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16. Abstract

This research was performed to study the safety and operational characteristics of two-lane rural highway curves. A series of interdependent research methodologies was employed, including (1) multivariate accident analyses; (2) simulation of vehicle/driver operations using HVOSM: (3) field studies of vehicle behavior on highway curves; and (4) analytical studies of specific problems involving highway curve operations.

Among the study findings are recommendations regarding design of the highway curves. The research indicated important trade-offs among curve radius, curve length and superelevation. The value of spiral transitions was demonstrated by the studies of driver behavior. Significant path overshoot was observed at all sites regardless of the curve radius; this behavior was also modeled by HVOSM.

Studies of accidents on highway curves showed single-vehicle run-off-road accidents to be of paramount concern. Roadside treatment countermeasures were found to offer the greatest potential for mitigating the frequency and severity of accidents on rural highway curves.

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I. INTRODUCTION

Curves are a necessary and important element of nearly all highways. Their form has evolved from what appeared to be reasonable to the builder's eye to the more modern geometrically designed form of a circular curve with superelevation, cross-slope transitions, and sometimes spiral transitions.

Despite a reasonably well conceived design procedure, which considers a tolerable level of lateral acceleration on the driver, highway curves continually show a tendency to be high-accident locations. Several studies over the years have indicated that highway curves exhibit higher accident rates than tangent sections, and that accident rate increases as curve radius decreases. But, curve radius may be just one element that is interdependent with other elements that together contribute to accident rate. For example, the sharpest curves tend to be located on lower quality highways; those with narrow roadways, narrow shoulders, marginal sight distance, hazardous roadsides, etc.

The highway curve is one of the most complex features on our highways. The several elements or aspects of highway curves listed in Table 1 are all potential candidates for study in relating highway design to safety.

A concern for highway safety requires that the elements in Table 1 be designed and coordinated to accommodate the demands and limitations of both vehicle and driver. Given these requirements, a series of questions arises concerning design and operation of highway curves:

(1) Do drivers correctly perceive the radius of curvature and superelevation in managing the position and speed of their vehicles?

(2) Are curves designed for the true paths of vehicles?

(3) What levels of lateral acceleration represent an upper threshold of demand imposed by the driver/vehicle system?

(4) Which types of vehicles present the most critical dynamics to be considered in the design of each element?

(5) How do drivers perceive and respond to the build-up of lateral acceleration?

(6) Do spirals give the driver a safer transition into the circular curve? If so, what is their proper length?

(7) Do vertical sight restrictions on curves lead to a higher incidence of accidents involving trucks?

(8) Is there a safety trade-off between radius and length of curvature for a given central angle?

(9) Do run-off-road vehicles travel farther off the road on curves than on tangents?

(10) Are run-off-road vehicles more apt to roll over on curves than on tangents?

All of the roadway elements listed in Table 1 together with the questions about driver and vehicle interactions (these lists are presumably much larger) are parts of the puzzle which describes the relationship of highway curve design to highway safety.

Project Objectives and Scope

The primary objectives of this research were:

(1) To establish relationships between highway operations and safety and the geometric aspects of highway curves;

(2) To investigate cost-effective combinations of elements for a variety of operating conditions; and

(3) To develop design criteria and guidelines for these elements and their combinations for the design of new highways, reconstruction of existing highways, and spot improvement of existing highways.

The study was limited to two-lane rural highways carrying average daily traffic (ADT) of at least 1500. Methods of research included:

(1) Literature synthesis;

(2) Accident studies;

(3) Computer simulation of vehicle dynamics; and

(4) On-site operational studies.

TABLE 1

ELEMENTS OF HIGHWAY CURVES

A. Horizontal Alinement Elements

1. Radius of Curvature
2. Length of Curve
3. Superelevation Runoff Length
4. Distribution of Superelevation Runoff Between Tangent and Curve
5. Presence and Length of Transition
6. Stopping Sight Distance Around Curve

B. Cross-Sectional Elements

1. Superelevation Rate
2. Roadway Width
3. Shoulder Width
4. Shoulder Slope
5. Roadside Slope
6. Clear-zone Width

C. Vertical Alinement Elements

1. Coordination of Edge Profiles
2. Stopping Sight Distance on Approach
3. Presence and Length of Contiguous Grades
4. Presence and Length of Contiguous Vertical Curves

D. Other Elements

1. Distance to Adjacent Highway Curves
2. Distance to Nearest Intersection
3. Presence and Width of Contiguous Bridges
4. Level of Pavement Friction
5. Presence and Type of Traffic Control Devices
6. Type of Shoulder Material

II. LITERATURE REVIEW AND RESEARCH PLAN

The initial research task involved an extensive search and review of literature describing previous studies that attempted to relate accidents to roadway and traffic elements. The search and review process was undertaken to fulfill three basic functions:

(1) To provide a broad background of existing knowledge and gaps in the existing knowledge on the accident causation effects of roadway elements;

(2) to gain insights into problems encountered by researchers in experimental design, data collection, and data analysis; and

(3) to assist the research team in developing efficient, reliable experimental alternatives.

The literature search and review was not intended to be an exhaustive, time-consuming search identifying all possible sources of information, as one had been recently completed and published in National Cooperative Highway Research Program (NCHRP) Report 197, "Cost and Safety Effectiveness of Highway Design Elements" Roy Jorgensen Associates, 1978 (1). Instead, the literature search was undertaken to provide a meaningful direct input to the formulation of the research plan. Thus, titles were selected either for their significant findings or for the methodologies (data collection, statistical procedures, analysis interpretations) employed. In addition, relevant research published subsequent to NCHRP 197 was reviewed to update that knowledge of accident research.

Literature Search Procedure

Four basic sources of titles were used in the literature search:

(1) Highway Research Information Service (HRIS) computer search of all recent titles dealing with accidents and roadway elements

(2) Bibliographies and summaries listed in NCHRP Report 197

(3) Bibliographies from "Traffic Control and Roadway Elements--Their Relationship to Highway Safety" published by the Highway Users Federation for Safety and Mobility

(4) Transportation library and topical files in the offices of Jack E. Leisch Associates

The large number of titles to be reviewed required a two-step process. The first step was a cursory review of the publication to determine the following:

- Does the reference seem to have a relationship to the project? (If no, eliminate it from further consideration.)
- Does the reference report any important or specific conclusions? (If no, eliminate it from further consideration.)

Annotations provided in NCHRP 197 and included with the HRIS computer search enabled a rapid initial review of most of the literature.

Each publication found relevant to the research was reviewed in detail. The object at this second step was to document and evaluate the quality of study results by analyzing the methodology employed, the data collected, and the interpretation of the results. The key to these in-depth annotations was the critical review of each publication. The goal here was not to merely note the author's quantitative results and/or conclusions, but to judge the validity of these results. In judging each publication a number of important concepts were applied:

(1) Were all relevant variables considered?

(2) Was sufficient control for data collection errors maintained?

(3) Was sufficient detail maintained in data collection?

(4) Was the data sample obtained large enough for establishing statistically reliable results?

(5) Were the assumptions necessary in the applying statistical procedures actually met?

(6) Were tests of statistical significance applied?

(7) Were the results properly interpreted?

Following completion of all annotations, research findings were aggregated by general topic area (e.g., roadway width, horizontal alinement, roadsides, etc.). These findings were evaluated based on overall judgments of the nature and quality of the research which was behind them, and on the validity of interpretations which resulted from statistical tests. The following overall findings relating accidents and each general topic formed the basis for decisions regarding priorities for the research plan.

Findings

Horizontal Alinement

Most research studies dealing with curvature have come to the same basic conclusion, namely that curves are hazardous. Such conclusions, however, are meaningless in themselves. What must be ultimately answered is how hazardous are various curve radii, and under what conditions are horizontal curves particularly hazardous. Findings from some studies give indications of the answers to these questions. Kihlberg and Tharp (2) discovered that curves in combination with intersections resulted in greater accident rates. Billion and Stohner (3) found that overall poor alinement resulted in higher accident rates. A number of authors examined curves in isolation to determine their accident effects. Babkov (4) and Coburn (5) reported accident rates for various degrees of curve, and found that curves sharper than 2° are 20 to 50 percent more hazardous than tangents. Jorgensen (1) separated highway sections into those with alinement sharper than and milder than 3° of curve. Jorgensen reported approximately a 15 percent higher accident rate for alinement in excess of 3°. Taylor and Foody (6), in a study of curve delineation found that length of curve as well as its degree has an influence on accident rates. Raff's (7) study of the effects of multiple curves found no significant results.

One constraint in much of the research was a lack of discrimination in the data which was carried over to the analyses. Billion and Stohner defined "poor" alinement merely in terms of curvature in excess of 5°. Jorgensen categorized curvature using 3° as the breakpoint. Undoubtedly, discrimination of incremental accident effects requires greater detail in the collection and analysis of data describing horizontal curvature.

Superelevation Rate and Curve Transition

Previous research shows little indication as to the accident effects of variable superelevation practices, transition of superelevation on approaches to curves, or the use of spirals. Dart and Mann (8) used pavement cross slope as an independent variable in a multiple regression analysis of accident rates on rural highways, and noted marginal interaction effects. Whatever accident effects are present are undoubtedly small, and would thus require a large amount of data to discern. When this consideration is coupled with the general unavailability of such data and the difficulty of gathering it in the field, it is not difficult to understand the lack of previous studies of superelevation and/or spirals.

Roadway Width

A great number of studies of the relative safety of variable roadway widths have been performed. A few authors, namely Stohner (9), Gupta and Jain (10) and Sparks (11) observed no accident reduction effects of wider pavements. However, Gupta and Jain and Sparks did not use a direct measure of pavement width as an independent variable, thereby losing much of whatever sensitivity may have existed. Other authors have reported an effectiveness of wider pavements with respect to accidents. Among these, Raff (7), Dart and Mann (8), Jorgensen (1) and Zegeer and Mayes (12) reported limited effectiveness of roadway or pavement width. Raff showed increased roadway width on curves to be effective, but not on tangents. Dart and Mann and Jorgensen discovered significant effectiveness of 20 to 22-foot (6.1 to 6.7 m) widths, but little or no incremental effectiveness at 24 feet (7.3 m).

Unfortunately, data problems preclude acceptance of these findings as accurate indicators of the incremental effectiveness of pavement or roadway widths. The Jorgensen study utilized study sections of insufficient length to assure that reasonable distributions of accidents were actually being sampled. The Dart and Mann data sample was too small to significantly note incremental effects of roadway width.

In any case, the effectiveness of roadway width in reducing accidents is undoubtedly interrelated with other roadway elements, such as shoulder width, horizontal alinement and roadside character, and with traffic elements such as volume and percent trucks. Zegeer and Mayes attempted to examine such interactions between traffic volume and roadway width and between roadway width and shoulder width. Their study design, however, was incapable of examining all possible interactions, and results were not statistically reliable.

Shoulder Width and Type

One of the most widely studied roadway elements has been the shoulder. A great number of authors have reported highly variable findings concerning the effects of shoulder width on accident rates. Schoppert (13), Perkins (14), Taragin (15,16) and Sparks (11) did not detect any significant effects of variable shoulder widths. Stohner (9) and Jorgensen (1) noted that wider shoulder widths resulted in lowered accident rates. The Jorgensen study in particular indicated that shoulder width was more sensitive to accident rates than roadway width. Shoulder type (paved vs. unpaved) was also noted as being an important determinant to accident rates. Foody and Long (17), Raff (7) and Billion and Stohner (3) noted only marginal effects, or effects discernable under certain situations (e.g., Billion and Stohner concluded that wider shoulders are most effective in reducing accidents on poor alinement).

A problem encountered by most researchers was the variability in both traffic volume and facility type which accompanies variable shoulder widths. Higher volume, primary-type facilities with a higher quality alinement and clear roadsides tend to have wider shoulders. Many of the studies did not consider these factors, hence any variation in accident rates could not necessarily be attributed to shoulder width alone. As with roadway width, the interactions of shoulder width with other elements, suggested by Raff and Foody and Long, may be significant.

Vertical Alinement

The safety effects of vertical alinement, including variable grades, lengths of grade and vertical curvature, are difficult to estimate given the available literature. Most studies which include vertical alinement within the set of independent variables have categorized vertical alinement very roughly (e.g., "good" alinement, with grades less than 5 percent vs. "poor" alinement, with grades greater than 5 percent); or have treated it indirectly, by focusing on variables such as sight distance restrictions. Examples of such studies include

Billion and Stohner's work (3), as well as Sparks (11), Cirillo (18), and Foody and Long (17). Difficulties in data collection undoubtedly have hampered efforts aimed at quantifying the effects of severe and/or long grades. Not surprisingly, most authors have concluded that such effects cannot be discerned.

A number of publications are useful in estimating the possible effects of vertical alinement and in treating this variable in a multivariate analysis. Cirillo (18), in a study of interstate accident rates, concluded that the total geometric effects of all elements of the highway account only for marginal accident impacts. On two-lane rural highways, with more variable alinement and speeds, such effects may be higher. Nevertheless, the individual effect of grades, or interaction of grade with other elements, is probably small.

Sight Distance

The presence of obstructions or poor alinement which result in restricted sight distance can contribute to a potentially unsafe condition. A number of studies have attempted to determine the degree to which such restrictions translate into higher accident rates. Cirillo (18) and Foody and Long (17) developed regression equations that included sight distance restrictions as an independent variable. Schoppert (13) judged sight distance to be relatively unimportant in explaining variations in accident rates. Agent and Deen (19) noted that a significant proportion of total accidents on two-lane rural highways are rear-end collisions, which suggests that sight distance may play a role in accident causation. However, it would seem that the hazardous effects of poor sight distance would be most likely to be observed at or near intersections, where turning and crossing maneuvers conflict with traffic along the highway.

The general lack of reliable study results on which to judge incremental effects of variable sight distance restrictions is in large part due to difficulties in collecting data which describe accurately the sight distance for large numbers of highway sections. Efforts to date have generally categorized sight distance or have found it necessary to utilize field studies to collect such data.

Roadside Design

The character of the area adjacent to the highway can have a significant impact on safety. Roadside design, which refers to the roadside slope, ditch sections and presence and type of obstacles adjacent to the roadway, has been studied by a number of authors. The relative hazard of steep roadside slopes has been documented by Glennon (20), Foody and Long (17), and Deleys (21), among others. Studies in Georgia (22) and Arizona (23) of single-vehicle fatal crashes on rural highways have noted a preponderance of rollovers on highway curves. More recently, Graham and Harwood (24) have evaluated clear-zone policies and found that accident rates decrease as the clear zone is widened and roadside slopes are flattened. Also, a Michigan study (25) found that, in general, both the frequency and severity of roadside accidents were higher on highway curves than on tangent sections.

Traffic Volume

Many studies have attempted to relate accident rates to traffic volume. Some studies indicate that accident rates increase with ADT, others indicate the reverse, and still others have concluded no effect. The reasons for these contradictory findings are as follows:

(1) Most studies have not considered the interaction of traffic volume with other elements. It has been highway development practice over the years to upgrade highway design as traffic volume increases. The greater the traffic volume on the road, the more likely that road will have wider roadway and shoulder widths, and milder horizontal and vertical alinement.

(2) There is an underlying dynamic relationship between accident rates and traffic volume which has not been considered in many studies. Highways with very low traffic volumes have a very high proportion of single-vehicle accidents. As traffic volume increases, vehicle to vehicle interaction increases, causing this proportion to decrease. What seems apparent from NCHRP Report 47 (2) is that the single-vehicle accident rate decreases with ADT and the multivehicle accident rate increases with ADT.

(3) Another dynamic effect related to the single to multivehicle accident ratio is the traffic distribution throughout the day. Two highways with equal ADT might have grossly different accident effects related to ADT if one highway has a much higher peaking characteristic.

As a result of ignoring these dynamic effects, most past studies have not truly explained the accident rate/traffic volume relationship. In some cases they have either combined urban and rural highways or attempted to predict the relationship with a straight line. Indications are that accident rates drop sharply between ADT's of 50 and about 2000. Between about 2000 and 3000 ADT the accident rate bottoms out and is fairly constant, whereupon accident rate increases gradually as volumes increase above 3000 ADT.

Summary of Literature on Accident Effects of Geometrics

Despite many years of effort, the current knowledge about the accident effects of incremental changes in roadway and traffic elements is extremely limited. While it is generally recognized that wide pavements are "safer" than narrow ones, and that sharp curves are "hazardous," the critical questions of just how wide roadways and/or shoulders need be, and how sharp a curve can be tolerated, remain unanswered. Research efforts to date have, in general, been unable to sort out the accident effects of variable roadway widths, curvature, and other roadway elements from the large number of other variables (traffic, geometric, topographic, etc.) which both act directly and interact to affect accident experience.

The reasons that true accident effects have yet to be discovered are basically threefold.

(1) Because accidents are rare events, very large data samples are generally required to discern accident effects. This requirement not only creates organizational and budget problems, but also conflicts with the dynamic character of a route or system being studied. Thus, in attempting to study the accident characteristics of a facility, researchers often have been faced with changes in traffic volumes, construction along the facility and even changes in local accident reporting practices. Such problems limit the amount and quality of data available for analysis.

(2) When sufficient data are available, the data are often not in a form to enable determination of incremental effects. For example, a number of authors utilized State highway department data to evaluate the effects of vertical and horizontal alinement on accidents. Unfortunately, the

data provided discrimination to only two or three levels, thus precluding any comparison of the effects of relatively small differences in either variable. Other problems also occur when attempts are made to combine data from two or more jurisdictions. Differences in quality of data, actual information collected, and reporting levels have contributed to inconclusive or contradictory findings.

(3) The fact that accidents are not only rare events, but are also extremely complex, has also confounded many attempts to study their causes. The highway itself contains a large number of individual elements (cross-sectional, such as roadway widths and roadside slopes; and longitudinal, including horizontal and vertical alinement) which all act in concert on the driver and vehicle. The human element alone is undoubtedly a major cause behind much of the variance in accidents that occurs. These considerations contribute greatly to the difficulties encountered by accident researchers.

The above discussion points to the need to design a study of incremental accident effects with great care. Unfortunately, much of the research in the field of accident causation has problems with either the plan, data collection procedure, or analyses. Basic problems encountered reflect limited understanding of the above concepts, underestimation of their importance, or difficulties with the data base itself.

Accident research problems that tend to recur include the following:

(1) Failure to account for or control for variables not being studied.--Examples of this major problem abound in studies of variable shoulder widths and curvature. Most highways with wide shoulders also have wider pavements, better roadside design and milder alinement, as well as higher traffic volumes. Studies in the past have collected and analyzed data which did not account for the effects these variables might have had on accident rates. Similarly, studies of horizontal curve effects have attributed higher accident rates to sharper curves, while apparently ignoring width and traffic variability on the curves.

(2) Insufficient Exposure Levels.--Many studies have utilized only one year of data, or have used injuries or injury rates as the dependent variable. In such cases, the distribution of accidents by section becomes skewed towards "zero" and "one," i.e., very few sections with more than one accident are found. Problems in the statistical reliability of findings tend to result from multiple regression analyses based on such data distributions. Exposure level problems can also occur if section lengths are too short or traffic volume levels too low. These problems occur when large numbers of sites are required. The researchers typically shorten the section length to increase the number of sites (and data points), but in so doing merely increase the number of sections with zero accidents.

(3) Failure to Recognize and Evaluate Interactions Among Variables.--Studies employing multiple regression techniques have encountered difficulties in developing meaningful results because the relationships investigated did not include interaction effects among the dependent variables. What results is a relationship which describes accident effects over the wide range of dependent variables, but which may be useless in predicting effects for a given set of conditions.

An understanding of these basic problems is essential to the design of a research plan and data collection procedure.

General Research Plan

The initial plan for this research was to study the incremental accident effects of various roadway design elements. However, following the critical analysis of literature, it appeared that sole dependence on such a plan would yield limited results. Therefore, decisions were made to limit the research effort to the geometric design aspects of highway curves, and to pursue more than one avenue of research. This section of the report addresses: (1) the hypothesis of safety problems on highway curves; (2) the identification of alternative research methods; (3) the selection of feasible research problem statements and attendant research methods; and (4) the development of an integrated strategy for data collection and analysis.

Hypothesized Safety Problems

Based on the critical review of literature and the experience of the research staff, a list of general hypotheses associated with the safety of highway curve geometrics was developed. This list, shown in Table 2 presumably covers the major concerns of highway engineering professionals who design and maintain highway curves.

TABLE 2

PRELIMINARY GENERAL HYPOTHESES ON THE SAFETY OF HIGHWAY CURVES

(1) Safety varies directly with curve radius.

(2) Safety varies directly with roadway width.

(3) Safety varies directly with shoulder width.

(4) Curves with spiral transitions are safer than those without.

(5) Safety is decreased as the 85th percentile operating speed exceeds the design speed.

(6) With respect to safety, there is an optimum distribution of superelevation runoff.

(7) There is a safety trade-off between length and radius of curve for a given central deflection angle. Stated differently, there is a net safety loss as the length of curve increases for a given curve radius.

(8) Approach conditions to a curve are an important safety consideration. Safety is decreased as sight distance becomes restrictive, as the curve becomes more isolated, and as approach alinement encourages higher speeds.

(9) Because of a tendency to produce a high proportion of single-vehicle accidents, roadside hazards are an important element on highway curves.

(10) Roadside slopes that are generally acceptable for tangent sections may promote vehicle rollover on curve sections.

(11) The safety of nominal shoulder encroachments varies inversely with the amount of cross-slope break between the shoulder and the roadway.

(12) Pavement settlement and/or washboarding on highway curves may produce unsafe conditions.

(13) A steep downgrade preceding a highway curve may produce unsafe conditions.

Research Methods

Four basic research methods were selected to study the hypothesized problems. Accident studies, computer simulation, field operational studies, and analytical studies were integrated in the research approach in order to cover the broadest range of specific research questions and in some cases to provide support or verification for one another.

Accident Studies.--These studies used the combination of a highway site data base and a corresponding accident data base to both study the direct incremental accident effects of individual geometric elements and the relative safety of various combinations of geometric elements. Specific techniques included: accident characterization of site categories, analysis of covariance, and discriminant analysis of high- and low-accident sites.

Computer Simulation.--The Highway Vehicle Object Simulation Model (HVOSM) methodology was used to measure the dynamic responses of vehicles to various highway curve geometrics using predetermined driver operating characteristics.

Field Operational Studies.--These included measurements of speed, path, and placement of vehicles on highway curves. The major intent was to provide indirect measures of driver behavior as input to the HVOSM. These studies were also intended to provide information on drivers' speed responses to various approach conditions.

Analytical Studies.--This method used basic physical laws and certain assumptions about vehicular operations and driver behavior in an attempt to gain insights about those research questions that could not be addressed within project constraints by one of the other methods.

Research Questions

From the hypothesized problems, a series of research questions about various aspects of highway curves was developed, and the feasible research method or methods were selected to address each question. For some of the questions, it was determined that none of the research methods was feasible either because of unavailability of data or because of the perceived cost-effectiveness within the total project scope. The questions and their proposed research methods are shown in Table 3.

The research plan was conceived as a multifaceted attack on the objectives of the research. This approach involved several stages of accident analysis, computer simulation of vehicle dynamics, and field studies of vehicular speeds and path-following on curves. These various techniques were integrated in an attempt to achieve the maximum knowledge on the effects of highway curve geometrics on highway safety. Site selection was integrated to provide information for an Analysis of Covariance, an Accident Characterization, and a Discriminant Analysis of high- and low-accident sites, and to provide a base of sites for operational field studies.

Research Approach

The research approach is shown schematically in Figure 1 and will be discussed more fully in later sections of this report. This figure illustrates the several stages of accident analysis, HVOSM computer simulation studies of vehicle dynamics, and traffic operations studies of vehicular speed and path-following on highway curves.

Figure 1 shows how maximum knowledge about the safety and operations of highway curves was obtained through an integrated approach that utilized project linkages to connect these research methods. Of particular importance are the stages of data collection and site selection which were designed to provide inputs into the multivariate accident analysis using Analysis of Covariance, the Discriminant Analysis of high- and low-accident sites, the selection of speed study sites and traffic operations study sites.

TABLE 3

RESEARCH QUESTIONS AND ATTENDANT RESEARCH METHODS

	Proposed Research Methods				
	Accident Studies	HVOSM Studies	Field Operations Studies	Analytical Studies	Not Feasible
RADIUS OF CURVE					
Do accident rate increase with decreasing radius of curve ?	●				
Is the probability of a high - accident location sensitive to degree or radius of curve ?	●				
What is the criticality of under - designed curves with respect to driver control ?		●	●		
ROADWAY WIDTH					
Does accident rate increase with narrower roads ?	●				
Is the probability of a high - accident location sensitive to roadway width ?	●				
Under what circumstances is roadway widening on curves justified ?		●	●		
SHOULDER WIDTH					
Does accident rate increase with narrower shoulders ?	●				
Is the probability of a high - accident location sensitive to shoulder width ?	●				

TABLE 3

RESEARCH QUESTIONS AND
ATTENDANT RESEARCH METHODS (continued)

	Proposed Research Methods				
	Accident Studies	HVOSM Studies	Field Operations Studies	Analytical Studies	Not Feasible
SHOULDER TYPE					
Is there a relationship between accident rate and shoulder type ?	●				
Is the probability of a high - accident location sensitive to shoulder type ?	●				
Does shoulder type affect the stability of traversal onto the shoulder ?					●
LENGTH OF CURVE					
Does accident rate increase with length of curve (for the same radius) ?	●				
Is the probability of a high - accident location sensitive to length of curve ?	●				
Is there a safety trade - off between length and radius of curve for a given central angle ?	●				
Do very short, sharp curves present significant, unusual driver control problems ?		●	●		
CURVE APPROACH CONDITIONS					
Is there a relation between approach conditions and accident rate ?					●
Is the probability of a high - accident location sensitive to approach conditions ?	●				
Are there approach conditions that encourage significant high speeds on the curve approach ?			●		

TABLE 3

RESEARCH QUESTIONS AND
ATTENDANT RESEARCH METHODS (continued)

	Proposed Research Methods				
	Accident Studies	HVOSM Studies	Field Operations Studies	Analytical Studies	Not Feasible
SUPERELEVATION RUNOFF					
Is accident rate sensitive to the length and distribution of superelevation runoff ?					●
Is the probability of a high - accident location sensitive to the length and distribution of superelevation runoff ?	●				
Are vehicle operations and control sensitive to the superelevation runoff design ?		●	●		
SPIRAL CURVES					
Is there a relationship between the presence of spirals and accident rate ?					●
Is the probability of a high - accident location related to the absence of spirals ?					●
What is the sensitivity of vehicle control to spiral presence for various speeds and curve radii ?		●			
Is vehicle operation sensitive to the length of spiral ?		●			
ROADSIDE FEATURES					
Is there a relationship between the degree of roadside hazard and accident rate ?					●
Is the probability of a high - accident location sensitive to the degree of roadside hazard ?	●				
Do normally acceptable roadside slopes contribute to a high incidence of rollovers for encroaching vehicles ?		●		●	
What is the relationship between lateral and longitudinal displacement of roadside encroachments and degree of curve ?				●	

TABLE 3

RESEARCH QUESTIONS AND ATTENDANT RESEARCH METHODS (continued)

Research Questions	Accident Studies	HVOSM Studies	Field Operations Studies	Analytical Studies	Not Feasible
SKID RESISTANCE					
Is there a relationship between skid resistance and accident rate ?					●
Is the probability of a high - accident location sensitive to skid resitance ?	●				
Does skid resistance deteriorate faster on sharper curves ?				●	
CROSS - SLOPE BREAK					
Is there a relationship between cross - slope break and accident rate ?					●
Is the probability of a high - accident location sensitive to cross - slope break ?	●				
What is the relationship between cross - slope break and vehicle stability and / or driver discomfort ?		●			
STOPPING SIGHT DISTANCE ON CURVE					
Is there a relationship between available sight distance and accident rate ?					●
Is the probability of a high - accident location sensitive to available sight distance ?	●				
Are AASHTO stopping sight distance requirements consistent for tangents and curves ?				●	

TABLE 3

RESEARCH QUESTIONS AND ATTENDANT RESEARCH METHODS (continued)

	Proposed Research Methods: Accident Studies	HVOSM Studies	Field Operations Studies	Analytical Studies	Not Feasible
GRADE					
Is there a combined effect of grade and curvature that relates to accident rate ?					X
Is there a combined effect of grade and curvature that affects vehicle operations and control ?		X			
PAVEMENT SETTLEMENT					
Does pavement settlement or "washboard" have a significant effect on vehicle control ?				X	

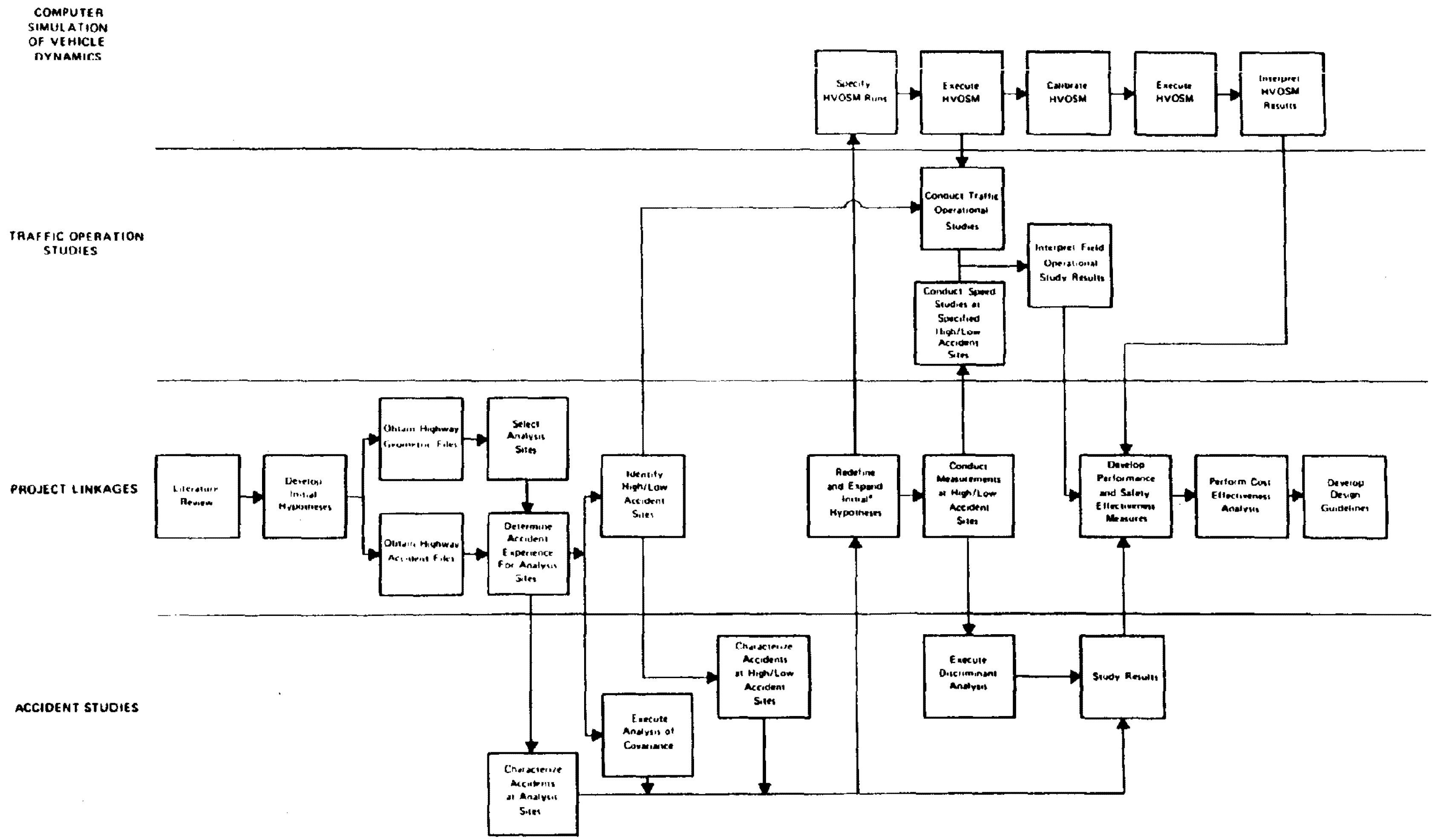

Figure 1. RESEARCH FLOW CHART

III. HIGHWAY CURVE SITE SELECTION

The objectives of selecting highway curve sites for this research were to:

(1) Identify a large site population of pure highway curve sections for general analysis of accident experience;

(2) Acquire an adequate data base for identifying a set of high- and low-accident sites for the determination of accident-prone combinations of geometrics; and

(3) Provide the necessary data for selecting a limited, but highly defined set of operational study sites.

Procedures and constraints were applied for ensuring a reliable data base, designating States from which data would be obtained, and selecting curve and tangent highway segments for analysis.

Selection of Candidate States

For reasons of both efficiency and economy, the site selection was limited to not more than five States. The following criteria were used in developing a preliminary list of candidate States:

(1) Extent of Highway Meeting Study Constraints - The study constraints as set forth in the work plan limited analyses to predominantly two-lane rural highways with at least 1500 ADT. Nationwide statistical summaries were consulted to determine the extent of rural highways meeting these constraints in each State. Table 4 shows the total mileage of rural highways in the range of 2,000-10,000 ADT for selected States. The first 13 States, listed in descending order by total surfaced mileage, are those with the most rural highway mileage in the United States. Virginia and Michigan (ranked 24th and 25th, respectively) are also included because of their high percentage of total surfaced mileage with ADT of 2,000-10,000.

(2) Geographic Distribution - It was desirable, if possible, to include States that represented the different climates and topography found throughout the United States.

(3) Availability of Geometric and Accident Data - The amount and quality of data, as well as the ease of retrieving certain information, varies among States. Those States with the most accessible and reliable data were considered the best candidates.

TABLE 4

SELECTED STATISTICS
STATE PRIMARY HIGHWAY SYSTEMS

State	State Primary Highway System - Rural: Total Surfaced Mileage	Mileage with 2,000-10,000 ADT	Percent of Mileage with 2,000-10,000 ADT
*Texas	61,809	10,480	17.0
*Ohio	16,019	6,135	38.3
Georgia	15,605	4,982	31.9
Arkansas	13,851	2,797	20.2
*Illinois	13,155	5,901	44.9
California	12,749	5,260	41.3
North Carolina	11,976	5,522	46.1
New York	10,894	5,117	47.0
Oklahoma	10,869	3,106	28.6
New Mexico	10,582	1,793	16.9
Wisconsin	10,197	4,333	42.5
Minnesota	10,061	2,907	28.9
*Florida	10,037	4,995	49.8
Virginia	8,147	4,241	52.1
Michigan	8,103	4,371	53.9
Total-U.S.	408,821	134,549	32.9
Total Candidate States	101,020	27,511	27.2
Percent of U.S.	24.7	20.4	

*Candidate States

Source: Reference (26)

(4) Willingness to Cooperate in the Study - While the States were not asked to actively participate in the study, their willingness to provide information and assist in interpretation was deemed to be imperative to success of the project.

Based on these criteria, the following preliminary candidate States were selected:

California	Ohio
Florida	Pennsylvania
Illinois	Texas
Michigan	Virginia

FHWA, through its regional and division offices, solicited and obtained the cooperation of each candidate State. Initial contacts with California and Virginia disclosed that data describing horizontal alinement were not in a form readily usable for this research. Pennsylvania was not considered after it was found that contiguous Ohio had an excellent, accessible data base.

Preliminary visits were made to State highway agencies in Florida, Illinois, Michigan, Ohio, and Texas. A checklist was used to insure completeness and uniformity of information obtained from each highway agency. Results of these initial state visits are summarized as follows:

Florida - Highway geometry and accident data are both available on automatic data processsing (ADP) files. "Straight line diagrams" (SLDs) are also maintained as a source of highway geometry information. Since the Florida ADP highway inventory system would be difficult to access for the specific needs of this project, it was concluded that the SLD would be the best source of information.

Illinois - Highway geometry and accident data are both available on ADP files. There are inconsistencies, however, in designation of milepost locations between the two files. With some assistance from state personnel, it was found that accident locations could be re-coded wherever necessary, to correspond with mileposts given in the highway inventory.

Michigan - Highway inventory data are available either from the State's computer file or from photo-logs. Accident records are also maintained in ADP files. The State uses a Burroughs computing system, however, which posed a problem in transferring information to the FHWA computer facilities.

Ohio - Highway inventory and accident records are both available on ADP files. The highway inventory, however, only identifies curves sharper than 3 degrees and grades steeper than 3 percent. There are also SLDs but these had insufficient geometry information for this research.

Texas - Highway geometry data would have to be obtained from SLDs. Some items are available in ADP files, but curve data can only be obtained from SLDs. Accident records are maintained in ADP files, but accident location is only recorded to the nearest 0.10 mile (0.16 km).

Although minor problems were expected in compiling data from any of the candidate States, all five were suitable for the study. Further analyses indicated, however, that sufficient coverage could be obtained through use of four, rather than five States. Michigan was classified as an alternate, leaving Florida, Illinois, Ohio and Texas as the recommended candidate States.

These four States represent a reasonable cross section of the nation in terms of both climate and topography. As indicated in Table 4, they account for nearly 25 percent of all rural highways on State primary highway systems, and approximately 20 percent of the rural highways with ADT from 2,000-10,000.

Field Inspection of Sample Curve Sections

A critical factor in research that uses a large data set is the reliability and accuracy of the data. One task within the site selection process, therefore, was a field check of a sample of curve sections identified from the State geometry files. The following information was field checked for representative curve sections in Illinois, Ohio and Texas to verify the quality of each State's geometric data base:

(1) Presence of the highway curve
(2) Roadway width at the curve
(3) Shoulder width and type at the curve
(4) Approach and departure grades (i.e., "flat," "steep upgrade," etc.)
(5) Curve radius and approximate length
(6) Presence of intersections and other landmarks in the vicinity of the curve

In every case where curve segments were inspected, the curve was found at the location indicated in the files. Recorded vertical alinement was also consistently accurate in each State. In Ohio, roadway and shoulder width measurements proved to consistently match those indicated in the State files. In Illinois, some differences were found in recorded vs. measured roadway width which were traced later to a lack of up-to-date data. The few width discrepancies found in Illinois were not considered to be serious enough, however, to invalidate the entire data base.

A check of the actual curve radius was also made using chord offset measurements to the inside edge of pavement. In almost all cases, the curve radii were verified within tolerances expected as a result of the survey procedures used in this preliminary check.

In summary, the field inspection indicated that horizontal and vertical alinement was accurately recorded in State files. For the most part, width measurements were also recorded accurately. At this point, it was reasonable to conclude that the selection of curve sections and the geometric characterization of those sections could be accomplished with the required accuracy using State inventory data.

Site Control Criteria

The selection of curve analysis segments was designed to produce a large set of rural highway curves (and tangents, for comparison) meeting the various constraints established by the work plan and unaffected by factors which might produce unexplained variances in accident experience. A curve analysis segment was defined as the full length of a highway curve with a minimum length of tangent on each end of the curve. The need to include tangent length as a part of a curve segment was dictated by the variance expected in reporting the location of curve-related accidents at the point of rest of the vehicle, either upstream or downstream from the curve itself.

It was also recognized that analysis segment lengths needed to be relatively consistent to ensure that the comparative accident experience among sites would be sampled from comparable Poisson distributions. A uniform segment length of 0.6 mi (1.0 km) was specified, therefore, unless longer segments were needed to accommodate minimum tangents.

The process used in selecting analysis segments from geometric highway inventory files is diagrammed in Figure 2 and described below.

Study Constraints

The work plan specified that the research would be concerned with rural, two-lane curves on highways where the traffic volume was at least 1500 vehicles per day.

The initial screening process eliminated all highways within urban areas or within 1.2 mi (2.0 km) of a city limit. Florida inventory data sometimes showed both city limits and urban limits, the latter being farther out from the central city. In such cases, for conservatism, the separation was measured from the urban limit. In Texas, some city limits were found to extend from their normal bounds in a narrow strip along the highway. Again, in the interests of conservatism, the separation was measured from the end of any such extension.

All highways with more than two lanes or traffic volume less than 1500 ADT were then deleted from further consideration. (In Texas, some highways of 1400 ADT or greater were included.)

External Influences

Sections of highway conforming with the study constraints, described above, were then tested to eliminate variances which would be associated with external influences such as intersections and structures.

All highway sections within 660 feet (200 m) of a major intersection were eliminated from further consideration. Major intersections were defined as those with marked U.S. and State Highways. In Texas, junctions with State Highways designated Farm-to-Market (FM) routes were also considered major intersections.

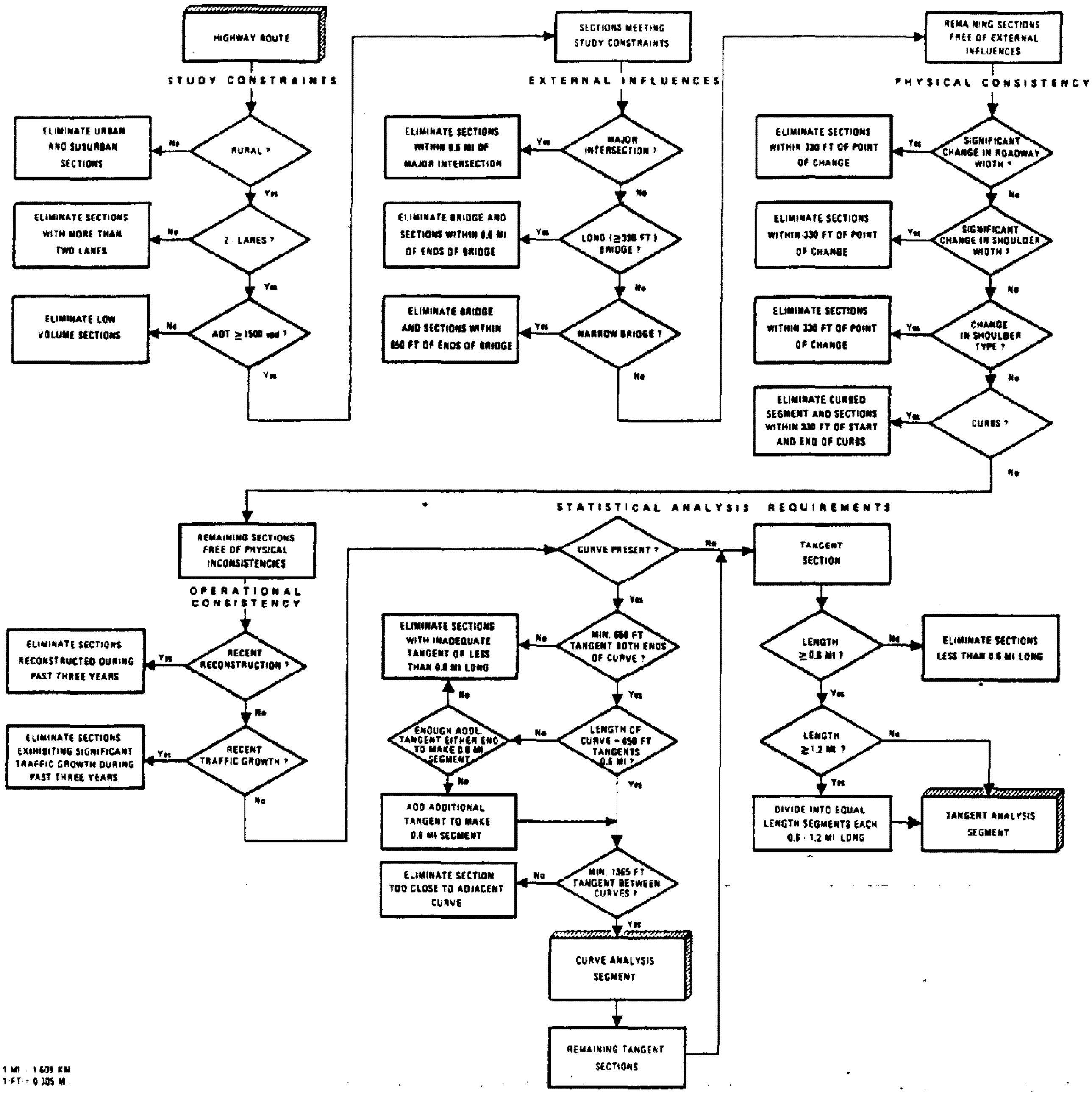

Figure 2. CURVE SEGMENT SELECTION PROCESS

To avoid variances associated with structures, major bridges and highway segments within 0.6 mi (1 km) of the ends of each major bridge were deleted. Potential analysis segments containing more than one minor structure were also deleted. The external influence constraints related to structures could not be applied, however, in treatment of Illinois data due to unavailability of the required information.

