circumstances on LVR roads, this application must remain discretionary because of the lack of explicit knowledge on their safety effectiveness. The remainder of this discussion will concentrate on five traffic control devices of particular concern on LVR roads. These are speed signs, stop signs, curve warning signs, centerline markings, and no-passing stripes.

Speed Signs

The MUTCD does not specifically require speed signs, but the language would strongly suggest this. The MUTCD states that speed limit signs are used to display either a limit established by law or by regulation according to the findings of a traffic engineering study. Factors to consider in an engineering study include the geometric design, traffic characteristics, and roadside development of the highway. For most highways, drivers tend to judge their appropriate safe speed range by these three factors. This would suggest for LVR roads that, because of minimum roadside friction and relatively infrequent encounters with other vehicles, geometric design elements are the major determinants of vehicle speeds. Without the other controls, however, drivers might tend to overdrive LVR roads except where directly influenced by physical constraints such as horizontal curvature. For this reason, speed limit signs keyed to the design speed of the highway appear to be an important adjunct to the safe operation of LVR roads.

Stops Signs

The MUTCD lists three general conditions where a two-way stop may be warranted. These are:

1. The intersection with a main road where application of the normal right-of-way rule is unduly hazardous.
2. The intersection with a through highway.
3. Other intersections where a combination of high speed, restricted view, and serious accident record indicate the need for a stop sign.

These warrants are general and do not lend themselves easily to a consistent application on LVR roads. Therefore, in Appendix H of this report, the cost-effectiveness of stop signs is evaluated at intersections with various LVR road traffic volumes.

Curve Warning Signs

The MUTCD says that curve warning signs are intended where engineering studies show the recommended speed on the curve to be between 30 and 60 mph and equal to, or less than, the speed limit of the highway. It also suggests that an advisory speed limit might be appropriate.

For many LVR roads, horizontal curvature may be the major control on maximum safe speed. For this reason, the curve warning signs and an advisory speed plate showing the curve design speed should be posted on all curves fitting the MUTCD specification.

Centerline Markings

The MUTCD does not require centerline markings, but does list some conditions where they are desirable. For LVR roads, the major condition is on pavements of 16 ft, or more, with prevailing speeds greater than 35 mph. Although this requirement is functionally related to safety performance by considering speed and highway width, it does not consider the frequency of head-on meetings, which are related to traffic volume. Appendix G considers the functional need of centerlines related to safety performance for various LVR road volumes.

No-Passing Stripes

The MUTCD requires no-passing stripes where sight distance is restricted, but only when centerline markings are present. The sight distances at which no-passing zones are required for various 85th percentile operating speeds are given on page 190 of the MUTCD. These values, which range from 500 ft at 30 mph to 1200 ft at 70 mph, are based on an evaluation of passing requirements but not in a critical mode (*15*).

In applying these standards to LVR roads, the question is whether the low probabilities of the three-car passing conflict situation even justify the use of the stripes at all. To answer this question, Appendix F of this report analyzes the frequency of passing conflicts for various LVR road volumes.

EXAMINATION OF AASHTO GEOMETRIC DESIGN POLICIES

With some exceptions, the geometric design criteria, guidelines, and standards for all types of highways can be found in the 1965 AASHTO Bluebook "A Policy on Geometric Design of Rural Highways." With a few changes, the major design aspects as they apply to LVR roads given in the 1965 AASHTO Bluebook are summarzied in the 1970 AASHTO publication, "Geometric Design Guide for Local Roads and Streets." The major changes in this AASHTO Local Road Guide deal with the specification of design speed. The AASHTO Local Guide specifies minimum design speeds related to the volume of traffic and type of terrain on LVR roads. In this process, a design speed of 20 mph is allowed. Therefore, the other major change is to specify design values at a 20-mph design speed for sight distance, horizontal curvature, vertical curvature, superelevation runoff, width of surfacing, and width of shoulders.

The considerations of geometric design pertinent to LVR road operations are discussed in the following paragraphs. In the discussion of each design aspect, the AASHTO standards are referenced, their direct relation to safety performance is analyzed, and a statement of suitability is made.

Design Speed

The 1970 AASHTO Guide for Local Roads, on page 5, specifies minimum design speeds ranging from 20 mph to 50 mph depending on the traffic volume and type of terrain on LVR roads. Lower ADT's and more severe terrain justify lower minimum design speeds, and higher ADT's and more level terrain justify higher minimum design speeds. Given that design engineers apply the basic premises of design consistency, these design speed specifications generally will allow the design engineer to balance the objectives of safety, service, and economy consistent with roadway function and expected operations.

Stopping Sight Distance

The minimum standards for stopping sight distance are given on page 6 of the 1970 AASHTO Local Road Guide. These minimum standards are based on a functional analysis of the requirements for safe stopping related to design speeds. As such, these standards are directly applicable to LVR roads using design speed as the basic performance criterion.

Passing Sight Distance

The minimum standards for passing sight distance are given on page 6 of the 1970 AASHTO Local Road Guide. These standards are based on a functional analysis of the requirements for safe passing related to design speed. Although these standards are directly applicable to LVR roads using design speed as the basic performance criterion, unlike stopping sight distance standards, they are not applied continuously along the highway. The following operational statement from page 146 of the 1964 AASHTO Bluebook guides the general application of passing sight distance to design:

> *On highways with high volumes that approach capacity, frequent and long passing sections are essential. On highways with intermediate to low volumes, the need is not as great but passing sections are still an important adjunct for efficiency and safety.*

Horizontal Alignment

The minimum standards for horizontal curvature related to superelevation rate are given on page 7 of the AASHTO Local Road Guide. These standards are based on a functional analysis of a vehicle's ability to avoid lateral skidding. As such, these standards are directly applicable to LVR roads using design speed as the basic performance criterion.

Other elements of horizontal alignment discussed by the two AASHTO publications are spiral curves and superelevation runoff. Spiral curves are discussed in the 1965 AASHTO Bluebook and are encouraged but not required. Minimum superelevation runoff values related to design speed on LVR roads are given on page 8 of the 1970 AASHTO Local Road Guide. These values appear to be reasonably related to safety performance.

Highway Grades

The guidelines for maximum highway grades are given on page 195 of the 1965 AASHTO Bluebook and repeated on page 6 of the 1970 AASHTO Local Road Guide. For low-volume roads, allowance is made for grades 2 percent higher than those for main highways. These values for LVR roads range from 5 percent for a 60-mph design speed in flat terrain to 12 percent for a 20-mph design speed in mountainous terrain.

Although percent and length of grade have some bearing on safety performance, the AASHTO values as based on acceptable traffic service levels related to the speed and acceleration performance of low-performance trucks on grades. The major safety consideration involving highway grades is to minimize rear-end accidents involving low-performance vehicles. But, as shown in Appendix A, rear-end accidents in general are not a major problem on LVR roads. Therefore, the AASHTO guidelines for maximum highway grades appear to be acceptable for safety on LVR roads.

Vertical Curvature

The minimum design controls for crest vertical curvature are given on page 6 of the 1970 AASHTO Local Road Guide. These minimum values are functionally based on safety performance by providing for minimum stopping sight distance at each design speed. For purposes of correspondence, the sight line is defined from a 3.75-ft driver eye height to the top of a 0.5-ft object on the road. Although the 0.5-ft object is somewhat arbitrary, it is abstractly based on a rational balance between the downstream height of the line of sight (safety) and the length of vertical curve (cost). Therefore, these design controls are directly applicable to LVR roads using design speed as the basic performance criterion.

Guidelines for the length of sag vertical curves are given on page 6 of the 1970 AASHTO Local Road Guide. Like crest vertical curves, these guidelines are functionally based on safety performance by providing headlight distances greater than minimum stopping sight distances at each design speed. As such, these guidelines are directly applicable to LVR roads using design speed as the basic performance criterion.

Cross Slope

Guidelines for cross slope are given on page 8 of the 1970 AASHTO Local Road Guide. These values are a reasonable compromise between two contradictory safety controls. A steep lateral cross slope is desirable to minimize ponding on the highway, whereas a flat lateral cross slope is desirable for ease of vehicle tracking.

The cross slope values vary according to surface type. They are lower on high-type surfaces where speeds are higher, and higher on low-type surfaces where speeds are lower and drainage is more difficult because of loose materials.

These guidelines appear as a rational balance between two safety-performance aspects common to LVR roads.

Curbs

Curbs have been used on rural roads in the past as expedient devices for handling pavement runoff. The 1965 AASHTO Bluebook offers several cautions against the use of curbs on rural highways. Not only are they hazardous because they contribute to ponding of water, but they also contribute to loss of vehicular control when inadvertently hit. As such, curbs should be avoided whenever possible on LVR roads.

Shoulder Widths

The 1965 AASHTO Bluebook lists 11 functions for highway shoulders. Among these, five are directly related to safety as follows:

1. Emergency stops.
2. Leisure stops.
3. Space for evasive maneuvers to avoid collisions.
4. Increased sight distance in cut sections.
5. Lateral clearance for signs and guardrails.

Although the Bluebook provides separate guidelines for shoulder width on LVR roads, it is inconsistent. On page 234, it says:

> *. . . The shoulder on minor rural roads with low traffic volume serves essentially as structural lateral support for the surfacing and as an additional width for the narrow traveled way.*
> *. . . It varies in width from only two feet or so on minor rural roads . . .*

On page 235, it says:

> *A minimum shoulder width of 4 feet should be considered for the lowest type highway, and preferably a 6- or 8-foot width.*

Not only is the Bluebook inconsistent regarding shoulder width, but it has not directly related it to safety performance in any way. The results of several studies show conflicting effects of shoulder width on accidents for two-lane highways in general (*17*). However, analysis of all the studies that show that accidents decrease with increasing shoulder width will show the lack of statistical control for traffic volume. This lack of control probably means that the accident trend was more directly related to traffic volume, because shoulders tend to be wider for higher traffic volume.

The foregoing discussion raises questions on whether shoulders are really needed for safety on LVR roads. Consequently, this need is evaluated in two appendixes of this report. In Appendix D, the need for shoulders and the width of shoulders are analyzed as an adjunct to the traveled way to accommodate reasonable tracking corrections and head-on clearances. In Appendix C, shoulders need is analyzed for emergency and leisure stops on the highway.

Shoulder Cross Slope

The guidelines for shoulder cross slope are given on page 237 of the AASHTO Bluebook. For the most part the considerations are similar to those for pavement cross slope. One additional consideration is given on page 251 as follows:

> *The algebraic difference in cross slopes at the pavement edge should not exceed about 0.07 in order to avoid a hazardous roll-over effect.*

The guidelines for shoulder cross slope are rationally determined on the basis of safety performance and are, therefore, directly applicable to LVR roads.

Roadside Obstructions

The 1965 AASHTO Bluebook does not treat the safety considerations of roadside obstructions in much detail, although some general guidelines are given for guardrail placement on pages 242-243.