Physical Consistency

To avoid variances associated with boundary conditions, no study sites were selected within 330 feet (100 m) of a significant change in roadway width, shoulder width or shoulder type. In addition, all curbed sections of highway and segments within 330 feet (100 m) of the start or end of curbed sections were deleted.

Operational Consistency

Since a three-year safety record was required for each analysis segment, it was necessary to screen out locations containing recent reconstruction during this period, or those which exhibited significant traffic growth over the past three years. This constraint was applied only when the inventory data clearly indicated reconstruction or traffic inconsistency.

Designation of Analysis Segments

Segments of highway passing all of the screens described above were then investigated for the presence of curves. Whenever a curve was found, a determination was made whether a minimum tangent length of 650 feet (200 m) was available at each end of the curve. In treating Ohio data, where only curves of 3° or sharper were recorded in the inventory file, the assumption was made that approaches to and departures from all recorded curves were tangents, but it was recognized that some might include mild curves. In subsequent field investigations of someof these sites, it was found that only about nine percent of the segments did not have a tangent approach and departure.

Separation between the point of tangency (PT) of one curve and the point of curvature (PC) of another had to be at least 1300 feet (400 m) in order to provide minimum tangents in each analysis segment. In application, however, the minimum separation was taken as 1365 feet (416 m) to insure that the milepost of the end of one segment was 0.01 mi (16 m) different than the starting milepost of the next segment. This step precluded double counting of an accident that occurred at the boundary between two analysis segments.

Finally, whenever possible, curves were centered within the 0.6 mi (1 km) long analysis segments. If necessary to meet various constraints, however, the curve was shifted within the analysis segment, but the minimum length of tangent between the extremity of the curve and the beginning or end of the analysis segment was never less than 650 feet (200 m). In rare cases, when the length of curve plus the minimum tangent lengths totaled more than 0.6 mi (1 km), the analysis segment length was increased in 0.3 mi (0.5 km) increments until all specifications were met.

Once the curve analysis segments had been identified, the remaining portions of the highway meeting all other constraints were available for designation of tangent analysis segments. Tangent segments, each 0.6 mi (1 km) long, were selected for nearly even distribution over volume, surface width, shoulder width and shoulder type classifications. To minimize data collection efforts, selection of tangent segments was restricted to Florida and Texas.

Site Characterization

Because of the way the curve segments were selected, the sample should roughly approximate the population of highway curves on main rural two-lane highways in the United States. The four-State sample consisted of 3557 analysis segments. Of these, 3304 were curve segments and 253 were tangent segments. The curve segments consisted of 2071 curves (63 percent) of less than 3° of curvature, and 1233 (37 percent) with degree of curvature of 3° degrees or greater.[1]

Average daily traffic (ADT) at the analysis segments was predominantly in the lower ranges. A total of 2240 sites (63 percent) had ADT less than 3100 vehicles per day. Table 5 summarizes the number of curve and tangent analysis segments in each ADT group.

The distribution of sites by degree of curvature and width of roadbed is shown in Table 6. Sites are summarized by ADT and width of roadbed in Table 7, and by length of curve and degree of curvature in Table 8.

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[1]Further characterization and discussion of highway curvature in this report will include "degree of curve" as a descriptor of highway curvature. Degree of curve is a commonly used and understood measure of curvature. The most common definition of degree of curve is the arc definition. According to arc definition, degree of curve is the central angle subtended by a 100-foot (30.5 m) arc.

This description of highway curvature was common to the data files of all four States in the study. For expediency in handling existing data bases and consistency with U.S. design practice, it was decided to retain degree of curve as the basic descriptor for highway curvature. Unfortunately, there is no metric equivalent to degree of curve which is universally accepted. Alternate definitions of degree of curve include arc definitions based on 20 m, 30 m and 100 m arcs (equivalent to arc lengths of 66 feet, 98 feet and 328 feet). More common metric design practice involves definition of the radius of curve.

The reader is referred to Appendix A, which describes the equivalent metric radius of curve for a range of curvature defined by a 100-foot (30.5 m) arc length. Also shown are alternative metric degrees of curve based on various definitions.

TABLE 5

DISTRIBUTION OF CURVE ANALYSIS SEGMENTS
BY TRAFFIC VOLUME

Traffic Volume (ADT)		Number of Analysis Segments		
		Curve	Tangent	Total
<2100		1061	76	1137
2100-3099		1037	66	1103
3100-4899		746	70	816
4900-9999		409	37	446
≥10,000		51	4	55
	All Volumes	3304	253	3557

TABLE 6

DISTRIBUTION OF CURVE ANALYSIS SEGMENTS BY DEGREE OF CURVE AND WIDTH OF ROADBED

Degree of	Number of Segments by Width of Roadbed (Feet)							
Curvature	<28	28-31.9	32-35.9	36-39.9	40-43.9	44-47.9	>48	Total
Tangent	6	38	51	39	54	65	0	253
<1°	6	67	68	47	178	153	13	532
1°-2°59'59"	25	184	238	210	414	443	25	1539
3°-4°59'59"	34	74	72	63	116	88	5	452
5°-7°59'59"	58	75	86	53	49	28	0	349
>8°	163	110	75	49	19	16	0	432
Total	292	548	590	461	830	793	43	3557

Note: Width of roadbed is total surfaced width plus width of shoulders.
1 ft = 0.305 m

TABLE 7

DISTRIBUTION OF CURVE ANALYSIS SEGMENTS BY TRAFFIC VOLUME AND WIDTH OF ROADBED

Traffic Volume	Number of Segments by Width of Roadbed (Feet)							
(ADT)	<28	28-31.9	32-35.9	36-39.9	40-43.9	44-47.9	>48	Total
<2100	114	272	222	138	203	181	7	1137
2100-3099	107	107	190	159	275	247	18	1103
3100-4899	55	107	96	99	198	252	9	816
4900-9999	15	56	66	64	135	103	7	446
>10,000	1	6	16	1	19	10	2	55
Total	292	548	590	461	830	793	43	3557

Note: Width of roadbed is total surfaced width plus width of shoulders.
1 ft = 0.305 m

TABLE 8

DISTRIBUTION OF CURVE ANALYSIS SEGMENTS
BY DEGREE AND LENGTH OF CURVE

Length of Curve (Miles)	Number of Segments by Degree of Curve						Average Curvature (Degree)†
	<1°00'00"	1°-00'-00"-2°-59'-59"	3°-00'-00"-4°-59'-59"	5°-00'-00"7°-59'-59"	≥8°-00'00"	Total	
<0.100	104	272	124	218	385	1103	5.8
0.100-0.199	236	571	198	108	40	1153	2.7
0.200-0.299	113	383	99	18	6	619	2.3
≥0.300	79	313	31	5	1	429	1.9
Total	532	1539	452	349	432	3304	3.6
Average Length*	0.20	0.20	0.15	0.10	0.05	0.15	

* Rounded to nearest 0.05 mi

† Rounded to nearest 0.1 degree of curvature

1 mi = 1.6 km

IV. ACCIDENT STUDIES

A three-year history of accident experience was compiled and analyzed for each of the curve and tangent analysis segments. The basic objectives of this phase of the research were to determine the relationships between accidents and highway geometrics and to identify those geometric elements that contribute the most to accidents.

Three separate, but interrelated, analysis steps were performed:

(1) **Characterization** of accident experience of both curve and tangent analysis segments;
(2) **Analysis of Covariance** (AOCV), a multivariate analysis to determine incremental effects of basic geometric and traffic variables; and
(3) **Discriminant Analysis**, a study of the comparative geometry of those sites with either high or low accident rates.

Accident Data Compilation

Following the identification of analysis segments, State accident records were interrogated to produce a three-year history of accident experience at each site. Selection of the accident analysis period required consideration of both statistical stability and consistency of geometric elements. On one hand, the statistical techniques which were applied have the intrinsic assumption that the dependent variable (accidents) is normally distributed. Satisfaction of this assumption required a study design that would produce the largest expected number of accidents considering other study constraints, so that the underlying Poisson distribution most closely approximated a normal distribution. On the other hand, roadway geometric conditions change over time due to reconstruction, new development, etc., and traffic volume and composition usually vary over a period of years. After careful consideration, a three-year period was selected to give the best tradeoff between maximum exposure and minimum exclusion of potential sites due to a study variable having changed during the study period.

Given analysis segment lengths of 0.6 mi (1 km) and an accident study period of three years, the number of accidents per site was estimated to vary largely between 2 and 10 accidents, with an expected number of about 4. A preliminary

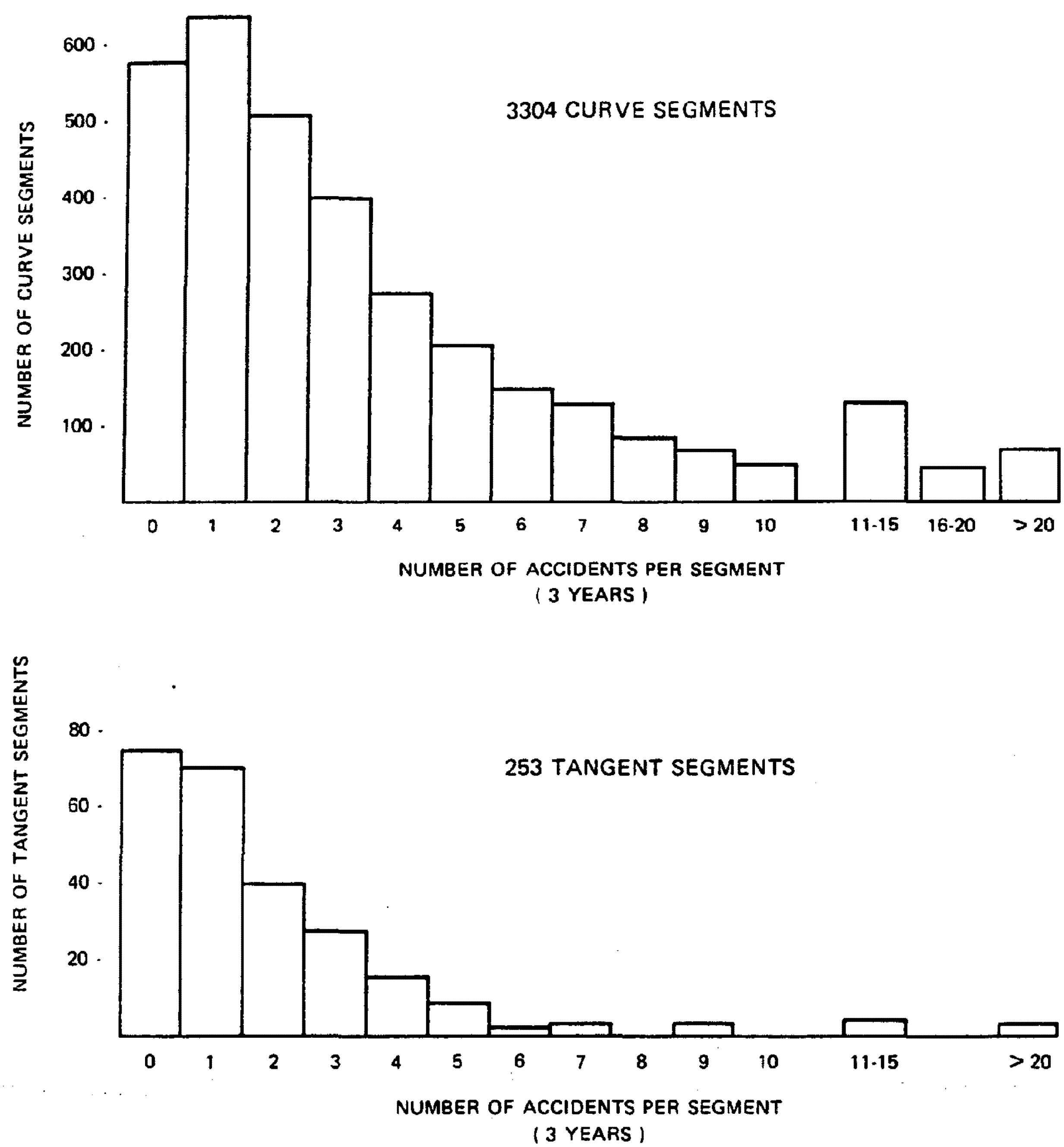


Figure 3. DISTRIBUTION OF ACCIDENT FREQUENCY FOR CURVE AND TANGENT ANALYSIS SEGMENTS

analysis of 77 curve sites in Florida and Illinois verified this assumption, and it was further verified in the comprehensive analysis, as is shown in Figure 3.

Accident histories obtained from each State were transformed to a common format for processing by FHWA computer personnel. Accident data included for each occurrence were as follows:

- Location
- Severity (Fatal, Injury Only, Property Damage Only)
- Vehicle type (Vehicles 1 and 2, if applicable)
- Accident type
- Surface condition
- Light condition
- Weather condition

Accident Characterization

There were 13,545 reported accidents on the designated analysis segments for the three-year analysis period. Mean accident rates were 3.93 accidents/segment on curve sections and 2.21 accidents/segment on tangent sections. Tabulation and analyses of this accident experience provide a valuable insight into the safety characteristics of rural highway curves.

Accident Type

Slightly more than one-half (54 percent) of accidents which occurred on the selected analysis segments involved only one vehicle. Of these, about two-thirds were single-vehicle run-off-road (ROR) accidents and the remainder were categorized "other" single-vehicle accidents (involving animals, objects in the road, pedestrians, etc.).

Single-vehicle vs. Multivehicle Accidents.--Traffic volume levels appear to affect the relative proportions of single-vehicle ROR and multivehicle accidents. As Figure 4 indicates, the proportion of single-vehicle ROR accidentson the lowest volume roads (less than 2099 ADT) was 42.5 percent. Multivehicle accidents were 33.6 percent of accidents on these roads. For roads with ADT of 10,000 or more, single-vehicle ROR accidents were only 14.9 percent of the total, while multivehicle were 76.9 percent.

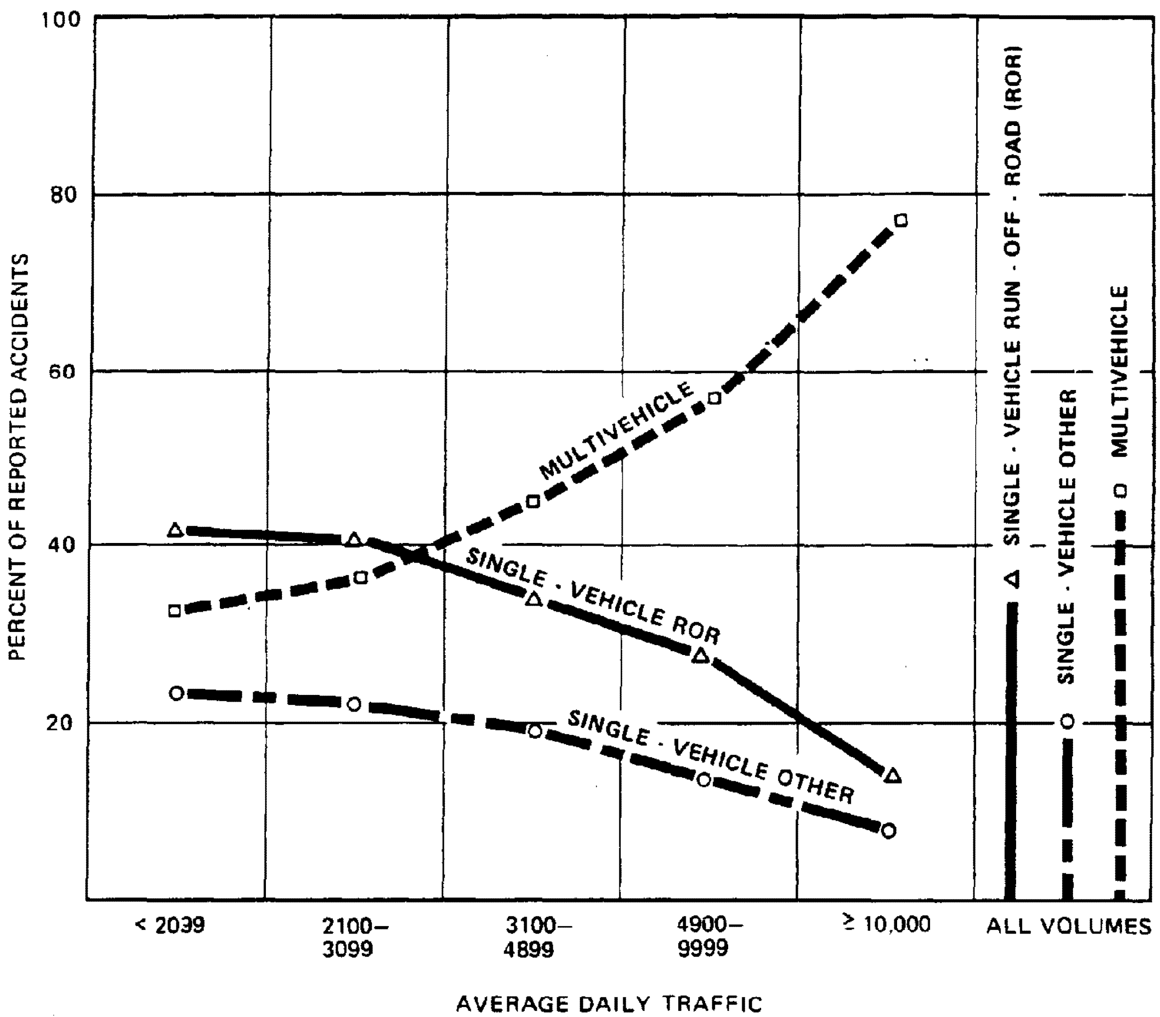


Figure 4. ACCIDENT TYPE VERSUS TRAFFIC VOLUME ON RURAL HIGHWAY CURVES

Multivehicle Accident Types.--Despite the careful exclusion of curve segments with major intersections, a significant number of angle and turning accidents were observed. Table 9 shows the number and rate of multivehicle accidents on curve segments. About 41 percent of multivehicle accidents were angle and turning types, indicating the significant presence of driveways and minor intersections. The other 59 percent included head-on, sideswipe and rear-end types. No apparent relationship was found between either degree of curve or roadway width and the incidence of head-on accidents on curves. All other categories of multivehicle accidents, however, exhibited rates which increased as curves became sharper and decreased with wider roads.

TABLE 9

TYPE AND NUMBER OF MULTIVEHICLE ACCIDENTS ON CURVE SEGMENTS

	Multivehicle Accidents			
Accident Type	Total Number	Per Analysis Segment	Percent of Total Multivehicle Accidents	Percent of Total Accidents
Head-on	909	0.28	15.3%	7.0%
Rear-end	1883	0.57	31.8%	14.6%
Sideswipe	681	0.21	11.5%	5.3%
Angle & Turning	2450	0.73	41.4%	19.0%
Total	5923	1.79	100.0%	45.9%

Note: Three-year accident experience at each analysis segment.

Surface Conditions

On curve analysis segments, 27.5 percent of all accidents occurred when the surface condition was wet or icy. On tangent analysis segments, 22.0 percent of all accidents occurred with wet or icy pavement conditions. Although exact exposure data are not available, average climatology information for the four study States (27) indicated that pavements are wet or icy about 10 to 12 percent of the time. The probability of an accident occurrence, therefore, is almost three times as high for wet or icy pavements than for dry pavements.

Types of accidents that occurred during periods of poor and normal surface conditions are summarized in Table 10. On curve analysis segments, proportionately more single-vehicle ROR accidents occurred when the surface was wet and icy than under normal surface conditions. Both total and single-vehicle accident rates in all volume groups increased significantly with degree of curve when the roadway surface was wet or icy. Accident rates during periods of poor roadway surface conditions also were generally lower as roadway width increased.

Light Conditions

Approximately 61 percent of all accidents on curve analysis segments occurred during the daytime, but more than one-half of single-vehicle ROR accidents took place at night. Table 11 shows that single-vehicle ROR accidents on curves constituted 46 percent of total nighttime accidents, but were only 28 percent of total daytime accidents.

A large majority of multivehicle accidents were daytime occurrences. This is expected since traffic volume is usually higher during the day than at night.

Severity

Accidents involving a personal injury or fatality were judged as "severe." A total of 5390 accidents (41.5 percent) on curve analysis segments during the three-year study period were severe. On tangent analysis segments, severe accidents accounted for 37.8 percent of the total. A summary of accident severity by type of accident is given in Table 12.

Single-vehicle ROR accidents on curve segments were more likely to be severe than multivehicle or other single-vehicle accidents. Regardless of roadway width or degree of curve, nearly one-half of all single-vehicle ROR accidents involved a personal injury or fatality. By contrast, only 41 percent of multivehicle accidents and 29 percent of other single-vehicle accidents on curves were severe.

TABLE 10

ACCIDENT EXPERIENCE RELATED TO SURFACE CONDITIONS

Type of Segment & Accident	Wet or Icy Surface Conditions		Normal Surface Conditions		Total	
	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment
Curves:						
Single-Vehicle Run-Off-Road	40.6%	0.44	33.4%	0.95	35.4%	1.39
Single-Vehicle--Other	15.8%	0.17	20.2%	0.58	19.0%	0.75
Multivehicle	43.6%	0.47	46.4%	1.32	45.6%	1.79
Total - Curves	100%	1.08	100%	2.85	100%	3.93
Percent by Surface Condition	(27.5%)		(72.5%)		(100.0%)	
Tangents:						
Single-Vehicle Run-Off-Road	43.1%	0.21	23.4%	0.40	27.8%	0.61
Single-Vehicle--Other	11.4%	0.06	24.5%	0.42	21.7%	0.48
Multivehicle	45.5%	0.22	52.1%	0.90	50.5%	1.12
Total - Tangents	100%	0.49	100%	1.72	100%	2.21
Percent by Surface Condition	(22.0%)		(78.0%)		(100.0%)	

Note: Three-year accident experience at each analysis segment.

TABLE 11

ACCIDENT EXPERIENCE RELATED TO LIGHT CONDITIONS

Type of Segment & Accident	Daytime		Nighttime		Total	
	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment
Curves:						
Single-Vehicle Run-Off-Road	28.4%	0.68	46.2%	0.71	35.4%	1.39
Single-Vehicle--Other	13.6%	0.32	27.4%	0.42	19.0%	0.75
Multivehicle	58.0%	1.39	26.4%	0.41	45.6%	1.79
Total - Curves	100%	2.39	100%	1.54	100%	3.93
Percent by Light Conditions	(60.7%)		(39.3%)		(100.0%)	
Tangents:						
Single-Vehicle Run-Off-Road	23.2%	0.32	35.4%	0.29	27.8%	0.61
Single-Vehicle--Other	12.9%	0.18	36.4%	0.30	21.7%	0.48
Multivehicle	63.9%	0.88	28.2%	0.24	50.5%	1.12
Total - Tangents	100%	1.38	100%	0.83	100%	2.21
Percent by Light Conditions	(62.5%)		(37.5%)	(100.0%)		

Note: Three-year accident experience at each site.

TABLE 12

SEVERITY OF ACCIDENTS

Type of Segment & Accident	Severe (Personal Injury-Fatality) Accidents: Percent of Accidents by Type	Severe (Personal Injury-Fatality) Accidents: Accidents/ Segment	Non-Severe (Property Damage Only) Accidents: Percent of Accidents by Type	Non-Severe (Property Damage Only) Accidents: Accidents/ Segment	Total: Percent of Accidents by Type	Total: Accidents/ Segment
Curves:						
Single-Vehicle Run-Off-Road	40.8%	0.67	31.5%	0.72	35.4%	1.39
Single-Vehicle--Other	13.6%	0.22	22.8%	0.53	19.0%	0.75
Multivehicle	45.6%	0.74	45.7%	1.05	45.6%	1.79
Total - Curves	100%	1.63	100%	2.30	100%	3.93
Percent by Severity	(41.5%)		(58.5%)		(100.0%)	
Tangents:						
Single-Vehicle Run-Off-Road	33.6%	0.28	24.2%	0.33	27.8%	0.61
Single-Vehicle--Other	10.9%	0.09	28.2%	0.39	21.7%	0.48
Multivehicle	55.5%	0.46	47.6%	0.65	50.5%	1.12
Total - Tangents	100%	0.83	100%	1.37	100%	2.21
Percent by Severity	(37.8%)		(62.2%)		(100.0%)	

Note: Three-year accident experience at each site.

No discernible relationships were found between accident severity and roadway width, degree of curve, or traffic volume for either single-vehicle or multivehicle accidents. The percentage of accidents which resulted in a personal injury or fatality remained nearly constant across the full range of roadway width and degree of curve.

Vehicle Type

Trucks or buses were involved in approximately 20 percent of both single-vehicle and multivehicle accidents on the curve analysis segments. Table 13 summarizes accidents by vehicle type.

Exposure data reflecting the number of trucks in the traffic stream at each analysis segment were not available. No firm conclusions could be drawn, therefore, about the relationships between incidence of truck/bus accidents and geometric elements. The available information however, indicates that there is little relationship between the rate of truck/bus accidents and either degree of curve or roadway width.

Summary of Accident Characteristics

Analyses of the number and types of accidents on curve and tangent analysis segments confirmed that curves are substantially more hazardous than tangents. The probability of accident occurrence was found to be about 75 percent greater on curve segments than on tangent segments. Also, the analysis strongly indicates a need to focus on single-vehicle ROR accidents on curves.

Single-vehicle ROR accidents accounted for about 35 percent of the total on curve analysis segments versus 27 percent on tangents. Furthermore, single-vehicle ROR accidents on curve segments were more likely to be severe than multivehicle or other single-vehicle accidents. Single-vehicle ROR accidents on curves were also proportionately greater than other types of accidents under poor environmental (wet/icy roadway) and light (nighttime) conditions.

TABLE 13

ACCIDENT EXPERIENCE RELATED TO VEHICLE TYPE

Type of Accident	Truck/Bus Involvement		Auto Only		Total	
	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment
Single-Vehicle Run-Off-Road	30.7%	0.27	36.9%	1.21	35.6%	1.48
Other Single-Vehicle and Multivehicle	69.3%	0.61	63.1%	2.06	64.4%	2.67
Total	100%	0.88	100%	3.27	100%	4.15
Percent by Vehicle Type	(21.2%)		(78.8%)		(100.0%)	

Note: No vehicle type data were obtained from Florida. Accident rates reported above, therefore exclude Florida data. Three-year accident experience at each site.

It should be recognized that the greater number of accidents on curve segments compared to tangent segments is an incremental difference directly associated with the presence of the curve. Considering that the curve itself is a relatively small proportion of the 0.6 mi (1 km) segment, this incremental difference is even greater. An average accident rate for the curve itself can be computed using curve segment and tangent segment rates shown in Tables 10, 11 and 12, and taking into account the average length of all curves in the data base. Assume that the accident effect of the curve is over the full length of curve plus some nominal transition length on each end. The following equation is then appropriate for computing the accident rate associated with an average curve:

$$R_c = \frac{R_s - R_t \left(\frac{L_t}{L_s}\right)}{\frac{L_c}{L_s}} \qquad [4.1]$$

where: R_c = Accidents per 0.6 mi (1 km) of curve plus transitions only

R_s = Accidents per 0.6 mi (1 km) of curve segments (from upper portion of Tables 10, 11 and 12)

R_t = Accidents per 0.6 mi (1 km) of pure tangent segments (from lower portion of Tables 10, 11 and 12)

L_c = Average length of highway curve plus two transitions

L_s = Length of analysis segment = 0.6 mi (1 km)

L_t = Average length of highway that is pure tangent = $L_s - L_c$

The average length of curve in the 3304-site data base is 0.17 mi (0.27 km). If a representative transition length is taken as 150 feet (46 m), or approximately 0.03 mi (0.05 km), then L_c is equal to 0.23 mi (0.47 km) and L_t is equal to 0.37 mi (0.63 km). Three-year average accident rates for alinement consisting of curve plus transitions are calculated as follows:

<u>R_c for Total Accidents</u>

[3.93 - 2.21 (0.37/0.60)] / (0.23/0.60) = 6.70

<u>R_c for Single-vehicle ROR Accidents</u>

[1.39 - 0.61 (0.37/0.60)] / (0.23/0.60) = 2.64

The above exercise demonstrates that, once the presence of tangent alinement is accounted for in the curve segment accident rates, the total accident rate for the average curve is (6.70/2.21) or over three times the average tangent rate. Similarly, the average single-vehicle ROR rate for curves is (2.64/0.61) or 4.3 times the average tangent rate.

Analysis of Covariance

The Analysis of Covariance (AOCV) was intended to investigate the incremental accident effects of highway traffic and geometric variables generally available from State data files. The site selection process yielded 3304 curve analysis segments with values for five basic variables -- ADT, degree of curve, length of curve, roadway width and shoulder width. The analysis, therefore, was limited to studying the incremental accident effects of these five basic variables. Although the literature review indicated the potential futility of this AOCV investigation, the integrated data base allowed the analysis to be performed with very little additional effort.

An analysis approach was needed to study the incremental accident effects of the five basic traffic and geometric variables in a framework which considers both the direct effects of each variable and all of the potential interactional effects between variables. AOCV can be regarded as an extension of standard multiple regression in that, in essence, a family of simple regression equations is determined. The individual members of the family exist in the different cells of the experimental design framework which are defined by the combinations of categorical variable levels.

Analysis Results

Preliminary analyses were conducted on the data base to determine the significance of the five basic variables in predicting accident rate. The procedure used was from the "Statistical Package for the Social Sciences" (SPSS)(28). An AOCV framework was established, using accident rate as the dependent variable, in which each independent variable was individually treated as a covariate while the other independent variables were treated as factors. This analysis indicated that all variables except ADT had a significant relationship with accident rate.

Subsequent analyses were conducted using the framework shown in Table 14. The results of this analysis were as follows:

(1) The multiple R^2 was about 0.19 (the AOCV framework explained 19 percent of the variance) for all matrices with total accident rate as the dependent variable, and was much lower for all other dependent variables.

(2) State, degree of curve, and their two-way interactions with other variables accounted for most of the explained variance.

(3) The raw regression coefficients for each of the covariates were:

Covariate	Regression Coefficient
Degree of Curve	0.056
Length of Curve (mi)	-0.141
Roadway Width (ft)	-0.023
Shoulder Width (ft)	-0.057

1 mi = 1.609 km

1 ft = 0.305 m

Although the AOCV did not indicate any strong relationships, the regression coefficients shown above are the best overall estimates of the incremental effects of each covariate. These coefficients indicate that accident rate increases as degree of curve increases and decreases as length of curve, roadway width, and shoulder width increase. These all appear to be logical relationships with the exception of length of curve. But, in reality, curve length is usually associated with degree of curve. Also, as shown later, the coefficient for length of curve accounts for almost no change in predicted accident experience over the practical range of curve lengths.

In attempting to be more explicit about the effects of degree of curve, cell regressions of degree of curve vs. accident rate were run for the 32 cell covariance matrix. This additional exercise was not fruitful in showing any logical trend in the relationship between degree of curve and accident rate over ranges and combinations of the other variables.

TABLE 14

FRAMEWORK FOR ANALYSIS OF COVARIANCE

Covariances	
Degree of Curve	
Length of Curve (mi)	
Roadway Width (ft)	
Shoulder Width (ft)	
Factors	
State	1, 2, 3, 4
Degree	$\leq$ 1.999, 2.000-3.999. 4.000-6.999, $\geq$ 7.000
Length of Curve (mi)	$\leq$ 0.149, $\geq$ 0.150
Roadway Width (ft)	$\leq$ 21.999, $\geq$ 22.000
Shoulder Width (ft)	$\leq$ 5.999, $\geq$ 6.000
Dependent Variables (Accidents/MVM)	
Total Accident Rate	
Single-Vehicle Accident Rate	
Multivehicle Accident Rate	
Night Accident Rate	
Fatal plus Injury Accident Rate	

1 mi = 1.609 km

1 ft = 0.305 m

Findings

Since no logical trends could be derived from the individual regression of the cells in the analysis of covariance matrix, the overall regression coefficients derived in the second step of the analysis were considered the best available predictors of the incremental accident effects of each covariate. In looking at the sensitivity of accident rates, it is informative to determine the predicted incremental differences over the practical range of each covariate as follows:

Covariate	Practical Range	Regression Coefficient	Difference in Accident Rate* Accidents per MVM
Degree of Curve	1°-20°	0.056	1.12
Length of Curve(mi)	0.05-0.40	-0.141	0.05
Roadway Width (ft)	18-24	-0.023	0.14
Shoulder Width (ft)	0-8	-0.057	0.46

* Difference in accident rate is regression coefficient multiplied by the difference in the practical range for the respective covariate.

1 mi = 1.609 km

1 ft = 0.305 m

From this exercise, degree of curve and shoulder width appear to have sizable effects on accident rate over the their practical ranges of usage. The effects of the other two covariates, however, appear to be relatively small. These effects, of course, are subject to the reservations previously stated concerning interactions between variables and the dubious validity of using regression coefficients as predictors for incremental effects of individual variables.

Analysis of High- and Low-Accident Sites

The difficulty of establishing direct links between accident occurrence and the dimensions of geometric features is demonstrated by the limited success of the AOCV described above and by previous research. The problems encountered by researchers and discussed in the review of literature are basically four-fold:

(1) Large data base requirements;
(2) Inadequate geometric and/or accident data;
(3) Extreme complexity of accident causation;
(4) Exclusion of important variables.

For these reasons, an analysis procedure was developed which maximized the potential for learning something about geometric/accident relationships. The analysis procedure involved a detailed study of the geometric characteristics of two distinct curve site populations. The populations were defined as accident outliers, i.e., the curve sites were selected on the basis of either a very high accident rate, or a very low rate. Differences in the geometric characteristics of these high- and low-accident site populations were then investigated.

The obvious advantage of such an approach is that, assuming the data are carefully collected, it insures discovering any safety/geometry relationships that may exist. This is because the study sites are selected on the basis of dissimilarities in their accident experience, rather than differences in geometric or other features which are only hypothesized as being related to accidents.

Site Selection

Proper definition of a large enough sample of true accident outliers required a fairly large data base. Proper evaluation of this data base necessitated consideration of factors such as variable accident reporting levels among the States, and possible accident effects of extraneous variables.

The sites obtained from the four States enabled identification of accident outliers with a high degree of confidence. The following procedures were followed in identification of these outliers:

(1) Accident rates for each site within the large (3304 curve site) data base were computed.

(2) The sites were partitioned into three ADT classes to control for any accident rate or accident type effects of traffic volume. The ranges of the ADT classes were based on two concerns. First, for efficiency in evaluating the data, it is desirable to have equal numbers of sites within each ADT class, second, it was necessary to limit the range of each ADT class to ensure similar traffic volume effects for all sites within a class. The following ADT classes were established:

ADT Class	Volume Range	Number of Sites
Low	1400-2099	1059
Medium	2100-3099	1034
High	3100-4899	745

(Note: There were 459 curve sites with ADT greater than 4899. These sites were not included in this phase of the research.)

(3) To control for differences in reporting levels among the States, high-accident site thresholds were computed separately for each State. Specifically, mean accident rates were computed for each of the three ADT classes in each of the four States. Sites with accident rates at least twice the mean rate for the appropriate State and ADT class were designated "high-accident sites."

(4) Low-accident sites were also identified. In all but the high ADT range, such sites had no reported accidents over three years.

Budget and time considerations limited the number of sites that could be studied further. There were generally many more low-accident sites available than could be studied. At this stage, it was hypothesized that, in general, greater variability in geometric and environmental conditions would be found at the high-accident sites. Hence, it was believed desirable to study more high-accident than low-accident sites. Table 15 shows the distribution of high- and low-accident sites by State that were selected for further, detailed study. These were selected randomly from the available samples of high- and low-accident sites.

TABLE 15

DISTRIBUTION OF HIGH- AND LOW-ACCIDENT SITES BY STATE

State	Number of High-Accident Sites	Number of Low-Accident Sites	Total
Florida	50	56	106
Illinois	44	31	75
Ohio	26	16	42
Texas	65	45	110
Total	185	148	333

Field Data Collection

Following identification of the high- and low-accident sites, field studies were performed. These studies, conducted in the fall of 1980, were designed to further define the geometric and environmental character of the sites. In addition to verifying the State geometry files, field crews observed and measured a number of important geometric and environmental elements at each curve site.

The field studies were performed with two-person crews equipped with special survey forms and instruments for measuring various geometric elements. Appendix B contains a description of field procedures followed by the crews, and depicts a sample set of survey forms. An important aspect of the survey were photographs taken by the crew of the approaches and roadsides.

The following information was obtained for each site in the data base:

Roadway Geometry

Degree of curve

Roadway width (on tangent and in curve)

Shoulder width (on tangent and in curve)

Superelevation in curve

Superelevation transition length

Superelevation distribution

Sight distance to the curve

Roadway Environment

Characteristic of horizontal alinement on approach to the curve

Characteristic of vertical alinement on approach to the curve

Relative hazard of roadside (i.e., slopes, objects, etc.)

Pavement condition

Pavement skid resistance

Signing

Pavement markings

Presence of driveways, structures, minor roads, etc.

Roadway geometry was described in terms of measured values. Environmental data were based on judgment of the survey crew, and tended to be categorical in nature. The approach conditions to the curve (in terms of both horizontal and vertical alinement) were categorized according to various levels of alinement, from "primarily tangent with flat grades" to "predominantly curvilinear and/or hilly." Two elements were described in terms of rating schemes. These were pavement friction and roadside condition.

Field crews were trained to judge the amount of pavement friction in the curve as a function of surface roughness and depth of asperities. A set of tables was developed (see Appendix B) and guidelines provided to assist the field crews. Crew members inspected the pavement in the curve and arrived at a consensus pavement rating. This rating was intended to approximate skid number at 60 mph, SN_{60} (97 km/h, SN_{97}).

Roadside hazard ratings were assigned to each of the sites by inspection of the pictures taken in the field. The roadside ratings were based on Glennon's model for roadside accidents (29), which considers frequency, type, and placement of roadside objects. Appendix C describes the derivation of the roadside rating scheme used in the research.

Characterization of Sites

Mean values of some important geometric characteristics of the selected high- and low-accident sites are given in Table 16. These data illustrate differences among sites by State as well as variations among high- and low-accident locations in each State.

Typically, high-accident locations were sharper curves with narrower shoulders, poorer roadsides, and lower pavement ratings than the low-accident sites. On the average, curve length, roadway width, superelevation rate, and transition characteristics were not materially different.

As indicated in Table 16, there were some significant differences among States in average geometric conditions. The sites selected in Ohio had generally sharper and shorter curves (of course, Ohio curve data were limited to 3° or sharper curves). Ohio sites also exhibited the worst roadside ratings and worst pavement ratings. High- and low-accident sites in Florida had the mildest and longest curves. High-accident sites in Florida also exhibited the best roadside ratings. Differences in shoulder width between high- and low-accident sites were greatest in Illinois.

Pavement marking and signing information were also collected and evaluated. In general, all sites were well marked with edge lines and center stripes. This was undoubtedly because the highways studied were State primary highways with moderate to high traffic volumes. Signing was more variable; however, the sharpest curves tended to be advance signed, and delineated with one or more types of reflectorized delineator.

TABLE 16

GEOMETRIC CHARACTERISTICS OF
HIGH- AND LOW-ACCIDENT SITES

Geometric Characteristic	Mean Values for High- and Low-Accident Sites									
	Florida		Illinois		Ohio		Texas		Four States	
	High	Low	High	Low	High	Low	High	Low	High	Low
Degree of Curve	2.41	1.44	3.21	1.73	11.77	6.50	2.68	1.32	4.13	2.07
Length of Curve (mi)	0.25	0.21	0.17	0.15	0.06	0.08	0.18	0.16	0.18	0.17
Roadway Width in Curve (Ft)	22.5	22.7	24.3	23.7	22.4	22.2	24.1	24.7	23.5	23.4
Shoulder Width in curve (Ft)	7.5	8.9	5.8	8.6	5.9	8.2	7.6	8.4	6.9	8.6
Ratio of Super. at the PC to Max. Super.	0.51	0.64	0.43	0.27	0.40	0.40	0.37	0.35	0.42	0.44
Rate of Change of Super (per 100 ft)	0.019	0.035	0.022	0.011	0.043	0.025	0.017	0.014	0.023	0.023
Roadside Rating	29.0	26.3	32.8	30.1	34.1	30.1	32.0	26.8	31.7	27.6
Pavement Rating	36.1	40.7	36.0	36.4	32.4	35.1	37.0	38.9	35.9	38.7

1 mi = 1.609 km
1 ft = 0.305 m

Accident Characteristics of High-Accident Sites

A total of 1,558 accidents were reported during the three-year analysis period at the 185 high-accident curve analysis segments, for an average of 8.42 accidents per segment. This is 2.25 times greater than the average for all other curve analysis segments in the data base. Single-vehicle ROR accidents accounted for 42 percent of all occurrences at the high-accident sites versus 35 percent at all other curve sites.

Tables 17, 18 and 19 compare accident experience related to surface and light conditions, and severity of accidents at high-accident sites and all other curve segments. High-accident sites had similar proportions of accidents at night and on wet or icy pavements as all other sites. The percentage of accidents with an injury or fatality was nearly the same for high-accident sites as for all others.

Discriminant Analysis

The formal analysis of the high- and low-accident sites used a statistical technique known as Discriminant Analysis. This procedure is useful for situations in which the researcher desires to statistically distinguish between two or more groups or populations. To do so, data describing the characteristics on which the groups are expected to differ are collected and analyzed. In this case, the defined populations are (1) highway curve segments with significantly high accident experience; and (2) highway curve segments with significantly low accident experience. The characteristics (or "discriminating variables") are the geometric and environmental variables discussed earlier.