The 1970 AASHTO Local Road Guide provides an additional suggestion for the clearance to roadside obstructions as follows:

> *Where the design speed is less than 50 mph, or ADT is less than 750, a clear roadside recovery area should be provided, preferably 10 to 15 feet or more from the edge of the through traffic lane. The clear roadside area should be an appropriate flat and rounded cross-section design. Exceptions may be made (1) in cut sections where fixed objects are located sufficiently up the cut slope so that there is little likelihood that they would be struck and (2) where guardrail protection is provided.*

These recommendations are recognized as idealistic when applied to existing roadways because of limited rights-of-way that were set years ago. The treatment of roadside hazards for LVR roads depends on achieving a balance between the cost and safety effectiveness of the treatment. Appendix I of this report analyzes the cost-effectiveness of various roadside safety treatments.

Side Slopes

The 1965 AASHTO Bluebook, on page 244, offers the general suggestion that side slopes should be 4:1 or flatter whenever possible. Although this is a reasonable general guideline for safety, it does not consider the marginal cost-effectiveness of different slopes on roadways of widely different traffic volumes. Therefore, Appendix I of this report offers a cost-effectiveness comparison of various side slopes for LVR roads.

Roadway Width

The current minimum standards for widths of surfacing and shoulders are given in the 1970 AASHTO Local Road Guide. Together, the two widths define minimum roadway widths. These roadway width standards do not appear to be functionally related to the safety performance of the highway. For example, the minimum total roadway width of 28-ft is the same for both a 50-vpd roadway at a 20-mph design speed and a 400-vpd roadway at a 50-mph design speed. Appendix D of this report functionally relates the safety of vehicle tracking and head-on vehicle clearances to the design speed and traffic volume on LVR roads.

Structure Width

The 1970 AASHTO Local Road Guide specifies minimum structure widths of 4 ft greater than the pavement width for new or reconstructed bridges. Minimum structure width for bridges to remain in place are specified as 20 ft on roadways carrying 250 vpd, or less, and 22 ft on roadways carrying 250 to 400 vpd. These widths appear to provide a reasonable balance between construction cost and safety performance.

Intersection Design

The most important aspect of intersection design on LVR roads is corner sight distance. The 1970 AASHTO Local Road Guide specifies sight distances related to design speed that would enable a stopped vehicle to cross a major highway. These values appear to be reasonably related to the safety performance of LVR roads.

APPENDIX C

ANALYSIS OF NEED FOR SHOULDERS ON LVR ROADS

This appendix considers only whether highway shoulders are needed to shadow and store stopped vehicles. The need for shoulders as additional roadway width for tracking corrections is treated in Appendix D, which analyzes total road width requirements.

The relative hazard of a highway with no shoulders can be estimated by evaluating the additional conflicts created by vehicles stopped on the traveled way rather than on a shoulder. Vehicles stopped on the traveled way present a hazard, first, to following vehicles and, second, to opposing vehicles when following vehicles pull into the left lane to pass the stopped vehicle. The hazard to following vehicles, per se, is judged as insignificant if adequate stopping sight distance has been designed for. With adequate stopping sight distance, the following vehicle driver should have more than enough time and distance to either stop or pull into the left lane. The critical situation, therefore, involves the head-on conflicts created by a stopped vehicle. An estimate of the number of head-on conflicts per mile per day can be calculated using the following dimensional relationship:

$$N_C = N_S \times N_{A/S} \times N_{C/A} \tag{C-1}$$

where: N_C = number of head-on conflicts per mile per day created by a stopped vehicle;
N_S = number of stopped vehicles per mile per day;
$N_{A/S}$ = number of following vehicle arrivals per stopped vehicle; and
$N_{C/A}$ = number of conflicts (opposing vehicle arrivals) per following vehicle arrival.

The following sections analyze the components of Eq. C-1, calculate the expected conflict rates for various LVR road volumes, and draw conclusions regarding the need for highway shoulders.

FREQUENCY AND DURATION OF STOPPED VEHICLES

The term, N_S, in the preceding equation can be estimated from data presented by Billion (*9*). He studied stops on the shoulder for both leisure and emergency purposes. For leisure stops, the frequency is one stop every 2,400 veh-mi (3,864 km-mi) with an average duration of 30 min. For emergency stops, the frequency is one every 1,140 veh-mi (1,835 km-mi) with no average duration reported. For this analysis, a safety-conservative estimate for the duration of emergency stops is 60 min.

Safety-conservative estimates of the number of stopped vehicles per mile per day can be derived from the Billion rates recognizing that the adaptive behavior of drivers on roads without shoulders would probably yield a lower frequency and duration of total stops on the roadway. Estimates of the number of stopped vehicles per mile per day for leisure stops, N_{SL}, and for emergency stops, N_{SE}, calculated from the Billion data, are as follows.

ADT	Number/Mile/Day N_{SL}	N_{SE}
50	0.02083	0.04386
100	0.04167	0.08772
200	0.08330	0.17540
300	0.12500	0.26310
400	0.16670	0.35090

Note: 1 mile = 1.61 km

NUMBER OF FOLLOWING VEHICLE ARRIVALS PER STOPPED VEHICLE

The number of following vehicle arrivals per stopped vehicle can be estimated as the average one-direction traffic volume over the duration of the stopped vehicle, as follows:

$$N_{A/S} = vD \quad \text{(C-2)}$$

where: v = average one-direction hourly traffic volume; and
D = duration of stopped vehicle, hours.

By using the safety-conservative assumptions that the total daily traffic occurs over an 18-hr period with a 50-50 directional split, the number of following vehicle arrivals per leisure stop, $N_{A/SL}$, and per emergency stop, $N_{A/SE}$, are calculated as follows.

ADT	$N_{A/SL}$	$N_{A/SE}$
50	0.694	1.389
100	1.389	2.778
200	2.778	5.556
300	4.167	8.333
400	5.556	11.111

NUMBER OF CONFLICTS PER FOLLOWING VEHICLE ARRIVAL

Calculating the number of head-on conflicts (opposing vehicle arrivals), while a following vehicle is in the left lane, requires an assumption on the duration of left-lane exposure. The following vehicle is assumed to be at risk for 5 sec. Assuming equal speeds for following and opposing vehicles, opposing vehicles having arrived 5 sec earlier can be within the hazard area defined by the following vehicle maneuver. Therefore, the arrival duration for opposing vehicles is estimated at 10 sec.

The number of conflicts per following vehicle arrival, therefore, is estimated using the following equation:

$$N_{C/A} = vt/3600 = 10v/3600 = v/360 \quad \text{(C-3)}$$

where: v = average one-direction hourly traffic volume; and
t = conflct exposure time, sec.

Again, by using the safety-conservative assumptions that the total daily traffic occurs over an 18-hr period with a 50-50 directional split, the number of head-on conflicts per following vehicle arrival are calculated as follows.

ADT	$N_{C/A}$
50	0.003858
100	0.007716
200	0.015432
300	0.023148
400	0.030864

NUMBER OF HEAD-ON CONFLICTS

To calculate the number of head-on conflicts associated with stopped vehicles, the basic equation given at the beginning of this appendix is restructured to account for the two kinds of vehicle stops, as follows:

$$N_C = N_{C/A}(N_{SL}\, N_{A/SL} + N_{SE}\, N_{A/SE}) \quad \text{(C-4)}$$

By using the previous developments of this appendix, the number of head-on conflicts per mile per day is estimated for various LVR road volumes as follows.

ADT (vpd)	Number of conflicts/mile/day (N_C)
50	.0003
100	.0023
200	.0186
300	.0628
400	.1489

Note: 1 mile = 1.61 km

Also interesting is to compare these expected rates with those calculated for higher volume roadways. For example, a two-lane road with no shoulders carrying 3,000 vpd is expected to have about 18 conflicts/day. This rate is 125 times that for the 400-vpd roadway and 60,000 times that for the 50-vpd roadway. Clearly then, the expected values for LVR roads are a large order of magnitude smaller than the expected values for major two-lane roadways.

EVALUATION OF SHOULDER NEED

The foregoing order of magnitude comparison suggests that the hazard associated with stopped vehicles on LVR roads is relatively insignificant. The conflict rate for 400 vpd, however, would qualify as justifying shoulders if the criterion (used in App. D) of at least one critical event every 2 weeks is applied. But this justification only holds for a very small percentage of LVR roads (those with volumes more than about 320 vpd). Then too, the requirements for total road width developed in Appendix D do allow for shoulders for the higher (more critical) design speeds. Therefore, although explicit requirements for shoulders based on the hazard of stopped vehicles on the traveled way do appear justified for volumes above 400 vpd, this kind of requirement can be treated as insignificant for LVR roads.

APPENDIX D

ROAD WIDTH SAFETY REQUIREMENTS

Table 7 of the AASHTO "Geometric Design Guide for Local Roads and Streets" indicates a previous lack of functional analysis regarding road width. To say that the road width requirement for 50 vpd at 20 mph is the same as for 400 vpd at 50 mph seems inconsistent both with relative safety and with economic efficiency. What is apparently needed is a safety analysis that would relate road width to design speed. Also, if road width was related to design speed, like the current requirements for horizontal and vertical alignment, the driver would be able to more readily relate his maximum safe speed to what he sees.

Two traffic conditions are apparent in analyzing safety requirements for total roadway width. These are (1) the clearances needed when two opposing vehicles pass, and (2) the lateral width needed to make a tracking correction without encroaching on the roadside.

The clearance requirement is the summation of two vehicle widths, two outside clearances, and one inside clearance. At very low speeds, the total roadway width need only be slightly more than the width of two vehicles. As speeds increase, the lateral margin for error is sensitive to the speed, requiring greater road width to accommodate the safe passing of opposing vehicles.

The tracking requirement is a function of the initial lateral position of the vehicle, the speed of the vehicle, the perception-reaction time of the driver, the skid resistance of the pavement, and the angle of the tracking correction needed. As speeds increase, the ability to avoid a roadside encroachment is very sensitive to speed, requiring greater road widths to accommodate safe vehicle tracking.

DERIVATION OF TRACKING MODEL

When a vehicle has a path that will lead it off the roadway, the driver must perform a tracking correction to stay on the roadway. Referring to Figure D-1, the vehicle having an encroachment angle, θ, is a distance, W, from the outside edge of the shoulder when the driver perceives the need for a tracking correction. Because the driver needs time for perception and reaction, the vehicle will be a distance, w, from the edge of the roadway at the point of initial tracking correction. The relationship between the two distances is a function of vehicle speed, V, the encroachment angle, θ, and the perception-reaction time, t, as follows:

$$w = W - 1.47\,Vt \sin\theta \qquad \text{(D-1)}$$

where the distances, w and W, are in feet, the speed, V, is in mph, and the time, t, is in seconds.

The least severe correction path of the vehicle is a circular curve, tangent to the outside edge of the shoulder and tangent to the encroachment line at the point of initial tracking correction. The radius of this path is developed as follows:

$$R = T \cot\frac{2}{\theta} \qquad \text{(D-2a)}$$

$$\cot\frac{\theta}{2} = \sin\theta/(1-\cos\theta) \qquad \text{(D-2b)}$$

$$R = T \sin\theta/(1-\cos\theta) \qquad \text{(D-2c)}$$

$$T = w/\sin\theta \qquad \text{(D-2d)}$$

$$T = (W - 1.47\,Vt\sin\theta)/\sin\theta \qquad \text{(D-2e)}$$

$$R = (W - 1.47\,Vt\sin\theta)/(1-\cos\theta) \qquad \text{(D-2f)}$$

The sharpest path correction is limited by the skid resistance of the tire-pavement interface. For any coefficient of friction, the minimum vehicular path radius is computed by the standard centripedal force equation:

$$R = \frac{V^2}{15(e+f)} \qquad \text{(D-3)}$$

where e is the roadway cross slope, and f is the maximum available coefficient of friction. Assuming −0.02 for the cross slope, substituting the previously developed relationship for R, and solving for W yield the following equation for computing the lateral distance needed for a tracking correction:

$$W = \frac{V^2(1-\cos\theta)}{15(f-0.02)} + 1.47\,Vt\sin\theta \qquad \text{(D-4)}$$

COMPUTATION OF TRACKING REQUIREMENTS

To compute the lateral distance needed for tracking correction as a function of speed and angle requires two assumptions. The first is an estimate of the maximum available coefficient of friction, f. And, of course, as the coefficient decreases, the tracking width increases. Here, the widely used value of 0.50 is assumed. Most pavements will have a higher value when dry. Most gravel surfaces have a value very close to 0.50 when dry. Although some surfaces have a lower value when wet, the wet surface conditions is infrequent enough to ignore for this analysis, particularly because the total tracking requirement is not extremely sensitive to the coefficient of friction within normal ranges. Substituting the 0.50 value for f yields the following revised equation for the tracking requirement:

$$W = \frac{V^2(1-\cos\theta)}{7.2} + 1.47\,Vt\sin\theta \qquad \text{(D-5)}$$

The other required assumption is for the time needed for perception-reaction. Of course, as this time increases, the tracking requirement increases. When a vehicle has an encroachment angle, the driver depends on sensing lateral acceleration or visually detecting a lateral rate of divergence to perceive the need for a path correction. The perception-reaction time should decrease with increasing encroachment angles to a point, at say about 6 deg, where the perception is almost instantaneous. Therefore, the following safety-conservative values are assumed for calculating tracking requirements.

Encroachment Angle (θ) (deg)	Perception-Reaction Time (t) (sec)
1	1.0
2	0.9
3	0.8
4	0.7
5	0.6
≥ 6	0.5

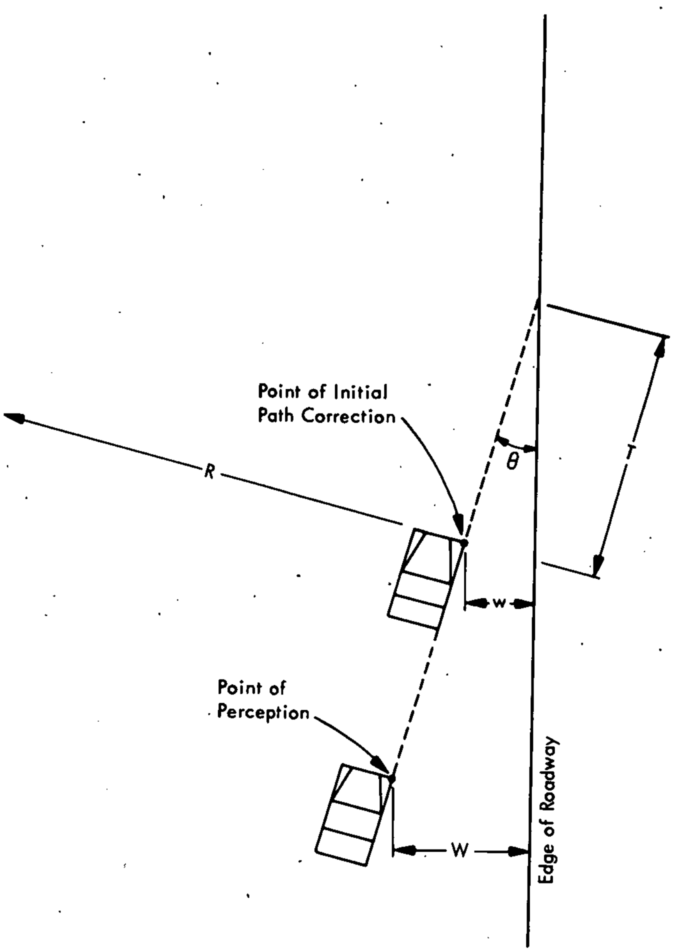

Figure D-1. Description of vehicle tracking.

The tracking requirements for various speeds and angles were calculated by using these perception-reaction times in the revised equations. These values are given in Table D-1.

DERIVATION OF ROAD WIDTH DESIGN BASED ON TRACKING REQUIREMENTS

The nominal condition for road width design to accommodate basic tracking corrections is the tangent highway section. Although tracking on horizontal curves may be more critical, tracking stability on curves is more sensitive to curve degrees, and this subject therefore is treated separately in this report.

To derive road width design requirements, some additional assumptions are necessary as follows:

1. Road width is used to mean the entire road usable for recovery, including the paved or stabilized traveled way as the primary recovery area and the paved or stabilized shoulder as the secondary recovery area.
2. The standard of tracking correction recovery is assumed at a safety-conservative level. It is that level of normal recovery provided by a 36-ft (11.0-m) roadway (two 12-ft lanes plus 6-ft shoulders) at 60 mph (96.6 kph).
3. An additional assumption needed to derive comparably safe widths for lower design speeds relates to the normal lateral positioning of vehicles on the roadway. Table D-2 shows the assumed positioning for various road widths. For very narrow roadways, the normal position is in the center of the roadway. As the widths increase, the distribution of lateral placements widens and the central tendency is more offset to the right.
4. By corresponding the tracking requirements given in Table D-1 with the lateral placement in Table D-2, the total percent of tracking corrections held for different roadway widths was calculated for each encroachment angle at various design speeds. For the total percent of tracking corrections held at each encroachment angle, the tracking corrections were assumed to be half to the left and half to the right. Table D-3 gives the total percent of tracking corrections accommodated for the 36-ft (11.0-m) road

TABLE D-1

LATERAL TRACKING WIDTH REQUIREMENTS

Angle (degrees)	Lateral Tracking Width (ft)[a/] Speed (mph)[b/] 20	30	40	50	60
1	0.52	0.78	1.06	1.34	1.62
2	0.96	1.46	1.98	2.52	3.07
3	1.31	2.02	2.76	3.56	4.38
4	1.58	2.45	3.41	4.45	5.53
5	1.75	2.79	3.93	5.16	6.51
6	1.84	2.99	4.30	5.74	7.35
7	2.20	3.62	5.24	7.07	9.11
8	2.56	4.28	6.25	8.49	10.99
9	2.97	4.98	7.33	10.00	13.03
10	3.39	5.73	8.48	11.65	15.25

a/ 1 ft = 0.305 m.
b/ 1 mph = 1.61 kph.

TABLE D-2

ASSUMED LOCATION AND DISTRIBUTION OF NORMAL LATERAL PLACEMENT OF VEHICLES

Road Width (ft)[a]	Central Tendency (clearance of 6.5 foot vehicle from either edge of roadway) ft[a] Left	Right	Percent Distribution of Lateral Position around Central Tendency -1.5 ft[a]	-1.0 ft	-0.5 ft	0	+0.5 ft	+1.0 ft	+1.5 ft
8	0.75	0.75				100			
9	1.25	1.25				100			
10	1.75	1.75			5	90	5		
11	2.25	2.25			10	80	10		
12	2.75	2.75			15	70	15		
13	3.25	3.25			20	60	20		
14	3.75	3.75			25	50	25		
16	5.00	4.50		5	20	50	20	5	
18	6.50	5.00		5	20	50	20	5	
20	8.00	5.50		10	15	50	15	10	
22	9.75	5.75		10	15	50	15	10	
24	11.50	6.00		10	15	50	15	10	
28	14.50	7.00	5	10	15	40	15	10	5
32	17.50	8.00	5	15	15	30	15	15	5
36	20.50	9.00	5	15	15	30	15	15	5
40	23.50	10.00	10	15	15	20	15	15	10

a/ 1 ft = 0.305 m

TABLE D-3

PERCENT OF TRACKING CORRECTIONS HELD AT VARIOUS ENCROACHMENT ANGLES FOR DERIVED DESIGNS

Encroachment Angle (degrees)	Percent of Tracking Corrections Held for Derived Road-Width, Design-Speed Combinations 36 ft[a] - 60 mph[b]	30 ft - 50 mph	22 ft - 40 mph	18 ft - 30 mph	14 ft - 20 mph
1	100	100	100	100	100
2	100	100	100	100	100
3	100	100	100	100	100
4	100	100	100	100	100
5	100	100	100	100	100
6	100	100	100	100	100
7	67	85	95	100	100
8	50	60	63	98	100
9	50	50	50	88	100
10	50	50	50	50	95

1 ft = 0.305 m
1 mph = 1.61 kph

width at 60 mph (96.6 kph) and for the comparably safe road widths for other design speeds. The road widths for design speeds other than 60 mph (96.6 kph) are the minimum widths that give equal or greater tracking correction performance than the 36-ft (11.0-m) width at 60 mph (96.6 kph).

DERIVATION OF ROAD WIDTH DESIGN BASED ON CLEARANCE REQUIREMENTS

Besides accommodating a certain level of tracking correction, road width also needs to be sufficient to allow reasonable clearances when two opposing vehicles pass. Given that the road has adequate sight distance for the design speed, the two opposing drivers will have enough time and distance at design speed to generally adjust their vehicle positions before meeting. The marginal clearances required therefore are those needed by the driver to track his vehicle past the opposing vehicle with minor path corrections to try to maintain optimal clearances.

For this analysis, the outside clearances are assumed as the tracking requirement for a 1-deg tracking correction. Because the driver is usually intimidated more by the opposing vehicle than by the roadside, the inside clearance is assumed as the sum of the tracking requirement for a 2-deg tracking correction by each vehicle. The equation for road width based on clearance requirements, therefore, is the sum of the two vehicle widths, two 1-deg tracking requirements, and two 2-deg tracking requirements for the design speed.

Besides the basic tracking requirements, a road width to accommodate reasonably safe head-on meetings is a function of both the total number of such meetings and the widths of the vehicles involved. The following nominal widths are assumed for various vehicle categories on LVR roads.

Vehicle Category	Types of Vehicles	Nominal Width (ft)*
car	automobiles, pick-ups	6.5
truck	large trucks, buses	8.5
farm machinery	all farm machinery	12.0

* 1 ft = 0.305 m

In the basic development of head-on clearance requirements that follows, the analysis uses cars and trucks only. The final development of total road width requirements using all the analysis of this appendix makes an allowance for designs to accommodate farm machinery.

Deciding which vehicle widths, cars at 6.5 ft (2.0 m) or trucks at 8.5 ft (2.6 m), to use in the road width determination depends on the frequency of the event. For this analysis, the critical event selected for design is an event that occurs more frequently than once every two weeks. With this criterion, the proportion of trucks can be determined for which a car-car, car-truck, or truck-truck combination of design vehicles is appropriate for each ADT category. The results of this analysis are given in Table D-4 using the head-on meeting rates determined in Appendix G. Considering that buses and large trucks usually account for more than 5 percent of the ADT on LVR roads, some design vehicle combinations can probably be ignored. These include the car-car combination for all LVR roads and the car-truck combination for ADT's between 201 and 400 vpd.