Discriminant analysis distinguishes between the populations being studied by forming a linear combination of the discriminating variables. The discriminant function is of the form

$$D = d_1Z_1 + d_2Z_2 + \ldots + d_pZ_p \qquad [4.2]$$

TABLE 17

ACCIDENT EXPERIENCE RELATED
TO SURFACE CONDITIONS AT
HIGH-ACCIDENT AND ALL OTHER SITES

Type of Segment & Accident	Wet or Icy Surface Conditions		Normal Surface Conditions		Total	
	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment
High-Accident Curve Segments:						
Single-Vehicle Run-Off-Road	46.2%	0.96	40.8%	2.58	42.2%	3.55
Single-Vehicle--Other	14.4%	0.30	17.2%	1.09	16.5%	1.39
Multivehicle	39.4%	0.82	42.0%	2.66	41.3%	3.48
Total - High-Accident Sites	100%	2.08	100%	6.33	100%	8.42
Percent by Surface Conditions	(24.8%)		(75.2%)		(100.0%)	
All Other Curve Segments:						
Single-Vehicle Run-Off-Road	39.9%	0.41	32.4%	0.86	34.6%	1.27
Single-Vehicle--Other	16.0%	0.16	20.6%	0.55	19.4%	0.71
Multivehicle	44.1%	0.45	47.0%	1.24	46.0%	1.69
Total- Other Sites	100%	1.02	100%	2.65	100%	3.67
Percent by Surface Conditions	(27.8%)		(72.2%)		(100.0%)	

Note: Three-year accident experience at each analysis segment.

TABLE 18

ACCIDENT EXPERIENCE RELATED
TO LIGHT CONDITIONS AT
HIGH-ACCIDENT AND ALL OTHER SITES

Type of Segment & Accident	Daytime		Nighttime		Total	
	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment
High-Accident Curve Segments:						
Single-Vehicle Run-Off-Road	32.6%	1.64	56.2%	1.91	42.2%	3.55
Single-Vehicle--Other	13.4%	0.67	21.2%	0.72	16.5%	1.39
Multivehicle	54.0%	2.71	22.6%	0.77	41.3%	3.48
Total - High-Accident Sites	100%	5.02	100%	3.40	100%	8.42
Percent by Light Conditions	(59.6%)		(40.4%)		(100.0%)	
All Other Curve Segments:						
Single-Vehicle Run-Off-Road	27.8%	0.62	44.8%	0.65	34.6%	1.27
Single-Vehicle--Other	13.6%	0.30	28.3%	0.41	19.4%	0.71
Multivehicle	58.6%	1.31	26.9%	0.38	46.0%	1.69
Total- Other Sites	100%	2.23	100%	1.44	100%	3.67
Percent by Light Conditions	(60.9%)		(39.1%)		(100.0%)	

Note: Three-year accident experience at each analysis segment.

TABLE 19

SEVERITY OF ACCIDENTS AT HIGH-ACCIDENT AND ALL OTHER SITES

Type of Segment & Accident	Severe (Personal Injury Fatality) Accidents		Non-Severe (Property Damage Only) Accidents		Total	
	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment	Percent of Accidents by Type	Accidents/ Segment
High-Accident Curve Segments:						
Single-Vehicle Run-Off-Road	46.8%	1.65	38.8%	1.90	42.2%	3.55
Single-Vehicle--Other	10.0%	0.35	21.2%	1.04	16.5%	1.39
Multivehicle	43.2%	1.52	40.0%	1.96	41.3%	3.48
Total - High-Accident Sites	100%	3.52	100%	4.90	100%	8.42
Percent by Severity	(41.8%)		(58.2%)		(100.0%)	
All Other Curve Segments:						
Single-Vehicle Run-Off-Road	40.0%	0.61	30.5%	0.66	34.6%	1.27
Single-Vehicle--Other	14.1%	0.21	23.1%	0.49	19.4%	0.71
Multivehicle	45.9%	0.70	46.4%	1.00	46.0%	1.69
Total- Other Sites	100%	1.52	100%	2.15	100%	3.67
Percent by Severity	(41.7%)		(58.3%)		(100.0%)	

Note: Three-year accident experience at each analysis segment.

where D is the score on the discriminating function, the d's are weighting coefficients and the Z's are the standardized values of the discriminating variables. In concept, the D values for each case within a population will be similar and will be significantly different from the D values of the other population(s).

The analysis aspect of this technique provides several tools for the interpretation of data. Among these are statistical tests for measuring the success with which the variable, when combined in the discriminant function, actually discriminates between groups.

The Statistical Package for the Social Sciences (SPSS) (28) program was used for the discriminant analysis. The program utilizes a stepwise procedure which first selects the variable with the highest discriminating power. It then chooses the variable with the next highest discriminating power, given the effects of the first variable. This process is continued until all variables are selected or the remaining variables are no longer able to contribute to further discrimination.

In exercising discriminant analysis, the measurements corresponding to each group (high- and low-accident sites) are assumed to be multivariate normal with a common covariance matrix but different mean vectors. In application, however, these ideal assumptions are never completely satisfied. Covariance and mean vectors are not known in advance, but have to be estimated from the observational material. Distributions are not exactly normal and covariances for different groups are not exactly equal. Nevertheless, the SPSS Manual indicates that the method is fairly robust even with substantial violations of the basic assumptions.

Twelve variables were selected for inclusion in the discriminant analysis as follows:

(1) Degree of Curve
(2) Length of Curve
(3) Maximum Superelevation
(4) Roadway Width
(5) Shoulder Width
(6) Shoulder Type
(7) Roadside Hazard Rating
(8) Pavement Skid Resistance Rating
(9) Rate of Change of Superelevation
(10) Ratio of Superelevation at the P.C. (point of curvature) to maximum superelevation (RATIO)
(11) Advance Sight Distance (composite for both directions)
(12) Approach Alinement (composite for both directions)

Signing and pavement marking data were collected and analyzed. Inspection of the data and consideration of the night accident characteristics of high-accident sites revealed that such data would not affect the analysis.

The initial run of the discriminant analysis using the above twelve variables developed a discriminant function in which seven of the variables were significant. One indication of the relative importance of the variables is given by the standardized coefficients associated with each variable. Standardizing the coefficients of the analysis removes differences in the variables associated with the units by which they were measured and analyzed. The SPSS output provides these standardized coefficients. The seven variables and their relative discriminating power in the discriminant function were as follows:

Variable	Standardized Coefficients	Relative Discriminating Power*
Roadside Rating	0.592	3.93
Shoulder Width	0.425	2.87
Degree of Curve	0.363	2.40
Length of Curve	0.347	2.27
Pavement Rating	-0.222	1.47
Shoulder Type	0.162	1.07
RATIO	-0.150	1.00

* Taken as ratio of the variable's standardized coefficient to the smallest standardized coefficient (absolute value).

The discriminant function derived with these variables correctly classified 74.4 percent of the high-accident sites, 63.8 percent of the low accident sites, and 69.8 percent of all sites.

Several other combinations of variables were tried in the discriminant analysis in an attempt to find a more efficient discriminant function, i.e., one with fewer variables that was almost as good in classifying sites. As each combination was tried, different numbers of sites were included because missing records for a variable required that site's exclusion from the analysis. The best derived discriminant equation was one that used 298 sites (the initial analysis had only 291) and the five most powerful discriminating variables in the initial analysis. The discriminant function, D, is:

$$D = 0.0713(DC) + 2.9609(LC) + 0.1074(RR) - 0.0352(PR) - 0.1450(SW) - 1.5454 \quad [4.3]$$

Where
D = Discriminant Function (non-dimensional)
DC = Degree of Curve
LC = Length of Curve (mi)
RR = Roadside Rating
PR = Pavement Rating
SW = Shoulder Width (ft)

NOTE: 1 mi = 1.609 km
1 ft = 0.305 m

Since a higher discriminant score means a higher likelihood that a site is a high-accident location, the variables contribute to that likelihood as expected. Greater degrees of curve, lengths of curve, and roadside hazard ratings all increase the discriminant score. Greater pavement skid ratings and shoulder widths decrease the discriminant score.

The relative discriminating power (which is based on standardized coefficients) of each of the five variables in Equation 4.3 is:

Variable	Standardized Coefficients	Relative Discriminating Power
Roadside Rating	0.594	2.11
Shoulder Width	-0.393	1.39
Length of Curve	0.393	1.39
Degree of Curve	0.325	1.14
Pavement Rating	-0.276	1.00

Equation 4.3 correctly classifies 75.9 percent of the high-accident sites, 60.2 percent of the low-accident sites, and 69.1 percent of all sites. Although the seven-variable equation is somewhat better in classifying all sites, Equation 4.3 is better at classifying the high-accident sites.

Equation 4.3 suggests that roadside hazard is the most important contributor to high-accident experience, and, therefore, at existing high-accident sites, roadside safety improvements may generally be the most effective countermeasures. Other practical and potentially cost-effective countermeasures would include shoulder widening and pavement resurfacing.

Although Equation 4.3 may be best for explaining causal relationships, it contains two variables (roadside hazard, pavement skid resistance) that are not specifically recorded in State inventory files. Therefore, some additional discriminant analysis runs were undertaken in an attempt to develop a relationship for readily identifying potential site improvement candidates. These relationships presuppose that the discriminating variables may be highly correlated with other variables not included and, therefore, may account for but not totally explain high-accident locations.

The best high-accident location identifier equation using generally available inventory data is as follows:

$$D = 0.3768(DC) + 3.2092(LC) - 0.2198(SW) + 0.2887 \qquad [4.4]$$

Where D = Discriminant Factor (non-dimensional)
DC = Degree of Curve
LC = Length of Curve (mi)
SW = Shoulder Width (ft)

NOTE: 1 mi = 1.609 km
1 ft = 0.305 m

This equation correctly classifies 75.9 percent of the high-accident sites, 53.3 percent of the low-accident sites, and 65.6 percent of all sites.

Application of Discriminant Analysis

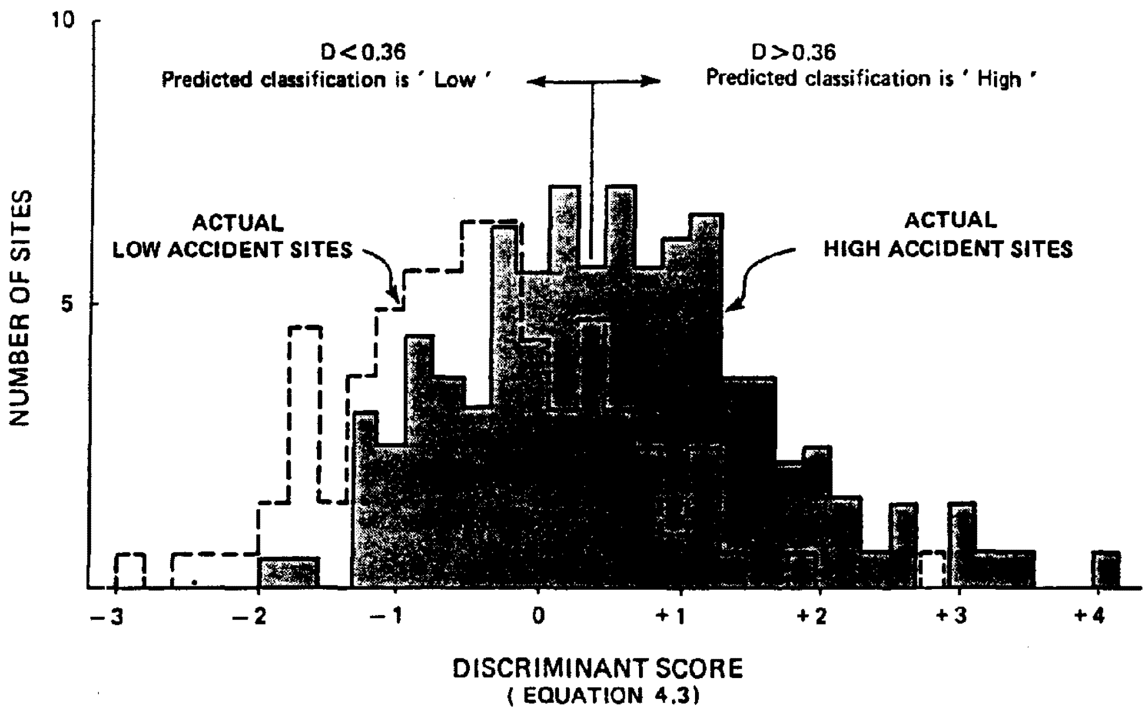
The value of the discriminant analysis procedure is primarily in the ability to predict or classify various combinations of the discriminating variables. In its application here, the analysis serves to identify combinations of geometric and other conditions that have a tendency to produce either very high or very low accident experience.

A measure of how well the discriminant function identifies differences between populations is given by the SPSS output. The procedure develops an optimal discriminant score relationship, which it then uses to compute D scores for each of the sites in the data base. The distributional characteristics (mean and variance) of the D scores for each of the populations are then used to test the accuracy or validity of the discriminant function. This is done by classifying each site according to its D score and the relative probability of belonging to one or the other distribution of D scores for the two populations. A site's predicted classification is then compared to its actual population membership.

To illustrate, consider Figure 5, which shows the histograms for D scores for the high- and low-accident sites as calculated by Equation 4.3. There is obviously considerable overlap between the two distributions, as evidenced by the small differences in mean D (-0.537 for low-accident sites and 0.404 for high-accident sites) and the large spread of both distributions. It appears that sites with D scores around zero could reasonably be placed in either classification. The actual classification is performed in the SPSS procedure by calculating probabilities that a D score belongs to one or the other class of sites, and selecting the highest probability. Thus, sites with a 50.1 percent probability of being high-accident sites (and, thus a 49.9 percent low-accident probability) are classified as high-accident sites.

Once this classification has been completed, the analysis compares actual group membership with predicted membership. Figure 5 reports this comparison for the Equation 4.3 analysis. The D score which represents a 50-50 probability is 0.36.

Evaluation of Figure 5 produces a series of conclusions. First, the discriminant equation predicts high-accident site membership better than low-accident site membership. Second, a significant number of low-accident sites are incorrectly identified as high-accident sites. Third, very few sites with high D scores (say, greater than 1.0) are actually low-accident sites. Further analysis of these conclusions is given below.



ACTUAL ACCIDENT CHARACTERISTICS	PREDICTED CLASSIFICATION		
	LOW Number Percent	HIGH Number Percent	TOTAL Number Percent
LOW	77 60.2 (65.2%)	51 39.8 (28.3%)	128 100.0 (43.0%)
HIGH	41 24.1 (34.8%)	129 75.9 (71.7%)	170 100.0 (57.0%)
TOTAL	118 39.6 (100.0%)	180 60.4 (100.0%)	298 100.0 (100.0%)

Figure 5. DISTRIBUTION OF DISCRIMINANT SCORES FOR HIGH- AND LOW-ACCIDENT SITES

Predictive Power for High-Accident Sites.--The discriminant function apparently predicts high-accident sites better than low-accident sites. This is a positive consideration, as one is generally more interested in identifying what is hazardous. The ultimate usefulness of the analysis will be in a demonstrated ability to identify hazardous combinations of geometry and other conditions, for which logical countermeasures can be developed. Prediction of low-accident site characteristics is useful only in that it provides a basis for comparison with high-accident sites.

Incorrectly Identified Low-Accident Sites.--As Figure 5 points out, the discriminant analysis predicted that 180 sites were high-accident sites. The probability-based criterion of $D \geq 0.36$ resulted in this prediction. Of the 180 predicted high-accident sites, 51 sites (28.3 percent) were actually low-accident sites. This suggests that the 50-50 probability criterion for selection of high-accident site characteristics may not sufficiently screen out those geometric/condition combinations of true interest. Put another way, the 28.3 percent of sites predicted incorrectly to be high-accident sites represents a measure of the error in the criterion for prediction. Inspection of the D distributions, which led to the third conclusion, suggests an alternative selection criterion.

High Probability Criterion for Identifying High-Accident Sites.--Selection of a higher probability level as a basis for characterizing high-accident sites results in reduced chance that sites would be incorrectly identified as being hazardous. The SPSS output enables selection of any probability criterion level. Figure 6 depicts the relationship between D and P(H) (the probability that a site is a high-accident site) for Equation 4.3. Selection of any P(H) level can be translated into a minimum D score (based on Equation 4.3) for analysis purposes. If an 80 percent criterion is adopted (which corresponds to a D score of 1.10), almost all sites selected from the data base actually would be high-accident sites. Consider Figure 7. A 50 percent criterion results in 71.7 percent of selected sites as being true high-accident sites. An 80 percent criterion, however, increases the percentage of correctly identified sites to 91.3 percent. This improved level of prediction is a direct measure of the confidence one might have in the use of the

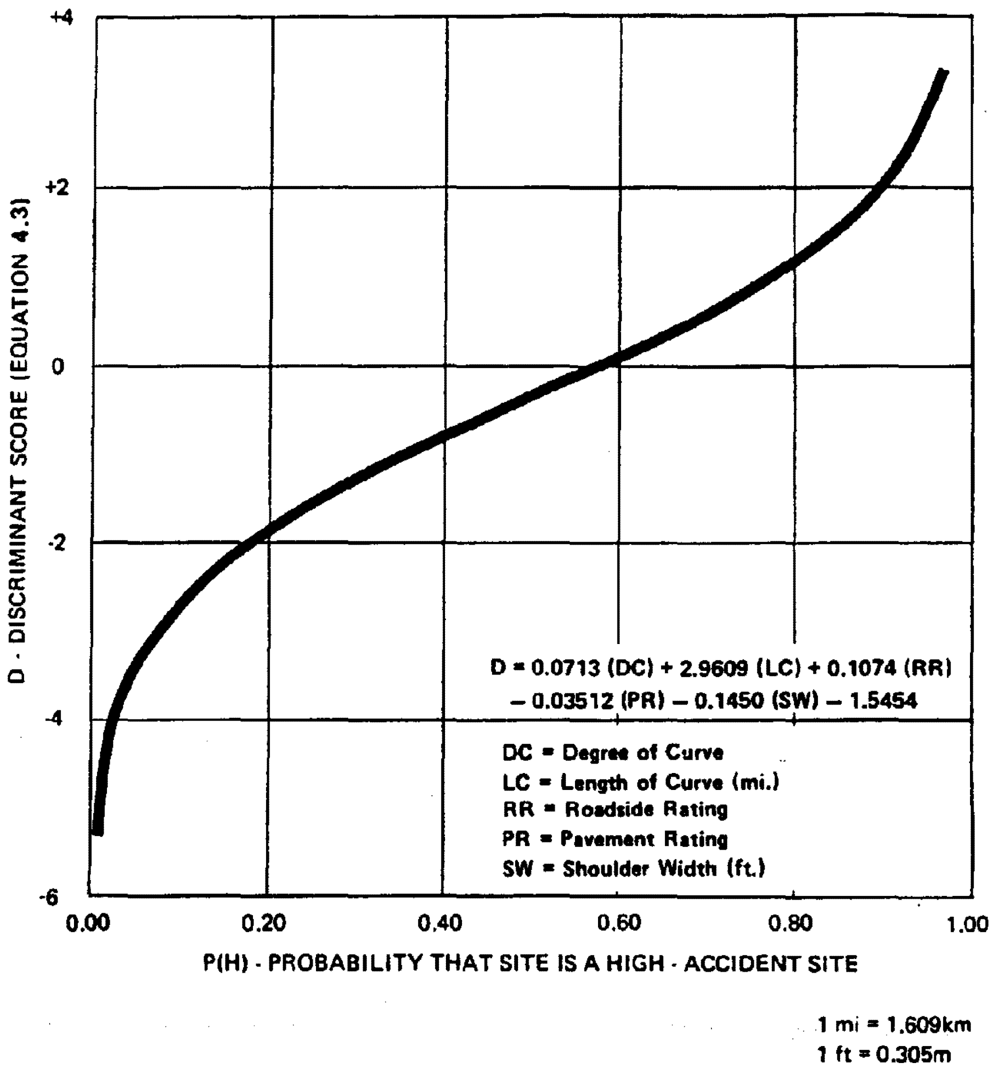
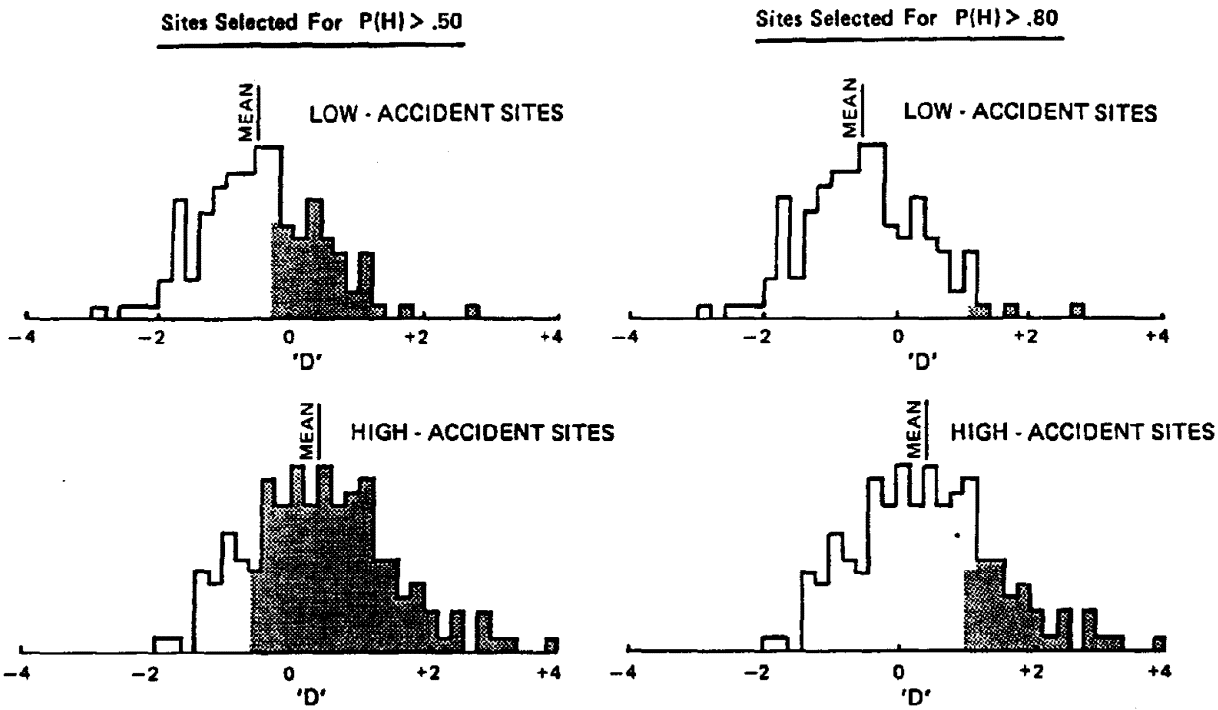


Figure 6. RELATIONSHIP BETWEEN DISCRIMINANT SCORE AND THE PROBABILITY THAT A SITE IS A HIGH-ACCIDENT SITE



Sites Selected For P(H) > .50

NUMBER OF SITES IDENTIFIED AS HIGH-ACCIDENT SITES	180
(PERCENT OF ALL SITES)	(60.4%)
NUMBER CORRECTLY CLASSIFIED	129
NUMBER INCORRECTLY CLASSIFIED	51
PERCENT OF IDENTIFIED SITES CORRECTLY CLASSIFIED	*71.7%*

Sites Selected For P(H) > .80

NUMBER OF SITES IDENTIFIED AS HIGH-ACCIDENT SITES	46
(PERCENT OF ALL SITES)	(15.4%)
NUMBER CORRECTLY CLASSIFIED	42
NUMBER INCORRECTLY CLASSIFIED	4
PERCENT OF IDENTIFIED SITES CORRECTLY CLASSIFIED	*91.3%*

Shaded area represents sites selected as high-accident sites according to appropriate P(H) criterion.

Figure 7. COMPARISON OF HIGH-ACCIDENT SITE SELECTION FOR VARIABLE P(H) CRITERION

relationships in combination with an 80 percent P(H) criterion for selection of sites for study.

Implications of High Probability Selection Criterion.--Selection of sites based on an 80 percent probability criterion has important implications. As Figure 7 indicates, the number of sites so identified is substantially reduced. With a 50 percent probability criterion, 180 sites (60.4 percent of the sample) were classified as high-accident sites. An 80 percent criterion results in only 46 sites (15.4 percent of the sample) as being high-accident sites worthy of further study. Consider also that the high- and low-accident data base represents a relatively small proportion of the total site population. Therefore, an 80 percent criterion will produce a smaller percentage of total sites for further study (say, 5 percent or less).

Table 20 shows the probabilities that sites defined by a range of geometric conditions would be high-accident sites. With the 80 percent criterion, it appears that almost all sites with high roadside hazard would qualify as high-accident sites. Likewise, almost all sites with low roadside hazard would not qualify.

With moderate roadside hazard, the result is more mixed. Generally, the moderate roadside hazard must be combined with either very sharp curvature or a combination of two or more other variables that are moderate or worse.

In summary, hazardous roadside design appears to be the greatest contributor to high-accident experience at highway curves. Other less prominent contributors are sharp curvature, narrow shoulders, low pavement skid resistance, and long curves.

Application of Results

For application at existing sites, Equation 4.3 and cost considerations indicate that improving roadside design, pavement skid resistance, and shoulder width may be the better accident countermeasures. Reducing curvature may not be practical or productive because of high costs and the apparent trade-off between degree and length of curve for a given central angle. This discussion is not to

TABLE 20

PROBABILITY THAT A SITE IS A HIGH-ACCIDENT SITE
(EQUATION 4.3)

L O W R O A D S I D E H A Z A R D
(Roadside Rating of 20)

Curve Length (mi)	Shoulder Width (ft)	Degree of Curve 1°	3°	6°	12°	20°
High Pavement Skid Resistance (Pavement Rating of 50)						
Long (0.30 mi)	0	50	53	58	-	-
	4	37	39	45	-	-
	8	22	24	27	-	-
Moderate (.17 mi)	0	42	45	50	-	-
	4	30	32	37	-	-
	8	18	20	23	-	-
Short (.05 mi)	0	34	37	42	52	64
	4	23	25	30	38	52
	8	14	16	19	26	38
Moderate Pavement Skid Resistance (Pavement Rating of 35)						
Long (0.30 mi)	0	61	65	68	-	-
	4	49	52	56	-	-
	8	36	38	42	-	-
Moderate (.17 mi)	0	54	58	61	-	-
	4	41	44	48	-	-
	8	29	31	35	-	-
Short (.05 mi)	0	46	50	54	64	75
	4	33	36	40	52	63
	8	22	24	28	39	50
Low Skid Resistance (Pavement Rating of 20)						
Long (0.30 mi)	0	75	77	80	-	-
	4	63	66	70	-	-
	8	50	53	60	-	-
Moderate (.17 mi)	0	68	71	75	-	-
	4	56	59	63	-	-
	8	42	45	52	-	-
Short (.05 mi)	0	61	64	68	77	85
	4	48	51	56	65	77
	8	35	38	44	53	65

Shaded area denotes curve conditions which result in at least an 80 percent probability that the site would be a "high-accident" site.

1 mi = 1.609 km
1 ft = 0.305 m

TABLE 20

PROBABILITY THAT A SITE IS A HIGH-ACCIDENT SITE (Continued)
(EQUATION 4.3)

M O D E R A T E R O A D S I D E H A Z A R D
(Roadside Rating of 35)

Curve Length (mi)	Shoulder Width (ft)	Degree of Curve 1°	3°	6°	12°	20°
High Pavement Skid Resistance (Pavement Rating of 50)						
Long (0.30 mi)	0	[illegible]	[illegible]	[illegible]	-	-
	4	76	77	[illegible]	-	-
	8	66	67	70	-	-
Moderate (.17 mi)	0	[illegible]	[illegible]	[illegible]	-	-
	4	69	71	74	-	-
	8	58	60	62	-	-
Short (.05 mi)	0	74	76	78	[illegible]	[illegible]
	4	62	64	66	77	[illegible]
	8	50	52	54	65	79
Moderate Pavement Skid Resistance (Pavement Rating of 35)						
Long (0.30 mi)	0	[illegible]	[illegible]	[illegible]	-	-
	4	[illegible]	[illegible]	[illegible]	-	-
	8	73	79	[illegible]	-	-
Moderate (.17 mi)	0	[illegible]	[illegible]	[illegible]	-	-
	4	78	[illegible]	[illegible]	-	-
	8	66	72	75	-	-
Short (.05 mi)	0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	4	71	76	79	[illegible]	[illegible]
	8	59	65	68	74	[illegible]
Low Skid Resistance (Pavement Rating of 20)						
Long (0.30 mi)	0	[illegible]	[illegible]	[illegible]	-	-
	4	[illegible]	[illegible]	[illegible]	-	-
	8	[illegible]	[illegible]	[illegible]	-	-
Moderate (.17 mi)	0	[illegible]	[illegible]	[illegible]	-	-
	4	[illegible]	[illegible]	[illegible]	-	-
	8	77	78	[illegible]	-	-
Short (.05 mi)	0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	8	70	71	79	[illegible]	[illegible]

Shaded area denotes curve conditions which result in at least an 80 percent probability that the site would be a "high-accident" site.

1 mi = 1.609 km
1 ft = 0.305 m

TABLE 20

PROBABILITY THAT A SITE IS A HIGH-ACCIDENT SITE (Continued)
(EQUATION 4.3)

HIGH ROADSIDE HAZARD
(Roadside Rating of 50)

Curve Length (mi)	Shoulder Width (ft)	Degree of Curve 1°	3°	6°	12°	20°
High Pavement Skid Resistance (Pavement Rating of 50)						
Long (0.30 mi)	0	[redacted]	[redacted]	[redacted]	-	-
	4	[redacted]	[redacted]	[redacted]	-	-
	8	[redacted]	[redacted]	[redacted]	-	-
Moderate (.17 mi)	0	[redacted]	[redacted]	[redacted]	-	-
	4	[redacted]	[redacted]	[redacted]	-	-
	8	[redacted]	[redacted]	[redacted]	-	-
Short (.05 mi)	0	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
	4	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
	8	79	[redacted]	[redacted]	[redacted]	[redacted]
Moderate Pavement Skid Resistance (Pavement Rating of 35)						
Long (0.30 mi)	0	[redacted]	[redacted]	[redacted]	-	-
	4	[redacted]	[redacted]	[redacted]	-	-
	8	[redacted]	[redacted]	[redacted]	-	-
Moderate (.17 mi)	0	[redacted]	[redacted]	[redacted]	-	-
	4	[redacted]	[redacted]	[redacted]	-	-
	8	[redacted]	[redacted]	[redacted]	-	-
Short (.05 mi)	0	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
	4	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
	8	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
Low Skid Resistance (Pavement Rating of 20)						
Long (0.30 mi)	0	[redacted]	[redacted]	[redacted]	-	-
	4	[redacted]	[redacted]	[redacted]	-	-
	8	[redacted]	[redacted]	[redacted]	-	-
Moderate (.17 mi)	0	[redacted]	[redacted]	[redacted]	-	-
	4	[redacted]	[redacted]	[redacted]	-	-
	8	[redacted]	[redacted]	[redacted]	-	-
Short (.05 mi)	0	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
	4	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
	8	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]

Shaded area denotes curve conditions which result in at least an 80 percent probability that the site would be a "high-accident" site.

1 mi = 1.609 km
1 ft = 0.305 m

suggest that other design deficiencies such as very poor approach sight distance, extremely narrow lanes, extremely poor transitions, extreme shoulder slope breaks, etc. might not be considered in an improvement program. Regardless, the discriminant analysis does give guidance about the effects of roadside, pavement surfaces, and shoulders.

Equation 4.4 is useful when applied to the problem of identifying candidate sites from a large data base. A suggested application of the discriminant analysis results is to use Equation 4.4 with a 70 percent criterion to identify candidate sites for improvement and, after inspection of these candidate sites, to use Equation 4.3 with an 80 percent criterion to decide if each site should remain on the list. Although Equation 4.4 will miss identifying those sites where all the variables but the roadside hazard and/or pavement skid resistance are good, under the limitations of typical State inventory data, it will be reasonably efficient.

Table 21 shows the calculation of the probability of a site having a high-accident experience based on Equation 4.4, which is the discriminant relationship that uses available inventory data to identify candidate improvement sites. Because this equation does not include the more powerful roadside hazard rating, and because the identification process is just a first step toward deciding which site to improve, the lower criterion level of 70 percent is appropriate. If this level is used, Table 21 indicates that sites with curvature of 6° or greater or with very narrow shoulders would be candidates. Also, sites with 4-foot shoulders and 3° curves, or sites with 4-foot shoulders and long curves would qualify as candidates.

After an improvement site is selected, Equation 4.3 can also be used as a general guide to evaluate the ability to change the high-accident experience. Again, an appropriate criterion level could be selected. For example, the goal of any improvement could be to reduce the probability to only 40 percent that the site would remain as a high-accident location after improvement. The parameter, "reduction in probability of being a high-accident site," however, must be interpreted only as a general guideline and not as an absolute measure.

TABLE 21

PROBABILITY THAT A HIGHWAY CURVE SITE IS A HIGH-ACCIDENT LOCATION

(Equation 4.4)

Curve Length (mi)	Shoulder Width (ft)	Degree of Curve 1°	3°	6°	12°	20°
Long (0.30 mi)	0	[illegible]	[illegible]	[illegible]	-	-
	4	[illegible]	[illegible]	[illegible]	-	-
	8	55	68	[illegible]	-	-
Moderate (.17 mi)	0	[illegible]	[illegible]	[illegible]	-	-
	4	63	[illegible]	[illegible]	-	-
	8	44	61	[illegible]	-	-
Short (.05 mi)	0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	4	58	[illegible]	[illegible]	[illegible]	[illegible]
	8	38	55	[illegible]	[illegible]	[illegible]

Shaded area denotes curve conditions which result in at least a 70 percent probability that the site would be a "high-accident" site.

1 mi = 1.609 km
1 ft = 0.305 m

Summary of Accident Studies

Characterization and multivariate analysis of accidents on rural highway curves revealed a number of important findings:

(1) Average accident rates on curves are three times the average rate for tangents.

(2) Single-vehicle run-off-road accidents are the most predominant type of accident on curves.

(3) The frequency of accidents on wet or icy highway curves is almost three times the frequency of accidents on dry pavements.

(4) The character of the roadside, degree and length of curve, shoulder width, and pavement skid resistance were all found to be related to the propensity for curves to experience high accident rates.

The types and relative success of the analyses performed lead to important conclusions regarding accidents on rural highway curves:

(1) Discriminant Analysis is a useful tool for classifying sites according to their potential as high-accident locations.

(2) The value of geometric data bases maintained by State highway departments and used for accident studies can be enhanced. Specifically, collection and maintenance of data on roadside character, including roadside slopes and clear-zone width, would improve the utility of such data bases.

(3) The number of highway curves that can be characterized as high-accident sites is a relatively small proportion of the total number of highway curves.

V. COMPUTER SIMULATION STUDIES

This task of the research used the Highway-Vehicle-Object Simulation Model (HVOSM) to study various aspects of vehicle operations and control on highway curves. The objectives of this task were to:

(1) Demonstrate the applicability of HVOSM as a tool for studying the dynamic responses of vehicles traversing highway curves;

(2) Study the sensitivity of tire friction demand, vehicle placement, and vehicle path for critical vehicle traversals to various highway curve design parameters;

(3) Study the sensitivity of tire friction demand and driver discomfort for moderate encroachments onto the shoulder of highway curves with various cross-slope breaks;

(4) Study the rollover potential of moderate vehicular encroachments onto various roadside slopes on highway curves.

HVOSM Methodology

The HVOSM is a computerized mathematical model originally developed and refined by Calspan Corporation, formerly Cornell Aeronautical Laboratories (30). The HVOSM is capable of simulating the dynamic responses of a vehicle traversing a three-dimensional terrain configuration. The vehicle is composed of four rigid masses; viz., sprung mass, unsprung masses of the left and right independent suspensions of the front wheels, and an unsprung mass representing a solid rear-axle assembly.

This study used the Roadside Design version of HVOSM that is currently available from FHWA. A 1971 Dodge Coronet was used as the test vehicle throughout the study. Certain modifications were necessary to perform the range of studies undertaken in this research. These modifications are described in Appendix D and in a separate report, HVOSM Studies of Cross-Slope Breaks on Highway Curves, (31) which gives the details of the HVOSM studies of cross-slope breaks.

These modifications included the following:

(1) Driver discomfort factor output;
(2) Friction demand output;
(3) Terrain table generator;
(4) Driver model inputs (damping, steer velocity, steer initialization);
(5) Wagon-tongue path following algorithm;
(6) Ground contact point interpolation; and
(7) Effective Range Angled Boundary Option (ERABO).

For the highway curve traversal studies, one of the more important aspects of the path following algorithm is the length of the wagon-tongue or probe length. The wagon-tongue is attached to the center of gravity and extends in front of the vehicle parallel to its x-axis. A probe at the end of the wagon-tongue monitors the error from the intended path and activates the driver model inputs. The probe length in essence simulates the complex interaction which occurs as a driver sees the roadway ahead and responds to what he sees. Selection of a probe length, therefore, actually amounts to a decision as to what type of driver is being modeled. Long probe lengths are indicative of "ideal" drivers, who prepare for the curve well in advance. The resulting simulated behavior closely follows that described by the centripetal force equation, with the simulated vehicle path tracking nearly exactly the center of the lane. Moderate probe lengths create minor path corrections just preceding the curve, and tend to allow the vehicle to track in a near optimum manner. Calculated friction values are somewhat higher than is predicted by the centripetal force equation. Very short probe lengths represent aggressive or inattentive driver behavior. Path corrections in response to the presence of the impending curve occur only as the vehicle actually enters the curve. The result is a dynamic over-shoot at the beginning of the curve, with high lateral friction demand generated by the vehicle and a distinctly noncircular path.

The above discussion emphasizes the need to carefully define the driver behavior being modeled. Highly variable results can be obtained running different probe lengths on the same simulated curve at the same speed.

Preliminary Curve Runs and Results

Twelve initial HVOSM runs were made to demonstrate and verify that the HVOSM yields reasonable dynamic responses for curve traversals. These runs were made on unspiraled highway curves with AASHTO (32) superelevation runoff lengths distributed 70 percent on tangent and 30 percent on curve. The basic idea was to select a long probe length that would allow the vehicle to track the center of the lane with very little path deviation. The resulting vehicle dynamics given by the HVOSM could then be compared to those predicted by the centripetal force equation.

Table 22 shows the calculated and simulated dynamic responses for running the vehicle at design speed for the twelve test curves using a probe that represented a 1.0 second driver preview. As can be seen, the calculated lateral acceleration, $V^2/15R$ ($V^2/127R$) and the simulated lateral acceleration are closely comparable for all tests. Also, the calculated tire responses, $(V^2/15R)-e$ $[(V^2/127R)-e]$ are comparable to the simulated tire responses.

It is noteworthy that, because of roll angle, the driver discomfort factor (centrifugal acceleration acting on the driver) is always higher than the lateral acceleration on the tires. Therefore, the design f values in the AASHTO process are not the centrifugal acceleration where the driver begins to feel discomfort, but represent the lateral friction on the tires that creates the threshold of driver discomfort.

Critical Curve Runs and Results

With the HVOSM verified for use on curve traversals, the model appeared to be a reasonable tool for studying curve traversals where the vehicle does not precisely follow the center of the lane. The purpose of this exercise was to use the HVOSM to study the sensitivity of vehicle dynamics to varying curve and operational parameters.

It was first necessary to define a nominally critical level of driver behavior. Behavior less critical, or near average, would result in simulations which tend to mirror dynamics predicted by the centripetal force equation. Highly critical

TABLE 22

INITIAL HVOSM TESTS

V Speed mph	V Speed (km/h)	R Roadway Radius ft	R Roadway Radius (m)	e Superelevation percent	Calculated Results*: Lateral Acceleration	Calculated Results*: Tire Friction	HVOSM Results: Maximum Lateral Acceleration	HVOSM Results: Maximum Tire Friction	HVOSM Results: Maximum Driver Discomfort Factor
20	(33)	108	(33)	8	0.25	0.17	0.25	0.17	0.20
20	(33)	128	(39)	4	0.21	0.17	0.20	0.14	0.18
31	(50)	230	(70)	10	0.26	0.16	0.26	0.17	0.20
31	(50)	272	(83)	6	0.22	0.16	0.22	0.17	0.20
42	(67)	469	(143)	8	0.23	0.15	0.23	0.16	0.18
42	(67)	574	(175)	4	0.19	0.15	0.19	0.16	0.19
52	(83)	650	(198)	10	0.26	0.16	0.27	0.17	0.21
52	(83)	850	(259)	6	0.20	0.14	0.20	0.14	0.18
62	(100)	1207	(368)	8	0.20	0.12	0.22	0.10	0.15
62	(100)	1529	(466)	4	0.16	0.12	0.18	0.12	0.16
73	(117)	1637	(499)	10	0.20	0.10	0.20	0.11	0.12
73	(117)	2083	(635)	6	0.16	0.10	0.16	0.11	0.13

* Calculated results are based on centripetal force equation

1 mph = 1.609 km/h
1 ft = 0.305 m

levels, on the other hand, may not produce realistic results, and thus may not provide a useful basis for comparing variable geometrics.

The selection of an appropriate level of criticality was based on previous vehicle operations research. Studies by Glennon (33) in Texas indicated that most drivers exceed the AASHTO design f, and that some exceed it greatly. The report relates maximum path curvature to highway curvature for various percentiles of the driving population. For purposes of this study the 95th percentile path was selected to represent nominally critical operations. This relationship is as follows:

$$R_V = 5820\ R_C/(R_C + 6780) \qquad [5.1]$$

Where

R_V = 95th percentile vehicle path radius (ft)
R_C = highway curve radius (ft)

NOTE: 1 ft = 0.305 m

Using the path described by Equation 5.1, the critical f factors were calculated by substituting path curvature for highway curvature in the centripetal force equation for any design speed combination of highway curvature and superelevation.

With this relationship between highway curve parameters and nominally critical f factors established, several preliminary HVOSM runs were made to select a probe length that best generated the intended critical operations. The selected probe length represents a 0.25 second driver preview, and is expressed as follows:

$$L = 0.25V$$

Where

L = Probe Length, ft (m)
V = Forward Velocity, ft/s (m/s)

With the probe length established, the HVOSM was ready for studying the sensitivity of vehicle dynamics to various highway curve design and operational parameters under nominally critical path following conditions. Of particular interest were:

(1) Vehicle speed
(2) Superelevation runoff length
(3) Superelevation runoff distribution
(4) Presence of spirals
(5) Length of spirals
(6) Presence of downgrade
(7) Length of curve

Twenty-four HVOSM runs were made using six AASHTO metricated curves. The results of these runs are shown in Table 23 and discussed below. Figure 8 shows examples of the HVOSM output.