TABLE D-4

DETERMINATION OF DESIGN VEHICLE COMBINATIONS DEPENDENT ON ADT AND PERCENT OF ADT REPRESENTED BY LARGE TRUCKS AND BUSES

Design ADT	Average Head-on Meetings Per Mile Per Day	% of ADT Represented by Large Trucks and Busses to Justify Given Design Vehicle Combinations: Car-Truck	Truck-Truck
0-50	1	≥ 4.0%	≥ 27%
51-100	5	≥ 0.7%	≥ 12%
101-200	16	≥ 0.3%	≥ 7%
201-300	38	> 0	≥ 4.5%
301-400	68	> 0	≥ 3.5%

The road widths for the head-on clearance requirement for the various design speeds are given in Table D-5 for both the car-truck and truck-truck combinations. These widths would appear to be safety-conservative because, before the nationwide 55-mph speed limit was imposed, many miles of primary two-lane roads in the width range of 20 to 24 ft were operated at 70 mph without a holocaust of head-on accidents.

TOTAL ROAD WIDTH REQUIREMENTS

The total road width requirements for design of LVR roads are the minimum widths to satisfy both the tracking requirement and the head-on clearance requirement. Table D-6 gives these minimum widths for the various design

TABLE D-5

COMPUTATION OF ROAD WIDTH REQUIREMENTS FOR HEAD-ON CLEARANCES

Design Speed (mph)b/	2 Times Tracking Requirement for 2 Degrees Correction (ft)a/	2 Times Tracking Requirement for 1 Degree Correction (ft)	Width of Design Vehicle 1 (ft)	Width of Design Vehicle 2 (ft)	Road Width Requirement for Clearance (ft): Computed	Rounded
20	1.04	1.92	6.5	8.5	17.96	18
			8.5	8.5	19.96	20
25	1.30	2.42	6.5	8.5	18.72	20
			8.5	8.5	20.72	22
30	1.56	2.92	6.5	8.5	19.48	20
			8.5	8.5	21.48	22
35	1.84	3.44	6.5	8.5	20.28	22
			8.5	8.5	22.28	24
40	2.12	3.96	6.5	8.5	21.08	22
			8.5	8.5	23.08	24
45	2.40	4.50	6.5	8.5	21.90	22
			8.5	8.5	23.90	24
50	2.68	5.04	6.5	8.5	22.72	24
			8.5	8.5	24.72	26

a/ 1 ft. = .305m
b/ 1 mph = 1.61 kph

TABLE D-6

MINIMUM ROAD WIDTH REQUIREMENTS

Design Speed (mph) c/	Total Road Width Requirements (ft) a/ b/			
	Lower % Busses & Trucks (as specified below)		Higher % Busses & Trucks (as specified below)	
	< 28% for 0- 50 ADT < 12% for 51-100 ADT < 7% for 101-200 ADT NA for 201-400 ADT		≥ 28% for 0- 50 ADT ≥ 12% for 51-100 ADT ≥ 7% for 101-200 ADT All% for 201-400 ADT	
	Infrequent Trips by Farm Machinery d/	Frequent Trips by Farm Machinery d/	Infrequent Trips by Farm Machinery	Frequent Trips by Farm Machinery
20 mph	18 ft	22 ft	20 ft	24 ft
25	20	24	22	26
30	20	24	22	26
35	22	24	24	26
40	22	26	24	28
45	26	26	26	28
50	30	30	30	30

a/ 1 ft = .305 m

b/ Widths above 24 ft (7.3m) include appropriate shoulder widths.

c/ 1 mph = 1.61 kph

d/ The determination of "frequent" and "infrequent" are at the discretion of the designer.

speeds for the two levels of trucks and bus percentages. Also under each level are two sets of values: one for infrequent trips by farm machinery and the other for frequent trips by farm machinery. The determination of "frequent" and "infrequent" is left to the discretion of the designer.

APPENDIX E

ANALYSIS OF TRACKING ON CURVES

This analysis is not intended to reinvent the AASHTO design requirements for horizontal curves. Rather, it is intended to: (1) evaluate the adequacy on curves of the road width requirements developed in Appendix D, and (2) evaluate the safety on curves that have design speeds lower than the operating speed of the highway.

DERIVATION OF TRACKING MODEL

The tracking model developed here is conceptually similar to that used in Appendix D. A vehicle is assumed to have an initial roadside encroachment path tangent to the highway curvature (see Fig. E-1). Because the driver needs time for perception and reaction, the vehicle will have a lateral offset, $\overline{CD}$, from the initial highway curvature path at the point of initial path correction. This offset, $\overline{CD}$, is a function of the highway curve radius, R, and the perception-reaction distance, $\overline{BD}$. The distance, $\overline{BD}$, is a function of the vehicle speed, V, and the perception reaction time, t. Solving the right-triangle ABS gives the following relationship for the offset $\overline{CD}$ (distances are in feet, speeds in mph, and times in seconds):

$$\overline{CD} = \sqrt{R^2 + (1.47\,Vt)^2} - R \qquad \text{(E-1)}$$

The least severe correction path of the vehicle is a circular curve, tangent to the outside edge of the shoulder and tangent to the inside encroachment line at the point of initial tracking correction. The sharpest path correction radius is limited by the skid resistance, f, of the tire-pavement interface and the superelevation rate, e; such that:

$$R_v = \overline{ED} = \overline{EG} = \frac{V^2}{15(e+f)} \qquad \text{(E-2)}$$

From Figure E-1, the maximum offset, $\overline{FG}$, is defined as:

$$\overline{FG} = \overline{AE} + \overline{EG} - R \qquad \text{(E-3)}$$

The values $\overline{EG}$ and R have been previously defined. The value for $\overline{AE}$ is calculated by applying the law of sines to triangle ADE, such that:

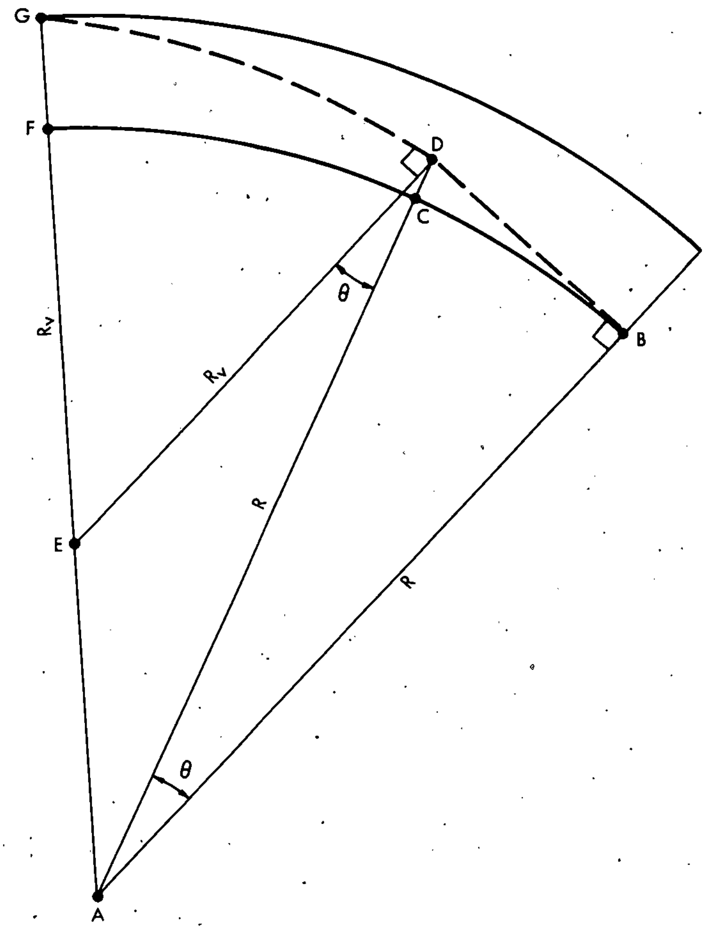

Figure E-1. Description of vehicle tracking on highway curves.

$$\sin\theta = \frac{\sqrt{(\overline{AE}+\overline{ED}+R+\overline{CD})(\overline{AE}+R+\overline{CD}-\overline{ED})(\overline{AE}+\overline{ED}-R-\overline{CD})(\overline{ED}+R+\overline{CD}-\overline{AE})}}{2\,\overline{ED}(R+\overline{CD})} \qquad \text{(E-4)}$$

but from right-triangle ABD, it is known that

$$\sin\theta = \frac{\overline{BD}}{R+\overline{CD}} \qquad \text{(E-5)}$$

therefore:

$$2\,\overline{BD}(\overline{ED}) = \sqrt{(\overline{AE}+\overline{ED}+R+\overline{CD})(\overline{AE}+R+\overline{CD}-\overline{ED})(\overline{AE}+\overline{ED}-R-\overline{CD})(\overline{ED}+R+\overline{CD}-\overline{AE})} \qquad \text{(E-6)}$$

By using trial and error, Eq. E-6 can be solved for $\overline{AE}$ with various values of vehicle speed, curve radius, perception-reaction time, friction coefficient, and superelevation to calculate all of the other terms using the previously developed equations.

COMPUTATION OF TRACKING REQUIREMENTS

To compute the maximum tracking offset, FG, as a function of vehicle speed and highway curve radius requires two assumptions. The first is an estimate of the nominal coefficient of friction, f. As in Appendix D, a coefficient of 0.5 is assumed. Substituting this value and a typical superelevation rate of 0.06 into the previous equation for minimum vehicle radius, R_v, gives:

$$R_v = \overline{ED} = \overline{EG} = \frac{V^2}{8.4} \qquad \text{(E-7)}$$

The other requried assumption is for the time needed for perception and reaction. Here the minimum value of 0.5 sec used in Appendix D is appropriate. The equation for maximum offset using these assumptions yields the values given in Table E-1.

EVALUATION OF TRACKING REQUIREMENTS

Given the minimum road width requirements for each design speed derived in Appendix D, the following relationships are assumed between road width and the maximum allowable offset to keep a 6.5-ft passenger vehicle traveling at design speed on the roadway.

Design Speed (mph)	Min. Road Width (ft)	Half of Min. Road Width (ft)	Nominal Vehicle Placement From Left Edge of Vehicle To Centerline of Highway (ft)	Maximum Allowable Offset From Edge of Roadway (ft)
20	18	9	0.0	2.5
30	20	10	0.5	3.0
40	22	11	1.5	3.0
50	30	15	2.5	6.0

For all design speeds, the design curvatures given by AASHTO produce vehicle offsets at design speed that are less than the maximum allowable offset calculated in Table E-1. Therefore, the tracking requirements on all curves of design specification do not require any wider roadway than on tangent sections.

The other consideration here is the operation of an existing highway curve that has a lower design speed than the operating speed of the roadway. Although the maximum operating speed (speed limit) should agree with the design speed, occasionally an older highway will have a highway curve that is substandard for the operating speed of the highway. When this occurs, the common practice is to use an advisory speed sign to warn drivers of the lower safe operating speed on the curve.

The tracking requirements developed previously define a maximum tolerance for the design and corresponding advisory speed of substandard curves keyed to the design and corresponding operating speed of the highway. This correspondence can be shown by using the safety-conservative assumption that the critical speed of a substandard curve is the general dseign speed of the highway (see Table E-2). The third column in this table shows the recommended maximum tolerable curve design speed below the general highway design speed. For example, for a highway design speed of 40 mph, the design curvature is 11.3 deg, but curves up to 20 deg are allowable with advisory speed signing of 30 mph. Any curves greater than 20 deg on a 40-mph roadway should be upgraded.

TABLE E-1

COMPUTED MAXIMUM VEHICLE TRACKING OFFSETS ON CURVES

Degree of Curve	Radius (feet)	Maximum Tracking Offset (feet) Vehicle Speed (mph) 20	30	40	50
10	573	0.2	0.6	1.0	2.5
15	382	0.3	1.0	1.8	7.0
20	287	0.5	1.4	3.0	-
25	229	0.8	2.0	10.0	-
30	191	1.2	2.6	-	-
35	164	1.5	3.2	-	-
40	143	1.8	-	-	-
50	114	2.3	-	-	-

1 ft. = 0.305 m.
1 mph = 1.61 kph

TABLE E-2

MINIMUM DESIGN OF HIGHWAY CURVES WITH ADVISORY SPEED SIGN RELATING TO HIGHWAY DESIGN SPEED

Operating (and Design Speed) of Highway (mph) a/	Design Curvature at 0.06 Superelevation (Degrees) b/	Minimum Advisory Speed (and Design Speed) of Curve (mph)	Maximum Curvature for Allowable Tracking (Degrees)
20	50	20	50
30	21	25	33
40	11.3	30	20
50	6.9	35	14

a/ 1 mph = 1.61 kph

b/ current AASHTO values

APPENDIX F

ANALYSIS OF NEED FOR NO-PASSING STRIPES

The Manual on Uniform Traffic Control Devices requires no-passing stripes on all highways where passing sight distance is inadequate. The question addressed in this appendix is whether passing conflicts on LVR roads are frequent enough to justify no-passing stripes.

For this analysis, passing maneuvers are initially assumed to occur without regard to oncoming vehicles in the zone of conflict. In other words, all passing maneuvers with an oncoming vehicle in the zone of conflict would result in a collision. This assumption is made to simplify the analysis and is, of course, very safety-conservative.

To analyze the three-car critical passing situation, the average speed differential between the passing vehicle and the passed vehicle is assumed as 10 mph (16.1 kph). The assumed speeds for the three vehicles are 40 mph (64.4 kph) for the passed vehicle and 50 mph (80.5 kph) for both the passing vehicle and the oncoming vehicle.

A pass occurs when, without regard for passing sight distance, the passing driver pulls into the opposing lane to pass. If an oncoming vehicle enters the zone of hazard while the passing vehicle is in the opposing lane, a collision will result. The expected number of this kind of conflict is calculated from the following general equation:

$$E(C) = E(P)P(A/P) \qquad \text{(F-1)}$$

where: $E(C)$ = expected number of conflicts per day;
$E(P)$ = expected number of passes per day; and
$P(A/P)$ = probability of an oncoming vehicle arrival that will conflict with the passing vehicle in the left lane.

EXPECTED NUMBER OF PASSES

The initiation of a pass (occupation of opposing lane) is assumed to occur when the passing vehicle arrives within 1 sec behind the passed car. Using the Poisson assumption, the probability of two or more arrivals within 1 sec is given by:

$$P(\text{2 or more arrivals in 1 sec or less}) = 1 - P(0) - P(1) = 1 - e^{-V/3600} - \frac{V}{3600}e^{-V/3600} \qquad \text{(F-2)}$$

where V equals average one direction volume in vehicles per hour. This equation is not the assumed probability of a pass, however, because it does not distinguish between the vehicles or their position in a pass. If the highway is divided into segments defined by the initial 1-sec separation with the passing vehicle behind, there will be 71.84 segments of 73.5 ft per mi. Considering a pass initiated with the passing vehicle 1-sec behind the passed vehicle, the separation of the two vehicles would proceed as follows.

Increment	Distance Relationship of Passing Vehicle to Passed Vehicle at Beginning of Increment (ft) *	Arrival Time Separation (sec)
1	−73.5	1.00
2	−55.1	0.75
3	−36.8	0.50
4	−18.4	0.25
5	0	0
6	+14.7	0.25
7	+29.4	0.50
8	+44.1	0.75
9	+58.8	1.00

* 1 ft = 0.305 m.

Looking at the time separation shows that there are 9 passing increments that have one second or less separation, but only one in nine that defines a pass initiation. Recognizing that each 73.5-ft highway increment has equal probability of each of the nine passing increments, if passes are accounted by 73.5-ft highway increments, the probability of a pass per one second for that interval is the probability of 2 or more arrivals within 1 sec divided by 9 or:

$$P(P) = \frac{1 - e^{-V/3600} - \left(\frac{V}{3600}\right) e^{-V/3600}}{9} \qquad \text{(F-3)}$$

To arrive at the expected number of passes per mile per day, the probability of a pass per increment is multiplied: (1) by the number of seconds in an 18-hr day; (2) by 2, to account for both directions of travel; and (3) by 71.84, the number of 1-sec passing increments. This calculation yields the following equation for the expected number of passes per mile per day:

$$E(P) = 1.0345 \times 10^6 \left[1 - e^{-V/3600}\left(1 + \frac{V}{3600}\right)\right] \qquad \text{(F-4)}$$

The following table gives the expected number of passes per mile per day for various LVR road volumes by using this equation.

ADT	Expected Number of Passes Per Mile Per Day, E(P)
50	0.031
100	0.165
200	0.693
300	1.552
400	2.695

PROBABILITY OF AN ONCOMING VEHICLE IN CONFLICT

The probability of one or more arrivals of oncoming vehicles for any interval is given by the following Poisson relationship:

$$P(A) = 1 - P(0) = 1 - e^{-Vt/3600} \quad \text{(F-5)}$$

where: V = average one direction volume, vph; and
t = arrival interval, sec.

Thus, the probability of an oncoming vehicle in conflict with the passing vehicle for any traffic volume depends on the conflict interval. Assuming the passing vehicle enters the opposing lane at the initiation of the pass and leaves the opposing lane 1 sec after it has achieved a 1-sec clearance ahead of the passed vehicle, the passing interval is 11 sec. Allowing for both the opposing vehicles within the conflict region when the pass is initiated and those that enter during the pass interval, the conflict interval for oncoming vehicles at 50 mph (80.5 kph) is 22 sec. The probability of a conflict given a pass, therefore, is calculated by substituting the 22-sec arrival interval into Eq. F-5 to yield:

$$P(A/P) = 1 - e^{-V/164} \quad \text{(F-6)}$$

By using this equation, the following table shows the probability of a conflict given a pass for various LVR road volumes.

ADT	Probability of a Conflict Given a Pass, P(A/P)
50	0.00630
100	0.01262
200	0.02509
300	0.03739
400	0.04954

EXPECTED NUMBER OF PASSING CONFLICTS

As previously stated, the expected number of passing conflicts is the product of the expected number of passes and the probability of an oncoming vehicle arrival during the pass. The following expected number of conflicts for various LVR road volumes is obtained by using the previously calculated values.

ADT	Unadjusted Rate for Expected Number Conflicts Per Mile Per Day *
50	0.00020
100	0.00210
200	0.01739
300	0.05803
400	0.13351

* 1 mile = 1.61 km.

Considering the assumptions of this development, these conflict rates are highly inflated. Recognizing that a driver will not pull out to pass when he sees an opposing vehicle within about 3 sec to collision (about 400 ft), and also when he does pull out he can easily abort within the first 3 sec, the foregoing conflict rates should be multiplied by 0.455. Also, one is interested in justifying no-passing stripes where sight distance is restricted. By using a safety-conservative assumption that 50 percent of the highway has passing sight distance restrictions, a more realistic conflict rate per mile of LVR road would be 0.455 × 0.50 = 0.227 of the previous table values. Using this multiplier, and also multiplying by 365 to compute an annual conflict rate, yields the following values.

ADT	Adjusted Rate for Expected Annual Number of Passing Conflicts Per Mile
50	0.02
100	0.17
200	1.44
300	4.81
400	11.06

If similar conflict rates are computed for higher traffic volumes, the expected number for 2000 vpd, for example, is 1440 per mile per year, or well over 100 times that for 400 vpd.

NEED FOR NO-PASSING STRIPES

Based on the conflict rates previously calculated and the order of magnitude comparison with higher volume roadways, no-passing stripes do not appear to be justified on LVR roads. This is particularly true because, as demonstrated in a study by Jones (*16*), drivers tend to decide to pass more on the availability of adequate passing sight distance than on the presence or absence of no-passing stripes. The Jones study indicates that of those drivers who have a passing opportunity in a passing zone with minimal sight distance, less than 10 percent will attempt a pass. Therefore, the conflict rates calculated earlier should probably be divided by at least 10. Of course, these conflict rates still only allow for minimal adaptive behavior by drivers. Therefore, accident rates involving passing vehicles are expected to be very low for LVR roads.

The need for no-passing stripes can also be generally evaluated on a benefit-cost basis using the following safety-conservative assumptions:

1. One-half of the roadway has restricted sight distance.
2. One-half of all head-on accidents involve passing maneuvers.
3. The presence of no-passing stripes will reduce the number of head-on-accidents involving passing maneuvers in restricted sight-distance areas by one-half.

Using these assumptions, the total accident rates generated in Appendix A, and the percent of head-on accidents generated in Appendix A yields the following estimates of accident reduction.

ADT	Total Accident Rate (#/mi/yr)	% Of Total Accidents That Are Head-On	Head-On Accident Rate (#/mi/yr)	Estimated Rate Of Head-On Accidents Reduced By No-Passing Stripes (#/mi/yr)
50	0.098	8.3	0.00813	0.0010
100	0.148	9.4	0.01391	0.0017
200	0.235	11.3	0.02656	0.0033
300	0.301	13.0	0.03913	0.0024
400	0.367	13.7	0.05028	0.0063

A recent research report by Bali et al. (7) gives current costs of pavement striping for several states. Considering all manpower, material, equipment, and overhead costs, one finds that the average unit cost of a 4-in. solid pavement stripe would be about \$0.05/ft. This same report estimates a maximum effective life of 1.5 yr for pavement striping on low-volume roads. If a roadway has no-passing stripes in each direction covering 50 percent of the mileage, the average annual striping cost per mile would be about $0.05 \times 5280/1.5 = \$176$/mi-yr.

On the basis of the previously developed safety-conservative unit cost of \$9,500 per accident and the estimated accident reductions previously calculated, the annual accident cost reduction on a 400-vpd roadway would be about \$60/mi. Because this benefit is only about one-third the striping cost, the no-passing stripes cannot be justified on cost-benefit break-even analysis. Even if all accidents reduced were fatal or non-fatal injury accidents, the \$19,600 average cost would still produce an annual benefit (\$123) on a 400-vpd roadway that is less than the striping cost.

On the basis of the foregoing analyses it is concluded that no-passing stripes cannot be justified on LVR roads.

APPENDIX G

ANALYSIS OF NEED FOR CENTERLINE STRIPES

This appendix analyzes the expected number of head-on meetings on LVR roads. This analysis has two purposes. The first is to provide the basic inputs in Appendix D for calculating the frequency of head-on meeting of various vehicle types for choosing the design vehicle widths for head-on clearance requirements. The second purpose is to evaluate the need for centerline stripes on LVR roads.