Vehicle Speed

The centripetal force equation demonstrates the sensitivity of vehicle dynamics to speed. For actual highway curve operations, it is reasonable to expect a portion of drivers to exceed the nominal design speed of the curve. (Of course, the frequency and amount of "excessive" speed behavior varies with the type of

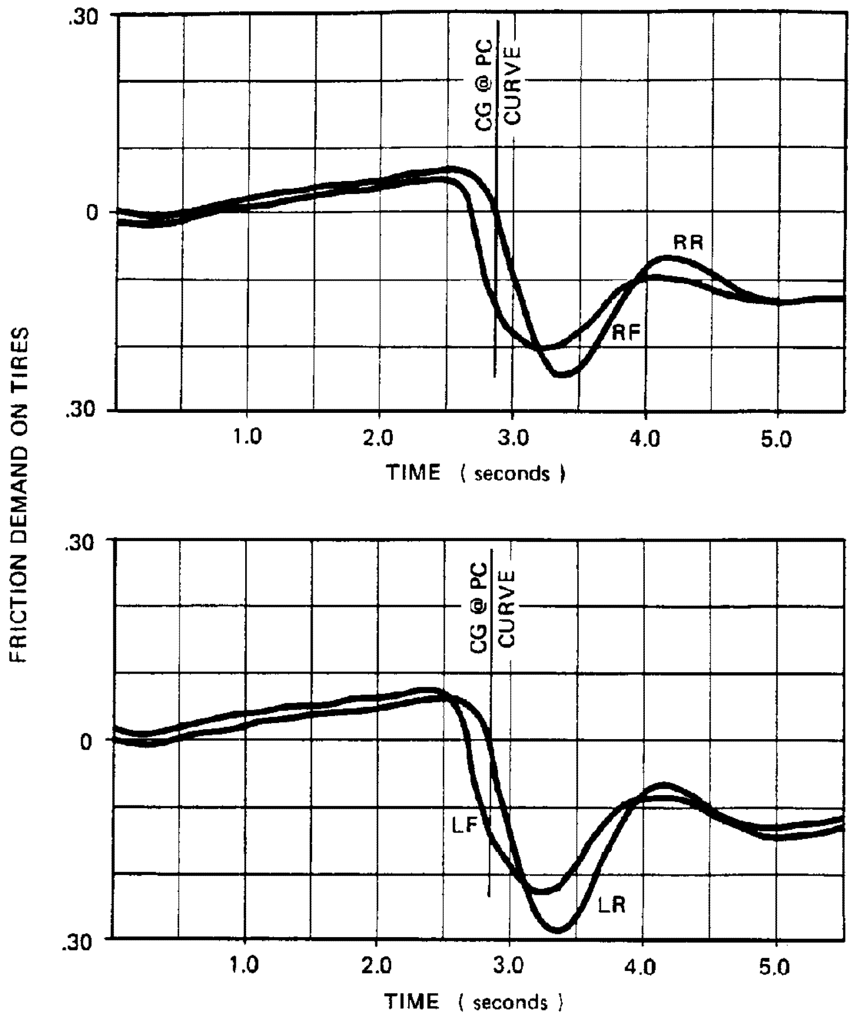
TABLE 23

CRITICAL HVOSM TESTS

TEST PARAMETERS								RESULTS	
Curve Radius ft (m)	Maximum Super-elevation (Percent)	Curve Design Speed mph(km/h)	Length of Super-elevation Runoff ft (m)	Percent of Maximum Super-elevation on Tangent	Presence and Length of Spiral	Grade (Percent)	Test Vehicle Operating Speed mph(km/h)	AASHTO Design f	HVOSM f
2461 (750)	6	75 (120)	200 (61)	70	None	0	87 (140)	0.092	0.190
2461 (750)	6	75 (120)	200 (61)	70	None	0	75 (120)	0.092	0.150
1968 (600)	10	75 (120)	302 (92)	70	None	0	87 (140)	0.092	0.230
1968 (600)	10	75 (120)	302 (92)	70	None	0	75 (120)	0.092	0.160
1968 (600)	10	75 (120)	302 (92)	20	None	0	75 (120)	0.092	0.190
1968 (600)	10	75 (120)	164 (50)	70	None	0	75 (120)	0.092	0.120
1345 (410)	8	62 (100)	216 (66)	70	None	0	75 (120)	0.116	0.260
1345 (410)	8	62 (100)	216 (66)	70	None	0	62 (100)	0.116	0.170
1345 (410)	8	62 (100)	108 (33)	70	None	0	62 (100)	0.116	0.140
1345 (410)	8	62 (100)	216 (66)	N/A	AASHTO	0	62 (100)	0.116	0.100
689 (210)	10	50 (80)	236 (72)	70	None	0	62 (100)	0.140	0.390
689 (210)	10	50 (80)	236 (72)	70	None	0	50 (80)	0.140	0.240
689 (210)	10	50 (80)	236 (72)	20	None	0	50 (80)	0.140	0.260
689 (210)	10	50 (80)	236 (72)	70	None	5	50 (80)	0.140	0.240
689 (210)	10	50 (80)	236 (72)	N/A	AASHTO	0	50 (80)	0.140	0.120
689 (210)*	10	50 (80)	236 (72)	70	None	0	50 (80)	0.140	0.200
689 (210)	10	50 (80)	236 (72)	20	None	5	62 (100)	0.140	0.430
426 (130)	8	37 (60)	164 (50)	70	None	0	50 (80)	0.152	0.400
426 (130)	8	37 (60)	164 (50)	70	None	0	37 (60)	0.152	0.200
426 (130)	8	37 (60)	164 (50)	70	None	5	37 (60)	0.152	0.210
426 (130)	8	37 (60)	164 (50)	N/A	AASHTO	0	37 (60)	0.152	0.120
164 (50)	10	25 (40)	164 (50)	70	None	0	37 (60)	0.164	0.520
164 (50)	10	25 (40)	164 (50)	70	None	0	25 (40)	0.164	0.200
164 (50)	10	25 (40)	164 (50)	70	None	5	25 (40)	0.164	0.200

* 164 ft (50 m) curve length

1 ft = 0.305 m
1 mph = 1.609 km/h



TEST CONDITIONS

Speed - - 50mph (80km/h)

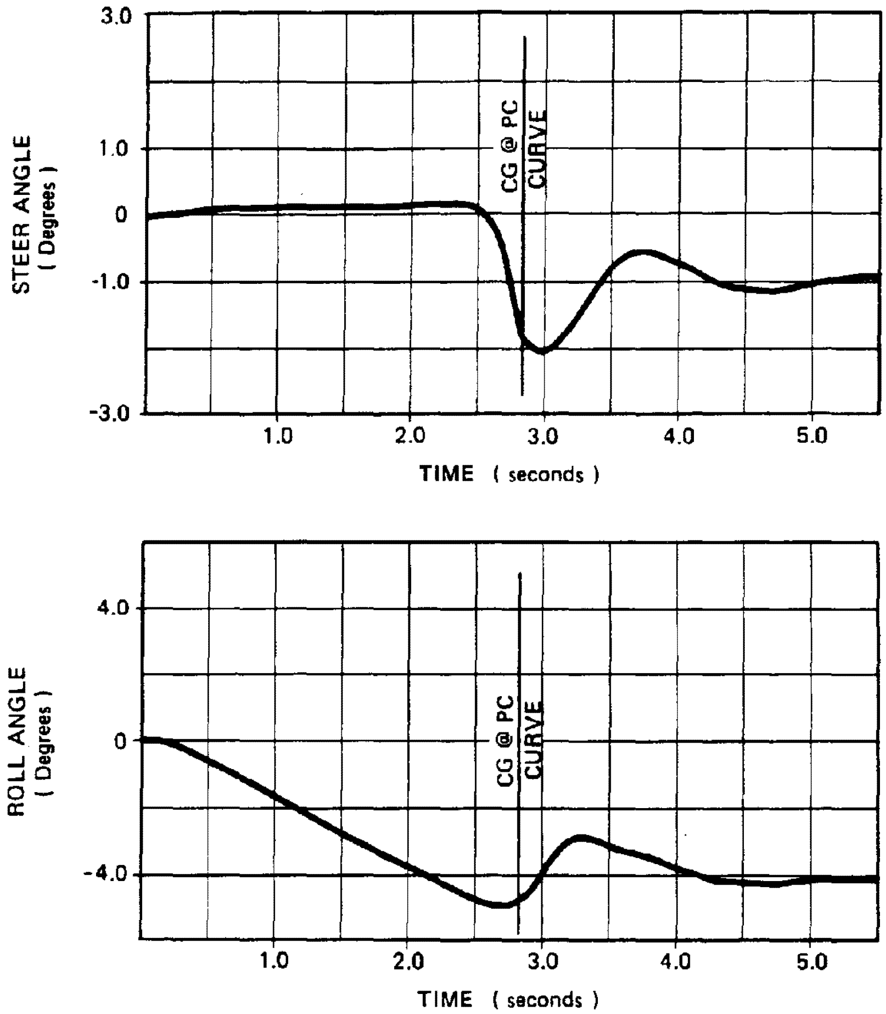
Roadway Geometry

Centerline Radius 689ft (210m)
Superelevation 10 percent
Super. Runoff 236ft (72m)
Super. Dist. 70% on tangent
Grade 0 percent

Vehicle and Driver Characteristics

Probe Length 17.7ft (5.4m)
P GAIN 5.8×10^{-6} rad / ft
(1.9×10^{-5} rad / m)
Q GAIN 5.8×10^{-7} rad · s / ft
(1.9×10^{-6} rad · s / m)
No Deceleration

Figure 8. EXAMPLE HVOSM OUTPUT



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Super. Runoff 236ft (72m)
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Probe Length 17.7ft (5.4m)
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(1.9×10^{-5}rad / m)
Q GAIN 5.8×10^{-7}rad · s / ft
(1.9×10^{-6}rad · s / m)
No Deceleration

Figure 8. EXAMPLE HVOSM OUTPUT (Continued)

highway, the curve itself, and environmental conditions.) Simulations of dynamic responses to speeds in excess of design were therefore believed valuable.

To test high-speed vehicle behavior, each of the six test highway curves was run at 12.5 mph (20 km/h) above design speed. This speed increment is slightly greater than is considered the "potential increase permissible within design speed" by Leisch (34), and thus represents an upper limit on reasonable speed expectations for almost all highway curves.

The tire friction for this speed increment was found to be most sensitive for the lower design speed curves. For the 25 mph (40 km/h) design speed curve, the friction demand was simulated to be 0.52 compared with a design f of 0.16. These results could also be similarly predicted with the centripetal force equation (thus providing one more verification of the HVOSM methodology.)

The implications of the test results for speed are very important. These suggest that an existing highway curve that is underdesigned for the prevailing operating speed could present a severe roadway hazard. This is particularly true for design speeds below 60 mph (about 100 km/h). At such lower design speeds, frequent vehicle operating speeds of 5 to 10 mph (8 to 16 km/h) above the curve design speed can be reasonably expected.

Superelevation Runoff Length

This parameter was evaluated for design speeds of about 50 mph (80 km/h) and 60 mph (100 km/h) by comparing the AASHTO runoff length with one that was half as long. For the comparison, the superelevation runoff length was distributed with 70 percent on the tangent and 30 percent on the curve.

The somewhat surprising result of these tests was that the shorter runoff length yielded slightly smaller friction demands. The only identifable explanation for this phenomenon is that the maximum simulated friction demands take place in the initial part of the curve where the shorter runoff length provided slightly higher superelevation.

Superelevation Runoff Distribution

This parameter was evaluated for 50 mph (80 km/h) and 75 mph (120 km/h) highway curves having AASHTO superelevation runoff lengths with 70-30 and 20-80 distributions. As expected, 70-30 distribution, where most of the superelevation transition is provided on the tangent, produced somewhat smaller friction demands. The differences can be explained almost entirely by the difference in superelevation in the initial part of the curve where the maximum friction demand was generated.

Presence of Spirals

This parameter was evaluated for highway curves with design speeds between 37 mph (60 km/h) and 62 mph (100 km/h). The comparison was between highway curves with and without AASHTO spirals.

This comparison provides the most dramatic results of the study. *In all cases, the presence of the spiral reduced the friction demand from a value significantly higher than the design f to one that was below the design f.*

The reason for this dramatic result seems readily evident. For the driver who is inattentive or for some other reason has limited notice of the upcoming curve, the spiral not only reduces his absolute path error over time but requires less severe steering to correct for the desired path because the path of a spiral is less severe than the path of a circular curve.

Length of Spiral

Although the initial plan was to test a spiral that was twice the length of an AASHTO spiral, this plan was not carried through after obtaining the dramatic results for the presence of AASHTO spirals.

Presence of Downgrade

This parameter was evaluated for highway curve design speeds of 25 mph (40 km/h) to 50 mph (80 km/h). In comparing a 5 percent downgrade with level terrain, no difference was found in the friction demand.

Short Curve Length

This parameter was evaluated by looking at the difference between vehicular response to the approach to a curve (i.e., the dynamics of proceeding from tangent to curve) and the response by the driver as he transitions in and immediately out of the curve. A 164-foot (50-metre) curve length of a 50 mph (80 km/h) design curve was selected for analysis.

The results of this test indicate that the inattentive driver will generate less dynamic overshoot on the very short curve because he begins sensing and adjusting for the upcoming tangent before he has to perform the maximum correction that would be necessary on a longer curve.

Summary of Critical Curve Runs

The critical analysis of highway curves provided two preliminary results with important implications. These results were subject to the field verification of the HVOSM driver inputs discussed in Chapter VI. The first important result is that the dynamic response of vehicles traversing a highway curve is very sensitive to speed. The implication of this result is that existing highway curves that are severely underdesigned for the prevailing highway speeds present serious hazards. The second important result is that the addition of spiral transitions to highway curves dramatically reduces the friction demand of critical vehicle traversals.

Cross-Slope Break Studies

Details of cross-slope break studies are reported in a separate report titled "HVOSM Studies of Cross-Slope Breaks on Highway Curves" (31). These studies and their results are summarized here.

The objective of these studies was to evaluate AASHTO (32,35) policy regarding the maximum recommended difference of 7 percent between the cross slopes of the pavement and the shoulder. This policy has existed since 1954 and is consistent with the AASHTO minimum pavement cross slope of 1 percent for high-type surfaces and the maximum AASHTO shoulder cross slope of 8 percent specified for turf shoulders.

When designing superelevated horizontal curves according to AASHTO, the cross-slope break requirement can constrain the shoulder cross-slope design on the outside of the curve. For example, with 6 percent superelevation, the cross-slope break requirement limits the maximum negative shoulder cross slope to 1 percent, which does not meet the AASHTO drainage requirements for even paved shoulders. The alternatives are to either design a positive shoulder slope or a rounded shoulder. A positive shoulder slope drains more runoff water across the pavement and creates problems with the melting of stored snow on the outside shoulder. The rounded shoulder design is more difficult to construct and maintain.

HVOSM Test Conditions and Performance Criteria

Table 24 shows the general highway geometrics, the parameters of vehicle operations, and the performance criteria selected for testing. The vehicle operating parameters were chosen to represent the design criteria of a moderate encroachment onto the shoulder. The performance criteria were selected as reasonable dynamic response thresholds for design.

HVOSM Runs

A series of initial HVOSM runs was made using the highest design speed and an extreme (16 percent) cross-slope break to study the dynamic differences between (1) four-wheel and two-wheel traversals onto the shoulder, and (2) entry to and exit from the shoulder. The results of these runs indicated that the four-wheel traversal and the entry to the shoulder produced the more extreme dynamic responses. In the main part of the experiment, 14 runs were made using design speeds of 50 mph to 75 mph (80 km/h to 120 km/h), shoulder slopes of 2 to 6 2 to 6 percent, and superelevation rates of 2 to 10 percent.

HVOSM Results and Design Implications

The results clearly show that the driver discomfort level (centrifugal acceleration) in a moderate shoulder traversal on highway curves is sensitive to speed, radius of curve, shoulder cross slope, and the lateral extent of movement onto the shoulder. For a given path and speed of shoulder traversal, therefore, the driver discomfort mainly increases with shoulder slope and very little, if any, with the amount of cross-slope break.

TABLE 24

HVOSM TEST CONDITIONS AND PERFORMANCE CRITERIA
FOR CROSS SLOPE BREAK STUDIES

Test Conditions	Specification
Highway Curve Radius	AASHTO Controlling Curves
Superelevation	AASHTO Controlling Curves (2 percent to 10 percent)
Shoulder Width	9.0 ft (2.7 m)
Shoulder Cross Slope	-2 percent to -6 percent
Vehicle	1971 Dodge Coronet
Initial Vehicle Speed	Design Speed
Vehicle Deceleration	Engine Braking @ 0.1 g
Vehicle Path Radius	95th percentile path as a function of highway curve radius measured by Glennon and Weaver (33)
Vehicle Path Radius Tangent Point	Corrective Curve 7.2 ft (2.2 m) from edge of roadway
Performance Criteria	
Tire-Pavement Friction	0.4
Driver Discomfort Factor	0.3

For paved shoulders with widths of 5.2 feet (1.6 m) or greater, where the shoulder cross slope is intended to accommodate up to a four-wheel traversal onto the shoulder, the research indicates a maximum tolerable cross-slope break of 8 percent. (Note: the tolerable cross-slope break is a function of design speed, design curvature, design superelevation and the maximum tolerable shoulder slope for these conditions.) For superelevation rates between 2 and 6 percent, this criterion allows maximum (negative) shoulder slopes ranging from 6 to 2 percent, respectively. For superelevation rates exceeding 6 percent, a different kind of shoulder cross-slope design is required.

For paved shoulders less than 5.2 feet (1.6 m) wide, which are implicitly designed to only accommodate two-wheel traversals within the bounds of the shoulder, the research indicates tolerable cross-slope breaks ranging from 8 to 18 percent. These greater cross-slope breaks do not further compromise safety beyond the initial decision of choosing the narrower shoulder.

Roadside Slope Studies

The sensitivity of vehicle dynamics to negative cross slopes shown in the cross-slope break studies raised some questions about vehicle dynamics on the more severe roadside slopes. Also, previous studies (36,37) had indicated that highway curvature was the most predominant factor in fatal rollover collisions. Therefore, a few HVOSM runs were undertaken to look at the severity of vehicle dynamic responses on roadside slopes of 4:1 and 6:1.

Since this exercise was an adjunct to the main research effort, a very limited study was done. The key purpose of these runs was to generally identify whether roadside slope design and embankment guardrail warrants might need to vary as a function of highway curvature.

Four HVOSM runs were performed in an identical manner to the cross-slope break runs using a -2 percent shoulder slope in place of the superelevation and either a 6:1 (i.e., a -16.7 percent) or a 4:1 (i.e., a -25 percent) roadside slope in place of the shoulder slope. The results of these tests are shown in Table 25.

TABLE 25

HVOSM ROADSIDE SLOPE TESTS

Side Slope Ratio	Side Slope Angle (Degrees)	Curve Design Speed mph	(km/h)	Curve Radius ft	(m)	Path Radius ft	(m)	Maximum Lateral Acceleration on Tires(g's)	Maximum Roll Angle (Degrees)
6:1	9.5	50	(80)	689	(210)	538	(164)	0.47	14.5
6:1	9.5	75	(120)	1968	(600)	1312	(400)	0.60	15.0
4:1	14.0	50	(80)	689	(210)	538	(164)	0.60	19.5
4:1	14.0	75	(120)	1968	(600)	1312	(400)	0.78	20.0

1 mph = 1.609 km/h

1 ft = 0.305 m

With a hard surface, these runs indicated a very severe lateral acceleration on the tires for even the 6:1 slope, which is considered a mild roadside slope. Therefore, for most well-stabilized roadside surfaces free of irregularities, skidding is very likely.

The test runs also showed fairly severe vehicle roll angles on the hard flat roadside surfaces. These vehicle roll tendencies in combination with tire-plowing on unstablized roadside surfaces or impact with surface irregularities would produce a high expectation of vehicle rollover.

Although these tests were simplistic in nature, they strongly indicate a need to review roadside slope design policies and highway guardrail warrants as they apply to highway curves.

VI. OPERATIONAL FIELD STUDIES

Evaluation of geometric design criteria requires knowledge about operations on the highway. For two-lane rural highways, the term operations refers to vehicle speeds, speed changes, and vehicle path behavior relative to the highway alinement.

A major element of this research was an investigation of operations on rural highway curves. Two separate sets of field experiments were conducted to measure important aspects of driver/vehicle behavior. The first, a study of vehicle speeds, determined the relationship of basic approach alinement features to speeds and speed change behavior in the vicinity of horizontal curves. The second, a study of vehicle curve traversals, examined the manner in which individual drivers track horizontal curves. The results of both studies, reported in this chapter, yield significant findings concerning geometric design policy and basic design assumptions regarding driver/vehicle behavior.

VEHICLE SPEED STUDIES

Vehicle speed is a critical consideration in design. The research team was interested in characterizing speeds of vehicles as they approached, transitioned into, and traversed through the curve. A number of basic hypotheses directed the design of these studies:

(1) Vehicle speeds in advance of highway curves are affected by the general character of the highway.

(2) With adequate sight distance, drivers approaching highway curves adjust their speed in advance of the curve to a comfortable level.

(3) The amount of speed reduction achieved by drivers is related to the sharpness of the highway curve.

Experimental Plan

The speed studies were conducted by field crews during their inspection of high- and low-accident sites. The 333-site sample was available to the crews to enable studying a range of curvature, sight distance to the curve, and character of approach alinement. Figure 9 shows the planned study matrix for site selection, along with sample sizes actually taken for each cell of the matrix. The field crews were restricted in their ability to observe sites in each cell due to availability of certain combinations of geometrics. Further restrictions were placed on site selection to insure consistency of sampling and mitigate the effects of extraneous conditions. Neither very short curves, nor sites near intersections, speed zones or city limits were studied.

Field Procedure

In the process of characterizing each of the curves studied in the high- and low-accident analysis, certain relevant approach and curve data were observed and recorded. These included:

° Sight Distance to the Curve --Field crews drove both approaches to the curve and judged whether there was at least 600 feet (183 m) of sight distance available to the point of curvature.

Type of Approach Alinement	Approach Sight Distance < 600 ft Highway Curvature			Approach Sight Distance > 600 ft Highway Curvature		
	≥ 6°	3 - 4°	1 - 2°	≥ 6°	3 - 4°	1 - 2°
<u>Class A</u> Primarily Tangent Primarily Level No Close by Intersection or City	2 (0)	2 (1)	2 (1)	2 (6)	2 (9)	2 (6)
<u>Class B</u> Moderate Mild Curvature Some Moderate Grades	2 (0)	2 (4)	0 (0)	2 (7)	2 (5)	0 (5)
<u>Class C</u> Predominantly Curvilinear Hilly, Multiple Grade Change	2 (0)	0 (2)	0 (0)	2 (6)	0 (3)	0 (0)

0 - - Number of Planned Speed Studies
(0) - - Number of Speed Studies Performed

1ft = 0.305m

Figure 9. VEHICLE SPEED STUDY MATRIX

- Approach Alinement --The highway alinement for one to two miles in advance of each approach was classified. Classification was in three basic categories. Class A alinement was primarily tangent with level grades. Class B included some moderate curvature and/or moderate grades. Class C was primarily curvilinear and/or hilly, with multiple significant grade changes.

- Degree of Curve --The roadway curvature given by state geometry files was checked in the field by ball bank indicator readings, in combination with field measurements of superelevation.

Field crews identified potential speed study sites using the study matrix shown in Figure 9. Once a site was selected for study, the following procedure was used:

- Range poles were set well off the highway at four points along the curve. These points were the tangent approach (TA), approximately 700 to 800 feet (200 to 250 m) in advance of the curve; the transition to the curve (TC), a point about 200 feet (60 m) before the PC; the point of curvature (PC); and a point about at the middle of the curve (MC), which was usually 200 to 400 feet (60 to 120 m) beyond the PC. See Figure 10.

- Free-moving vehicles were observed using radar guns. Samples of 25 to 30 vehicles were taken. Two observers recorded measurements for two points each and included a description of the vehicle for later use in matching observations in the office. The field crews voided observations of drivers who were obviously aware of the study and therefore reduced their speeds dramatically.

Summary of Data Obtained

A total of more than 1400 observations of vehicle speed behavior were recorded at 60 curve approaches. Data collection procedures enabled calculation of speed distribution data by site for each point along the curve, as well as speed change data between any desired sets of points for each vehicle studied. All site data and speed observations were recorded and coded. Computer summaries and statistical analyses were produced by FHWA personnel.

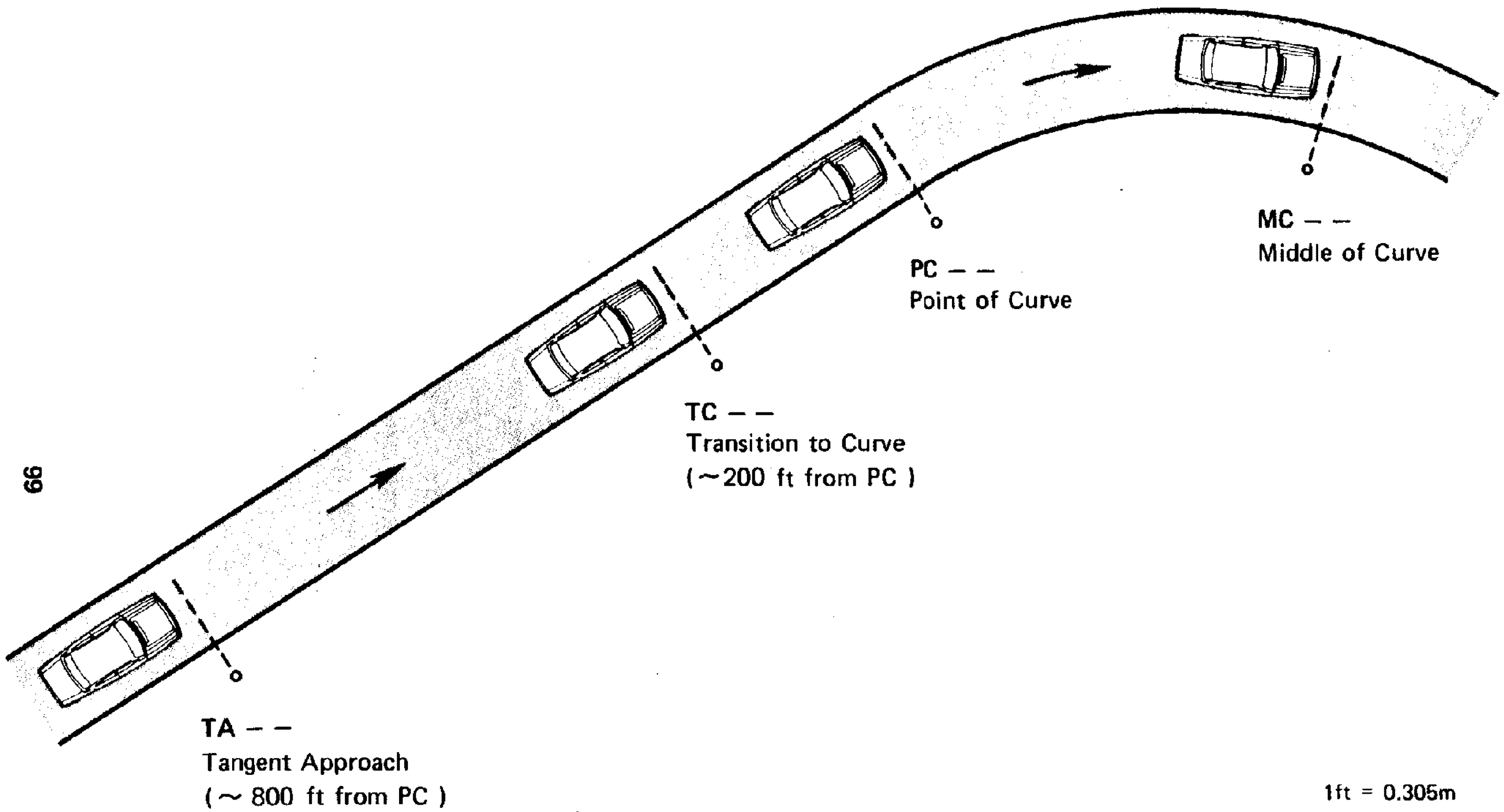


Figure 10. REPRESENTATIVE SPEED STUDY PLAN

Effect of Approach Alinement on Speeds

Speed distribution data were evaluated to determine the influence of roadway geometry and conditions on vehicle speeds while both approaching and traversing highway curves.

Sight Distance to the Curve

Curves with less than 600 feet (183 m) of sight distance on the approach were considered to have restricted sight distance. Only eight curve approaches were so identified. Analysis of speeds at the TA and the PC, controlling for alinement class and curvature, showed negligible differences in driver behavior compared to curves with unrestricted sight distance. Mean speeds at the tangent approach (TA) to curves with unrestricted sight distance were generally on the order of 2 to 3 mph (3.2 to 4.8 km/h) higher than for curves with restricted sight distances when approach alinement and curvature were similar. This small difference was also found for mean speeds at the transition to curve (TC). The observed difference is not considered significant given the limited number of sites studied and vehicles observed.

Approach Conditions

Free vehicle speeds on the approach to curves were found to be somewhat influenced by the overall character of the preceding alinement. Table 26 summarizes mean speeds at the TA for all vehicles grouped by type of approach and impending curvature. The effect of approach alinement on mean speed on the approach to a curve appears to be on the order of 2 to 5 mph (3.2 to 8 km/h). The 55 mph (89 km/h) speed limit undoubtedly had some effect on mean speed at sites with Class A approach conditions.

TABLE 26

MEAN SPEEDS OF VEHICLES ON APPROACHES TO CURVES
(UNRESTRICTED SIGHT DISTANCE TO THE CURVE)

Impending Curvature	Characteristic of Highway Alinement on Approach to Curve: Mostly Tangent; Flat Grades (Class A)	Mild Curvature and/or Grades (Class B)	Predominantly Curvilinear and/or Hilly (Class C)
	Mean Speed at Tangent Approach (mph)		
Mild (1°-2°)	57.6	58.1	N/A
Moderate (3°-4°)	56.3	58.3	51.7
Sharp ($\geq$ 6°)	54.0	52.2	52.2

1 mph = 1.609 km/h
N/A - No sites in this category

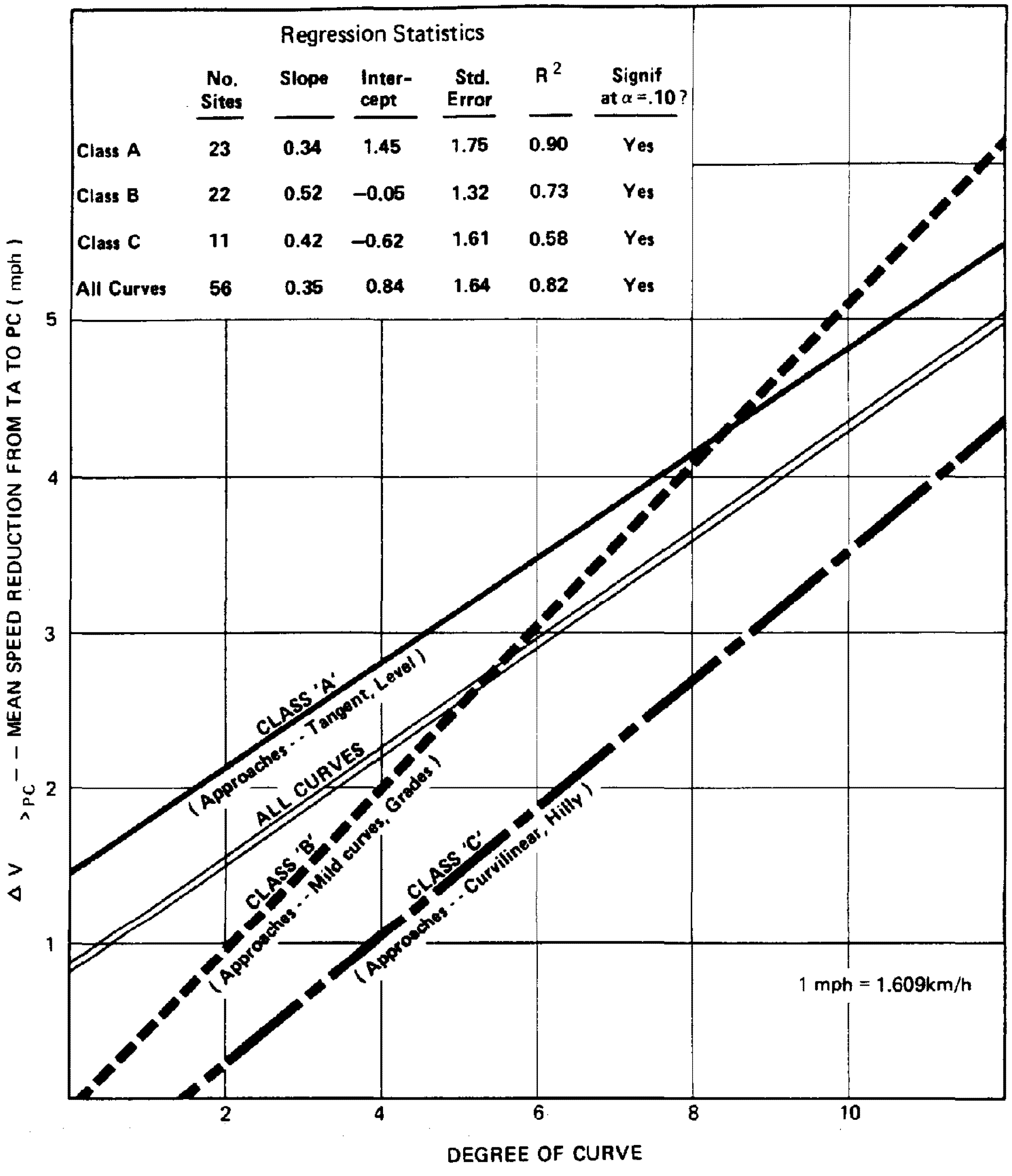
Speed Transition Behavior

The study design enabled detailed investigation of the way in which drivers adjust their speed as they approach a curve. Figure 10 provides reference for the following discussion of speed transition behavior.

For each vehicle observed, speed changes were computed for the following pairs of readings: TA to PC, TC to PC, PC to MC, and TA to MC. Distributional statistics for each site were calculated for these measurements of speed change behavior.

Speed Changes from Tangent Approach (TA) to Point of Curve (PC)

Approach sight distance and approach alinement had little effect on speed change behavior on the approach to a curve. The greatest factor in explaining driver behavior was found to be the sharpness of the impending curve. Figure 11 shows results of linear regression analyses which revealed the relationship between speed change behavior on the curve approach and the degree of curve. Mean speed reductions of up to 8 mph (12.9 km/h) were observed. As Figure 11 indicates,



Regression Statistics

	No. Sites	Slope	Intercept	Std. Error	R^2	Signif at α =.10 ?
Class A	23	0.34	1.45	1.75	0.90	Yes
Class B	22	0.52	−0.05	1.32	0.73	Yes
Class C	11	0.42	−0.62	1.61	0.58	Yes
All Curves	56	0.35	0.84	1.64	0.82	Yes

Figure 11. SPEED CHANGE BEHAVIOR ON APPROACHES TO HIGHWAY CURVES

most of the variation in mean speed reduction is explained by the sharpness of the impending curve. The figure also shows results of an analysis performed on a more critical measure of speed change behavior. For each site, the 85th percentile speed change value was determined from that site's speed change distribution. Regression analysis of the 85th percentile speed change against degree of curve showed higher overall speed reductions across all degrees of curve.

Speed Changes from Transition of Curve (TC) to Point of Curve (PC)

Further investigation of the speed behavior on the approach led to significant findings. Generally, 60 to 80 percent of the speed change observed from the TA to PC actually occurred closest to the PC--between the TC and PC. In other words, drivers do not gradually reduce speed before entering a curve, but rather accomplish such reductions only seconds before reaching the point of curvature.

Speed Changes from Point of Curve (PC) to Middle of Curve (MC)

The data indicated significant driver speed change behavior past the PC on sharper curves. For curves greater than 6°, drivers undergo mean speed reductions of 5 to 6 mph (8.0 to 9.7 km/h) beyond the PC. Table 27 shows mean speed reductions for curves with different approach conditions. The table indicates that significant speed reductions in the curve itself occur on curves of 6° or greater. Regression analysis was performed for all sharp curves ($\geq$ 6°) to determine the relationship between mean speed reduction and curvature as follows:

$$\begin{aligned} \Delta\overline{V} &= [\text{Mean speed at PC - Mean speed at MC}] \\ &= 0.13\ (DC) + 3.74 \end{aligned} \qquad [6.1]$$

Where $\Delta\overline{V}$ = Mean speed reduction for curves 6° or greater (mph)

DC = Degree of Curve

$R^2 = 0.34$

Standard Error = 2.60

t = 3.09

N = 19, Significant at $\alpha = 0.10$

NOTE: 1 mph = 1.609 km/h

TABLE 27

MEAN SPEED REDUCTIONS OF VEHICLES BETWEEN THE POINT OF CURVATURE (PC) AND THE MIDDLE OF CURVE (MC)

	Characteristic of Highway Alinement on Approach to Curve		
Curvature	Mostly Tangent; Flat Grades (Class A)	Mild Curvature and/or Grades (Class B)	Predominantly Curvilinear and/or Hilly (Class C)
	Mean Speed Reduction between PC and MC--mph		
Mild (1°-2°)	0.2	0.5	-
Moderate (3°-4°)	1.2	1.2	0.5
Sharp ($\geq$ 6°)	4.9	6.0	6.1

1 mph = 1.609 km/h

Vehicle Speeds on Curves

Data for speeds of vehicles in the curve (at the MC) provided a measure of driver comfort in traversing highway curves. Effects of curvature and roadway width were studied.

Effect of Curvature

Speed data for the MC of all curves were used in a series of simple linear regression analyses. Figure 12 demonstrates the effect of curvature on two measures of speed distribution--mean speed and 85th percentile speed (approximated by mean plus one standard deviation). Expected speeds for both measures of speed distribution are about 1.5 mph (2.4 km/h) for each one degree increase in curvature.

Regression Statistics

	No. Sites	Slope	Inter-cept	Std Error	R^2	Signif at α =.10 ?
Mean Speed	56	−1.50	58.3	2.57	0.84	Yes
Mean + Std Dev.	56	−1.61	64.6	2.76	0.84	Yes

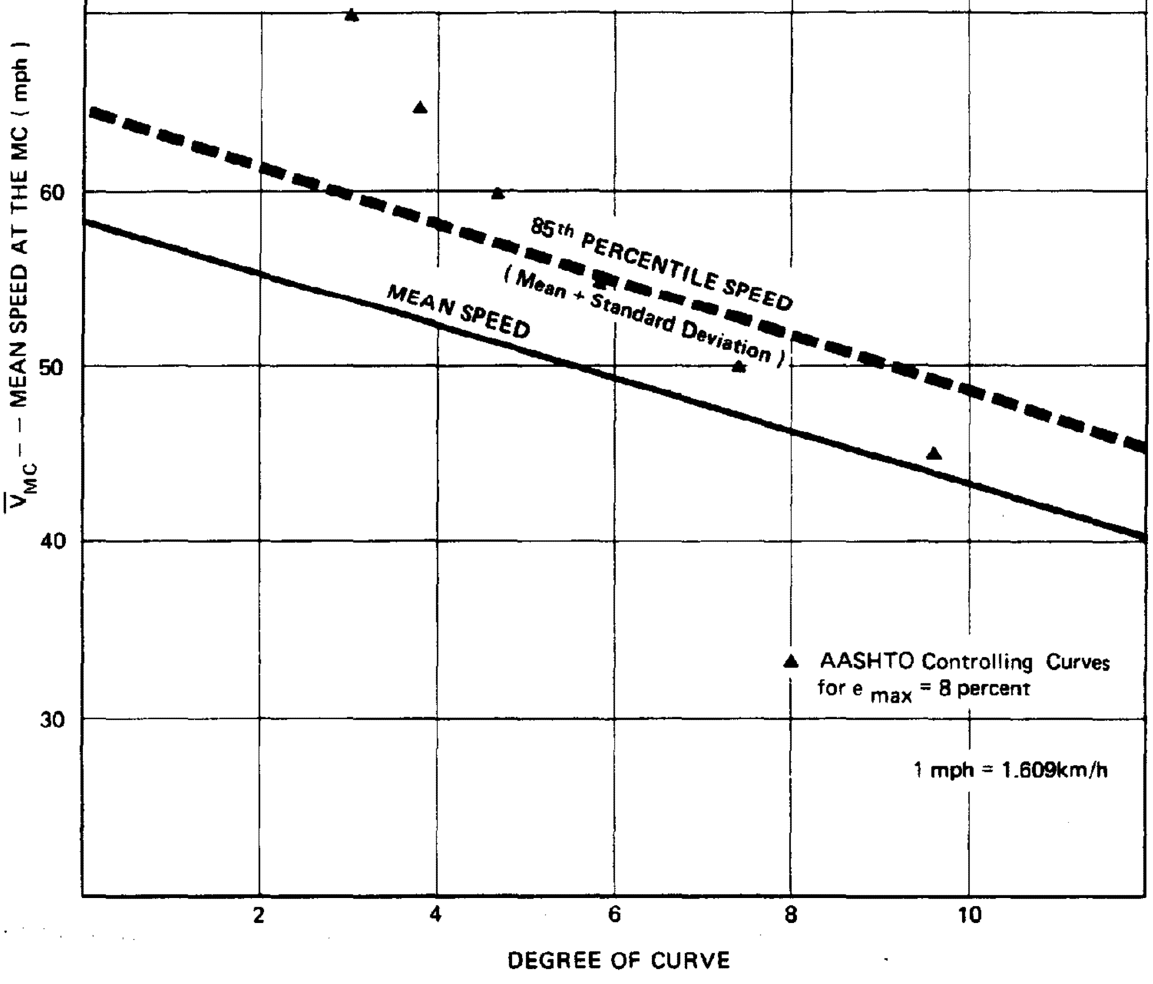


Figure 12. MEAN SPEEDS IN HIGHWAY CURVES

Analysis of the sensitivity of speed variance to curvature was also performed. Speed variance decreases with curvature (significant at $\alpha = .10$), indicating more freedom to operate on milder curves (producing higher variance) and more restricted operations on sharper curves (resulting in a narrowing of the speed distribution).

Effect of Roadway Width

The effect of roadway width on vehicle speeds in the curve was also studied. Two subsamples of sites were selected to observe any differences attributable to width. One subsample included all sites with roadway widths of 24 feet (7.3 m) or more. The second subsample was comprised of sites with widths no greater than 19 feet (5.8 m).

Figure 13 shows regression analyses for the two subsamples of sites. The figure is based on speeds at the MC. Only a slight difference in speeds is predicted for narrow vs. wide roadways. In addition, the effect of curvature on speeds is uniform for narrow and wide roadways.

Analysis of the effect of roadway width on approach speeds (at the TA) showed no significant difference between narrow and wide roadways.

Summary and Implications

The studies of speed behavior of vehicles approaching and traversing horizontal curves answered at least three basic questions regarding speed and highway design:

(1) For the range of speeds studied, generally 50-65 mph (80-105 km/h), alinement conditions in advance of the curve only marginally affected free vehicle approach speeds.

(2) Drivers tend to begin adjusting their speeds only as the curve becomes imminent. For milder curves ($< 4°$) speed changing is slight and is accomplished for the most part prior to the PC. Vehicle speed behavior on sharper curves ($\geq 6°$) is significantly different. The amount of speed reduction increases linearly with increasing degree of curve. Furthermore, about one-half of the total reduction in speed is typically achieved after the vehicle passes the PC.

(3) Sharpness of the curve has by far the greatest influence on vehicle speed and speed change behavior.

Regression Statistics

	No. Sites	Slope	Inter-cept	Std. Error	R^2	Signif at α =.10 ?
Wide Roadways	27	−1.40	59.0	2.48	0.75	Yes
Narrow Roadways	8	−1.50	56.2	1.01	0.97	Yes

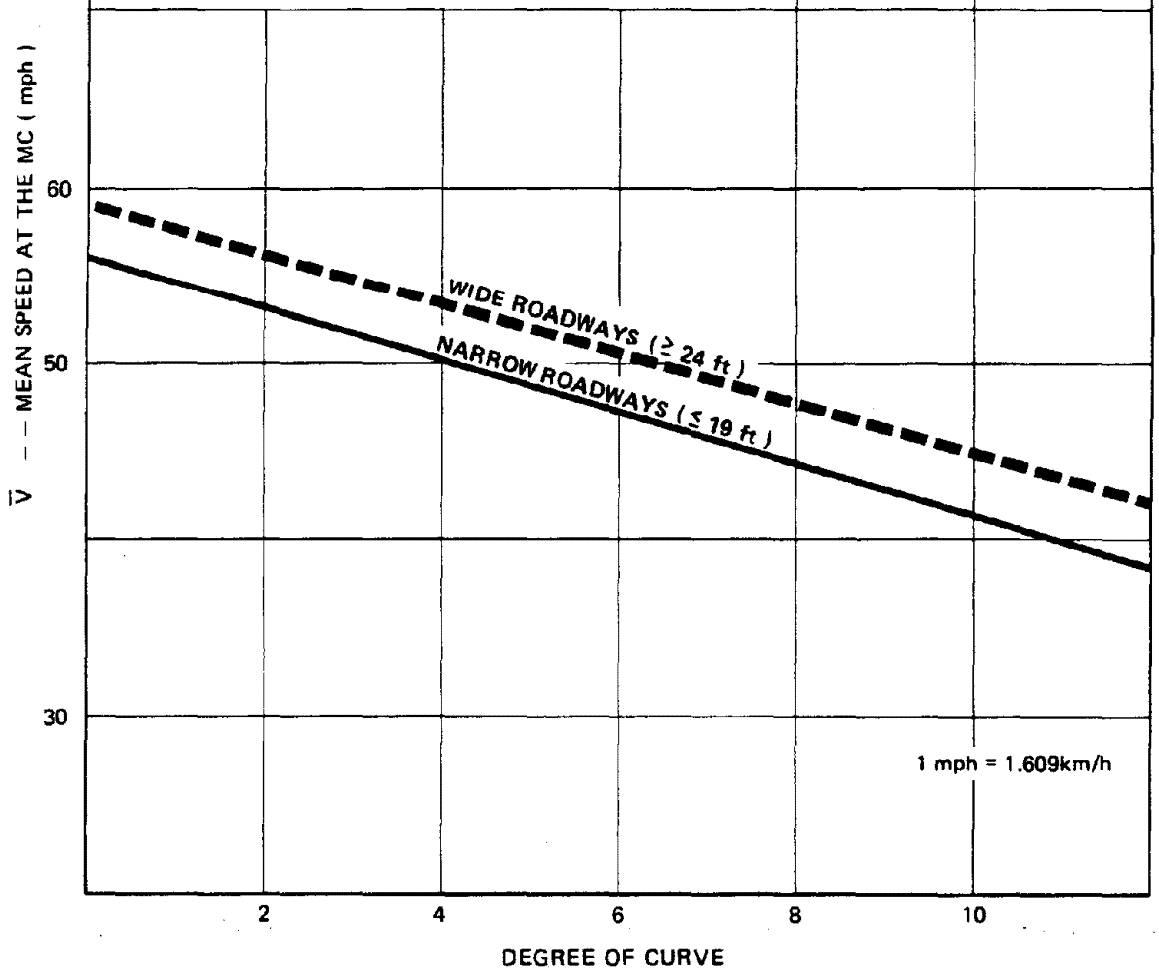


Figure 13. EFFECT OF ROADWAY WIDTH ON MEAN SPEEDS IN HIGHWAY CURVES

These three conclusions are illustrated by Figure 14. This figure depicts speed profiles of driver behavior throughout the approach and curve. The profiles, which are based on mean speeds at the sites observed, show the relative effects of approach conditions and curvature on vehicle speed. Curves greater than 6° are shown to produce different speed behavior than milder curves.

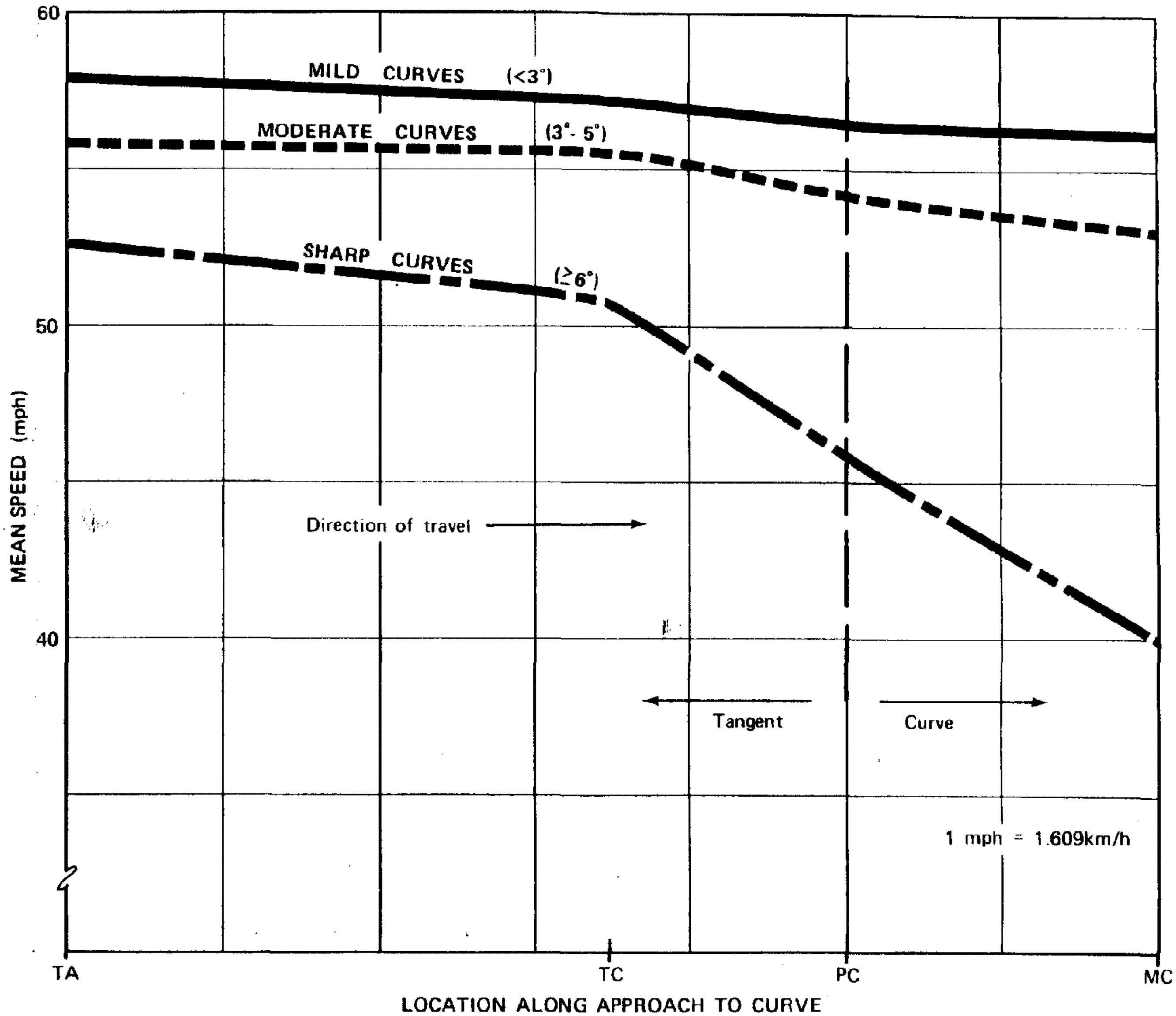


Figure 14. PROFILES OF MEAN SPEEDS ON HIGHWAY CURVES

VEHICLE TRAVERSAL STUDIES

The final major phase of the research involved an intensive study of vehicle operations on rural highway curves. The intent of this study, was to identify and quantify combinations of geometry which produce variable driver behavior. Particular focus was placed on studying vehicle path behavior.

The overall objective of these studies was to determine the operational characteristics of horizontal curvature on high speed, two-lane highways. Driver behavior as a function of curvature and approach conditions was of paramount interest. The following specific objectives defined the design and conduct of the studies.

(1) Investigation of the validity and accuracy of the HVOSM driver model in simulating critical driver behavior;

(2) Development of detailed descriptions of speed and path behavior for drivers as they approach, transition into and drive through the curve;

(3) Identification of the effects of highway curvature (both degree and length), roadway width, and transition design on driver behavior;

(4) Comparison of driver behavior in negotiating right-hand and left-hand curves; and

(5) Comparison of AASHTO design criteria for highway curves with actual observed driver behavior.

Fulfillment of these objectives required careful site selection, study design, and equipment selection.

Site Selection

Budget and schedule considerations limited the number and scope of feasible study sites. Initial planning focused on variations in curvature, width of roadway, transition design and curve approach conditions.

Candidate sites were selected from the file of 333 high- and low-accident locations previously surveyed for the accident analyses. As planning proceeded, it became apparent that a necessary budget constraint was the proximity of the sites to the office where the research was being conducted (Evanston, Illinois). Thus, only sites in Illinois and Ohio were considered for study.

Table 28 shows the significant variables of interest for the five locations selected for study. As the Table indicates, the plan attempted to address ranges of curvature and variable approach conditions. An effort was also made to select at least one narrow roadway with intermediate curvature. Also note that extremely poor transition designs were not studied. This variable was considered the least important in terms of operational effects. When sites with poor transition design in combination with other appropriate characteristics were not found in Ohio or Illinois, this element was dropped from the study design. A photograph of each site and further information on its conditions and characteristics are shown in Figure 15.

TABLE 28

CHARACTERISTICS* OF SITES SELECTED FOR VEHICLE TRAVERSAL STUDIES

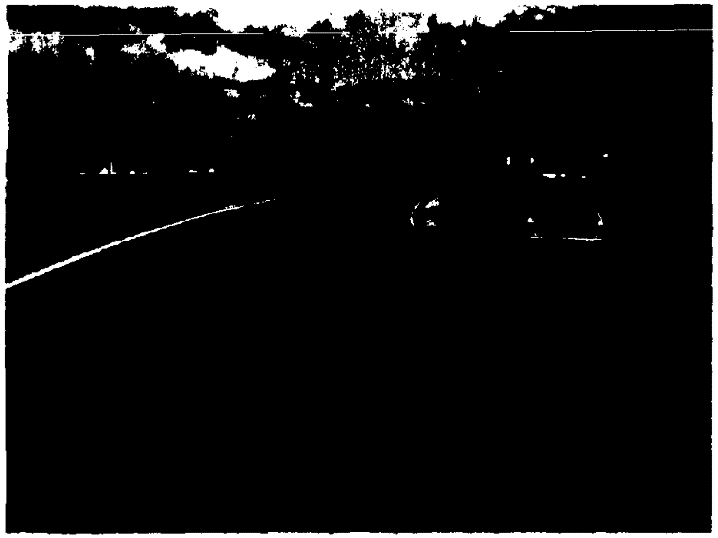
Site No.	Curvature	Roadway Width (feet)	Transition Design	Curve Approach Conditions
188	Mild	>22	Good	Open
198	Sharp	>22	Good	Moderately Restricted
204	Intermediate	≤20	Good	Open
206	Intermediate	>22	Good	Open
212†	Sharp	>22	Good	Restricted

* As indicated from State files and previous field studies.

† Both approaches to be studied.

1 ft = 0.305 m

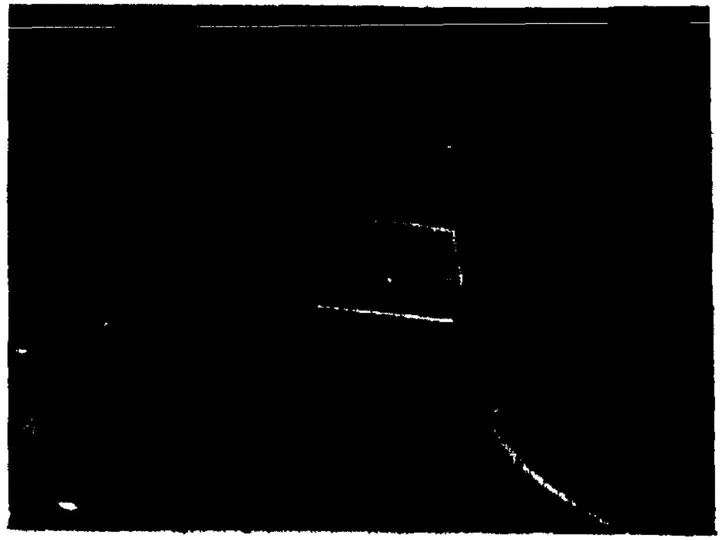
SITE 188
S. H. 47, Kane County, Illinois



ROADWAY CHARACTERISTICS

Geometry		Environment
Curvature	3.8 degrees	Approach Conditions
Direction of Curve	Left	Primarily Tangent
Length of Curve	0.27 mi.	Primarily Level
Roadway Width	23.5 ft.	Roadside Conditions:
Shoulder Width	6.0 ft.	Rating = 26
Superelevation @ P. C.	0.015 percent	Pavement Conditions:
Maximum Superelevation	0.070 percent	Rating = 32
Rate of Superelevation		Traffic Volume Class:
Runoff	0.034	High

SITE 198
S. R. 67, Wyandot County, Ohio



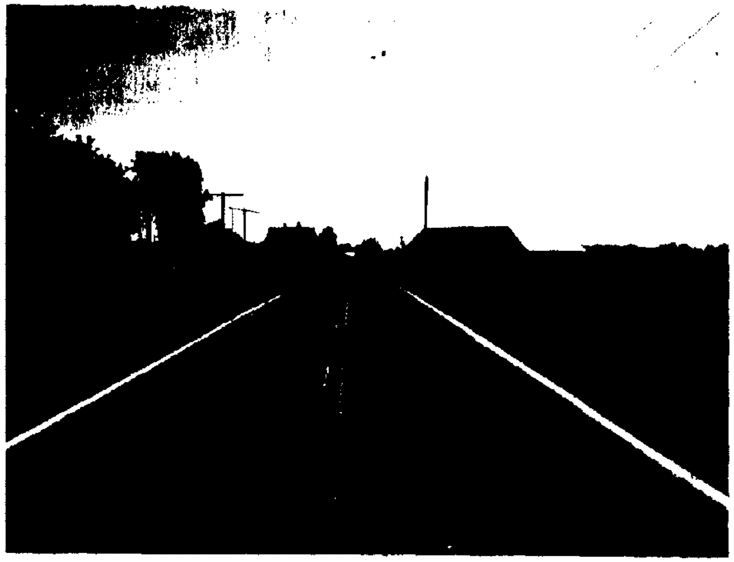
ROADWAY CHARACTERISTICS

Geometry		Environment
Curvature	12.9 degrees	Approach Conditions
Direction of Curve	Right	Mild Curvature
Length of Curve	0.04 mi.	Primarily Level
Roadway Width	22.0 ft.	Roadside Conditions:
Shoulder Width	4.0 ft.	Rating = 32
Superelevation @ P. C.	0.028 percent	Pavement Conditions:
Maximum Superelevation	0.060 percent	Rating = 36
Rate of Superelevation		Traffic Volume Class:
Runoff	0.017	Low

1 mi = 1.609km
1 ft = 0.305m

Figure 15. CHARACTERISTICS OF VEHICLE TRAVERSAL STUDY SITES

SITE 204
S. R. 571, Drake County, Ohio



ROADWAY CHARACTERISTICS

Geometry		Environment
Curvature	6.3 degrees	Approach Conditions
Direction of Curve	Left	Primarily Tangent
Length of Curve	0.05 mi.	Primarily Level
Roadway Width	24.0 ft.	Roadside Conditions:
Shoulder Width	10.0 ft.	Rating = 26
Superelevation @ P. C.	0.042 percent	Pavement Conditions:
Maximum Superelevation	0.086 percent	Rating = 34
Rate of Superelevation		Traffic Volume Class:
Runoff	0.032	Low

SITE 206
U. S. 6, Sandusky County, Ohio

ROADWAY CHARACTERISTICS

Geometry		Environment
Curvature	5.3 degrees	Approach Conditions
Direction of Curve	Left	Primarily Tangent
Length of Curve	0.05 mi.	Primarily Level
Roadway Width	24.7 ft.	Roadside Conditions:
Shoulder Width	6.0 ft.	Rating = 31
Superelevation @ P. C.	0.049 percent	Pavement Conditions:
Maximum Superelevation	0.083 percent	Rating = 33
Rate of Superelevation		Traffic Volume Class:
Runoff	0.034	Medium

1 mi = 1.609km
1 ft = 0.305m

Figure 15. CHARACTERISTICS OF VEHICLE TRAVERSAL STUDY SITES (Continued)

SITE 212R
S. R. 39, Columbiana County, Ohio

ROADWAY CHARACTERISTICS

Geometry		Environment
Curvature	9.6 degrees	Approach Conditions
Direction of Curve	Right	Predominantly Curvilinear
Length of Curve	0.13 mi.	Predominantly Hilly
Roadway Width	23.4 ft.	Roadside Conditions:
Shoulder Width	2.5 ft.	Rating = 36
Superelevation @ P. C.	0.048 percent	Pavement Conditions:
Maximum Superelevation	0.085 percent	Rating = 20
Rate of Superelevation		Traffic Volume Class:
Runoff	0.064	Low

SITE 212L
S. R. 39, Columbiana County, Ohio

ROADWAY CHARACTERISTICS

Geometry		Environment
Curvature	9.6 degrees	Approach Conditions
Direction of Curve	Left	Predominantly Curvilinear
Length of Curve	0.13 mi.	Predominantly Hilly
Roadway Width	23.4 ft.	Roadside Conditions:
Shoulder Width	2.5 ft.	Rating = 36
Superelevation @ P. C.	0.048 percent	Pavement Conditions.
Maximum Superelevation	0.085 percent	Rating = 20
Rate of Superelevation		Traffic Volume Class:
Runoff	0.064	Low

1 mi = 1.609km
1 ft = 0.305m

Figure 15. CHARACTERISTICS OF VEHICLE TRAVERSAL STUDY SITES (Continued)

Field Procedure

Study Techniques

Extensive investigation and field trials were made to determine the most expeditious and efficient manner of conducting the vehicle traversal studies. The criteria used to evaluate potential study procedures included:

(1) A need to record large samples (100 to 200) of vehicle traversals within reasonable time and cost constraints;

(2) A requirement that the procedure minimize the possible effect on the behavior of the drivers being studied;

(3) A need to minimize the safety hazard to both the driving public and study crews;

(4) A requirement that the measurements taken allow for sufficient accuracy in characterizing speed and lateral placement of surveyed vehicles throughout their traversal of the curve.

Earlier studies of this type undertaken in Texas(33) had been accomplished using motion pictures taken from a following vehicle. Photographs of a vehicle's left rear tire when adjacent to standard-sized markers placed near the roadway centerline were used to measure lateral placement. Speed was determined by counting the number of elapsed frames between successive markers placed a set distance apart. This procedure was the first one evaluated in this research. Both motion picture and video cameras were tested. The main drawbacks were found to be the time required for surveyors to identify and close in behind a candidate vehicle; and the high speeds sometimes required when following the fastest drivers.

Various methods of measuring vehicle lateral placement and speed from a stationary position on the roadside were analyzed and tested. These procedures relied on either tape switches, still photography, or motion pictures. The method finally selected for this research used a stationary, high-speed motion picture camera located on the roadside opposite the traffic lane to be studied. As in the earlier Texas experiments, markers placed on the roadway served as references to measure vehicle lateral placement and speed. The survey procedures, described below, were all field tested and verified in the office prior to commencing the studies.

Site Layout Requirements

The basic study objectives defined the layout requirements. Because observations of speed and lateral placement as vehicles approached and transitioned into the curve were of interest, the course layout required centering roughly around the Point of Curvature (PC). Reference markers were required of sufficient number and spacing to enable calculation of transient speed and path behavior. The distance studied in advance of the curve and into the curve was restricted by limitations of the camera equipment used, and the amount of data that could be reduced in the office.

Site Layout and Measurement

The first task was to place and measure the exact location of white reflective tape markers that would serve as reference points in subsequent photographic observations.

A reflective road tape was placed on or adjacent to the roadway centerline at the PC. Other markers were then placed on or adjacent to the painted centerline at equal intervals of 25 feet (7.6 m) (or 20 feet (7.1 m) as dictated by conditions at one site) in advance of and beyond the PC. Wherever possible, the line of markers extended 175 feet (53.3 m) along the tangent approach and 225 feet (68.6 m) into the initial portion of the curve. These distances were sometimes shorter or longer to fit site conditions, but the length of marked tangent was never less than 80 feet (24.4 m) and the length of marked curve was never less than 175 feet (53.3 m).

When the centerline markers had been placed, companion markers were set opposite each centerline marker, at a fixed radial distance away from the edge of the traffic lane to be studied. These markers aided in selecting the appropriate frame to be measured when the motion pictures were analyzed in the office.

The location of the upstream right corner of each centerline marker was then determined using triangulation. The procedure consisted of establishing a baseline on the roadside and measuring angles with a surveyor's transit to each marker from each of two points on the baseline. These measurements allowed subsequent office analyses to determine the relative coordinates of each centerline marker.

In addition to the centerline and edge of pavement reference markers, a pair of remote markers was set about 500 feet (about 150 m) in advance of the PC on the tangent approach. These were used to study driver lateral position before entering the influence area of the curve.

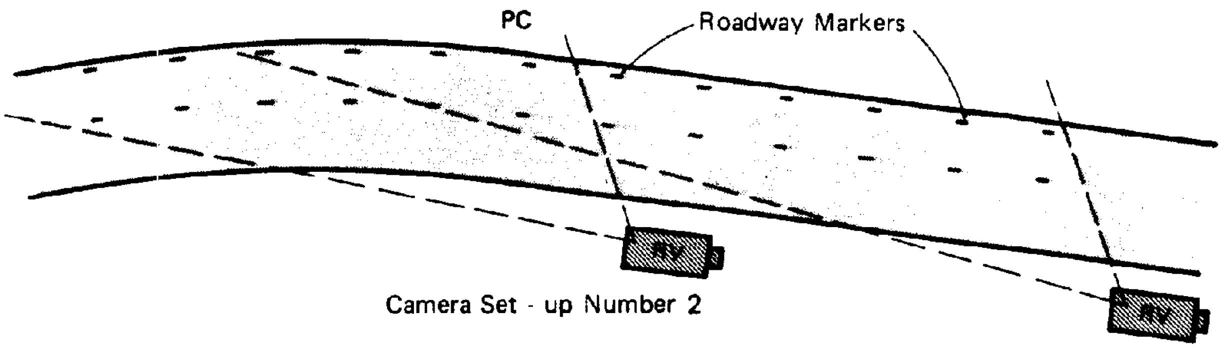
Finally, using a transit and level rod, the elevation of each centerline marker was determined from an assumed elevation on the baseline. The measured differences in elevation provided detailed data on superelevation, transition, and gradient.

Photography

Free moving, unopposed vehicles were filmed from a stationary camera mounted on the opposite side of the roadway. Figure 16 depicts a typical course set-up. The camera was hidden from view of the vehicles being filmed, as shown by Figure 17. A recreational vehicle (RV) was used to mount the camera, with the camera hidden by a frame-supported tarpaulin rigged to look like a luggage carrier on top of the vehicle. The RV was considered a common enough sight on the highway and afforded the height needed to gain photographic perspective. The overall appearance to oncoming traffic was that of a motor home parked off the opposite shoulder.

Film equipment, supplied by FHWA, was a Canon Scoopic 16-millimetre camera with a 12.5-75 mm (6:1) zoom lens. Film speed was 48 frames per second.

An observer inside the RV alerted the photographer as a vehicle approached. Only free flowing four-wheel vehicles, unaffected by traffic in the same or opposing direction, were to be filmed. Therefore, photographs were taken only after the observer reported a lone four-wheel vehicle approaching and the photographer observed no opposing vehicles. Filming began as the vehicle approached the first centerline marker of the course being studied. The photographer then followed the vehicle through the course, carefully keeping both the centerline and edge of pavement markers in the camera's field of view.



Camera Set - up Number 2

Camera Set - up Number 1

Typical View From Camera on RV

Figure 16. TYPICAL SET-UP FOR VEHICLE TRAVERSAL STUDIES

Figure 17. RECREATIONAL VEHICLE USED TO FILM VEHICLE TRAVERSALS

To enable office measurement of lateral offset at each centerline marker, a calibration scale was held perpendicular to the roadway centerline at each marker and photographed from the stationary RV. (See Figure 18.) Since the camera remained in a fixed postion, the photographed scale could be used in later analysis to accurately calibrate the nondimensional photographed offset to an actual dimension at each reference point.

Because the course was usually too long to photograph a vehicle traveling the entire length from just one position, two separate camera set-ups were used at most sites. The first set-up was upstream from the tangent centerline marker located farthest from the PC. For the second set-up, the camera was moved toward the PC about 150 to 200 feet (about 45 to 60 m). An overlap of about 100 feet (about 30 m) between the first and second set-ups was always included in the photography. From 100 to 150 vehicle traversals were photographed at each set-up.

In addition, still 35-mm slides were taken of a representative sample of surveyed vehicles as they passed the remote markers approximately 500 feet (150 m) upstream of the PC. The calibration scale was also photographed at these markers to allow measurement of lateral offset.

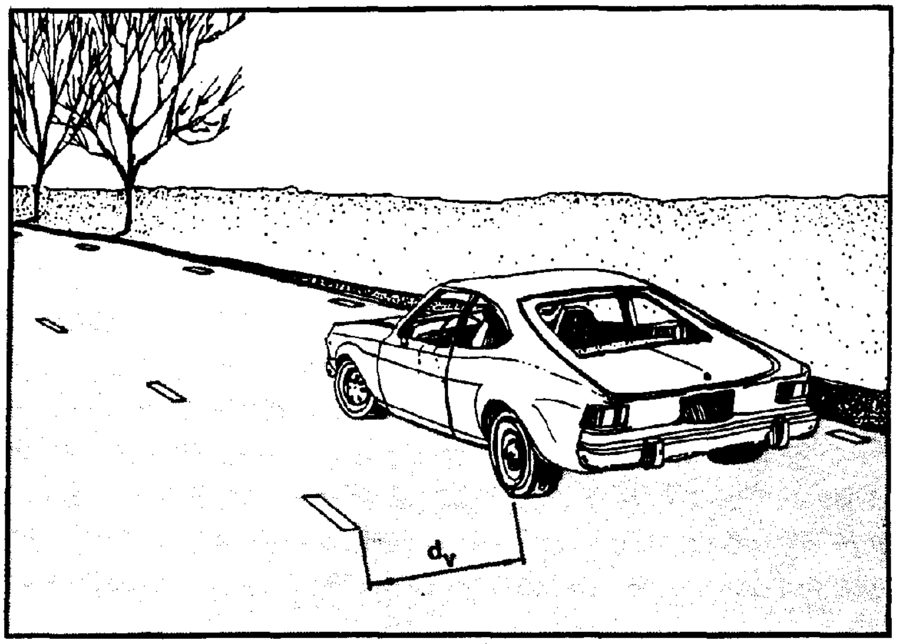
Data Reduction

Given the research objectives and the desirability of limiting the data reduction effort, it was found most efficient to reduce the data in two phases. In phase one, lateral placement and frame count readings for the beginning and ending points on the course were taken for all vehicles at each set-up. This enabled recording of data to describe the following:

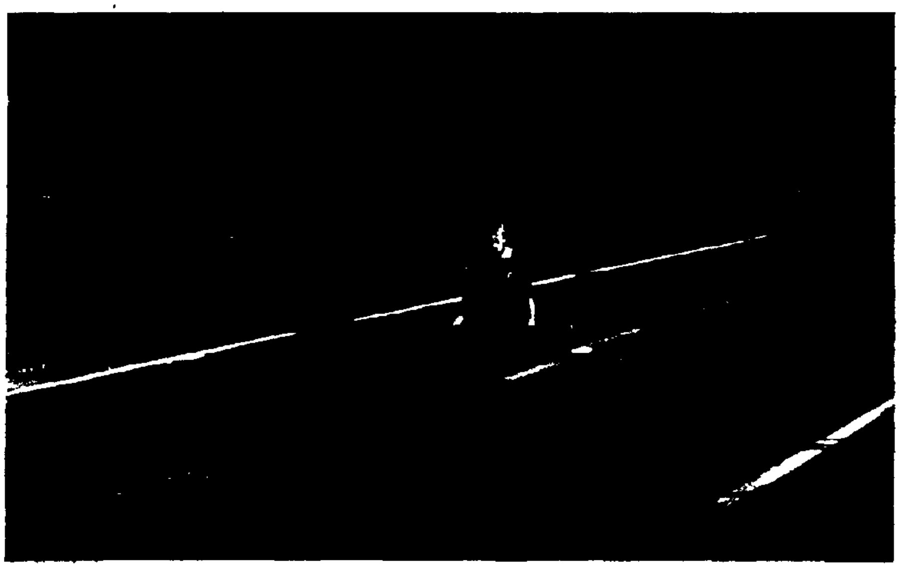
(1) Speed distribution (average speeds over the set-up length)

(2) Overall lateral movement through the course

(3) Erratic behavior (encroaching on opposing lanes or shoulder; braking; excessive speeds)



Schematic of Vehicle Opposite Markers



Calibration Picture of Scale

$$O_A = \text{ACTUAL OFFSET} = \left(\frac{d_v}{d_s}\right) L_s$$

Where d_s and d_v are scaled readings from film
and L_s is actual length of scale

Figure 18. CALIBRATION OF FILMED READINGS OF VEHICLE PLACEMENT

The second phase used distributional data from phase one to identify specific vehicle samples for more complete study. The vehicles of greatest interest were those thought to produce the most extreme behavior, defined by high levels of lateral acceleration. Because these levels could be produced by either high speeds or small path radii, two subsets of vehicles were identified for the phase two analysis. One subset was comprised of vehicles with the highest speeds; the other included those vehicles with the greatest lateral movement from the first to last point on the course. The latter subset would presumably reveal severe path radii.

For each site, vehicles comprising the highest 20 percent of the speed and lateral placement distributions were selected for phase two analysis. Because some overlap between subsets was expected, this sample was estimated to include roughly one-third of the total number of vehicles surveyed. The sample was selected to ensure identification of the top 10 percent of the lateral acceleration distribution with a high degree of confidence.

To measure the variability of lateral acceleration, speed, and path for each site, two small additional subsets were also identified for complete analysis. These two were comprised of about 10 to 15 vehicles with median speeds, and 10 to 15 vehicles with median lateral movements.

The second phase of data reduction was performed for each vehicle in the four subsets, which are identified as 'high speed,' 'high offset,' 'median speed' and 'median offset.' Data reduction for these vehicles included scaled offset readings and frame counts for the left rear tire opposite every coordinated pavement marker.

Data Reduction Procedures

Motion pictures of the vehicle traversals were analyzed in the office using a 16 mm stop-action motion analyzer. Pictures were projected onto an analysis board and advanced until the left rear tire of the vehicle was opposite the first pair of pavement markers. A straight edge extending between the centerline and companion roadside marker was used to aid the technician in aligning the tire with the centerline marker. The distance between the outside edge of tire and the right upstream corner of the marker was scaled and recorded. The frame count was also recorded before proceeding to the next reading. The readings for the filmed calibration scale at each course point enabled conversion of scaled data to real dimensions.

The important characteristics of driver/vehicle path behavior include its general form, the amount of similarity in behavior among drivers, the effects of the roadway geometry on this behavior, and driver/vehicle path relationships to speed. The data collected in phase two were of sufficient scope and detail to study these characteristics. This was accomplished by developing behavior profiles for each vehicle.

Driver/Vehicle Profiles of Behavior on Curves

Profiles of driver/vehicle behavior describe the way in which vehicles adjust their path and speed relative to the roadway. Figure 19 illustrates the output of such a profile from offset and frame count readings. The following procedure was used:

(1) The course centerline markers were coordinated from field survey data.

(2) A "best-fit" center of curve described by the coordinated points was calculated.

(3) Vehicle path coordinates for each vehicle opposite each point on the course were calculated off the coordinated markers as shown in Figure 20. Film readings of the offset were used for these calculations.

(4) Local vehicle path radii were calculated for each point N. The procedure used the principle that any three points uniquely define a circular arc (See Figure 21). The vehicle path radius at point N was computed from vehicle path coordinates for points (N - 2), N, and (N + 2)*. The calculation was performed every 25 feet (7.6 m) by "stepping" around the curve.

*Sensitivity of error analyses indicated the need to use these points, rather than points (N-1), N, and (N+1). Errors in surveying the course, film reading and data reduction are thus distributed over an approximately 100 foot (30.5 m) vehicle arc, rather than a 50-foot (15.2 m) arc which would result from using consecutive points.

140-1 FHWA

VEHICLE OPERATING CHARACTERISTICS

RUN DATE 11/5/82

SITE 212 LA.

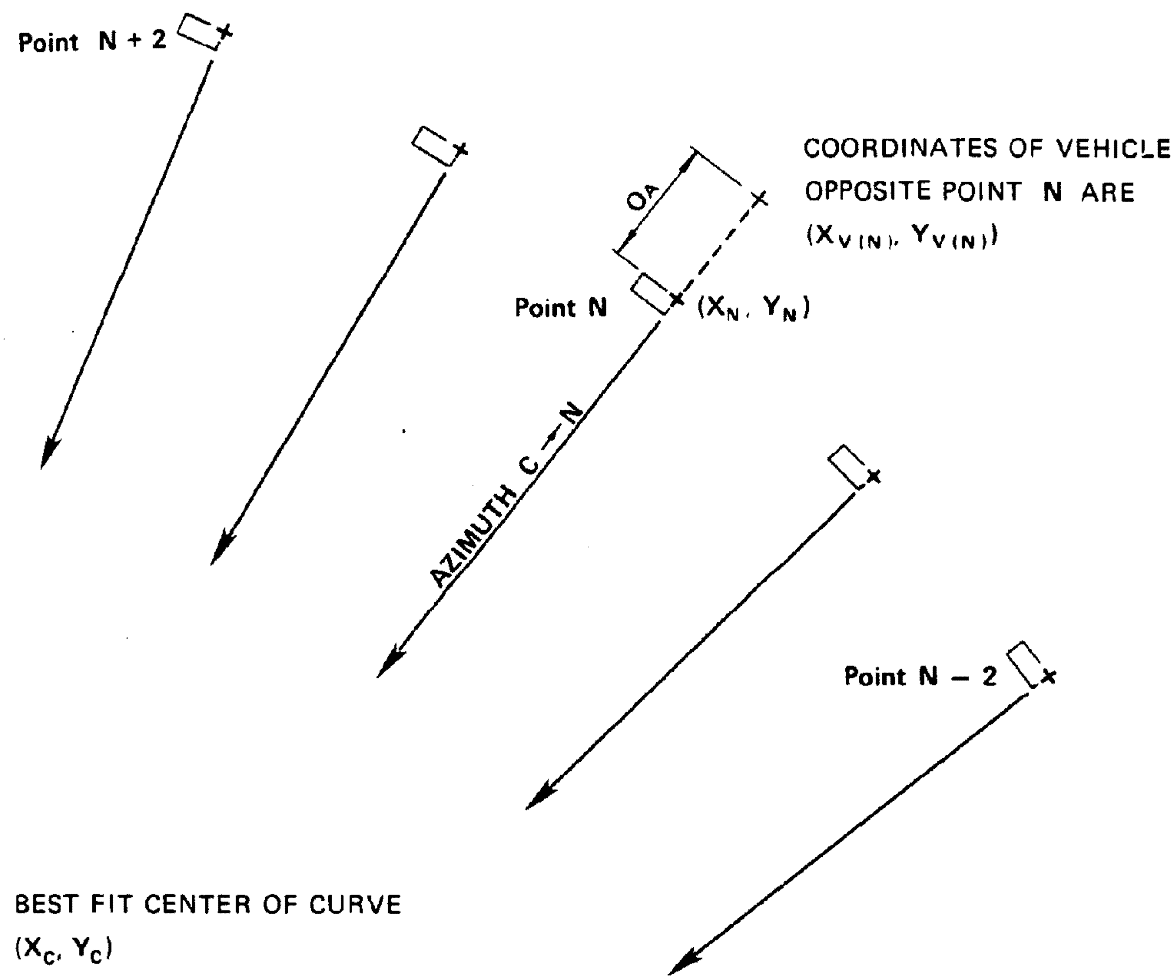
VEHICLE 41

DATA ERRORS

POINT NUMBER	CENTERLINE OFFSET (FEET)	VEHICLE SPEED (MPH)	EFFECTIVE VEHICLE RADIUS-FT	SUPER-ELEV AT POINT	FRICTION FACTOR
1	3.408	0	0	-0.003	0
2	3.745	0	0	-0.002	0
3	3.824	53.8	7490.	0.007	0.019
4	4.003	54.0	5909.	0.010	0.023
5	4.192	54.0	2737.	0.025	0.046
6	4.113	54.0	2293.	0.032	0.053
7	3.788	53.1	1621.	0.043	0.073
8 F1	3.449	53.0	1205.	0.053	0.103
9	2.760	51.3	826.	0.063	0.149
10	2.794	50.5	588.	0.080	0.209
11	2.104	51.3	807.	0.090	0.127
12	1.882	49.7	904.	0.088	0.094
13	2.296	0	0	0.088	0
14	2.106	0	0	0.086	0

1ft = 0.305m
1mph = 1.609km/h

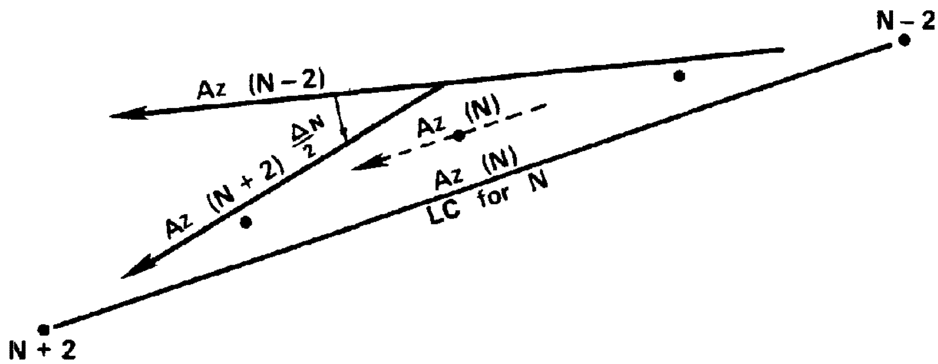
Figure 19. EXAMPLE OF DRIVER/VEHICLE PROFILE



$(X_{V(N)}, Y_{V(N)})$ are calculated by traversing from (X_N, Y_N) using azimuth C→N and actual offset O_A.

$(X_{V(N)}, Y_{V(N)})$ opposite points on tangent are calculated based on ±90° from azimuth of tangent.

Figure 20. CALCULATION OF VEHICLE PATH COORDINATES



Az (N) = Local azimuth of vehicle at point N, calculated by inversing coordinates of vehicle at points N – 2 and N + 2

Δ_N = Central angle of curve defined by points N – 2, N and N + 2

R_N = Local radius of curve for vehicle at point N

LC_N = Long chord of curve for vehicle at point N, calculated by inversing coordinates of vehicle at points N – 2 and N + 2

$$LC_N = 2 R_N \sin \frac{\Delta_N}{2}$$

Solving for R_N :

$$R_N = \frac{LC_N}{2 \sin \frac{\Delta_N}{2}}$$

Figure 21. CALCULATION OF INSTANTANEOUS VEHICLE PATH RADII

(5) The vehicle speed at point N was based on the difference in frame counts between points (N - 2) and (N + 2) (See Figure 22).

(6) Side friction generated at point N was calculated using the centripetal force equation, superelevation readings from the field, and the previously calculated speed and path radius values for point N.

The actual calculations were performed by programs written for the Apple II computer. Appendix E lists and discusses these programs.

Results of Vehicle Traversal Studies

Data describing how drivers approach and proceed through a horizontal curve revealed a number of significant findings that have implications with respect to design criteria and safety of horizontal curves.

General Characteristics of Vehicles Traversing Curves

Initial analysis of all free moving, unopposed vehicles showed significant general patterns in driver/vehicle behavior. Table 29 summarizes speed and lateral placement data for the six curve sites studied. The mean speed for all sites was about 50 to 55 mph (80 to 89 km/h). Significant individual vehicle speed reductions did not occur except for the sharpest curves. Mean lateral placement (the measured distance from the centerline marker to the left rear tire) was generally 3 to 4 feet (0.9 to 1.2 m) as the vehicles approached each curve, followed by a noticeable drifting from that position toward the inside of the curve. Thus, as Table 29 indicates, vehicles moved toward the centerline of left curves, and toward the shoulder of right curves. Mean lateral movement was as great as 2.8 feet (0.8 m). The increased standard deviation of lateral placement in the curve itself is also significant. This indicates greater variability in vehicle behavior in the curve as compared with that on the tangent.

Deviant Behavior.--As a part of the general characterization of vehicle behavior, the film data were studied for measures of deviant or undesirable operational behavior. This was defined as braking, encroaching on either the centerline or shoulder, or exceeding the apparent design speed of the curve by 10 mph (16 km/h) or more. Table 30 summarizes all such undesirable behavior.

$$\text{AVERAGE VEHICLE SPEED BETWEEN ANY TWO POINTS } N-2 \text{ AND } N-2 \text{ (Miles per hour)} = \underset{\text{(Frames per second)}}{FS} \div \underset{\text{(No. Frames)}}{\Delta FC} \times \underset{\text{(Feet)}}{LC} \times \underset{\text{(Seconds per hour)}}{3600} \div \underset{\text{(Feet per mile)}}{5280}$$

Where

FS	=	Film Speed (48 Frames per second)
ΔFC	=	Difference in Frame Counts Between Points N−2 and N + 2
LC	=	Long Chord For N (See Figure 21)

For Film Speed of 48 frames per second

$$\text{AVERAGE VEHICLE SPEED (miles per hour)} = 32.73 \times LC \div \Delta FC$$

1ft = 0.305m
1mph = 1.609km/h

Figure 22. CALCULATION OF VEHICLE SPEED FROM FILM DATA

TABLE 29

SUMMARY OF SPEED AND LATERAL PLACEMENT FOR
ALL VEHICLES OBSERVED

Site No.	Radius of Curve[1] (feet)	Speeds[2] at (mph) Tangent Approach	Speeds[2] at (mph) PC of Curve	Speeds[2] at (mph) Curve	Lateral Placement[3] at (feet) Tangent Approach	Lateral Placement[3] at (feet) PC of Curve	Lateral Placement[3] at (feet) Curve
Left Curves							
188	1516 (3.8°)	-	53.8(6.4)	60.8(6.0)	3.0(0.7)	1.5(0.8)	2.2(0.9)
206	1089 (5.3°)	56.9	52.8(5.7)	50.4(5.7)	3.1(1.1)	3.2(1.1)	0.4(1.7)
204	912 (6.3°)	52.3	52.3(6.0)	51.1(6.1)	4.0(1.0)	2.6(0.9)	1.2(1.6)
212	595 (9.6°)	52.1	46.0(7.4)		2.9(1.1)	-	3.3(1.3)
Right Curves							
212	595 (9.6°)	50.4	46.0(7.4)		2.9(1.1)	-	3.3(1.3)
198	444 (12.9°)	53.7	45.5(7.3)		2.2(1.0)	-	3.3(1.1)

[1] Number in parentheses represents Degree of Curve (English 100-foot arc definition)

[2] Speed shown is mean speed; number in parentheses represents standard deviation of speed distribution

[3] Lateral placement shown is the distance between the left rear tire and centerline marker; number in parentheses represents standard deviation of distribution of offsets

1 ft = 0.305 m
1 mph = 1.609 km/h

TABLE 30

SUMMARY OF UNDESIRABLE BEHAVIOR
FOR ALL VEHICLES OBSERVED

Site No.	Radius of Curve (feet)	Design Speed[1] (mph)	Number of Vehicles Observed	Proportion of Vehicles Observed: Braking	Proportion of Vehicles Observed: Encroaching	Proportion of Vehicles Observed: Exceeding Design Speed by > 10 mph
Left Curves						
188	1516	64	135	0.7%	14.8%	28.9%
206	1089	54	150	20.7%	21.3%	8.7%
204	912	56	138	1.4%	28.3%	4.3%
212	595	46	108	0.0%	11.1%	13.0%
Right Curves						
212	595	29	126	6.4%	10.3%	1.6%
198	444	46	87	35.6%	11.4%	78.2%

[1]Design speed is speed of equivalent AASHTO controlling curve

1 ft = 0.305 m
1 mph = 1.609 km/h

What is interesting about Table 30 is the apparent lack of consistency between these measures of undesirable behavior and curvature. At least 10 percent of vehicles were found to have encroached on either the centerline or shoulder at all sites. Excessive speeds were noted for the mildest curves as well as the sharpest. Drivers seemed to be negotiating all curves significantly different from implicit design assumptions of uniform speed and placement.