EXPECTED NUMBER OF HEAD-ON MEETINGS

The number of head-on meetings per mile per day can be estimated using the following equation:

$$N_M = N_A \times N_{O/A} \quad \text{(G-1)}$$

or

$$N_M = V \times \frac{vt}{3600} \quad \text{(G-2)}$$

where: N_M = number of head-on meetings per mile per day;

N_A = number of arrivals in one direction per day;

$N_{O/A}$ = number of opposing arrivals per 1 mi travel of a one-direction arrival;

V = one-direction daily traffic volume; vpd;

v = average one-direction hourly traffic volume, vph; and

t = exposure interval for head-on meetings within the 1-mi analysis interval, sec/mi.

To exercise this equation for LVR roads, the following safety-conservative assumptions are made:

1. The roadway has a 50-50 directional traffic split so that V equals one-half the ADT.
2. The roadway has traffic on it for only 18 hr of the day.
3. The average speed of vehicles is 45 mph.

Therefore, if the analysis increment is 1 mi, the exposure interval of a vehicle to head-on meetings per mile is calculated as twice the time it takes a vehicle at 45 mph to travel 1 mi. The factor of 2 accounts for both the opposing vehicles within the 1-mi segment when a subject vehicle arrives and those opposing vehicles that arrive after that time but before the subject vehicle leaves the 1-mi segment. This total exposure time is $5280 \times 2/45 \times 1.47 = 160$ sec.

Using these assumptions yields the following equation for calculating the number of meetings:

$$N_c = V^2/405 \quad \text{(G-3)}$$

From Eq. G-3, the following estimated number of head-on meetings is calculated for the various LVR road volumes.

ADT	Estimated Number of Head-On Meetings Per Mile Per Day
50	1.5
100	6.2
200	25.0
300	56.0
400	100.0

* 1 mi = 1.61 km

Another way of looking at these rates is to compute the expected number of miles to be driven by a single vehicle before a head-on meeting. This ranges from 2 mi at 400 vpd to 16.7 mi at 50 vpd. With these kinds of rates, many vehicles will travel on LVR roads without ever meeting an opposing vehicle.

EVALUATION OF THE NEED FOR CENTERLINE STRIPES

From the head-on meeting rates calculated, it is difficult to judge what rate would justify a centerline stripe. On the basis of the fact that for the lower volume LVR roads each vehicle or driver may account for a high percentage of the trips, the calculated head-on meeting rates for less than, say, 100 vpd may be excessively high because of the safety-conservative assumption of a 50-50 directional traffic split. Also, by using these calculated rates and judging from Bali et al. (*7*) an average cost of $200/mi for 4-in. centerline striping with an effective service life of 1.5 yr, the cost per head-on meeting is about $0.25 on a 50-vpd roadway. Similarly, for a 100-vpd road, this cost is about $0.06 per head-on meeting. When the cost of striping is put in this perspective, it seems excessive for these lower volume roadways. The reader is reminded also that these rates are just for head-on meetings and do not represent traffic conflicts, per se.

The need for centerline stripes can generally be evaluated by a benefit-cost trade-off analysis. For this purpose, the safety effectiveness of centerline stripes is taken as a 5 percent reduction in all accidents as reported in *NCHRP Report 162* (*21*). Applying this percentage to the expected annual number of accidents calculated in Appendix A gives the following estimates of accident reduction.

ADT	Estimated Number of Accidents Reduced per Mile per Year by Centerline Stripes *
50	0.0049
100	0.0074
200	0.0118
300	0.0151
400	0.0184

* 1 mi = 1.61 km

Using the $200 cost per mile of centerline striping with an average service life of 1.5 yr yields an annual striping cost of about $133. With the previously developed unit cost of $9,500 per accident, the benefit-cost trade-off point to justify centerline striping would be 133/9500 = 0.014.

On the basis of the foregoing analysis, it is concluded that centerline stripes are justified by a benefit-cost analysis for LVR roads with ADT's above 300 vpd. The reader is cautioned, however, that the exact decision point is sensitive to the assumed accident costs and the obtainable accident reduction.

APPENDIX H

INTERSECTION STOP CONTROL

For low-volume intersections, the MUTCD allows only for engineering judgment to warrant stop signs under the following statement "other intersections where a combination of high speed, restricted view, and serious accident record indicates a need for control by a stop sign." The purpose of the following analysis is to determine whether a clearer justification is apparent based on the estimated number of right-angle accidents.

NUMBER OF RIGHT-ANGLE CONFLICTS

The initial step in determining the estimated number of accidents is to determine the number of conflicts. From this determination, the number of accidents can be estimated using conflict/accident ratios from empirical studies.

For the purpose of analysis, the following seven assumptions are made:

1. Average speed is 40 mph (64.4 kph) and no intersection stop control is provided.
2. Sight distance restrictions are not considered in the analysis.
3. All vehicles arrive during an 18-hr period during the day.

4. A conflict occurs when the maneuver of a vehicle on one intersection approach causes the driver of a vehicle on a perpendicular approach to brake or change direction to avoid collision.

5. Any two vehicles approaching an intersection from perpendicular directions within a 2-sec arrival are considered in conflict.

6. Only one arrival per approach is possible during one 2-sec interval. In other words, all approaches are single lanes, and all headways are greater than 2 sec.

7. The possibility of vehicles arriving on three or four approaches within a 2-sec interval is ignored because of the extremely small probability of this occurrence.

The number of intersection conflicts per day is the sum of the number of daily conflicts for each of four pairs of perpendicular approaches. The number of intersection conflicts, therefore, can be estimated by the following equation:

$$N_C = 4 \times N_A \times N_{C/A} \tag{H-1}$$

or

$$N_C = 4 \times V_A \times \frac{v_p t}{3600} \tag{H-2}$$

where: N_C = number of intersection conflicts per day;
N_A = number of daily arrivals on the analysis approach;
$N_{C/A}$ = number of conflicting arrivals on a perpendicular approach per arrival on an analysis approach;
V_A = one-direction daily traffic volume on analysis roadway, vpd;
v_p = average one-direction hourly traffic volume on perpendicular roadway vph; and
t = conflict exposure interval, sec.

Substituting the assumed 2-sec analysis interval and the 18-hr day yields the following equation for computation

$$N_C = \frac{V_A V_P}{8100} \tag{H-3}$$

Table H-1 gives the estimated number of conflicts for various ADT's in the low-volume range. The number of conflicts per day ranges from 0.077 for the 50-50 ADT combination to 4.938 for the 400-400 ADT combination. (Note that these values do not correspond with those normally counted in a 2½-hr period with the standard Traffic Conflicts Technique.) If higher volume intersections were uncontrolled, the number of conflicts would be as follows.

ADT's	Conflicts Per Day
1,000-1,000	30
2,000-2,000	120
4,000-4,000	490

NUMBER OF RIGHT-ANGLE ACCIDENTS

The number of accidents at low-volume road intersections can be estimated by multiplying the number of conflicts per day in Table H-1 by the following empirically derived accident/conflict ratios.

Source	Accidents/Conflict (2½-hr count)
Baker (*6*)	0.00016
Perkins and Harris (*23*)	0.00033
Cooper (*11*)	0.00040

These ratios are computed from a 2½-hr daytime conflict count interval and, therefore, are inflated for estimating a ratio based on daily conflicts. Also, most of the intersections in these studies probably had stop control on one intersecting roadway. To arrive at a safety-conservative estimate, the highest empirical ratio, 0.00040, is multiplied by 2 to estimate an uncontrolled conflict situation and divided by 5 to normalize on a daily ratio. This yields an accident/daily conflict ratio of 0.00016. The expected number of accidents per year is derived by multiplying the expected conflicts by the accident/conflict ratio and by 365. Table H-2 gives the expected annual number of right-angle accidents for low-volume intersections, ranging from 1 every 222 yr for an intersection with a 50-50 ADT combination to 1 every 3.5 yr for an intersection with a 400-400 ADT combination. If higher volume intersections were uncontrolled, the expected number of accidents would be as in the following.

ADT's	Accidents Per Year
1,000-1,000	1.75
2,000-2,000	7.00
4,000-4,000	27.00

VERIFICATION OF EXPECTED NUMBER OF ACCIDENTS

The accident rates in Table H-2 are verified by comparing the average annual numbers of right-angle accidents at local-rural road intersections generated by first using the developments of Appendix A and second by using the accident rates developed here.

The total annual number of right-angle intersection accidents can be generated from the developments of Appendix A as follows:

1. Assume one intersection for every 2 mi (3.22 km) of roadway. Therefore, allowing that every intersection has two roadways, the 2,208,607 mi (3,555,857 km) of local-rural roadways is divided by 4 to yield 552,152 intersections.

2. The total annual number of accidents on local-rural roads is the annual number (160,827) of fatal plus injury accidents divided by the average percent (47 percent) of fatal plus injury accidents. This yields 342,185 accidents per year.

3. Assume that the average percent of total accidents that are right-angle is 4.0 percent. This yields 13,687 right-angle accidents per year on local-rural roads.

TABLE H-1

ESTIMATED NUMBER OF RIGHT-ANGLE INTERSECTION CONFLICTS PER DAY

Road A ADT	Road B ADT 50	100	200	300	400
50	0.077	0.154	0.309	0.463	0.617
100	0.154	0.309	0.617	0.926	1.234
200	0.309	0.618	1.236	1.852	2.468
300	0.463	0.926	1.852	2.778	3.704
400	0.618	1.234	2.468	3.704	4.938

TABLE H-2

EXPECTED ANNUAL NUMBER OF RIGHT-ANGLE INTERSECTION ACCIDENTS

Road A ADT	Road B ADT 50	100	200	300	400
50	0.0045	0.0090	0.0180	0.0270	0.0360
100	0.0090	0.0180	0.0360	0.0540	0.0720
200	0.0180	0.0360	0.0720	0.1080	0.1440
300	0.0270	0.0540	0.1080	0.1620	0.2160
400	0.0360	0.0720	0.1440	0.2160	0.2880

4. The average annual number of right-angle accidents per intersection is the total number (13,687) of right-angle accidents divided by the total number (552,152) of intersections. Therefore, the average annual number of right-angle accidents per local-rural intersection is estimated as 0.0248 acc./yr.

A comparative average rate of right-angle intersection accidents can be generated by using the developments of this appendix and the volume distribution generated in Appendix A as follows:

1. By using the traffic volume distribution identified in the verification step of Appendix A, the distribution classes are altered slightly to yield the following breakdown.

ADT Class	Percent of Total Local-Rural Roadway Mileage in Each ADT Class
50	40
100	35
200	15
300	7
400	3

2. The percentage of intersections represented by each ADT combination (e.g., 50-50, 100-200, etc.) is estimated as the product of the percentages of mileage in each ADT class normalized by the sum of all such products. For example, the percent of intersections that have both roadways in the 50 ADT class is $P_1P_2/\Sigma P_1P_2$ or $40 \times 40/67.03 = 23.87$. In addition to the intersections representing all combinations of the ADT's in the previous table, each of these ADT's was linked with all high-volume crossroads estimated at 2,000 vpd. The percent of total intersection represented by each of the low-volume ADT roads intersecting high-volume roads was assumed as half of those intersections with the same lower ADT and 400 vpd as the higher ADT.