Driver/Vehicle Transitioning.--The general information presented in Tables 29 and 30 indicates a basic characteristic of driver behavior. Typical drivers do not center themselves in the lane as their vehicles move from the tangent into the curve. Instead, they seem to drift or slowly "spiral" into the curve, gradually adjusting their vehicles paths to match the roadway curvature. Figure 23 schematically depicts this behavior, which is significantly different from the designed path of the highway. To explicitly follow the designed path requires the vehicle to instantaneously change from the tangent path to the highway curve path. The initial data findings indicate this does not actually occur, partly because it is physically impossible to change vehicle radius instantaneously, and partly because drivers apparently desire to do otherwise.

Analyses were made of curve traversals using four subsets of vehicle behavior -- high speed, median speed, high offset and median offset. Vehicles were grouped in their respective subset(s) and composite profiles were computed. These were based simply on arithmetic averages of lateral placement and frame count.

Graphic analyses were also performed to determine the driver/vehicle behavior pattern of the high speed and median speed composite groups at the six sites where field surveys were made. The calculated offset from centerline markers was plotted on a 1:120 scale plan of each curve. Combinations of spiral and circular curves with various radii were then visually fitted to form a smooth path for both the composite high speed and composite median speed driver/vehicle.

The field surveys only produced average offset and speed for a segment of tangent in advance of the curve PC and a short section of curve beyond the PC. In an effort to better understand driver/vehicle behavior throughout the entire length of curve, this initial driver/vehicle behavior was graphically

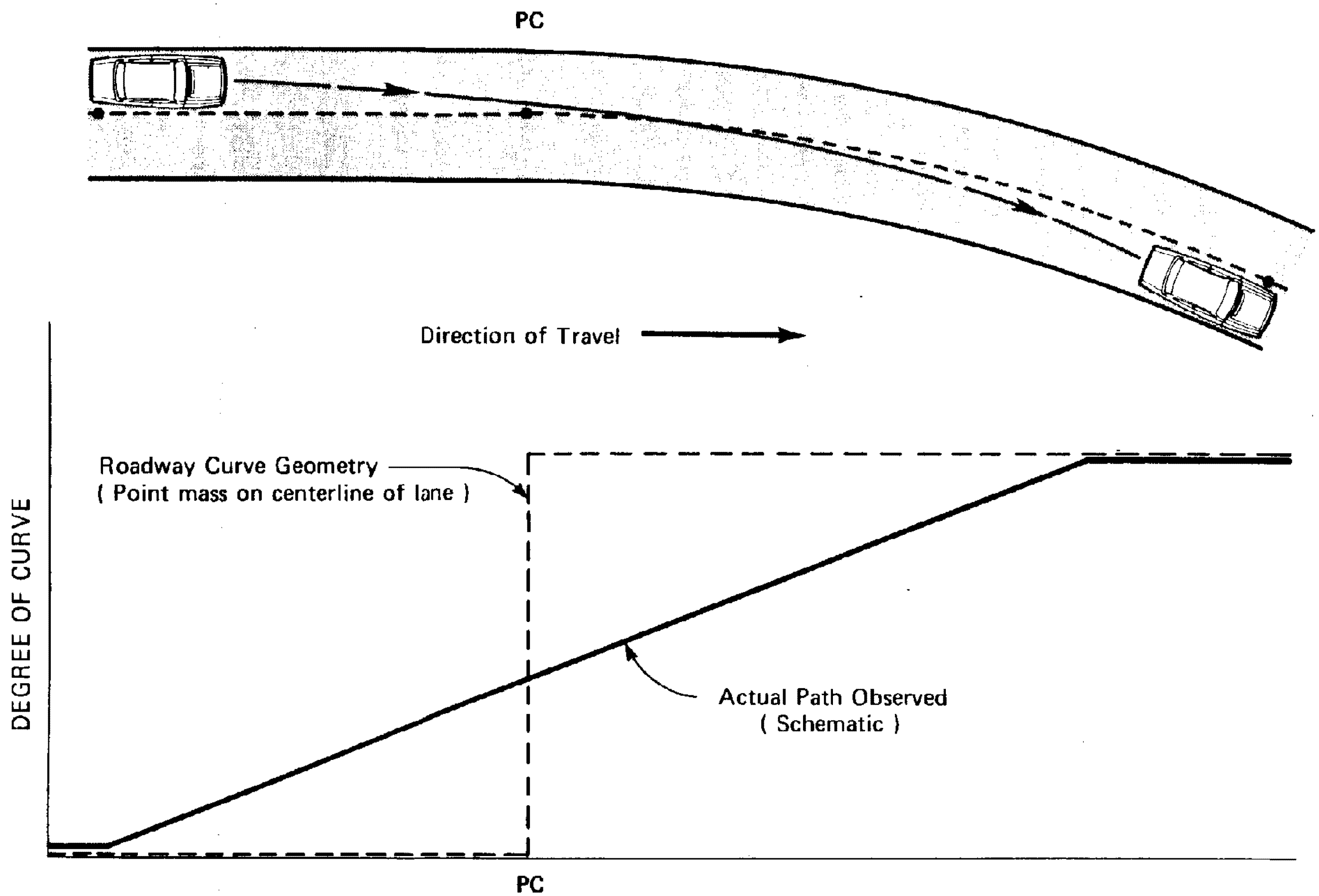


Figure 23. COMPARISON OF OBSERVED, SCHEMATIC PATH WITH CENTERLINE CURVE GEOMETRY

extrapolated beyond the area where average lateral offset and speed could be calculated. While this extrapolated path is believed to be representative of average driver/vehicle behavior, it acknowledgably is subject to refinement or modification whenever more extensive field studies are made.

Analyses of composite driver/vehicle behavior profiles indicated that, except on very short curves, drivers apparently spiral into the curve until reaching a radius equal to or slightly less than that of the roadway. The path then becomes approximately circular and concentric with the roadway until the driver senses an upcoming tangent at the end of the curve and begins spiraling out. On very short curves, the path appears to be one of spiral in and spiral out without the central circular arc as on longer curves. The maximum curvature reached on very short curves would also be less than that of the roadway.

The length of roadway over which the composite driver transitioned from tangent to full curvature was found to be in the general range of 250 to 300 feet (75 to 90 m) regardless of degree or length of curve. About one-half of the spiral path occurred on the tangent in advance of the PC. The remainder of the spiral transition was accomplished on the curve beyond the PC. At the PC, therefore, the composite vehicle had already achieved approximately one-half of the roadway curvature (0.5 D_c).

The extrapolated driver/vehicle behavior when leaving the curve indicated characteristics similar to those found when entering, except that the total spiral path would be shorter -- in the range of about 150 to 250 feet (about 45 to 75 m). Again, however, about one-half of the transition would have occurred in advance of the PT, and the remainder on tangent beyond the PT.

Analyses of composite driver/vehicle profiles also confirmed that drivers make maximum use of the available roadway to produce the smoothest possible transition. The lateral placement of the composite vehicle on the tangent approach was found to be nearly the same for both right-hand and left-hand curves (about 2.5 to 3.5 feet (0.8 to 1.1 m) right of the roadway centerline). On right-hand curves, however, the path through the curve drifted to the right until the right-hand tires were only about 1.0 to 1.5 feet (0.3 to 0.5 m) from the edge of roadway. Similarly, on left-hand curves, composite driver/vehicle profiles indicated a path coming within 1.5 feet (0.5 m) or less of the centerline. An example analysis of composite driver behavior is shown in Figure 24. The figure summarizes the spiraling behavior of the high speed drivers for Site 198.

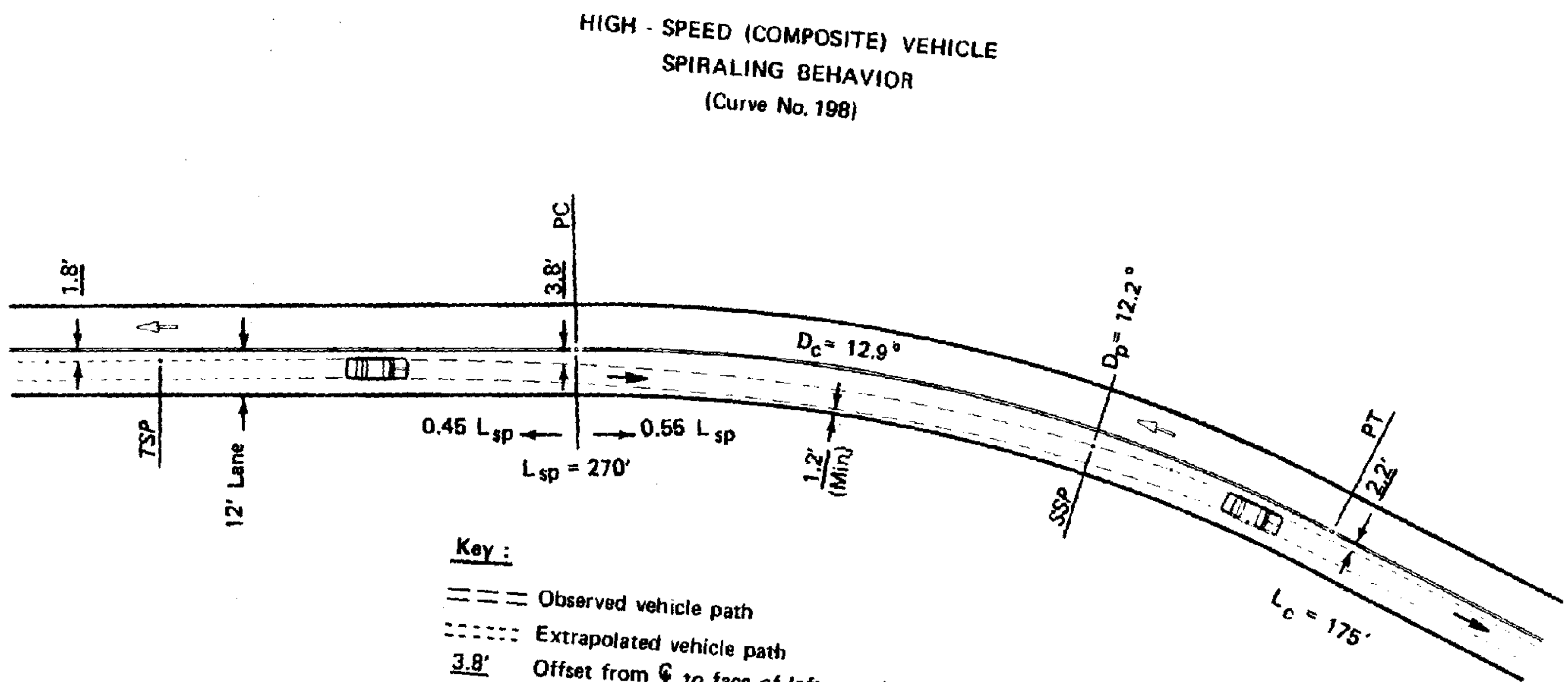
Significant findings which can be rationalized from analyses of composite driver/vehicle behavior profiles are as follows:

- The length of curve over which drivers track the curvature (D_c) of the roadway is less than the total length of curve (L_c) due to spiral path transitions at the beginning and end of the curve.
- On short curves, drivers do not reach a degree of curvature as great as that of the roadway.
- At both the PC and PT, path curvature (D_p) is approximately one-half of the roadway curvature (D_c).
- The length of transition path entering a curve is approximately 250 to 300 feet (about 75 to 90 m). The transition path when leaving the curve appears to be slightly shorter.

Summary of General Characteristics.--Drivers approach and transition into curves with a range of path transitions and speed change behavior. Significant numbers of vehicles encroach on opposing lanes or shoulders, or apply brakes as they traverse the curve. It is apparent that speed, path, and highway curvature all combine to characterize driver behavior. Analysis of composite profiles showed some differences in behavior of vehicles grouped by speed and path parameters.

Extreme Driver Behavior on Highway Curves

Cursory review of the range of vehicle profiles revealed highly variable behavior. Initial efforts at studying this range of behavior involved development of "composite" or average vehicle profiles, based on the four subsets of vehicles defined by speed and offset. While these composite profiles provide some indication of path and speed relationships, they do not fully describe individual driver/vehicle behavior. Further analyses were performed, therefore, to study the distributions of path, speed, and their relationship for each highway curve.



Key :

═ ═ ═	Observed vehicle path
∷∷∷	Extrapolated vehicle path
3.8'	Offset from ℄ to face of left rear tire
PC	Begin highway curvature
PT	End highway curvature
TSP	Begin entering vehicle spiral
SSP	End entering vehicle spiral, begin exiting vehicle spiral

Note: 1ft = 0.305m

Figure 24. EXAMPLE OF COMPOSITE DRIVER ANALYSIS OF SPIRALING BEHAVIOR

A basic design principle governed the study of the distributions of driver/vehicle behavior. This principle concerns the level of operational service to be provided in relation to the demands or desires of drivers. A well recognized principle is that proper design should provide for more than the average driver. Economic concerns dictate, however, that design should not accommodate the most severe demands. In terms of design for highway curvature, the roadway should enable almost all drivers to successfully negotiate the curve under nearly all conditions. To the extent that driver behavior in negotiating curves is variable, it becomes essential to measure that variation.

Further study focus was thus directed toward the extreme driver, who produces behavior significantly worse than average. What was needed was an appropriate measure of extreme behavior on highway curves. Because curve operations involve both speed and cornering, this should be an explicit measure of behavior, combining speed and vehicle path.

Clearly, the single best descriptor of driver/vehicle behavior is lateral acceleration. Lateral acceleration is felt by the driver, and manifested at the four tires, where it becomes a direct factor in the degree of control or stability of the vehicle. Highway curve design is directly linked to lateral acceleration. Design controls selected by AASHTO and summarized in Table 31 express allowable maximum side friction coefficient for a given design speed. These controls presumably correspond to levels of lateral acceleration that are tolerable to a reasonable driver. They also assume a margin of safety from generally available pavement friction levels under wet pavement conditions.

The studies of actual vehicle behavior provided an opportunity to observe extreme behavior expressed in terms of lateral acceleration. Distributions at each location, and comparisons of this behavior across all conditions studied were possible.

TABLE 31

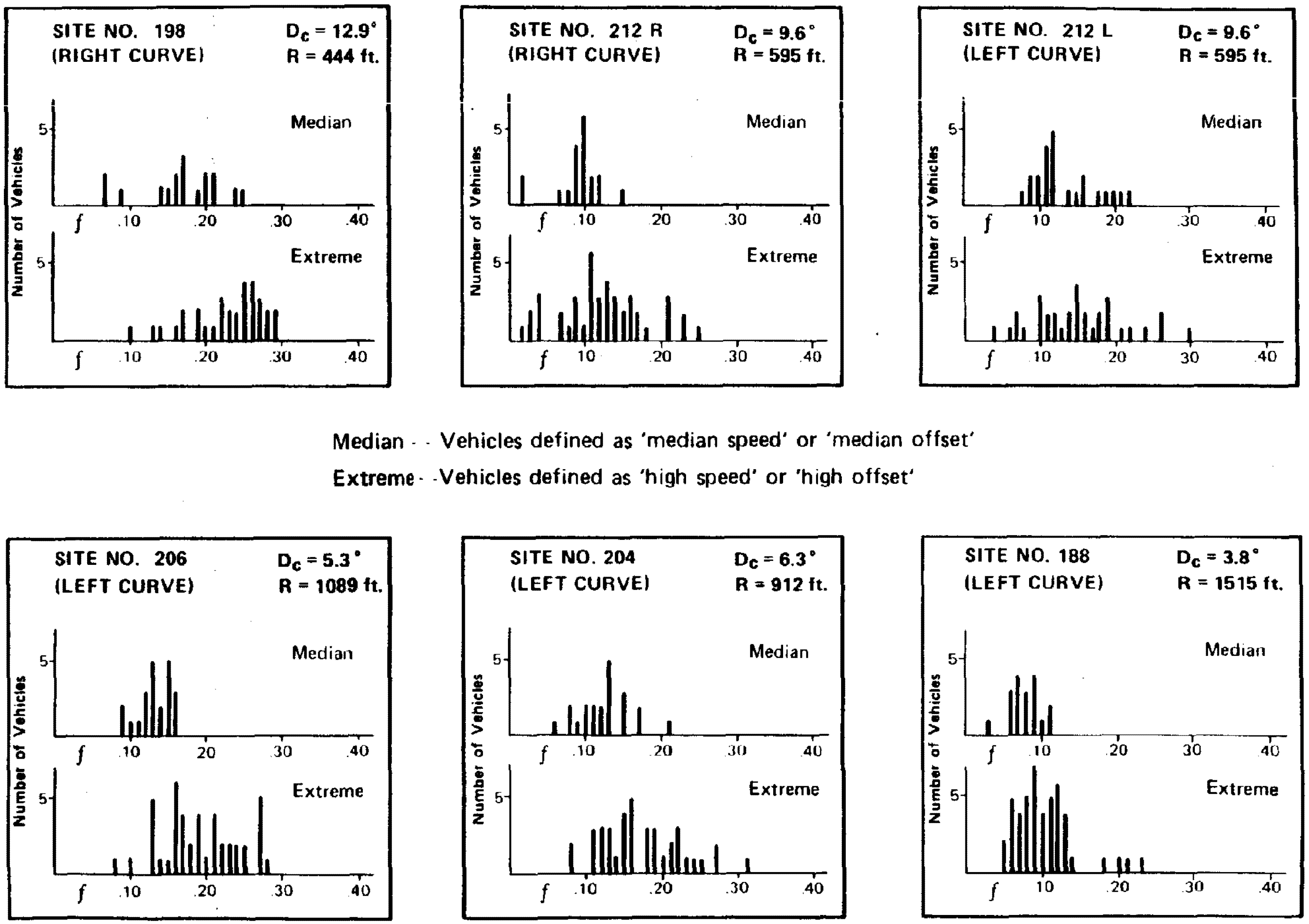
FRICTION FACTORS RECOMMENDED
FOR DESIGN OF HIGHWAY CURVES

Design Speed (mph)	Friction Factor
80	0.08
70	0.10
60	0.12
50	0.14
40	0.15
30	0.16

Source: Reference (32)
1 mph = 1.609 km/h

The selection of vehicles for complete study was intended to characterize a portion of the distribution of lateral accelerations. Of interest in terms of extreme behavior were those vehicles which produced high levels of lateral acceleration (expressed in terms of f), as well as those which produced average or typical levels of f. For each site, the profiles of all high speed and high offset vehicles were examined, and the maximum f values obtained. Histogram plots were prepared. These are summarized in Figure 25. The extreme (high speed and/or high offset) vehicles represented approximately the top 30 percent of all vehicles observed in terms of maximum f developed.

Thus, the histogram of f for these vehicles actually represents the tail of the cumulative frequency distribution of maximum f for all vehicles at that site. Similarly, the median speed and median offset vehicle histograms provide a reasonable estimate of median or 50th percentile maximum f for each site. Figure 26 illustrates how these subsets of vehicles were used to synthesize the upper (critical) portion of the cumulative frequency curve of f for each site.



1ft = 0.305m

Figure 25. FREQUENCY DISTRIBUTIONS OF MAXIMUM SIDE FRICTION DEMAND (f) OBSERVED AT STUDY SITES

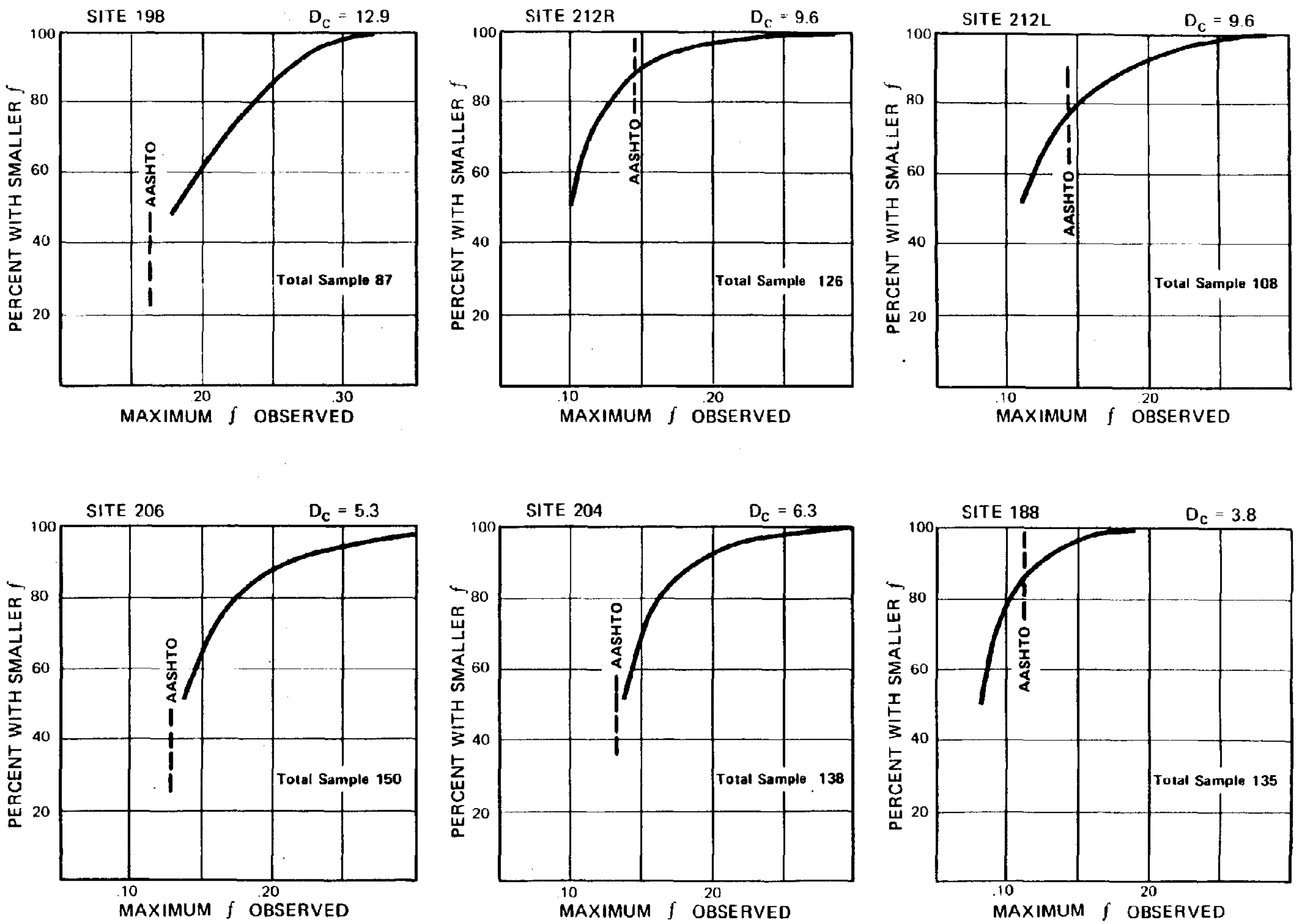


Figure 26. CUMULATIVE FREQUENCY DISTRIBUTIONS OF f SYNTHESIZED FROM OBSERVATIONS OF EXTREME AND MEDIAN VEHICLES

Figure 26 reveals that a significant proportion of vehicles exceeded the design f at all sites. It is noteworthy that excessive f levels are not restricted to just the sharpest curves. If AASHTO controls for curvature design are proper in terms of safety, important questions arise given that many drivers choose to exceed them.

How Extreme Driver Behavior Occurs

The following questions address important aspects of driver behavior on curves:

- How well do drivers track the geometry of the highway curve?
- Do higher speed vehicles tend to track sharper curve paths than lower speed vehicles?
- Do higher speed vehicles spiral into the curve at faster rates than lower speed vehicles?
- Does the spiral rate selected by a driver have an effect on the sharpness of the vehicle's path?

These questions concern relationships between speed and path radius at the point of maximum lateral acceleration. Hereafter, the term **critical path radius** is used to refer to the radius developed by the vehicle at the point of maximum f.

The vehicle profiles were used to quantify and study relationships involving critical path radius. For each vehicle studied, critical path radius was identified and recorded. The vehicle speed at the point of critical radius was also recorded. These data were analyzed and are discussed below.

Relationship Between Curve Geometry and Critical Path Radius.--
At five of the six sites studied, a majority of vehicles were found to be generating path radii sharper than that of the curve. Table 32 summarizes these findings.

TABLE 32

COMPARISON OF CRITICAL PATH RADIUS
AND HIGHWAY CURVE RADIUS FOR
VEHICLES GROUPED BY SPEED

Site	Radius of Highway Curve (Feet)	Vehicles with Critical Path Radii Sharper than Highway Curve Radius: All Vehicles	Vehicles with Critical Path Radii Sharper than Highway Curve Radius: High Speed Vehicles
Left Curves			
188	1516	58% [38/65]	57% [16/28]
206	1089	97% [59/61]	100% [29/29]
204	912	57% [34/60]	58% [14/24]
212	595	55% [22/40]	50% [8/16]
Right Curves			
212	595	28% [17/60]	28% [5/18]
198	444	2% [1/41]	0% [0/19]

1 ft = 0.305 m

Two points are significant. First, the sharpness of highway curvature does not seem to explain this type of behavior. Drivers apparently "overshoot" the highway curvature regardless of the degree of curve. Second, speed does not appear to explain why this overshoot occurs.

Site 198, a sharp and short curve, exhibited markedly different behavior. Most vehicles tracked minimum radii <u>less severe</u> than that of the highway. As will be seen in further analyses, driver/vehicle behavior at this site proved to be significantly different than the other locations.

Relationship Between Speed and Critical Path Radius.--Although speed did not seem to explain the propensity to overshoot a curve, it might explain severity of the overshoot. To test this hypothesis, simple linear regression analyses were performed for each site using the vehicle speed and critical path radius data.

Analyses were performed for all data together, as well as for data grouped by individual subsets (high speed, median speed). At all but one site, no relationship was found between speed and critical path radius. As Table 33 indicates, both the slopes of the regression lines and the R^2 values show the two variables to be independent. Again, Site 198 proved to be the lone exception. While the relationship is not strong, it was found that higher speed vehicles tracked less severe paths then lower speed vehicles at this site.

TABLE 33

RELATIONSHIP BETWEEN SPEED AND CRITICAL PATH RADIUS

		Regression Statistics*				
Site	Radius of Highway Curve (Feet)	Slope a	Intercept b	R^2	Std. Error	Signif. at α = 0.10?
Left Curves						
188	1516	0.0037	53	0.03	4.15	No
206	1089	-0.0086	61	0.06	5.15	No
204	912	0.0007	54	<0.01	5.13	No
212L	595	0.0100	40	0.02	4.89	No
Right Curves						
212R	595	0.0228	30	0.05	5.58	No
198	444	0.0530	18	0.27	5.47	Yes

* Critical Path Radius = a (Speed in mph) + b

1 ft = 0.305 m
1 mph = 1.609 km/h

Relationship Between Speed and Spiral Rate.--Figure 27 shows a plot of an individual vehicle's path profile as the vehicle transitions into the curve. What is shown is the instantaneous computed radius (and degree of curve) of the vehicle at each point of the course. Inspection of other similar plots for vehicles at each site showed a range of curve transition behavior.

Both the total change in vehicle curvature and distance over which the change occurs are parameters of interest in characterizing path behavior. These two parameters can be combined into a single measure of such behavior in the following manner:

call R_s = **Spiral Rate** for the vehicle

then, $R_s = L \div D_c$

where L is a length of highway over which the vehicle's curvature changes significantly;

and D_c is the change in the vehicle's curvature over distance L.

To analyze spiraling rates (R_s), profile plots were made similar to the one in Figure 27 for all vehicles. A number of different measures of R_s were tested. Ideally, characterization of a spiral rate would include as much of the vehicle's transition from tangent to final curvature as possible. However, four of the six sites required two set-ups, with the second set-up beginning near the PC. This precluded observing any one vehicle throughout its entire transition at these sites. At one site with a single set-up, a range of possible spiral rate definitions was tested to determine the best one, given the limitations of the data. The definition which was chosen, shown in Figure 28, is based on the length of highway over which the vehicle spirals from 50 percent of the highway curvature to 90 percent of the highway curvature. For all but one site, vehicles filmed in the second set-up could be observed achieving this transition. Therefore, adequate spiral rate observations and overshoot measurements could be made for the same vehicles.

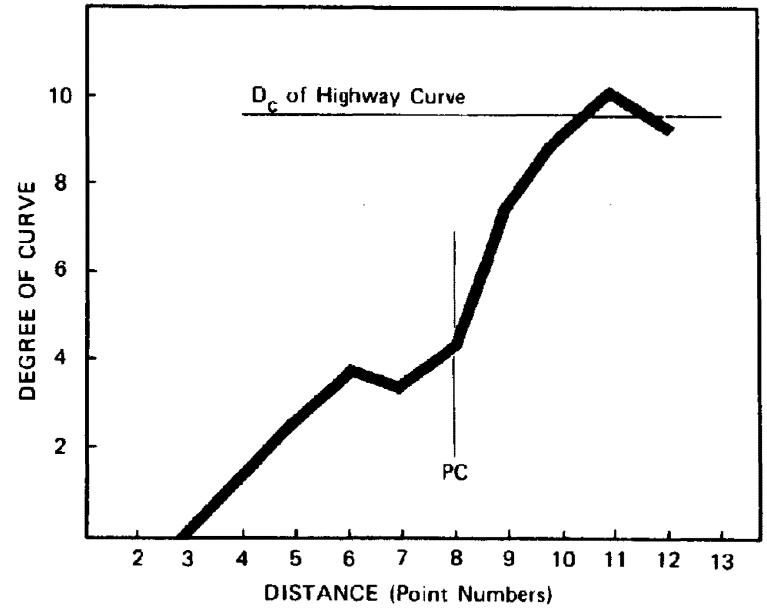
Initial interest focused on any relationships between a vehicle's speed and its spiral rate. Simple linear regression amd scatter-plot analyses were performed to test for such a relationship at each site. No correlation was found,

190-1 FHWA | SITE 212LB
VEHICLE OPERATING CHARACTERISTICS
RUN DATE 11/9/82 | VEHICLE 26

DATA ERRORS

POINT NUMBER	CENTERLINE OFFSET (FEET)	VEHICLE SPEED (MPH)	EFFECTIVE VEHICLE RADIUS-FT	SUPER-ELEV AT POINT	FRICTION FACTOR	DEGREE OF CURVE
1	3.129	0	0	-0.003	0	
2	3.174	0	0	-0.002	0	
3	3.352	46.2	134146	0.007	-0.006	0.04
4	3.765	47.6	4202.	0.010	0.026	1.36
5	3.927	46.3	2249.	0.025	0.039	2.55
6	4.006	45.1	1502	0.032	0.058	3.81
7	3.526	44.5	1621	0.043	0.038	3.53
8 PI	2.891	43.8	1333	0.053	0.043	4.30
9	2.500	43.8	744.	0.063	0.109	7.70
10	1.989	43.7	620.	0.080	0.145	9.24
11	1.508	42.5	560.	0.090	0.125	10.23
12	1.060	42.5	609	0.088	0.110	9.41
13	0.032	0	0	0.088	0	
14	0.040	0	0	0.086	0	

CALCULATED VEHICLE PROFILE



PLOTTED VEHICLE PROFILE

1ft = 0.305m

1mph = 1.609km/h

Figure 27. EXAMPLE OF VEHICLE CURVATURE PROFILE

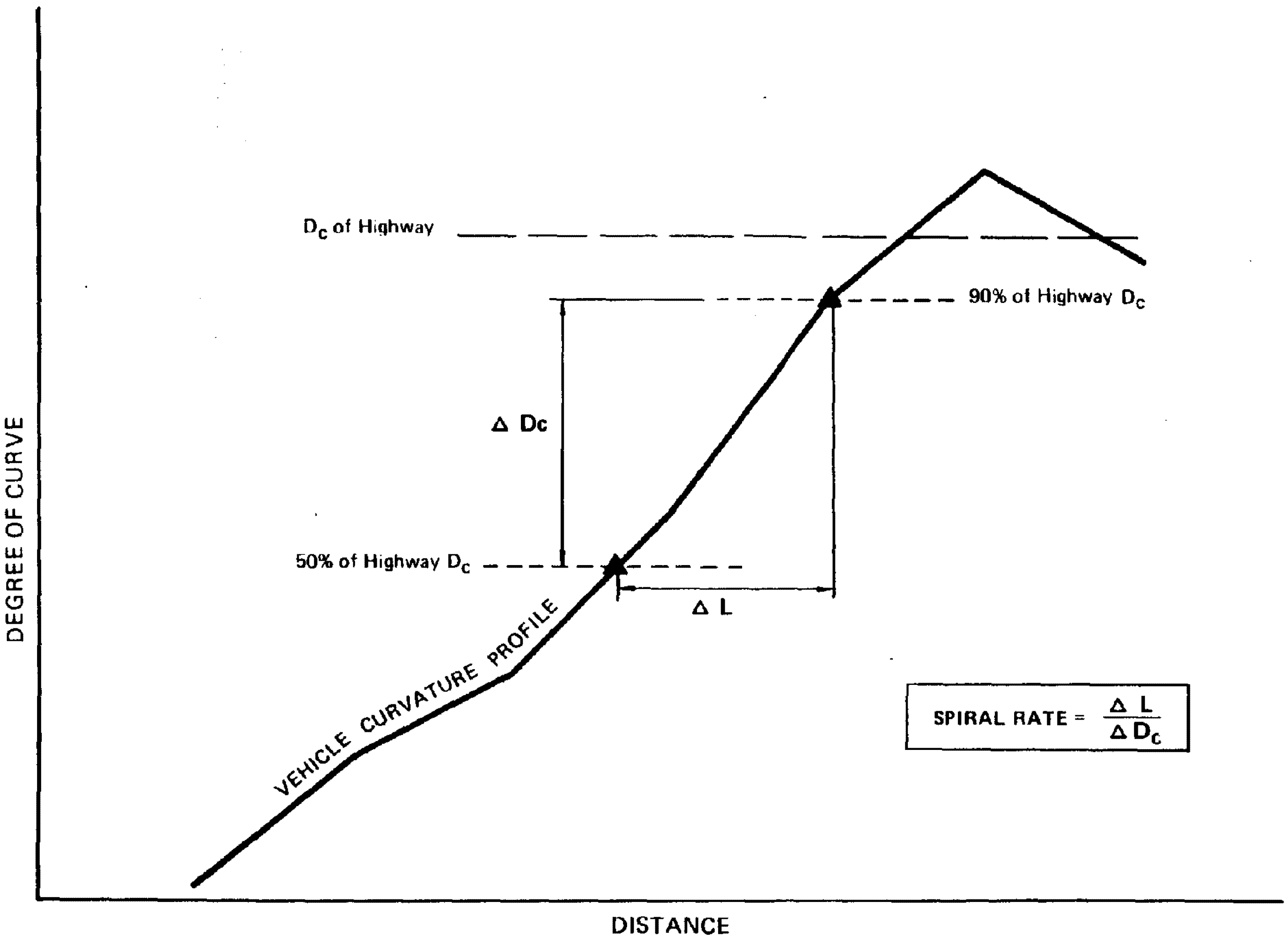


Figure 28. DEFINITION OF SPIRAL RATE

indicating drivers' desired speed and desired rate of change of curvature are essentially independent.

Relationship Between Spiral Rate and Critical Path Radius.--A third hypothesis was that a vehicle's spiral rate is related to its critical path radius. Table 34 reports results of simple linear regression analyses to test the association between spiral rate and critical path radius. The findings are both important and logical. Drivers who effect more gradual transitions (thereby producing a high spiral rate expressed as length per degree) tend to produce less severe critical path radii. This finding seems generally true regardless of curvature or speed.

TABLE 34

RELATIONSHIP BETWEEN SPIRAL RATE AND CRITICAL PATH RADIUS

		Regression Statistics*				
Site	Radius of Highway Curve (feet)	Slope a	Intercept b	R^2	Std. Error	Signif. at α = 0.10?
Left Curves						
188	1516	2.38	1317	0.11	198	Yes
206	1089	1.56	859	<0.01	151	No
204	912	10.43	640	0.47	109	Yes
212L	595	8.55	436	0.38	59	Yes
Right Curves						
212R	INSUFFICIENT DATA OBTAINABLE FOR SPIRAL RATE					
198	444	3.28	465	0.55	41	Yes

* Critical Path Radius = a (Vehicle Spiral Rate) + b

1 ft = 0.305 m

Summary.--Driver behavior on horizontal curves can be described in terms of speed, path radius achieved at the point of maximum lateral acceleration (critical path radius), and the rate at which the vehicle changes its circular path (spiral rate). A large number of drivers produce critical path radii smaller than that of the curve. While the critical path radius does not appear related to speed, it does appear related to spiral rate. Drivers who spiral more gradually tend to produce less severe critical paths.

Severity of Critical Path Radius

The findings that speed and critical path radius are independent is not surprising, as it was previously discovered by Glennon and Weaver (33). Also, the finding that, regardless of highway curvature, many drivers voluntarily track paths more severe than the roadway curvature is highly significant. The extent of the severity of driver/vehicle behavior was the subject of further study.

Data previously recorded for each site included critical path radius for each vehicle. These data potentially described the total population of drivers relative to their critical path radius. One possible problem in using this sample of vehicles was the original built-in bias in selecting the subsets of vehicles for study. Two subsets were selected on the basis of vehicle speed, and two on the basis of vehicle offset or lateral placement. Careful study of the data and previous research findings allowed the conclusion that the sample selected did indeed produce an unbiased estimate of critical path radius for the total population. The mean critical path radius for those vehicles selected on the basis of vehicle offset was not found to be different from the mean critical path radius of vehicles selected on the basis of speed. And, previous analyses indicated that speed and critical path radius are independent.

Figure 29 summarizes cumulative frequency distributions of the critical path radii for all six sites. These plots show that large numbers of vehicles track path radii significantly smaller than the actual curve radius. Site 198, a short, right curve, was an exception.

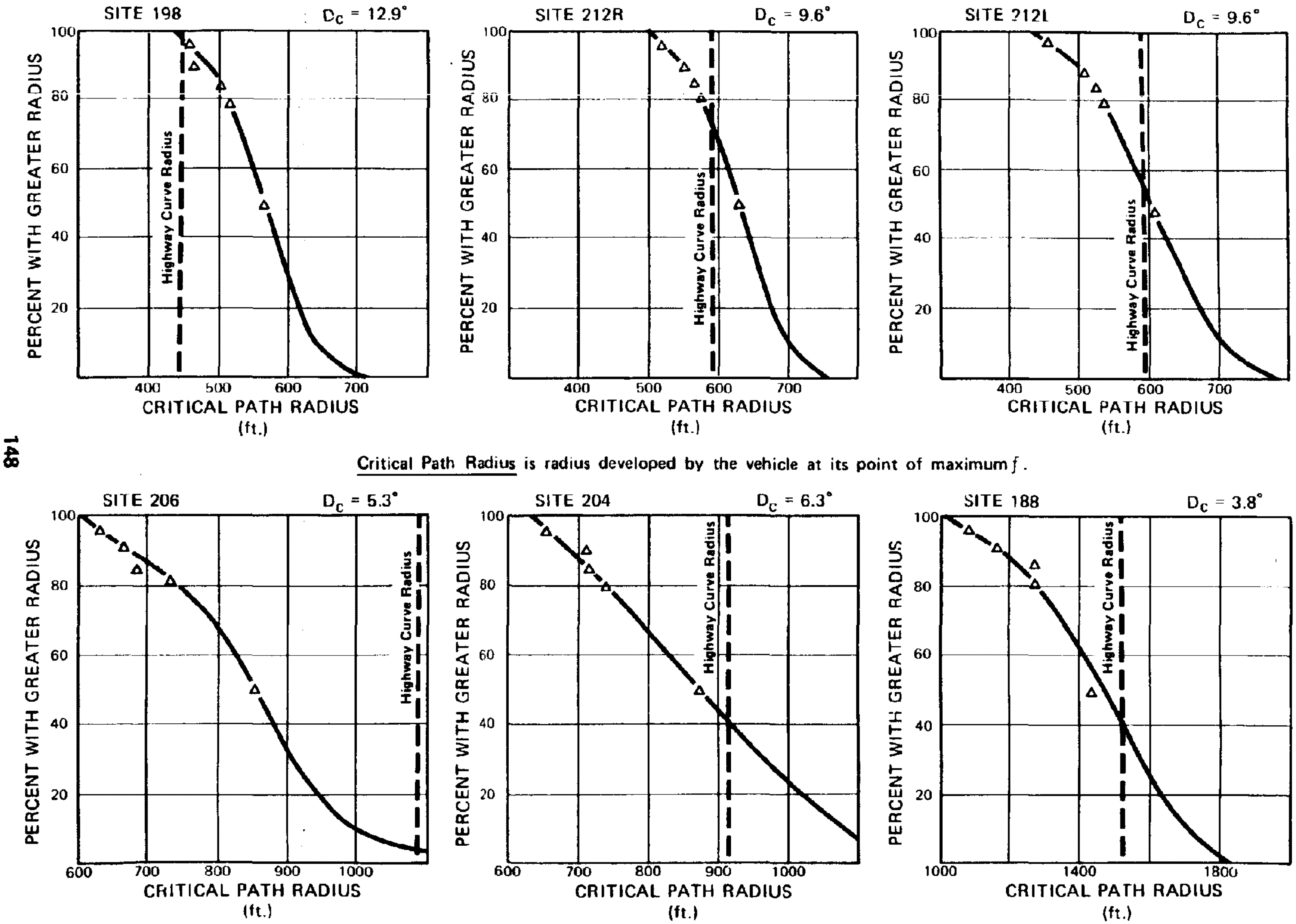


Figure 29. **CUMULATIVE FREQUENCY DISTRIBUTIONS OF CRITICAL PATH RADIUS OBSERVED AT STUDY SITES**

1ft = 0.305m

Relationship of Critical Path Radius to Highway Curve Radius.--Because the severity of driver behavior appeared uniform across all curvature, a series of analyses was performed to determine the presence of a relationship between critical path radius and highway curve radius. The cumulative frequency distributions were used to provide measures of critical path radius for various percentiles of the driver population. These values are summarized in Table 35. Values for the four left curves were used in simple linear regression analyses, with the following results shown in Table 36.

TABLE 35

PERCENTILES OF CRITICAL PATH RADIUS*
OBSERVED AT SIX STUDY SITES

Site	Radius of Highway Curve (Feet)	Critical Path Radius (feet) for Percentiles of Drivers			
		95th	90th	85th	50th
Left Curves					
188	1516	1085	1155	1265	1435
206	1089	635	665	685	855
204	912	655	705	715	875
212	595	465	505	525	605
Right Curves					
212	595	525	555	565	625
198	444	455	465	505	565

1 ft = 0.305 m

TABLE 36

RELATIONSHIP BETWEEN HIGHWAY CURVE RADIUS AND CRITICAL PATH RADIUS

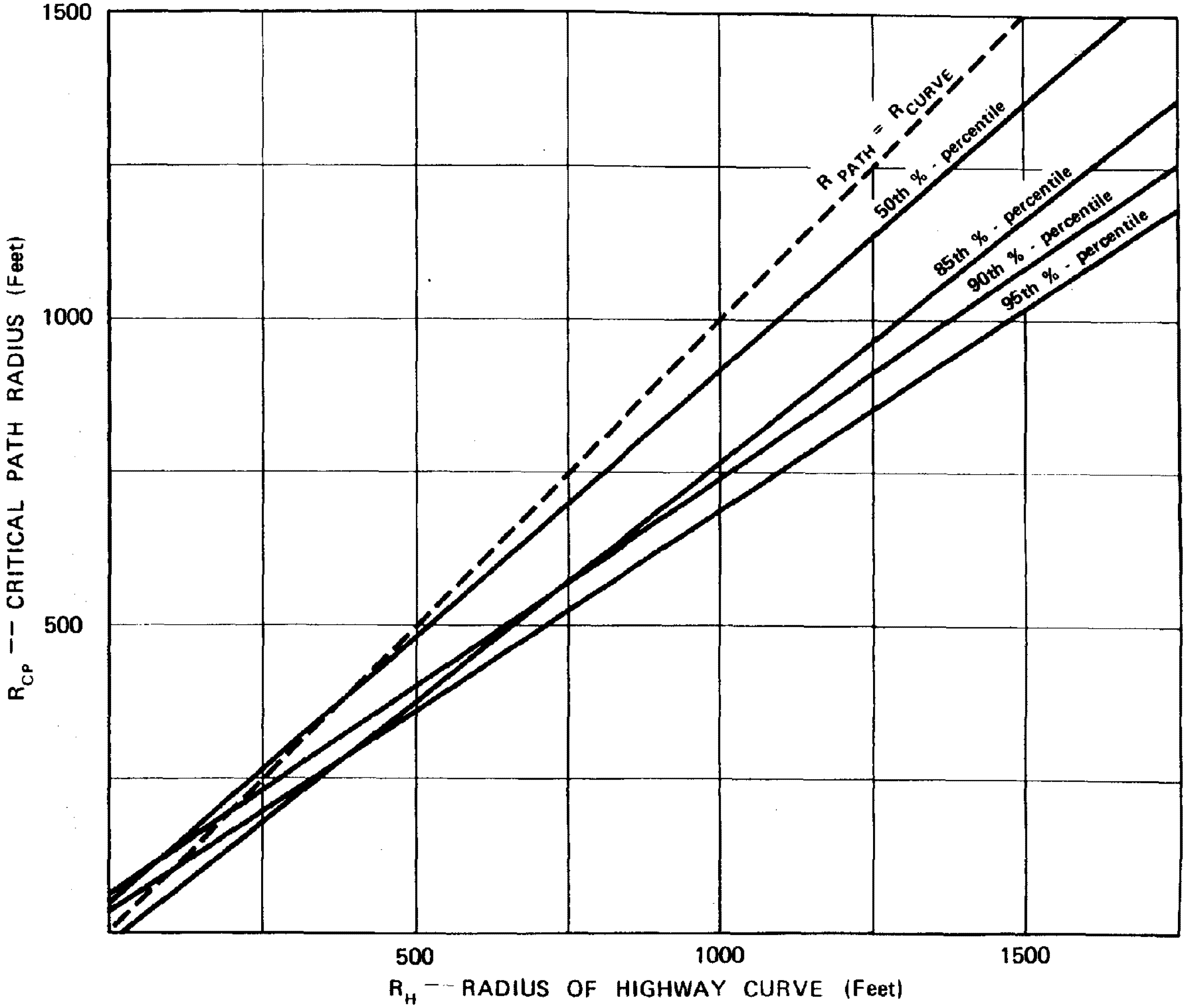
Percentiles of Driver Population	Slope (a)	Intercept (b)	R^2	Std. Error	t	Signif. at α = .10?
95th	0.66	35	0.91	95.4	4.58	Yes
90th	0.69	52	0.89	111.1	4.11	Yes
85th	0.79	-12	0.88	136.9	5.83	Yes
50th	0.88	39	0.93	116.2	5.03	Yes

Critical Path Radius = a (Highway Curve Radius) + b

The results of this analysis compare closely to previous research on highway curve traversals (33). Despite the limited number of sites and data points, the relationship between critical path radius (for any selected percentile of behavior) and curve radius appears strong. The implications are clear. Actual driver/vehicle path behavior is substantially and uniformly (across highway curvature) more severe than implicit design policy assumptions for highway curves. This is graphically illustrated by Figure 30, which shows the reported relationships compared to the implicit design assumption that the vehicle path follows the highway curve. Furthermore, because it was found that critical path and speed are essentially independent, high speed vehicles are just as likely to produce very severe (say, 90th or 95th percentile) path radii as are low or average speed vehicles.

Effects of Other Geometry on Driver Behavior

The vehicle traversal studies provided insights to the relationships of geometrics other than curve radius to driver behavior. Elements of interest include length of curve, direction of curve, width of roadway and design for superelevation runoff.



Note: 1ft = 0.305m

Figure 30. RELATIONSHIP BETWEEN HIGHWAY CURVE RADIUS AND PERCENTILES OF CRITICAL PATH RADIUS

Length of Curve.--Of the six sites studied, one was a short, sharp right curve, which exhibited noticeably different vehicle behavior. The following points summarize the differences in vehicle behavior at this curve that were attributed to its short length:

(1) Most vehicles tracked the curve with minimum path radii milder than that of the curve. As shown in Table 35, the 95th percentile critical path radius at this site (Site 198) was greater than the curve radius. All other sites exhibited more severe path behavior.

(2) This was the only site that showed a significant correlation between vehicle speed and critical path radius. The indicated relationship is that faster vehicles track less severe paths, that is, generate larger minimum path radii.

Further inspection of the film explained the difference in vehicle behavior on short curves. When the curve is short enough, drivers position themselves and traverse the curve in a manner that "cuts" or reduces their central angle. Their behavior is best characterized as "spiral-in and spiral-out" with little or no path overshoot.

Direction of Curve.--One site was studied in both directions, enabling a comparison of driver/vehicle behavior by direction. A review of the placement and speed characteristics for Site 212 shows no difference between left and right directions. A path overshoot was observed on both directions (see Table 35), although it was less severe for the right-hand approach.

Width of Roadway.--The original study plan was to observe driver behavior on a narrow roadway. Site 204, with an indicated width of 19 ft (5.8 m), was selected as a study site. Unfortunately, the roadway had been widened and repaved just prior to our traversal studies. Time and budget constraints did not allow filming of an alternate location. However, some conclusions about vehicle operations on narrow roadways can be drawn from the study. Following review of the film and analyses of transition behavior, it seems clear that different driver behavior would be expected on narrow roads. Lane widths are used by drivers to position themselves and effect a spiral path, even though the centerline geometry is tangent-to-curve. Reducing the lane width decreases or eliminates this freedom to position and spiral. It is logical, therefore, to expect sharper spiral rates which in themselves are less desirable, and sharper critical path curvature, also less desirable.

Superelevation Distribution and Runoff.--Observed driver behavior points out the effects of variable combinations of superelevation distribution and runoff length. In general, drivers produce path transitions of 200 to 300 ft (60 to 90 m) approximately centered around the PC of the highway curve. This transitioning behavior, which is independent of vehicle speed, leads to the following conclusions:

(1) Superelevation runoff lengths of 200 to 300 ft (60 to 90 m) tend to match vehicle path transitioning behavior, and are therefore desirable.

(2) At least 50 percent of full superelevation is desirable at the PC. This distribution tends to match average driver path curvature at the PC.

Optimal placement and length of superelevation runoff should result in a gradual, steady build-up of lateral acceleration for the majority of drivers. It is particularly important that full superelevation be provided by the time drivers reach their maximum path curvature. This was found to occur about 100 to 150 ft (30 to 45 m) past the PC.

Summary and Implications of Vehicle Traversal Studies

Driver vehicle behavior observed on the approaches to and traversing through horizontal curves is complex. Vehicle speed, path and roadway geometry combine to exhibit a wide range of behavior. A number of concepts stand out as significant in terms of design for variable vehicle behavior.

(1) Drivers tend to overshoot the curve radius, producing minimum vehicle path radii sharper than the highway curve. Furthermore, the tendency to overshoot is independent of speed.

(2) Drivers position themselves in advance of the curve to effect a spiral transition. Drivers who spiral gradually tend to produce less severe path radii.

(3) The tangent alinement immediately in advance of the curve is a critical region of operations. At about 200 feet (about 60 m) before the PC, which is about 3 seconds of driving time, drivers begin simultaneously adjusting both their speed and path. Such adjustments are particularly large on sharper curves.

(4) Points (2) and (3) demonstrate the significant operational benefits of spiral transitions to highway curves. Spirals of sufficient length enable the driver to adjust both speed and path in a manner that reduces or eliminates severe overshoot of the curve radius, thereby preventing the build-up of excessive levels of lateral acceleration.

(5) Both the speed studies and vehicle traversal studies point out the criticality of sharp, underdesigned curves on high-speed highways. The combination of high speeds and overshoot path behavior produces highly critical for much of the vehicle population dynamics on underdesigned curves.

(6) Present highway curve design policy presumably equalizes the dynamic effects of curve radius and superelevation. However, drivers tend to overshoot the curve radius. This behavior effectively increases the importance of curvature relative to superelevation. Therefore, under present design policies for curves, milder curves with lesser superelevation produce lower friction demands than presumably equivalent sharper curves with greater superelevation.

VII. COMPARISON OF HVOSM AND VEHICLE TRAVERSAL STUDIES

A primary objective of the vehicle traversal studies was to provide a basis for evaluating the previously completed HVOSM simulations (Chapter V). HVOSM has already been proven an accurate, cost-effective tool for studying vehicle behavior under highly unstable (i.e., loss of control, high speed impact) situations. Using controlled, full-scale tests for calibration, HVOSM can accurately predict the dynamic responses and consequences for a range of conditions.

In such critical applications, dynamic vehicle responses are essentially a function of vehicle properties and test conditions (e.g., speed at impact, angle of impact). Application of HVOSM to the evaluation of highway curve traversals, however, involves an additional important dimension. If the simulations are to have any real meaning, driver behavior must be reasonably modeled.

Driver Model

Modeling the driver is a particularly difficult problem, as it entails consideration of human factors such as perception and reaction time, psychological attitudes, and interaction with the vehicle. The task is more difficult given that a useful simulation tool must not be overly complex, and should be reasonably valid over the range of possible test conditions.

A complete discussion of development work is given in Appendix D. Previous research on modeling the driver (31) was adjusted and tested. Elements of the driver model employed in the simulations included a "wagon-tongue" algorithm, a neuromuscular filter, and steering parameters such as damping, steer velocity, and steer initialization.

One element of the driver model was particularly important to calibrate. Earlier discussion of HVOSM in Chapter V emphasized the importance of establishing a reasonable probe length. To review, probe length is one part of the wagon-tongue control algorithm. Its function is to simulate the driver preview of the alinement ahead. Previous research on actual driver behavior formed the basis for selection of a speed-sensitive probe length function for the initial set of simulations reported in Chapter V.

The importance of properly selecting probe length is illustrated by Figure 31, which shows results of early calibration runs for probe length, for which various length functions were tested. Variations in probe length from 0.20 V to 0.40 V produce significantly different levels of simulated lateral acceleration (expressed as maximum f developed on the rear tires). Given this sensitivity of probe length to resultant vehicle dynamics, efforts to validate the previous runs focused on validating the probe length function. Driver behavior observed in the vehicle traversal studies formed the basis for this validation.

Comparison of Results

Insights concerning driver/vehicle behavior on curves can be obtained from evaluation of both the HVOSM curve runs and the results of the vehicle traversal studies. In order to gain these insights, it is first important to understand what each type of analysis represents.

Characteristics

As Table 37 shows, the two types of analysis are not directly comparable. HVOSM was applied to a series of AASHTO controlling curves for a range of design speeds. The field studies involved a range of highway curvature with generally less than full superelevation. Variations in both speed and path were observed, and used to determine distributions of lateral acceleration or friction factor. The accuracy and meaning of the field data were limited by collection and data reduction methodology employed. Thus, transient behavior observed in the field actually represents average friction demand for the vehicle, averaged over 1.0 to 1.7 seconds of real time. This compares with the reported friction results for HVOSM, which relate more closely to actual loss of control (0.25 seconds of real time; friction demand at the critical axle).

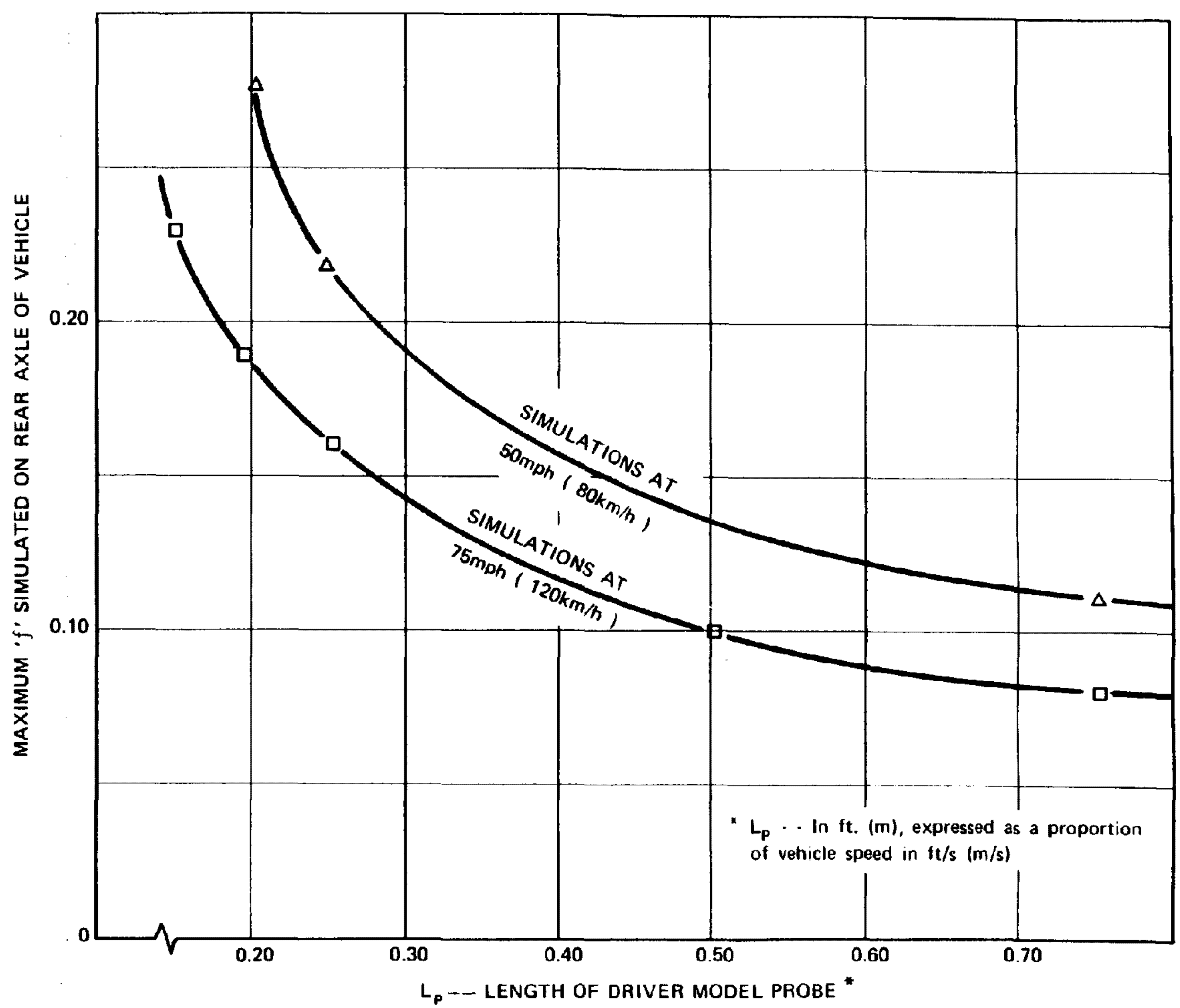


Figure 31. SENSITIVITY OF HVOSM VEHICLE DYNAMICS TO DRIVER MODEL PROBE LENGTH

TABLE 37

CHARACTERISTICS OF HVOSM ANALYSES AND VEHICLE TRAVERSAL STUDIES

	HVOSM Curve Runs	Vehicle Traversal Studies
Curves Analyzed	AASHTO Controlling Curves for Range of Design Speeds	Range of Curvature (No controlling curves)
Data Collected	Friction demand on 4 tires; Driver Comfort Factor; Roll and Steer Angle	Average Friction Demand (point mass)
Time Sensitivity of Data	Transient behavior observable to 0.25 seconds of real time	Measurements based on 100 ft. (30.5 m) arc--1.0 to 1.7 seconds real time
Results Reported	Maximum friction factor on 2nd highest tire of rear axle; average over 0.25 seconds	Friction factor and radius at point of maximum friction; average over 100 ft. (30.5 m) arc

Findings

Given the differences between the analyses, direct comparisons are difficult. However, because both analyses measured transient, extreme behavior across a range of speed and curvature, it is possible to compare overall levels of friction demand, and trends across the range in speeds.

The upper portion of Figure 32 contains a plot of reported maximum friction demand vs. design speed for a sample of the HVOSM runs. The points plotted represent those simulations at which the vehicle was run at design speed on the appropriate controlling curve, with AASHTO superelevation and transition design. Initial inspection of these points shows a consistent trend for f vs. design speed, with one striking exception. Simulated f for 50 mph (80 km/h) is greater by 0.04 to 0.05 than the overall trend seems to indicate.

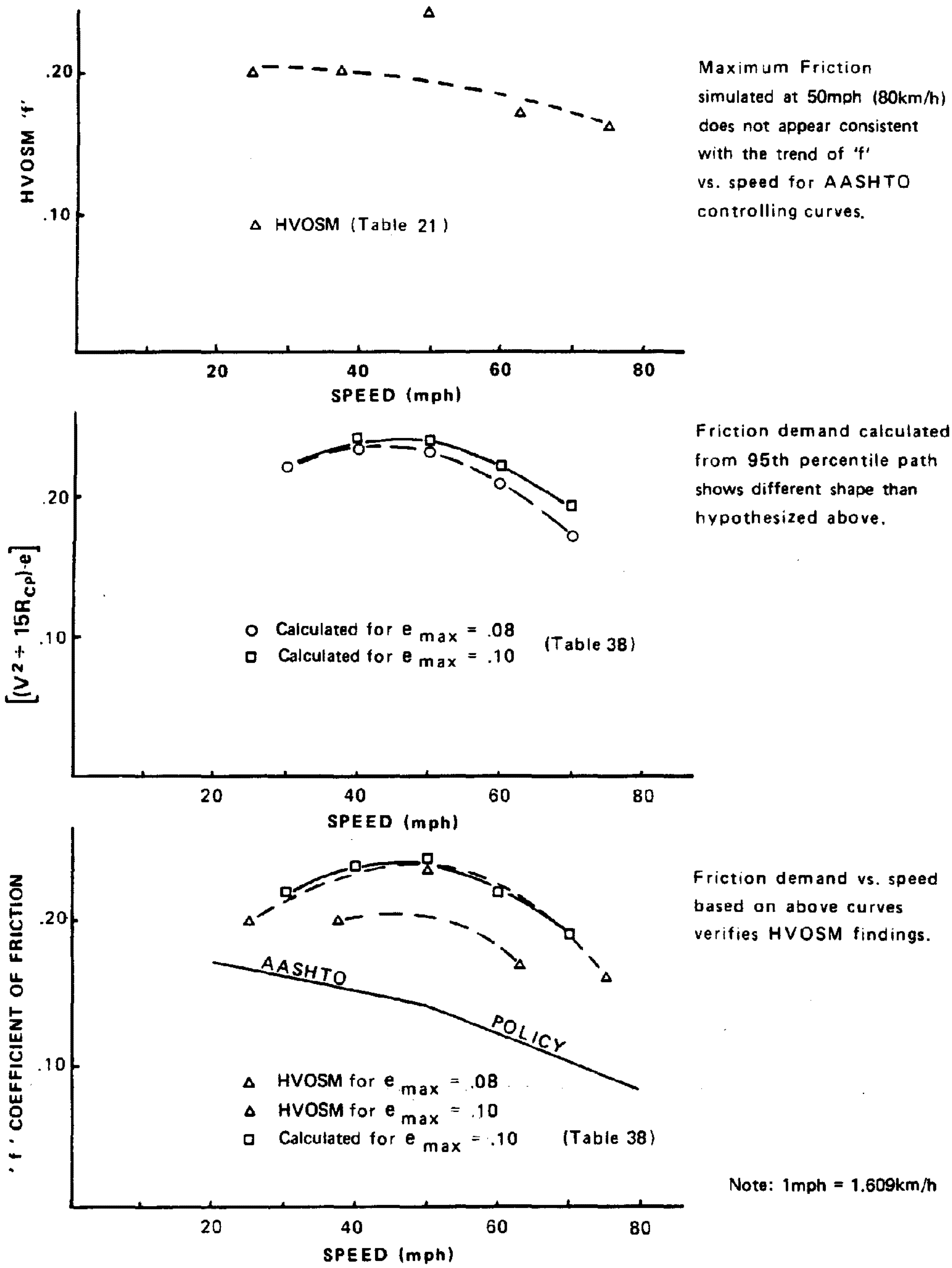


Figure 32. COMPARISON OF VEHICLE DYNAMICS FROM HVOSM AND VEHICLE TRAVERSAL STUDIES

Review of the vehicle traversal studies provides an explanation for the apparent anomaly. It was shown previously that vehicles tend to overshoot highway curves, producing path radii smaller than that of the curve. If this behavior is considered within the framework of AASHTO design policy, it results in an explanation for the HVOSM runs, and reveals important findings regarding design of highway curves. Consider AASHTO design controlling curves for a range of design speeds and maximum superelevation rates. If one calculates friction demand at design speed assuming overshoot driving behavior, an interesting picture of vehicle dynamics emerges. Table 38 shows such calculations, with an assumed 95th percentile driver path. As the table indicates, calculated friction demand varies for a given design speed depending on the superelevation policy (and resulting controlling curve) used. Design policies based on maximum superelevation rates (say, e_{max} of 10 percent) result in greater calculated friction demand at design speed than policies based on lower maximum rates (say, e_{max} of 6 percent), assuming the same overshoot driving behavior.

What Table 38 says is, assuming one is interested in nominally critical driver behavior as given by a 95th percentile driver, friction demand vs. speed relationships are not consistent for the range of superelevation policies. The middle portion of Figure 32 illustrates these side friction vs. speed relationships.

While the above discussion is relevant in itself in terms of design for curves, it is of particular value in understanding the HVOSM curve runs. As the bottom portion of Figure 32 shows, the family of points that were believed to simulate one relationship in fact represent two separate curves. The two curves describe simulated friction vs. speed for controlling curvature as defined by superelevation rate policies of 8 percent and 10 percent. Furthermore, the shape and values of the calculated curves based on the vehicle traversal studies very closely match the relationship described by the HVOSM points based on e_{max} of 10 percent.

TABLE 38

RELATIONSHIPS AMONG SPEED, SUPERELEVATION AND FRICTION DEMAND FOR 95TH PERCENTILE DRIVING BEHAVIOR

Design Speed (mph)	e_{max} (percent)	Radius of Highway Curve(ft)	Radius of Vehicle Path[1](ft)	f at Design Speed From Vehicle Path[2]	AASHTO Criteria
70	10	1637	1117	0.192	0.10
	8	1910	1295	0.172	0.10
	6	2083	1409	0.172	0.10
60	10	1091	755	0.218	0.12
	8	1206	831	0.209	0.12
	6	1348	924	0.200	0.12
50	10	694	493	0.238	0.14
	8	758	535	0.232	0.14
	6	833	584	0.225	0.14
40	10	427	317	0.237	0.15
	8	464	341	0.233	0.15
	6	508	370	0.228	0.15

[1] $R_{path} = 35 + 0.66\ R_{curve}$ (From Table 36)

[2] Calculated friction demand assuming nominally critical path behavior at design speed. In other words, $f_{path} = [V^2_{design} / (15\ R_{path})] - e_{max}$

1 mph = 1.609 km/h
1 ft = 0.305 m

One additional finding of both analyses is the relationship between speed and friction demand, given nominally critical driving behavior. Present design policy calls for decreasing design friction factor with increasing speed. As the lower portion of Figure 32 shows, however, friction demand does not decrease with speed, but rather peaks in the range of 45 to 55 mph (72 to 89 km/h), before decreasing for higher speeds.

Verification of Probe Length Function.--Figure 32 and the above discussion demonstrate the validity of HVOSM in simulating nominally critical vehicle dynamics expressed in terms of maximum friction demand on highway curves. Furthermore, the probe length function used in the simulations is shown to be sensitive and accurate across the range of speeds that were simulated.

Path Radius Simulation.--Simulation of nominally critical f levels was achieved with reasonable correlation to the field studies. Questions were raised, however, as to whether the simulated friction demand was a function of path overshoot similar to that observed in the field, or whether some hidden dynamic response was being simulated. These questions were answered by analyzing sample outputs from two of the runs. Among the data produced by HVOSM are X, Y coordinates for the tires and center of gravity. A simple algorithm was developed to calculate vehicle path coordinates for these data sets. The results of minimum calculated vehicle path radius from the HVOSM output are almost identical to predicted 95th percentile path radius as given by the vehicle traversal study results (see Table 36).

Radius of Highway Curve		Speed		Nominally Critical Path Radius: Simulated (HVOSM)		Nominally Critical Path Radius: Calculated Field Studies	
ft	(m)	mph	(km/h)	ft	(m)	ft	(m)
689	(210)	49.7	(80)	481	(147)	490	(149)

Vehicle Transitioning.--While HVOSM successfully simulates critical levels of f, and does so through nominally critical path radii, it does not exactly replicate the manner in which the f and critical radius are generated. Figure 33 shows plots for two vehicles--one observed in the field, and one simulated. Each vehicle's instantaneous curvature is plotted at various locations along the transition and into the curve. Simulated vehicle behavior, represented by vehicle 'A', shows almost all vehicle curvature developed after the PC, but with extremely rapid, severe spiraling. Vehicle 'B' is the vehicle which most closely represents 95th percentile path behavior at Site 212 L. The amount of vehicle path curvature at the PC, and the indicated rate of spiraling, are typical of most observed vehicles.

Short Curve Vehicle Dynamics.--One interesting verification of the HVOSM driver model was provided by the field observations for Site 198, a short, right-hand curve. Observed vehicle paths were much less severe than would be predicted by the path vs. curve relationships derived previously. Inspection of the individual vehicle paths provided a clue as to what was different about this site. Because the curve length was so short, drivers literally did not have the opportunity to overshoot the highway curve radius. Instead, they spiraled into and out of the curve, with a minimum path radius generally greater than that of the highway. This same behavior was simulated previously in a run specifically designed to study short curve dynamics. At the time of the simulation it was hypothesized that very short curves produced additional dynamics due to rapid changes of roll angle, steering, etc. The results (see Table 21) produced the surprising (at the time) conclusion that vehicles generated less friction demand on very short curves. It was left to the field studies to verify and explain why this was so.

Knowledge Obtainable Exclusively From HVOSM

HVOSM has been proven to accurately simulate nominally critical vehicle behavior on curves. There are obvious cost and time advantages in simulating rather than studying vehicle dynamics in the field. Also, there is a wealth of information provided by HVOSM which could not be obtained in a field experiment such as was

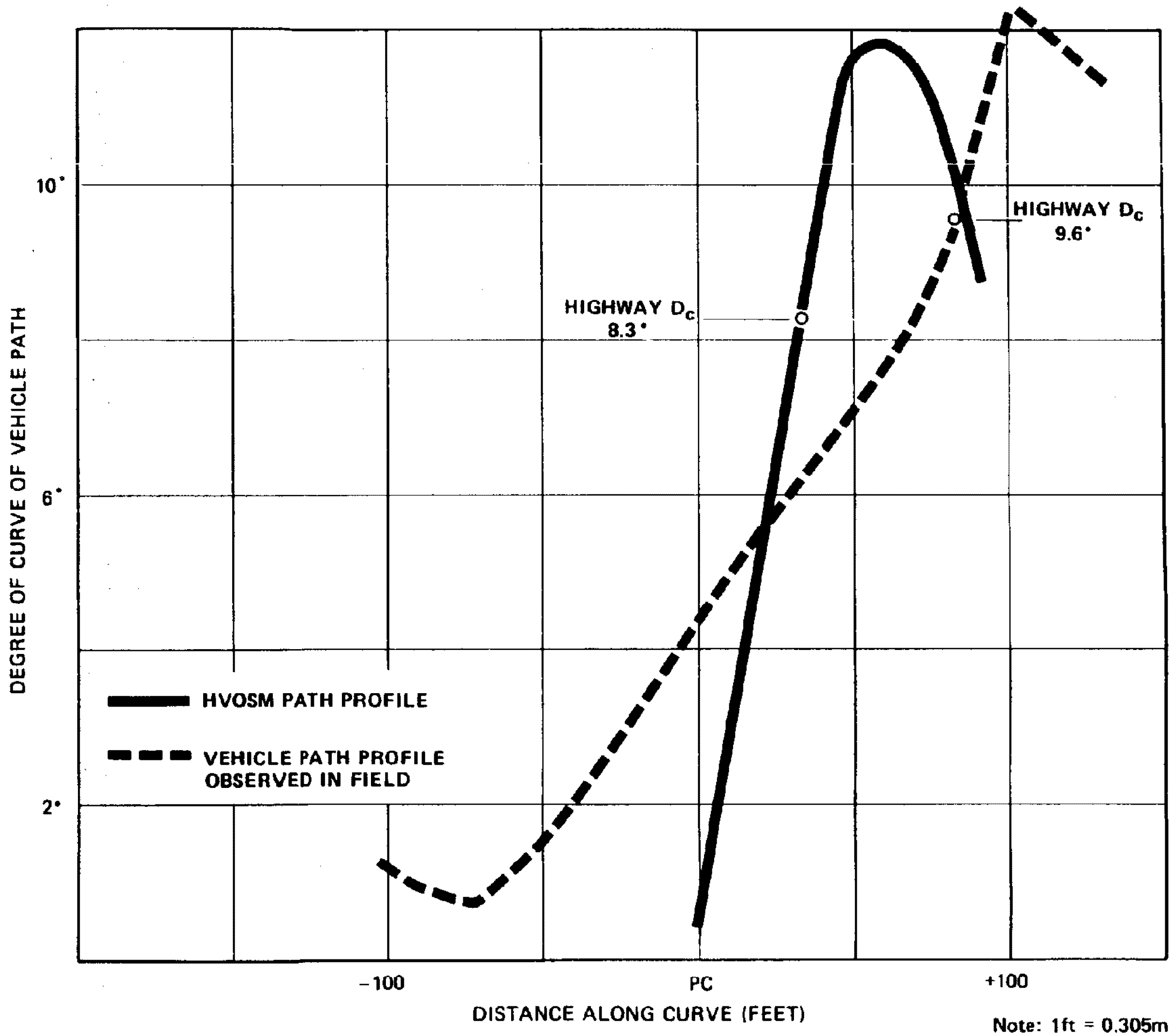


Figure 33. COMPARISON OF VEHICLE PATH TRANSITIONING BEHAVIOR FROM HVOSM AND VEHICLE TRAVERSAL STUDIES

performed for this research. Through simulation, not only can lateral acceleration be modeled, but also the distribution of lateral acceleration to the four tires. This is important in identifying thresholds of loss of control, which is dependent on friction demands on individual axles. Roll and steer angle data are also obtainable. Perhaps the most useful aspect of simulation is the ability to study dynamic effects on various vehicle types (e.g., trucks, semi-trailers, buses), or ranges of vehicle characteristics (e.g., front-wheel drive).

HVOSM has limited applications and usefulness, which are a function of the assumptions that are required to initiate the simulation. The assumptions generally relate to driver behavior. They include initial speed, acceleration/deceleration, and brake applications. HVOSM is also limited by its inability to address variable driver behavior as a function of changing environmental conditions.

Knowledge Obtainable Exclusively From Field Studies

The following discussion concerns crucial areas of vehicle operations for which actual observations of driver behavior are required. Knowledge obtained from field studies, combined with HVOSM or other simulations, can answer important questions about driver/vehicle behavior on highway curves.

Vehicle Speed Characteristics

Drivers' desired speed characteristics can only be determined by field measurements. The studies of speed and speed transition behavior showed that approach conditions and curvature have variable effects on desired speeds. Other factors such as weather or light conditions also can influence driver behavior. Field observations of vehicle speeds provide distributional data which enable more meaningful analysis of the criticality of a particular set of conditions. For example, one can simulate the vehicle dynamics resulting from a curve being "overdriven" by 10 mph (16 km/h). However, field measurements are required to determine what sets of conditions produce overdriving, and what percentage of the vehicles do in fact overdrive the curve.

Effect of Geometry on Path Behavior

Observed driver behavior in curve tracking is complex. Adaptation of the driver model in HVOSM to replicate this behavior requires extensive field data. One important design element which affects driving behavior is lane width. The vehicle traversal studies showed that drivers use the full lane to position their vehicles for spiraling into the curve. Given that this behavior is universal, one could expect highly variable spiraling behavior on 9- or 10-foot (2.7 or 3.0 m) lanes vs. 12- or 16-foot (3.7 or 4.9 m) lanes. Because the HVOSM driver model in its present form assumes that drivers desire to track the center of the lane, any effect of variable lane width on path would not be simulated.

Environmental Conditions

It is generally assumed that adverse weather conditions affect driving behavior. While changes in driving behavior are usually characterized in terms of lower speeds, it is possible that path-following behavior is also altered. Poor or limited visibility during rain, fog, or night time may have significant effects on the overshoot characteristics of drivers. Such effects could only be measured or estimated from actual observations of drivers.

Summary of HVOSM and Field Study Vehicle Dynamics

The total research effort demonstrated (1) the ability of HVOSM to predict vehicle dynamics across a range of curve conditions; and (2) the need to study actual vehicle behavior in order to assess the validity of the simulations. Both field studies and simulation work described driver behavior in a similar manner.

Spiraling Transitions

The studies of actual driver/vehicle behavior revealed that drivers spiral into horizontal curves. This spiraling behavior occurs at rates which vary with highway curvature. Simulated driver/vehicle behavior using HVOSM was generally similar in character. However, the simulated rate of spiraling was more severe than observed rates. This severe rate is attributed to the short probe length function which was a part of the HVOSM driver model.

Dynamic Overshoot

With selection of an appropriate, speed-sensitive probe length function, observed driver/vehicle overshoot can be simulated. The severity of overshoot can be related to a desired percentile of driver behavior. The HVOSM curve runs demonstrated the ability to then select a probe length that results in comparable simulation of path overshoot. In addition, the research validated the probe length function across the full range of speeds.

Vehicle Path

Simplifying assumptions in the driver model and the resulting overly severe simulated spiraling rates result in vehicle path simulations that differ from observed paths. The thrust of the research was to demonstrate nominally critical behavior in terms of maximum friction demand achieved under a range of conditions. HVOSM simulations successfully replicated friction demands calculated from observed vehicle paths. Moreover, the simulations were shown to produce similar minimum path radii as were observed in the field. However, the transient path behavior was not simulated.

VIII. ANALYTICAL STUDIES

In the project planning phase, a large number of research questions were identified that were judged feasible to accomplish within the overall project context. Of these, the following three questions were addressed with analytical studies.

(1) Are AASHTO stopping sight distance requirements consistent for tangents and curves?

(2) What is the relationship between lateral and longitudinal displacement of roadside encroachments on curves?

(3) Does pavement settlement or "washboard" have a significant effect on vehicle stability?

Analytical studies are defined here as those problems that begin with a question to be addressed, to which hypotheses, assumptions, and known physical relationships are applied in an attempt to gain some new insights on the problems.

Stopping Sight Distance on Curves

This discussion of the consistency of AASHTO stopping sight distance (SSD) requirements for curves summarizes a portion of a separate project report titled "Stopping Sight Distance -- An Operational and Cost-Effectiveness Analysis." (38) This separate report presents a complete functional analysis of stopping sight distance. It also contains a general evaluation of the cost-effectiveness of several potential countermeasures for ameliorating sight-distance related accidents at locations with restricted sight distance. In the functional analysis, the report identifies two aspects of highway curve operation that may be critical in terms of supplying safe stopping sight distance on highway curves. These two aspects, which are described below, are (1) the increased friction demand of a vehicle that is both cornering and braking, and (2) the loss of eye height advantage for truck drivers on highway curves when the horizontal sight restriction is caused by either a row of trees, a wall, or a vertical rock cut.

Effect of Horizontal Curvature on Stopping Distance

The stopping ability of vehicles is a basic input to AASHTO policy for SSD. Present policy assumes that full (design) pavement friction is available to a vehicle forced to brake in an emergency situation. (It is noted that braking friction design values were selected by AASHTO from actual pavement friction tests.)

This basic assumption is particularly important when considering vehicle stopping requirements on highway curves. The following discussion shows that, because of the added pavement friction demands created by vehicles during cornering, design braking distances, and hence, design stopping sight distance, should be greater on highway curves than on tangents.

Figure 34 demonstrates that friction available for braking on curves is the vector resultant of both available friction and cornering demand. Mathematically, this is given as:

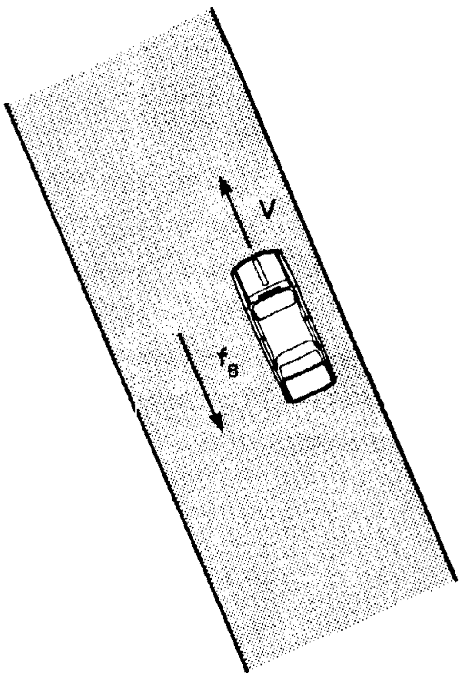
$$f_{b'} = \sqrt{f_b^2 - f_c^2} \qquad [8.1]$$

where

$f_{b'}$ = Coefficient of braking friction available on curve

f_b = Coefficient of braking friction on tangent--AASHTO design value

f_c = Coefficient of side friction demand on curve--AASHTO design values



BRAKING ON LEVEL TANGENTS

$$d_B = \frac{V^2}{30 f_B'}$$

Where d_B = Braking distance (ft)

V = Initial speed (mph)

f_B = Coefficient of friction available for braking (AASHTO design values assumed)

BRAKING ON LEVEL CURVES

$$d_B = \frac{V^2}{30 f_B'}$$

Where d_B, V, f_B as above

f_B' = Coefficient of friction available for braking

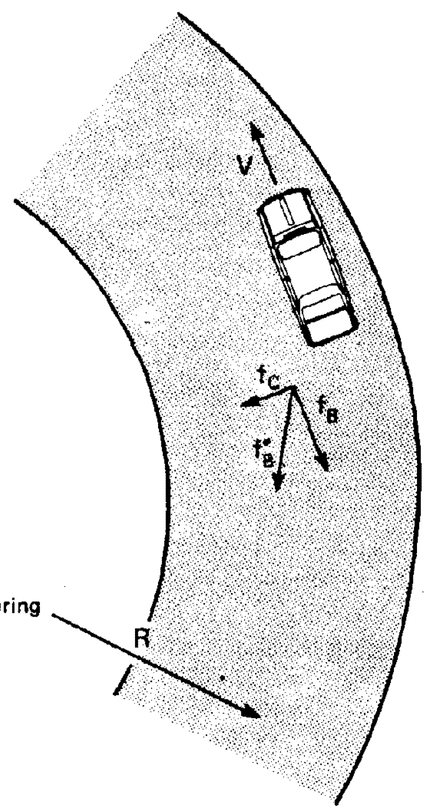
and $$f_B' = \sqrt{f_B^2 - f_C^2}$$

$$f_C = \frac{V^2}{15 R} - e$$

Where f_C = Coefficient of side friction required for cornering (AASHTO design values assumed)

R = Radius of curve (ft)

e = Superelevation (percent)



1ft = 0.305m
1mph = 1.609km/h

Figure 34. **FRICTION REQUIREMENTS FOR STOPPING ON HORIZONTAL CURVES**

Obviously, longer stopping distances on curves are indicated by the above equation. These greater stopping distances are particularly significant at higher speeds, as indicated in Table 39.

Even greater braking distances are required on horizontal curves if the "design event" is further defined in terms of nominally critical driver behavior. The field operational study results described in Chapter VI indicate that a large proportion of vehicles corner on horizontal curves at path radii significantly shorter than the roadway radius. This sharper cornering requires even greater side friction, further reducing available friction for braking on the pavement. Clearly, the effect of horizontal curvature on SSD requirements can be considerable.

TABLE 39

STOPPING SIGHT DISTANCE REQUIREMENTS FOR PASSENGER CARS ON CURVES

(e_{max} = 10 percent)

		Braking on Tangents (Wet Conditions)				Braking on Curves (Wet Conditions)		
Design Speed (mph)	Percention/ Reaction Distance (ft)	f_b	Braking Distance (ft)	Total Distance (ft)	f'	f_c	Braking Distance (ft)	Total Distance (ft)
30	110	0.35	86	196	0.311	0.16	96	206
40	147	0.32	167	314	0.283	0.15	189	336
50	183	0.30	278	461	0.265	0.14	314	497
60	220	0.29	414	634	0.264	0.12	455	675
70	257	0.28	583	840	0.262	0.10	625	882
80	293	0.27	790	1083	0.256	0.08	827	1120

f_b = AASHTO design friction factors

$f' = \sqrt{f_b^2 - f_c^2}$; f_c = cornering friction required at Design Speed on controlling curve. (AASHTO design values)

1 mph 1.609 km/h
1 ft = 0.305 m

Sight Distance for Trucks on Horizontal Curves

Vehicle characteristics also play a major role in design for SSD. Braking distances are a function of vehicle type, tire condition, and brake conditions. Vehicle type is by far the most important of these. Trucks require much greater stopping distances than do passenger cars. The height of eye of the driver is also a function of the vehicle. This dimension is critical in establishing the sight line from the driver to an object in the road over a crest vertical curve.