3. The average annual number of right-angle collisions is calculated as the sum of the percent represented times the right-angle accident rate for each volume combination. This number is 0.0323 acc./yr. Although this value is about 30 percent higher than the value previously calculated, the first value probably includes several controlled intersections. If in the calculation just made, all intersections with a combined ADT of 500 vpd or more were assumed to have stop control, and their right-angle accident rate was reduced to half, the calculated average rate for all roads would be within 1 percent of the previously calculated rate.

WARRANTS FOR STOP CONTROL

Justification for stop signs on LVR roads depends both on the effectiveness of stop signs in reducing intersection accidents and on whether stop-sign installation is an efficient use of highway funds. From a benefit-cost comparison, stop signs should be warranted where the benefits of accident reduction exceed the combination of stop-sign installation costs and increased operating costs associated with stopping vehicles.

~~The most~~ recent information on vehicle operating costs is published in the NCHRP Project 2-12/1 final report (*3*). For a stop from 40 mph, the average increased operating cost per vehicle is \$0.021. In stopping vehicles on a 400-vpd roadway, this represents an annual cost of \$3,066. If the annualized cost of the stop-sign installation is \$20, the total annual cost of stop signs on a 400 vpd roadway would be \$3,086. By using the previously developed unit cost of accidents (\$9,500), the required annual accident reduction to justify stop signs is 3086/9500 = 0.325 acc./yr. But, because this number is more than 100 percent of the largest expected accident reduction given in Table H-2, stop signs cannot generally be justified on a benefit-cost basis for the intersection of two LVR roads.

Because the foregoing analysis is based on average expected values, it should be recognized that the present discretionary warrants for stop control in the MUTCD are appropriate to LVR roads. Special problems with sight restrictions and with the designation of right-of-way, particularly when an LVR road intersects a higher volume through highway, should warrant consideration of stop control on LVR roads.

APPENDIX I

ROADSIDE HAZARD EVALUATION

The degree of hazard associated with any single roadside feature is generally a function of the number of vehicles passing by, the speeds of vehicles, the geometrics of the roadway, the size of the roadside feature, the lateral placement of the roadside feature, and the collision severity of the roadside feature. Therefore, high hazard is associated with large, rigid objects close to the edge of high-volume, high-speed highways. Even with this very general description, it is readily apparent that the hazard of roadside features on LVR roads is generally relatively low.

The predominant roadside features on existing LVR roads and their probable ranking as to total contribution to the over-all roadside hazard are side slopes, trees, utility poles, bridgerails, guardrails, culverts, fences, signs, and mailboxes.

Several strategies are available for the improvement of roadside safety. The proper evaluation of roadside design strategies, whether for new construction or for reconstruction, looks at the relative hazard of alternative designs for each roadside feature. Considering an in-place feature, the alternatives are:

1. Remove.
2. Move laterally.
3. Redesign for lower severity.
4. Use guardrail or attenuation devices.

Because of limited right-of-way, relatively high costs, and limited effectiveness on LVR roads, the second strategy is usually not cost-effective and, therefore, is ignored in the remainder of this analysis.

Two research reports by Glennon (*12, 13*) provide the tools for evaluating the hazard of various roadside features. Hazard is given by the following model:

$$H = \frac{E_f S}{5{,}280}\left[\ell P[y \geq s] + 6 \csc\theta\, P[y \geq s + 3\cos\theta] + \frac{w \cot\theta}{n}\sum_{i=1}^{n} P\left[y \geq s + 6\cos\theta + \frac{w(2j-1)}{2n}\right]\right] \qquad \text{(I-1)}$$

where: H = hazard of the roadside feature, number of fatal plus non-fatal injury accidents per year;

E_f = roadside encroachment frequency, number of encroachments per mile per year;

S = severity index of roadside feature, ratio of fatal and non-fatal injury accidents to total accidents;

ℓ = longitudinal length of the obstacle, ft;

w = lateral width of the obstacle, ft;

s = lateral placement of the obstacle, ft;

θ = angle of encroachment, deg;

$P\, y \geq \ldots$ = probability of a vehicle lateral displacement greater than some value;

n = number of analysis increments for hazard associated with the obstacle width; a reasonable subdivision is one increment for each 2.5 ft (0.76 m) of width; and

j = the number of the obstacle-width increment under consideration starting consecutively with 1 at the lowest lateral placement.

To exercise the model here, the following safety-conservative assumptions were made:

1. All roadside features of concern are assumed to have a lateral placement within 15 ft (4.6 m). For simplicity, the probability, $P\, y \geq \ldots$, is assumed as 1.0. This yields the following simplified model (which gives slightly higher hazard indices than expected for lateral placements greater than zero).

$$H = \frac{E_f S}{5{,}280}(\ell + 6\csc\theta + w\cot\theta) \qquad \text{(I-2)}$$

2. From the Glennon report (*12*), the average encroachment angle, θ, for two-lane roads is 7.2 deg and the encroachment rate, E_f, for one side of a two-lane highway is $\frac{5.23\, Rsv}{2}$, where R_{sv} is the single-vehicle accident rate per mile per year. Making these substitutions yields:

$$H = \frac{R_{sv} S}{2{,}019}(\ell + 6.05 + 7.92\, w) \qquad \text{(I-3)}$$

3. The severity indices for two-lane roads are taken from the Glennon (*12*) report and are given in Table I-1. These indices tend to reflect the speed factor in that they are lower than those for freeways and higher than those for urban arterial streets.

4. The hazard model as constituted only accounts for the average hazard of all roadside sections. The one highway feature that obviously has a higher encroachment rate is the horizontal curve. Although the relative roadside encroachment rate for highway curves has not been determined, several studies (*5, 19, 24*) indicate that the accident rate on curves is higher than the average highway rate. Another study (*10*) indicates that the severity of roadside accidents is significantly higher on curves. From these studies, a safety-conservative estimate for the hazard index on curves is four times the average hazard index. This yields the following hazard model for roadside features on highway curves:

$$H_c = \frac{R_{sv}S}{505}\,(\ell + 6.05 + 7.92\,w) \tag{I-4}$$

5. To arrive at a comparable model for tangent sections requires an estimate of the proportions of mileage (for the study sample in Ref. 12) that are tangent and curve. A safety-conservative assumption is 0.95 for tangents and 0.05 for curves. This yields an encroachment rate for tangents that is 0.842 times the average encroachment rate. The modified model for tangent sections, therefore, is:

$$H_T = \frac{R_{sv}S}{2{,}398}\,(\ell + 6.05 + 7.92\,w) \tag{I-5}$$

6. For roadside features that are essentially point obstacles, the dimensions are assumed as 1 ft × 1 ft. These include utility poles, trees, single-post wooden signposts, and mailboxes. Making these dimensional substitutions in the foregoing models yields the following models for point obstacles on curves and tangents:

$$H_{PC} = \frac{R_{sv}S}{33.7} \tag{I-6}$$

$$H_{PT} = \frac{R_{sv}S}{160} \tag{I-7}$$

7. For roadside features that have a more continuous nature, a unit analysis is made using a 100-ft × 1-ft dimension. These include sideslopes, bridgerails, guardrails, culverts, and fences. Substituting these dimensions in the general models gives the following models for the hazard per foot of continuous roadside features on curves and tangents:

$$H_{CC}/\mathrm{ft} = \frac{R_{sv}S}{3{,}610} \tag{I-8}$$

$$H_{CT}/\mathrm{ft} = \frac{R_{sv}S}{17{,}165} \tag{I-9}$$

8. One roadside feature that does not fit either of the foregoing categories is a 2-post sign. This is assumed to have a 1-ft × 5-ft dimension, yielding the following hazard models for 2-post signs on curves and tangents:

$$H_{SC} = \frac{R_{sv}S}{10.8} \tag{I-10}$$

$$H_{ST} = \frac{R_{sv}S}{51.4} \tag{I-11}$$

TABLE I-1

SEVERITY INDICES FOR ROADSIDE FEATURES ON LOW-VOLUME ROADS (*12*)

Roadside Feature	Severity Index (Fatal + Injury Accidents/ Total Accidents)
Fill Slopes	
2:1 or steeper	0.60
3:1	0.45
4:1	0.35
5:1	0.25
6:1	0.15
Cut Slopes	
1:1 or steeper	0.60
1-1/2:1	0.45
2:1	0.35
3:1	0.25
4:1 or flatter	0.15
Trees (greater than 6 in. diameter)	0.50
Utility Poles	0.45
Bridgerails	
Smooth rail	0.35
parapet rail	0.50
bridge end	0.50
Guardrail (100 ft minimum, safety end)	0.30
Culverts	0.45
Fences	0.35
Signs (wood post)	0.30
Signs (breakaway)	0.20

COST-EFFECTIVENESS OF POINT OBSTACLE SAFETY IMPROVEMENTS

As stated earlier, the point obstacles of interest are utility poles, trees, single-post wood signs, and mailboxes. With these obstacles on LVR roads, all potential improvement strategies are not possible, effective, or cost-effective. Given that LVR roads have limited rights-of-way, the strategy of moving obstacles laterally has minimal effectiveness and usually a higher cost than other strategies. Placing guardrails in front of point obstacles also has minimal effectiveness and is generally cost prohibitive. The remaining strategies are to (1) remove large trees and (2) make utility poles, signposts, and mailboxes breakaway. Breakaway wooden signposts are effected by drilling holes in the center of the post parallel to the sign face. Breakaway utility poles are effected by sawing cuts perpendicular to the utility line and modifying the crossbuck. The reference to mailboxes considers replacing any kind of rigid support with a yielding support.

For all of these improvements, the hazard reduction is a direct function of the severity reduction. The severity index of the improvement is the higher of that for the remaining obstacle or for the contiguous roadside. A safety-conservative estimate for the severity index of the contiguous roadside is taken from Table I-1 as 0.15 for 6:1 side slope or flatter. Therefore, the maximum severity reductions for the candidate improvements are as follows.

Roadside Improvement	$S_1 - S_2$	ΔS
Remove large trees	0.50 — 0.15	0.35
Make utility poles breakaway	0.45 — 0.20[a]	0.25
Make wood signpost breakaway	0.30 — 0.20	0.10
Make rigid mailboxes breakaway	0.35[a] — 0.15	0.20

[a] Estimated

Although the hazard reduction is the basic unit of analysis, for small values obtained, a more meaningful presentation is the inverse, or the number of improvements necessary to reduce one fatal or non-fatal injury accident per year. Table I-2 gives these numbers for the candidate improvements at various traffic volumes on LVR roads. As can be seen, the number of improvements to reduce one fatal or non-fatal injury accident per year ranges considerably from 498 trees removed on curves of 400-ADT roadways to 25,000-breakaway signposts on tangents of 50-ADT roadways.

The only objective way to decide if these improvements are worthwhile is to use a benefit-cost analysis to determine the maximum number of improvements that can be implemented to save one fatal or non-fatal injury accident per year at a total installation cost equal to the cost of the accidents saved. Using the previously developed safety-conservative unit cost of $19,600 for a fatal or non-fatal injury accident, assuming a 20-yr life for the improvements, and conservatively assuming the present worth of one accident reduced per year for 20 yr as 20 × $19,600 = $392,000, the cost-effectiveness of each kind of improvement can be determined for various traffic volumes and various unit costs of improvement. Table I-3 shows the cost-effectiveness of the candidate improvements for a range of unit costs of various traffic volumes for LVR roads. For the unit costs indicated, the point obstacle improvements are cost-effective on curves for most traffic volumes. On tangents, these improvements are only cost-effective for the lower unit costs and the higher traffic volumes.

COST-EFFECTIVENESS OF SAFETY IMPROVEMENTS OF CONTINUOUS ROADSIDE FEATURES

The continuous roadside features of interest are side slopes, bridgerails, guardrails, culverts, and fences. Again, all potential improvement strategies are not possible, effective, or cost-effective. For LVR roads, the strategy of moving continuous roadside features laterally is of minimal effectiveness and cost prohibitive. Then too, none of the continuous features can really be removed with the exception of unwarranted guardrail. The possible strategies and their estimated severity reductions are given in Table I-4.

Since the hazard reduction per foot of improvement is so small, a more meaningful analysis is the inverse, or the number of feet of improvement necessary to reduce one fatal or non-fatal injury accident per year. Table I-5 gives these footages for the various severity reductions of candidate improvements at various traffic volumes on LVR roads. These footages vary considerably from 41,300 ft (12,500 m) of a 0.45-severity-reducing improvement on curves of a 400-ADT roadway to 5,040,000 ft (1,540,000 m) of a 0.05-severity-reducing improvement on tangents of a 50-ADT roadway.

Again, in deciding if these improvements are worthwhile, a benefit-cost analysis is used to determine the maximum footage of each improvement that can be implemented to save one fatal or non-fatal injury accident per year at an installation cost equal to the cost of the accidents saved. Using a 20-yr life for the improvements, the total accident

TABLE I-2

NUMBER OF POINT OBSTACLE IMPROVEMENTS NEEDED TO REDUCE ONE FATAL OR INJURY ACCIDENT PER YEAR

ADT (vpd)	RSV (acc/mi/yr)[a/]	Tree Removal $\Delta S = 0.35$	Breakaway Utility Pole $\Delta S = 0.25$	Breakaway Mailbox $\Delta S = 0.20$	Breakaway Sign $\Delta S = 0.10$
Highway Curves					
50	0.068	1,408	2,000	2,500	5,000
100	0.101	952	1,333	1,667	3,333
200	0.141	685	952	1,190	2,380
300	0.170	565	794	990	1,980
400	0.194	498	694	870	1,740
Highway Tangents					
50	0.068	6,667	9,091	11,111	25,000
100	0.101	4,545	6,250	7,692	16,667
200	0.141	3,226	4,545	5,556	11,111
300	0.170	2,703	3,704	4,762	9,091
400	0.194	2,381	3,333	4,167	8,333

a/ 1 mile = 1.61 km.

TABLE I-3

COST-EFFECTIVENESS OF POINT OBSTACLE IMPROVEMENTS

ADT	Improvement											
	Tree Removal			Breakaway Utility Pole			Breakaway Mailbox			Breakaway Signposts		
(vpd)	$100	$200	$400	$25	$50	$100	$25	$50	$100	$25	$50	$100
Highway Curves												
50	X	X	O	X	X	X	X	X	X	X	X	O
100	X	X	X	X	X	X	X	X	X	X	X	X
200	X	X	X	X	X	X	X	X	X	X	X	X
300	X	X	X	X	X	X	X	X	X	X	X	X
400	X	X	X	X	X	X	X	X	X	X	X	X
Highway Tangents												
50	O	O	O	X	O	O	X	O	O	O	O	O
100	O	O	O	X	X	O	X	X	O	O	O	O
200	X	O	O	X	X	O	X	X	O	X	O	O
300	X	O	O	X	X	X	X	X	O	X	O	O
400	X	O	O	X	X	X	X	X	O	X	O	O

(X indicates improvement is cost-effective at unit price given)

TABLE I-4

SEVERITY REDUCTIONS OF CANDIDATE IMPROVEMENTS FOR CONTINUOUS ROADSIDE FEATURES

Roadside Improvement	$S_1 - S_2$	ΔS
Flatten fill slopes		
2:1 to 6:1	0.60 - 0.15	0.45
2:1 to 5:1	0.60 - 0.25	0.35
3:1 to 6:1	0.45 - 0.15	0.30
2:1 to 4:1	0.60 - 0.35	0.25
3:1 to 5:1	0.45 - 0.25	0.20
4:1 to 6:1	0.35 - 0.15	0.20
2:1 to 3:1	0.60 - 0.45	0.15
3:1 to 4:1	0.45 - 0.35	0.10
4:1 to 5:1	0.35 - 0.25	0.10
5:1 to 6:1	0.25 - 0.15	0.10
Place guardrail at fill slopes		
on 2:1	0.60 - 0.30	0.30
on 3:1	0.45 - 0.30	0.15
on 4:1	0.35 - 0.30	0.05
Place guardrail at cut slopes		
on 1:1	0.60 - 0.30	0.30
on 1-1/2:1	0.45 - 0.30	0.15
on 2:1	0.35 - 0.30	0.05
Place guardrail at tree grove	0.50 - 0.30	0.20
Place guardrail at bridge ends	0.50 - 0.30	0.20
Place guardrail at culverts	0.45 - 0.30	0.15
Place guardrail on parapet bridgerail	0.40 - 0.30	0.10
Remove unwarranted guardrail on fill slopes		
on 6:1	0.30 - 0.15	0.15
on 5:1	0.30 - 0.25	0.05

TABLE I-5

FOOTAGE OF CONTINUOUS ROADSIDE IMPROVEMENT NEEDED TO REDUCE ONE FATAL OR INJURY ACCIDENT PER YEAR

ADT	R_{SV}	Severity Reduction (ΔS)							
(vpd)	(acc/mi/yr) a/	0.45	0.35	0.30	0.25	0.20	0.15	0.10	0.05
Highway Curves									
50		118,000 b/	152,000	177,000	212,000	265,000	354,000	531,000	1,060,000
100		69,500	102,000	119,000	143,000	178,000	238,000	357,000	714,000
200		56,900	73,200	85,300	102,000	128,000	171,000	256,000	512,000
300		47,200	60,700	70,800	84,900	106,000	142,000	212,000	424,000
400		41,300	53,200	62,000	74,400	93,000	124,000	186,000	372,000
Highway Tangents									
50		561,000	721,000	841,000	1,010,000	1,260,000	1,680,000	2,520,000	5,040,000
100		378,000	486,000	567,000	680,000	850,000	1,130,000	1,700,000	3,400,000
200		271,000	348,000	406,000	487,000	610,000	812,000	1,220,000	2,440,000
300		224,000	288,000	337,000	404,000	505,000	674,000	1,010,000	2,020,000
400		197,000	253,000	295,000	354,000	442,000	590,000	885,000	1,770,000

a/ 1 mile = 1.61 km.
b/ 1 foot = 0.305 m.

benefit is again estimated at $392,000. The cost-effectiveness of the candidate improvements for a range of unit costs at various traffic volumes on highway curves for LVR roads is given in Tables I-6 and I-7. Tangent sections are not shown because none of the improvements were cost-effective for even the lower unit costs. Guardrail placement is only effective on fill slopes of 2:1 and cut slopes of 1:1 on the higher volume roads if the cost per foot of guardrails is $5.00 or below. In flattening fill slopes, there are more potentially cost-effective improvements, particularly for the lower embankment heights and the lower unit costs of fill material.

COST-EFFECTIVENESS OF BREAKAWAY POSTS FOR DOUBLE-POSTED WOOD SIGNPOSTS

The estimated severity reduction for making wood signposts was given previously as 0.10. Tables I-8 and I-9 show

TABLE I-6

COST-EFFECTIVENESS OF GUARDRAIL PLACEMENT OR REMOVAL ON HIGHWAY CURVES

ADT	Place Guardrail at 1:1 cut slope at 2:1 fill slope		Place Guardrail at bridge end at clump of trees	Place Guardrail at culvert at 3:1 fill slope at 1-1/2:1 cut slope	Remove Guardrail at 6:1 cut slope		Remove Guardrail at 5:1 cut slope
(vpd)	$5.00/ft	$7.50/ft	$5.00/ft	$5.00/ft	1.00/ft	2.00/ft	1.00/ft
50	0	0	0	0	X	0	0
100	0	0	0	0	X-	0	0
200	0	0	0	0	X	X	0
300	X	0	0	0	X	X	0
400	X	0	0	0	X	X	X

(X means improvement is cost-effective for unit price given)

1 ft = 0.305 m

TABLE I-7

COST-EFFECTIVENESS OF FLATTENING SIDE SLOPES ON HIGHWAY CURVES

	Improvement																			
	Flatten 2:1 to 6:1		Flatten 2:1 to 5:1		Flatten 3:1 to 6:1			Flatten 2:1 to 4:1			Flatten 3:1 to 5:1 Flatten 4:1 to 6:1			Flatten 2:1 to 3:1			Flatten 3:1 to 4:1 Flatten 4:1 to 5:1 Flatten 5:1 to 6:1		
ADT	≤$2.50 ft	$5.00 ft	≤$2.50 ft	$5.00 ft	≤$1.00 ft	$2.50 ft	$5.00 ft	≤$1.00 ft	$2.50 ft	$5.00 ft	≤$1.00 ft	$2.50 ft	$5.00 ft	$1.00 ft	$2.50 ft	$5.00 ft	$0.50 ft	$1.00 ft	$2.50 ft
50	X	0	X	0	X	0	0	X	0	0	X	0	0	X	0	0	X	0	0
100	X	X	X	0	X	X	0	X	X	0	X	0	0	X	0	0	X	X	0
200	X	X	X	X	X	X	0	X	X	0	X	X	0	X	0	0	X	X	0
300	X	X	X	X	X	X	X	X	X	0	X	X	0	X	X	0	X	X	0
400	X	X	X	X	X	X	X	X	X	X	X	X	0	X	X	0	X	X	0

(X means improvement is cost-effective for unit price given)

1 ft = 0.305 m.

(1) the number of improvements to save one fatal or non-fatal injury per year, and (2) the cost-effectiveness of the improvements on curves and on tangents for various traffic volumes and a range of unit costs. On highway curves, the breakaway improvement appears cost-effective for all traffic volumes and all reasonable unit costs. On tangent sections, the breakaway improvement is only cost-effective for the lower unit costs.

TABLE I-8

COST-EFFECTIVENESS TO MAKE DOUBLE-POSTED WOOD SIGNS BREAKAWAY ON HIGHWAY CURVES

ADT	R_{SV}	Number of Improvements To Reduce One I+F Acc/Year	Cost-Effectiveness (X Means Cost-Effective)		
			$50	$100	$200
50	0.068	1,588	X	X	X
100	0.101	1,069	X	X	X
200	0.141	766	X	X	X
300	0.170	635	X	X	X
400	0.194	557	X	X	X

TABLE I-9

COST-EFFECTIVENESS TO MAKE DOUBLE-POSTED WOOD SIGNS BREAKAWAY ON HIGHWAY TANGENTS

ADT	R_{SV}	Number of Improvements To Reduce One I+F Acc/Year	Cost-Effectiveness (X Means Cost-Effective)		
			$50	$100	$200
50	0.068	7,559	X	O	O
100	0.101	5,089	X	O	O
200	0.141	3,645	X	X	O
300	0.170	3,024	X	X	O
400	0.194	2,649	X	X	O

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