AASHTO policy does not directly treat the multitude of vehicle characteristics in design for SSD. Basic SSD design values are a function solely of passenger car braking ability and eye heights of passenger car drivers. Some general reference to SSD requirements for trucks is made. It has been assumed that the greater eye heights (and, hence, longer sight lines) afforded truck drivers tend to balance out the greater truck braking distances.

Clearly, however, a variety of geometric conditions can negate the advantages of greater eye heights for truck drivers. Horizontal sight obstructions such as retaining walls, rock cuts or tree lines restrict the view ahead from trucks and passenger cars alike. Furthermore, because such situations occur frequently on curves, highway curves with horizontal sight restrictions present particularly severe problems to trucks. Their greater braking distances, loss of eye-height advantage, and friction demands for cornering all contribute to SSD requirements that are greater than is indicated by AASHTO design policy.

Encroachment Characteristics of Run-off-Road Vehicles on Highway Curves

In light of the accident studies that showed the importance of roadside features to the accident experience on highway curves, some analysis of potential roadside encroachment characteristics is appropriate. Using certain assumptions, this section compares (1) the average outside lateral displacements on various highway curves to the average outside lateral displacement on highway tangents for various linear displacements; (2) the average effective vehicle angle on the side slope of various highway curves vs. highway tangents as a function of lateral displacement; and (3) the maximum inside lateral displacement as a function of initial angle and radius of highway curve.

Lateral Displacement Versus Linear Displacement for Outside Encroachments

This analysis attempts to generally compare the effect of clear-zone widths for the outside of highway curves of various radii with those for highway tangents. For this analysis, the following assumptions are made:

(1) Encroachment trajectories are tangent from both highway curves and highway tangents.

(2) The average encroachment trajectory for highway curves is tangent to the outside edge of pavement for encroachments starting in the outside lane.

(3) The average encroachment angles for highway tangents are 6.1° for a right side encroachment and 11.5° for a left side encroachment (20).

(4) The width of a highway lane is 12 feet (3.6 m).

For these assumptions, Figures 35 and 36 show the relationships between lateral and longitudinal displacements of encroaching vehicles for various highway curves and for highway tangents.

Figure 35 shows this relationship for encroachments from the outside lane on highway curves and for a right side encroachment on highway tangents. The appropriate equations are:

$$\text{For Curves:} \quad S = \sqrt{(R + W)^2 + L^2} - (R + W) \qquad [8.2]$$

$$\text{For Tangents:} \quad S = L \sin \Theta \qquad [8.3]$$

Where

S = Lateral displacement from the edge of the roadway (ft or m)

W = Lane width (ft or m)

L = Linear displacement along the encroachment path measured from the edge of the roadway (ft or m)

R = Radius of highway curve (ft or m)

Θ = Encroachment angle on tangent (degrees)

1 ft = 0.305 m

Figure 35 indicates that outside encroachments from the outside lane may result in lesser clear-zone width requirements for flat highway curves than for tangents; and greater clear-zone width requirements for sharp highway curves than for highway tangents.

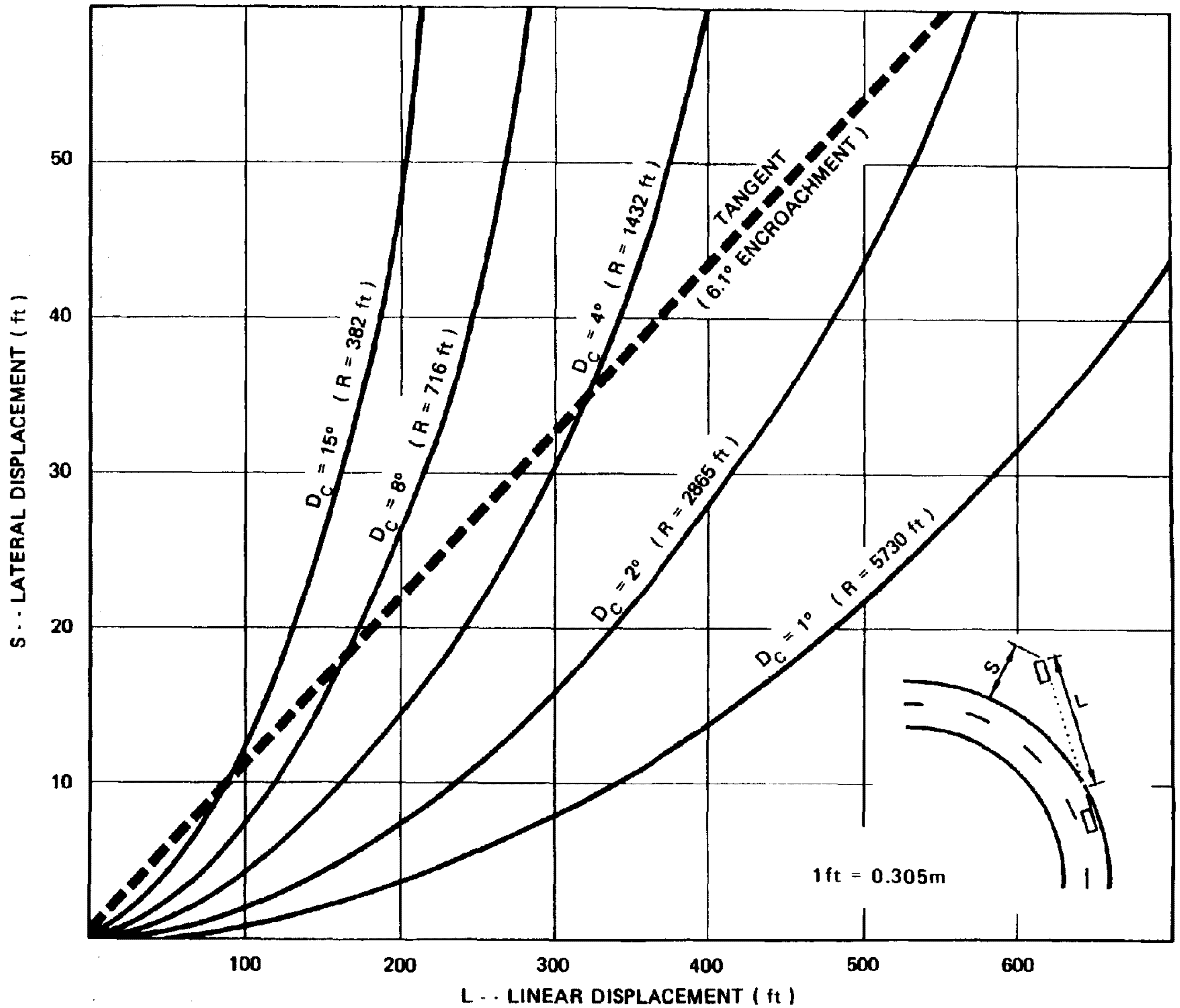


Figure 35. LATERAL DISPLACEMENT VERSUS LINEAR DISPLACEMENT FOR OUTSIDE ENCROACHMENTS FROM THE OUTSIDE LANE ON HIGHWAY CURVES

Figure 36 shows the relationship between linear and outside lateral displacements from the inside of highway curves and the left side of highway tangents. The appropriate equations are:

$$\text{For Curves: } S = \sqrt{R^2 + L^2} - (R + 12) \qquad [8.4]$$

$$\text{For Tangents: } S = L \sin \Theta \qquad [8.5]$$

With S, L, R and Θ as before

The relationships shown in Figure 36 for encroachments from the inside lane are similar to those shown in Figure 35, except that the clear-zone requirements are not quite as critical for any particular highway radius.

Effective Vehicular Angle for Outside Encroachments

Under the assumption of a linear roadside encroachment path, the effective vehicular angle at any point on the roadside of a highway tangent is equal to the initial encroachment angle. Therefore, on a right-side encroachment with a 6.1° angle, the effective slope of the vehicle traversal on a roadside slope is that slope times the side of the encroachment angle, or 10.6 percent of the roadside slope. For roadside slope traversals on curves, however, the effective angle and consequently the effective traversal slope increase as the vehicle proceeds across the slope. Because collision severity generally increases as the effective traversal slope increases, it is useful to analyze this effective angle as a function of highway curvature.

Using the same assumptions about outside roadside encroachments used in the previous analysis, Figures 37 and 38 show the relationship between lateral displacement and effective slope angle for various highway curve radii.

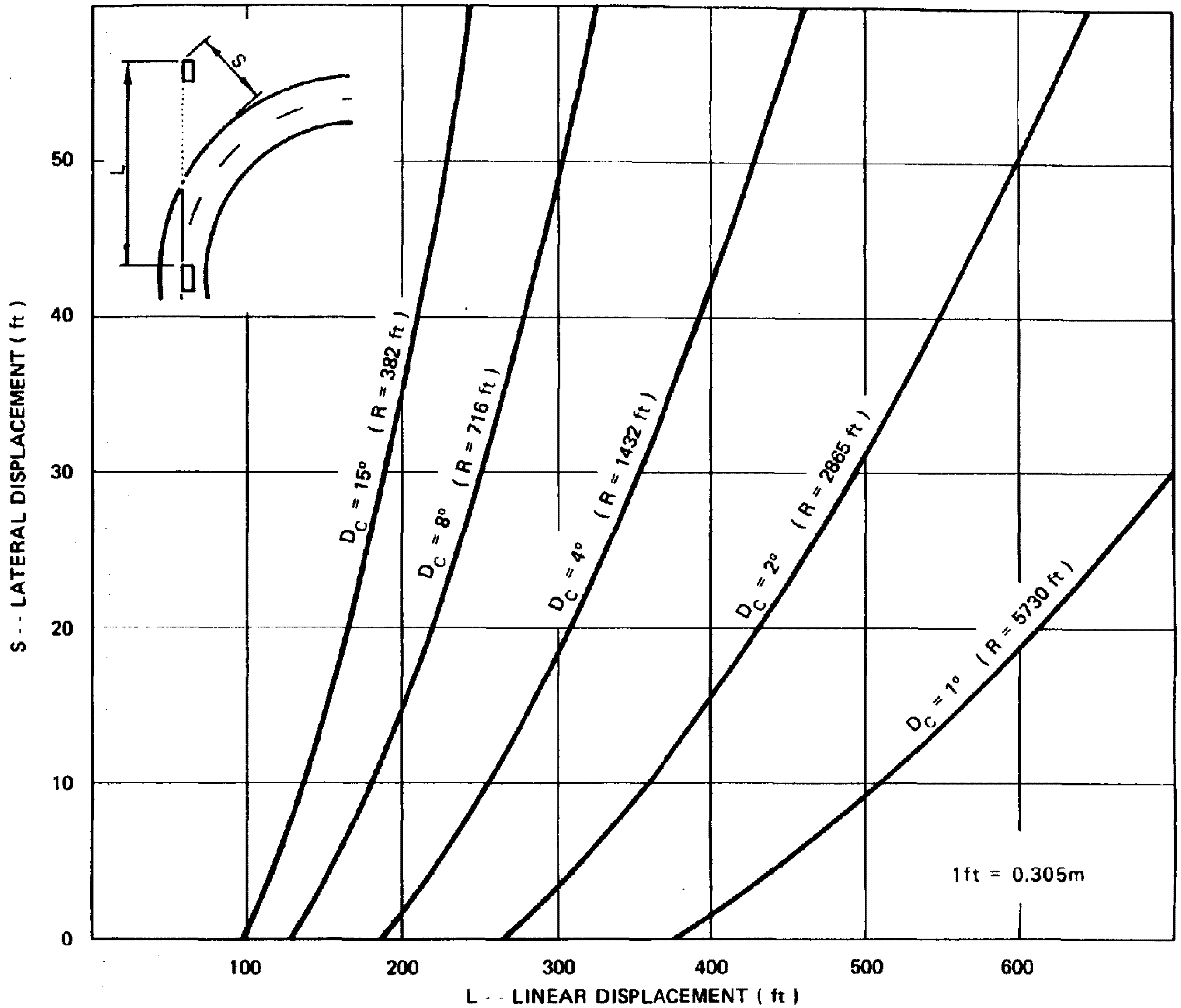


Figure 36. LATERAL DISPLACEMENT VERSUS LINEAR DISPLACEMENT FOR OUTSIDE ENCROACHMENTS FROM THE INSIDE LANE ON HIGHWAY CURVES

Figure 37 shows this relationship for outside encroachments from the outside lane. The appropriate equation is:

$$\theta_e = \cos^{-1}[R/(R + S)] \qquad [8.6]$$

Where θ_e = Effective slope angle (degrees)

R and S as before

This figure indicates that outside encroachments from the outside lane on flat highway curves may traverse flatter effective slopes than for right side encroachments on highway curves. Also, Figure 37 indicates the converse for sharp highway curves.

Figure 38 shows the relationship between effective slope angle and lateral displacement for outside encroachments from the inside lane. The appropriate equation is:

$$\theta_e = \cos^{-1}[R/(R + S + W)] \qquad [8.7]$$

Where θ_e, R, S, and W as before

The relationships shown in Figure 38 for encroachments from the inside lane are similar to those shown for the outside lane in Figure 37, except the effective angles are more severe for a given lateral displacement. However, the differences between effective slope angles from the inside lane of a highway curve and the left side of a highway tangent indicate a less critical comparison than that shown in Figure 37.

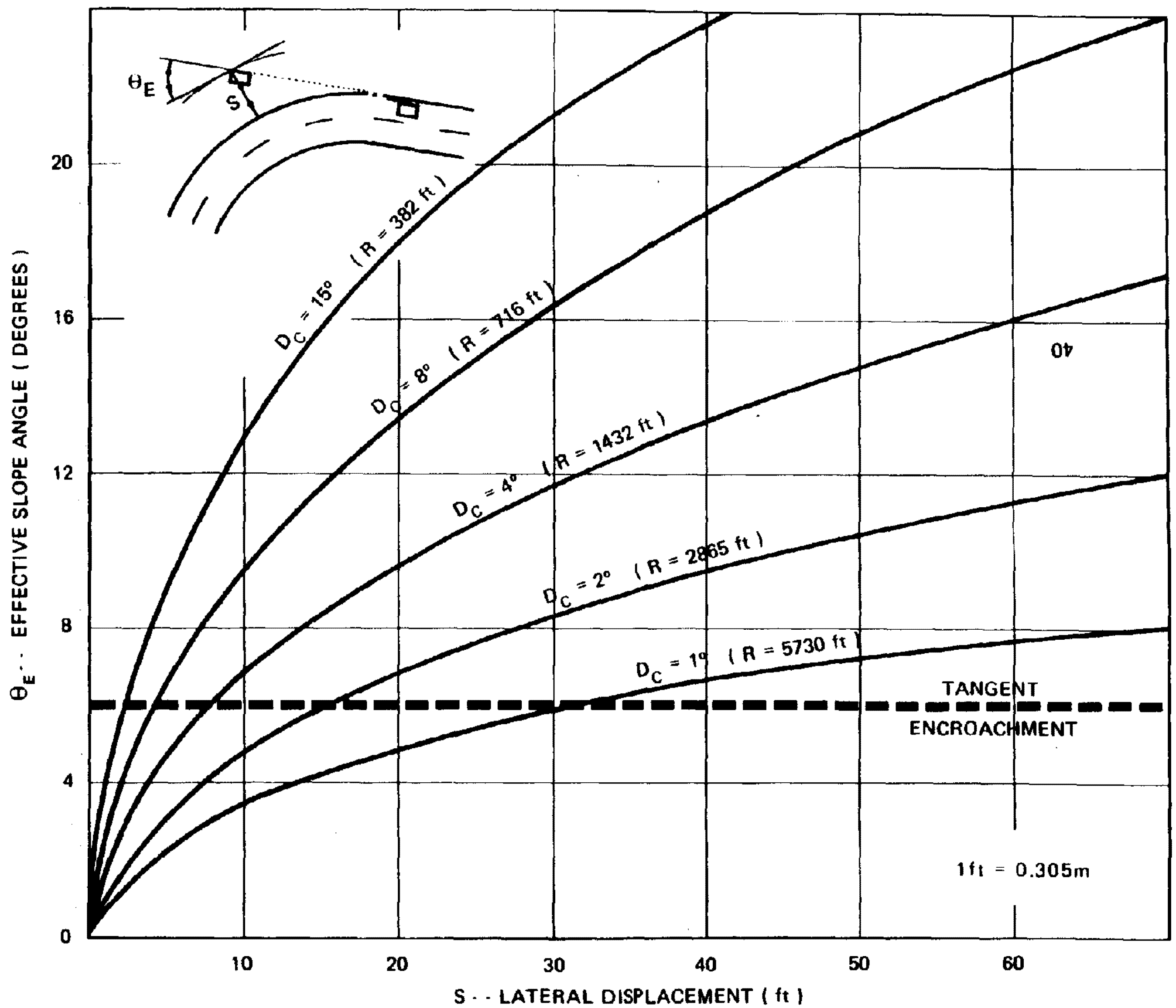


Figure 37. EFFECTIVE SLOPE ANGLE VERSUS LATERAL DISPLACEMENT FOR OUTSIDE ENCROACHMENTS FROM THE OUTSIDE OF A HIGHWAY CURVE

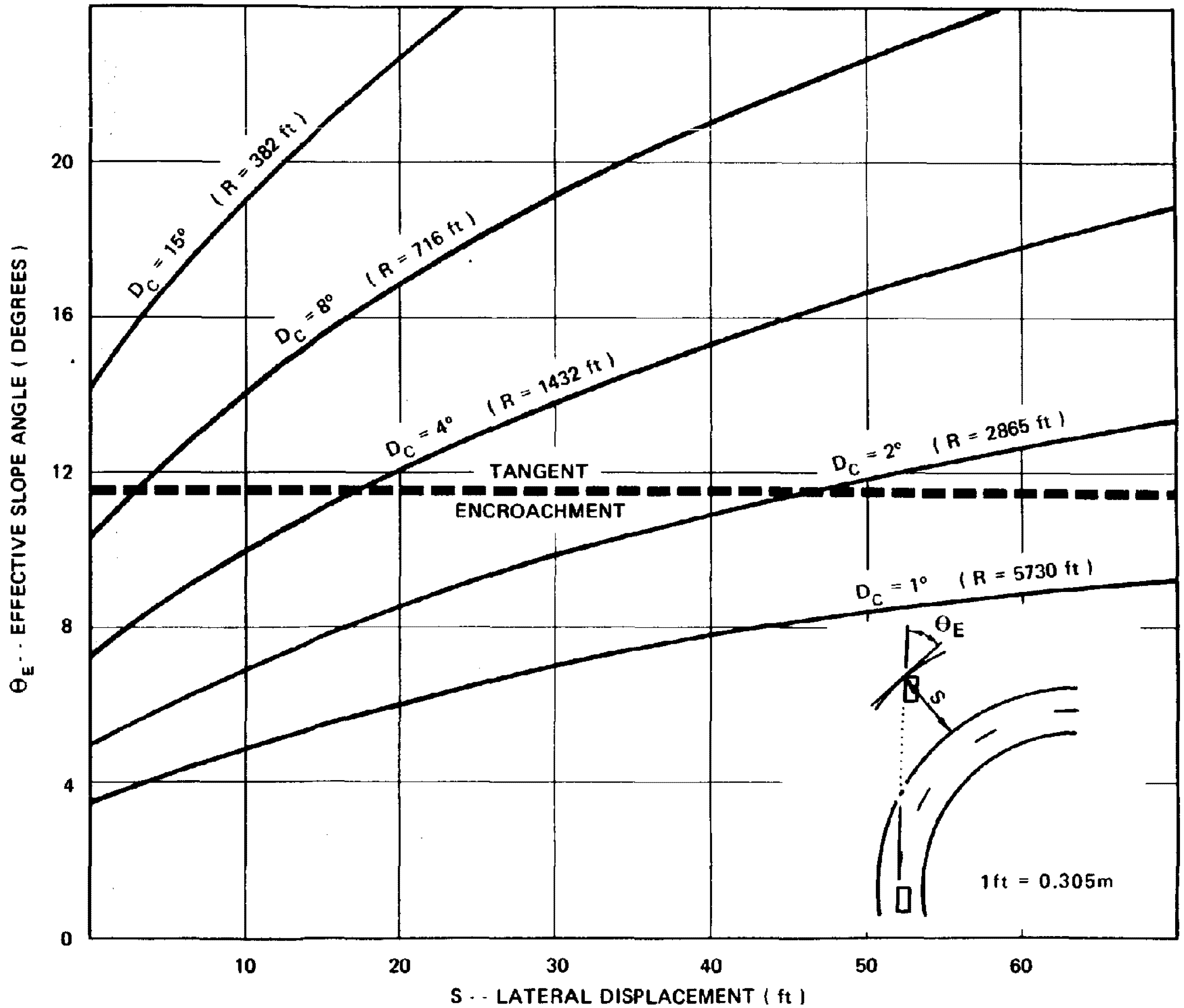


Figure 38. EFFECTIVE SLOPE ANGLE VERSUS LATERAL DISPLACEMENT FOR OUTSIDE ENCROACHMENTS FROM THE INSIDE LANE OF A HIGHWAY CURVE

Maximum Lateral Displacement Versus Initial Traversal Angle
For Inside Encroachments

This analysis attempts to show the effect of clear-zone widths for the inside of highway curves of various radii as a function of initial traversal angle. Although this analysis assumes a linear path and the same initial lateral placement and lane width as before, the path must have an initial angle to the local tangent on the highway curve. For these assumptions Figures 39 and 40 show the relationships between maximum lateral displacement and initial traversal angle for various highway curve radii.

Figure 39 shows this relationship for inside encroachments from the inside lane on highway curves. The appropriate equation is:

$$S = (1 - \cos \Theta_I)\ (R - W) \qquad [8.8]$$

Where Θ_I = Initial traversal angle with local tangent to highway curve (degrees)

S, R, and W as before

The figure indicates that, even with very extreme initial angles, the maximum inside lateral displacement on sharp highway curves will be relatively small. Conversely, for flat highway curves the maximum lateral displacement can be relatively large for even small initial angles (although fairly large linear displacements are necessary to achieve maximum lateral displacements).

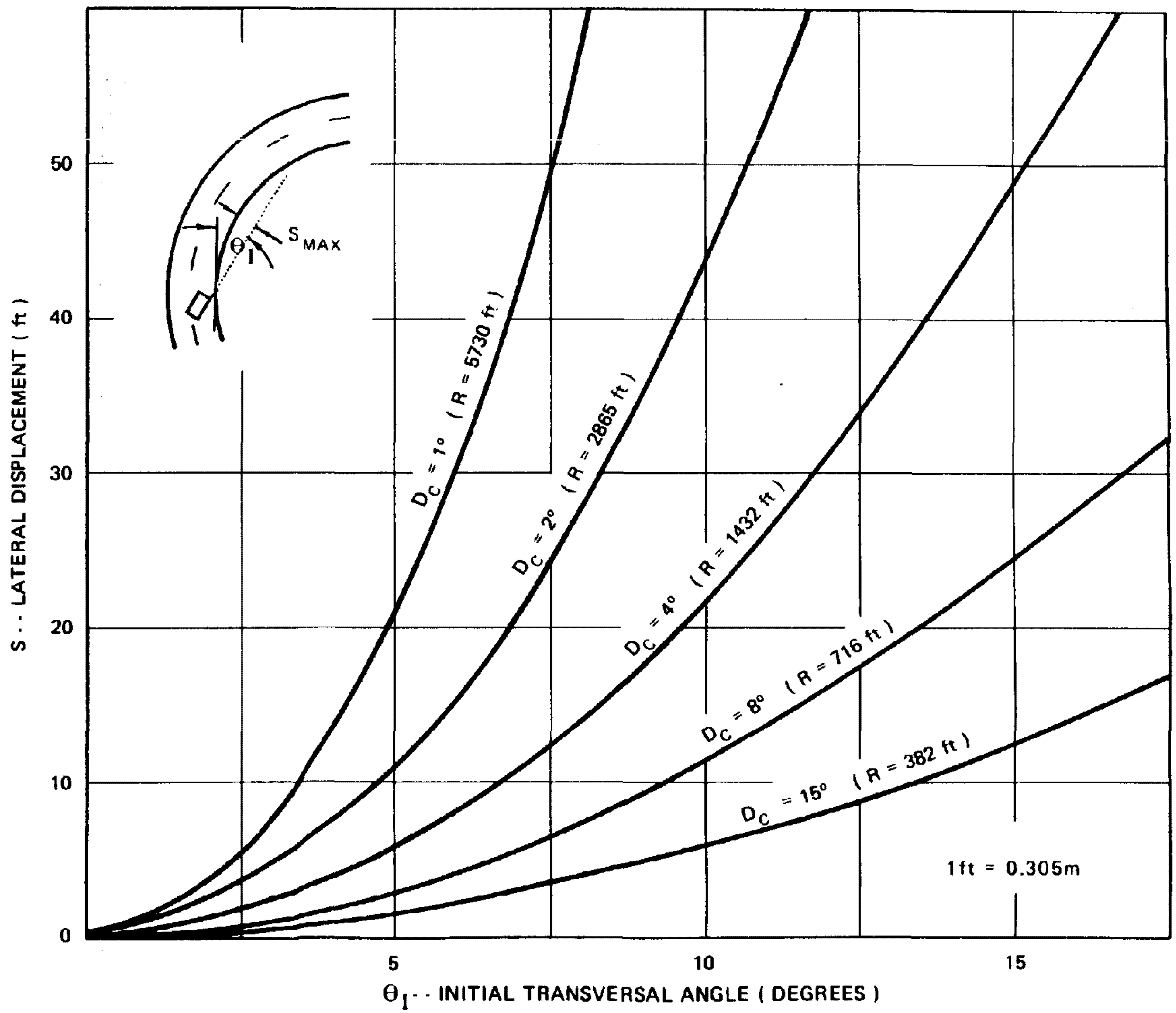


Figure 39. MAXIMUM LATERAL DISPLACEMENT VERSUS INITIAL TRAVERSAL ANGLE FOR INSIDE ENCROACHMENTS FROM THE INISDE LANE ON HIGHWAY CURVES

Figure 40 shows the relationship between lateral displacement and initial traversal angle for inside encroachments from the outside lane of various highway curves. The appropriate equation is:

$$S = R\ (1 - \cos \Theta_I) - W \qquad [8.9]$$

Where S, R, Θ_I, and W as before

As expected, this relationship shows less extreme maximum lateral displacements than for encroachments from the inside lane.

<u>Summary of Roadside Encroachments on Highway Curves</u>

Although this series of analyses does not begin to explain the very extreme complexity of roadside encroachments on highway curves, it does illustrate some important general principles:

(1) Encroachments on the outside of sharp highway curves appear to require greater clear-zone widths than do encroachments on highway tangents. For flat highway curves, the converse may be true.

(2) Encroachments on the outside of sharp highway curves appear to require flatter roadside slopes than on highway tangents. Again the converse appears true for very flat highway curves.

(3) Encroachments on the inside of sharp highway curves may require less clear-zone width than on highway tangents or on the outside of the same curves. Again, the converse appears true for flat highway curves.

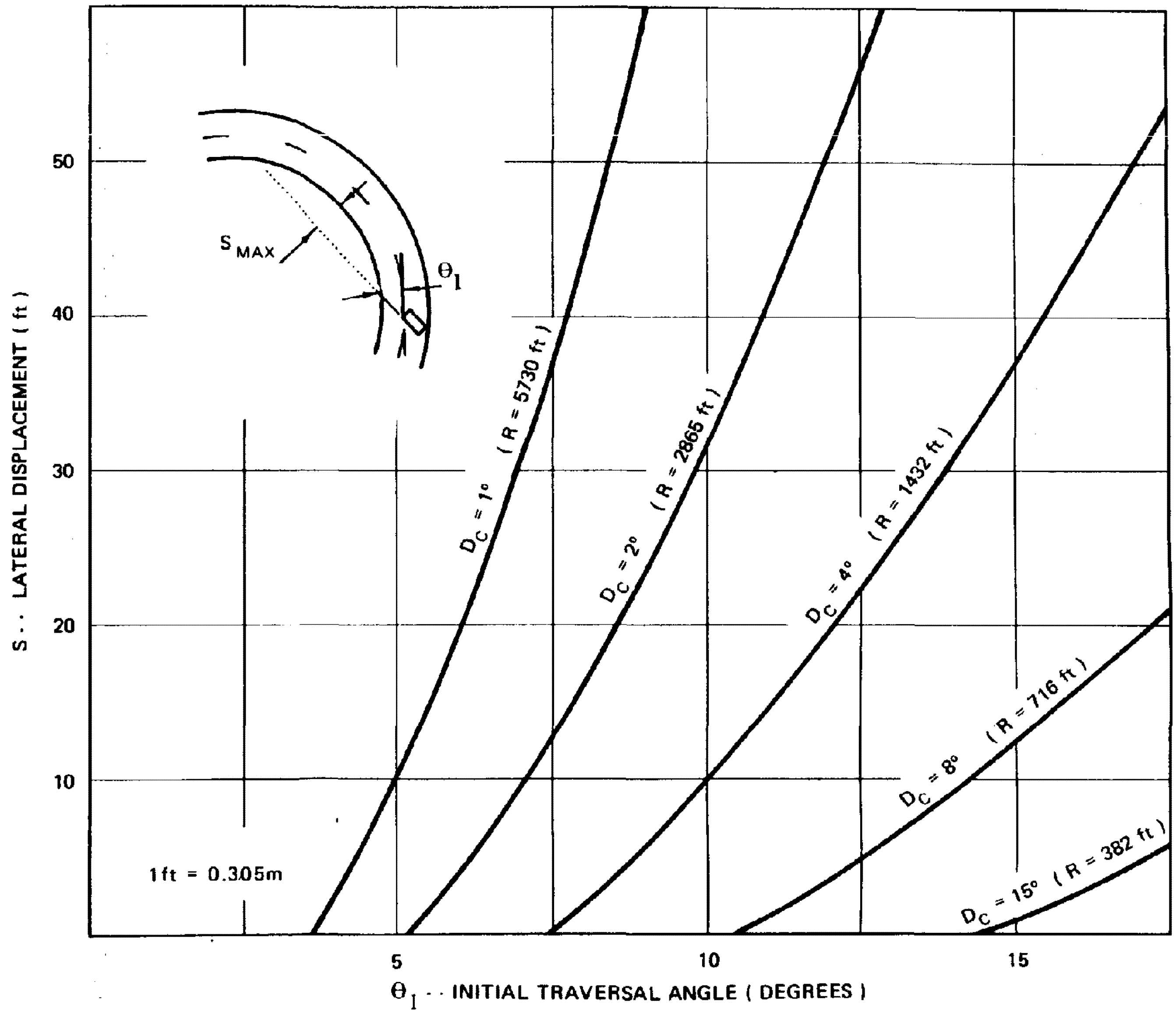


Figure 40. MAXIMUM LATERAL DISPLACEMENT VERSUS INITIAL TRAVERSAL ANGLE FOR INSIDE ENCROACHMENTS FROM THE OUTSIDE LANE OF HIGHWAY CURVES

Dynamic Effects of Pavement Settlement or Washboard on Highway Curves

Anyone driving on a highway curve with pavement washboard or a short bump quickly realizes that the vehicle control stability is affected by these vertical irregularities. Very short, high-amplitude bumps cause both vertical and lateral wheel hop. Successive loading and unloading of first front and then rear tires, with contingent wheel hop, greatly increases the effective lateral acceleration on the tire. In addition, as reported by Klein, et al. (39), loss of steering authority also occurs, which forces the driver to input larger steering angles than expected.

Appendix F presents a simplistic analytical exercise aimed at achieving a better understanding of vertical irregularities on highway curves. If the irregularity is described as having an effective radius, then the standard centripetal force equation can be expanded to include a term accounting for the additional lateral acceleration component created by the irregularity.

The derived form of the centripetal force equation stated in highway design terminology is as follows:

$$(e + f)/(1 - ef) = V^2/15R \qquad [8.10]$$

Where V = Vehicle Speed (mph)
R = Vehicle Speed Radius (ft)
e = Superelevation Rate (percent $\div$ 100)
f = Coefficient of tire friction

NOTE: 1 mph = 1.609 km/h
1 ft = 0.305 m

Note the term, $1 - ef$, in the denominator of the left side of the equation. In AASHTO design policies, this term is rounded numerically to one, with the assumption that the product ef is a very small term. However, in using the equation to analyze more extreme dynamics, this term can have a significant effect on the resulting calculations.

Applying the same derivation of procedure for the vehicle dynamics of cornering on a horizontal curve with a vertical irregularity, Appendix F shows the derivation of the centripetal force equation with a component for the vertical irregularity. Using an effective circular radius for the irregularity, the derived equation is as follows:

$$(e + f)/(1 - ef) = V^2/[R_p(15 \pm V^2/R_v)] \qquad [8.11]$$

Where R_p = Vehicle Path Radius (ft)
R_v = Radius of vertical irregularity (ft)
V, e and f as before

NOTE: 1 ft = 0.305 m

Note the dual sign in the denominator of the right side of the equation. This sign is negative for a bump and positive for a dip.

This equation generally explains the lateral acceleration on the tires for the steady-state condition up to the point of take-off, at which point it no longer applies. Therefore, for very large vertical radii, very little extra dynamic effect on the lateral acceleration will be experienced. But, for a given vehicular speed, the extra dynamic effect on lateral acceleration will be substantial as the take-off radius is approached. The derived relationship between take-off radius and take-off speed is as follows:

$$V_{to} = \sqrt{15R_{to}} \quad [8.12]$$

Where R_{to} = Take-off radius of vertical irregularity (ft)
V_{to} = Take-off speed (mph)

NOTE: 1 ft = 0.305 m
1 mph = 1.609 km/h

A similar relationship expressed in the dimensions of a parobolic vertical curve is as follows:

$$V_{to} = \sqrt{1500\ L/A} \quad [8.13]$$

Where L = Length of vertical curve (ft)
A = Algebraic difference in grades, percent

NOTE: 1 ft = 0.305 m

Although no attempt is made here to calculate the effects of washboard, it can be surmised that, for a nominally critical vehicle traversal at design speed on a highway curve, the driver might easily lose control of the vehicle. Single irregularities become critical for nominally critical traversals at design speed when their dimensions reduce the normal forces on the tire to almost zero. With a constant side force applied in cornering, a normal force close to zero will produce very high lateral acceleration on the tire.

IX. COST EFFECTIVENESS OF COUNTERMEASURES TO SAFETY PROBLEMS ON CURVES

An important task is evaluation of the effects of improving certain roadway elements to achieve safety and operational benefits. In a traditional analysis, comparison of marginal benefits and costs provides a means for judging the merits of improvements. Such analysis requires reasonable measures of effectiveness for given improvements. These measures should desirably be in the form of benefits associated with incremental levels of improvement.

Evaluation of highway safety improvements usually focuses on accident reductions as the primary benefit. In the case of two-lane rural highway curves, one might wish to know the effectiveness of widening the roadway from 20 to 22 or 24 feet (6.1 to 6.7 or 7.3 m); or of reducing curvature from 8° to 5° or 3°; or of instituting a range of clear-zone policies; or of various combinations of all of these or other improvements.

Figure 41 diagrams the process required to perform cost-effectiveness analysis of the incremental effects of highway safety improvements. In the case of accident relationships, it is crucial that each step of the process be carefully followed. It is particularly important that the proper analysis technique be used to sort out interactions among important variables and identify incremental effects of all variables.

The process outlined in Figure 41 was followed in the development and execution of the data collection and analysis plan described earlier. A large data sample was required to measure interactions and effects of variables such as roadway and shoulder width, curvature, and traffic volume. Analysis of Covariance (AOCV) was chosen as the evaluation tool because it offered the only clear way to identify incremental effects of the variables. However, as was discussed in Chapter IV, the AOCV results were of limited value because important variables such as roadside and pavement conditions were not available from state data files.

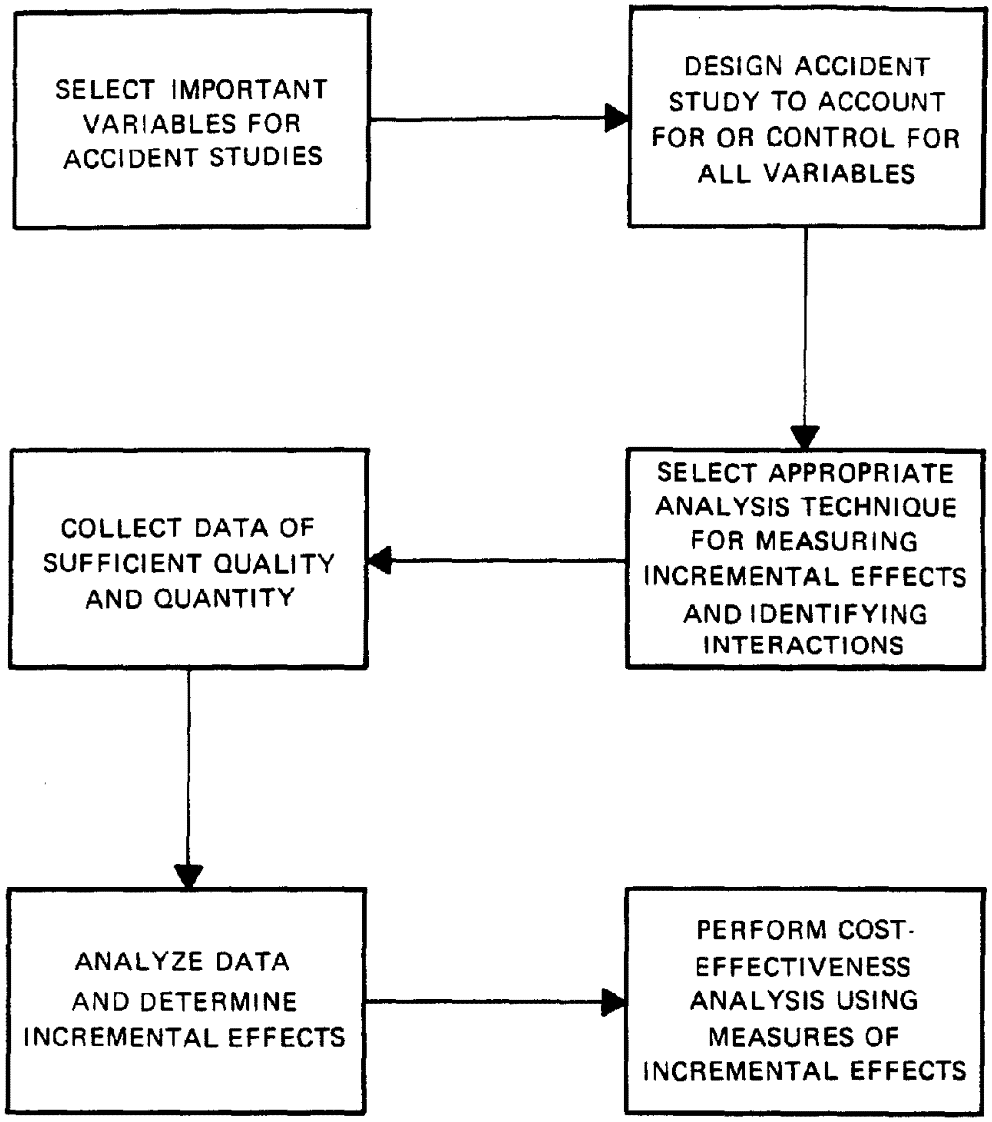


Figure 41. COST-EFFECTIVENESS ANALYSIS PROCESS

Due to the limited success of AOCV, the type and scope of cost-effectiveness analysis originally planned was not possible. Meaningful safety/geometric relationships discovered in the Discriminant Analysis, however, provided a means for a more general estimate of the cost-effectiveness of certain highway improvements.

Discriminant Analysis Applied to Effectiveness of Highway Improvements

The results of the Discriminant Analysis form the basis for evaluating the effectiveness of changes in geometrics with respect to changes in expected accident occurrence. The best equation for describing safety/geometric relationships was found to be:

$$D = 0.0713\ (DC) + 2.9609\ (LC) + 0.1074\ (RR) - 0.0352\ (PR) - 0.1450\ (SW) - 1.5454 \qquad [9.1]$$

Where D = Discriminant Function (nondimensional)

DC = Degree of curve

LC = Length of curve (mi)

RR = Roadside rating

PR = Pavement rating

SW = Shoulder width (ft)

NOTE: 1 mi = 1.609 km
1 ft = 0.305 m

The discriminant score is directly related to an associated probability that a site is a high-accident location. This relationship, which was shown in Figure 6, is a function of the D distributions for the high-accident and low-accident site populations.

It was found that certain dimensional combinations of the above characteristics produce a high probability that a site will have a very high accident experience. Conversely, other dimensional combinations of the same characteristics produce very low accident experience probabilities.

If the D score relationship is to be used in a cost-effectiveness evaluation of highway curves with a wide range of characteristics, a reasonable estimate of accident rates across the entire range must be derived. For example, if a highway curve's characteristics correspond to a D score which produces a 90 percent probability that the curve is a high-accident site, it would be reasonable for analysis purposes, to assign a high accident rate to that location. The actual rate used could be consistent with the typical or average accident rates which comprised the high-accident data base. Similarly, a very low rate (say, zero) could be assigned to curves with very low probabilities of being high accident sites.

Cost-effectiveness analysis, however, requires reasonable estimates of accident rates for curves with any probability of being high- or low-accident sites. Put another way, it is necessary to develop a means for estimating accident experience across the full range of geometrics that occur.

The basic task, then, lies in attempting to characterize accident rates for locations that neither are clearly "high" or "low" accident sites. This is necessary because the Discriminant Analysis did not include, and was not intended to measure, characteristics of "average" accident sites. Nevertheless, it is believed that the underlying geometric/accident relationships discovered in the analysis are applicable in some fashion to <u>all</u> curves. Two assumptions must be met if the Discriminant Analysis findings apply to all sites in a meaningful way:

(1) The discriminant score is assumed to describe cause/effect relationships (rather than merely correlative ones).

(2) Relationships between geometric elements and accident rate are continuous in nature. A continuous relationship has been identified through investigation of its "tails" (high- and low-accident sites).

Both assumptions appear logical and appropriate. The task thus becomes to define the accident rate versus discriminant score relationship for the full range of curve conditions.

Consider Figure 42, a schematic representation of relationships between accident rate and the probability of a curve being a high-accident site. Three possible and reasonable general forms are shown. Line A says that as the probability of being a high site (P(H)) increases, its expected accident rate increases proportionately. Line B hypothesizes that, as geometric conditions worsen slightly (producing a modest increase in P(H)), the expected accident rate sharply increases. Line C says that accident rates would only moderately increase until P(H) reaches some critical level, whereupon further degrading of geometry produces rapidly increasing accident rates.

Accident Rate Versus D Score for All Sites

Investigation of the form of the accident rate relationship required consideration of all curve conditions -- not just those characterized as high- and low-accident sites. This was accomplished by evaluation of Equation 9.2 which was a second discriminant function derived from the same high- and low-accident data base. Equation 9.2 was based on only data generally available from State geometry files. Because these geometry data were available for all sites, it is possible to test the hypothesis that discriminant analysis in some way predicts accident experience across the full range of geometrics.

$$D = 0.378(DC) + 3.209(LC) - 0.220(SW) + 0.289 \qquad [9.2]$$

Where D, DC, LC, SW are as before

Equation 9.2 was based on data from Florida, Illinois and Texas. A total of 2484 sites from the total data base for these three States were investigated. D scores were computed and their associated P(H) values obtained for all 2484 sites, and their accident rates recorded. The sites were partitioned according to percentile ranges of the P(H) distribution for Equation 9.2. For each

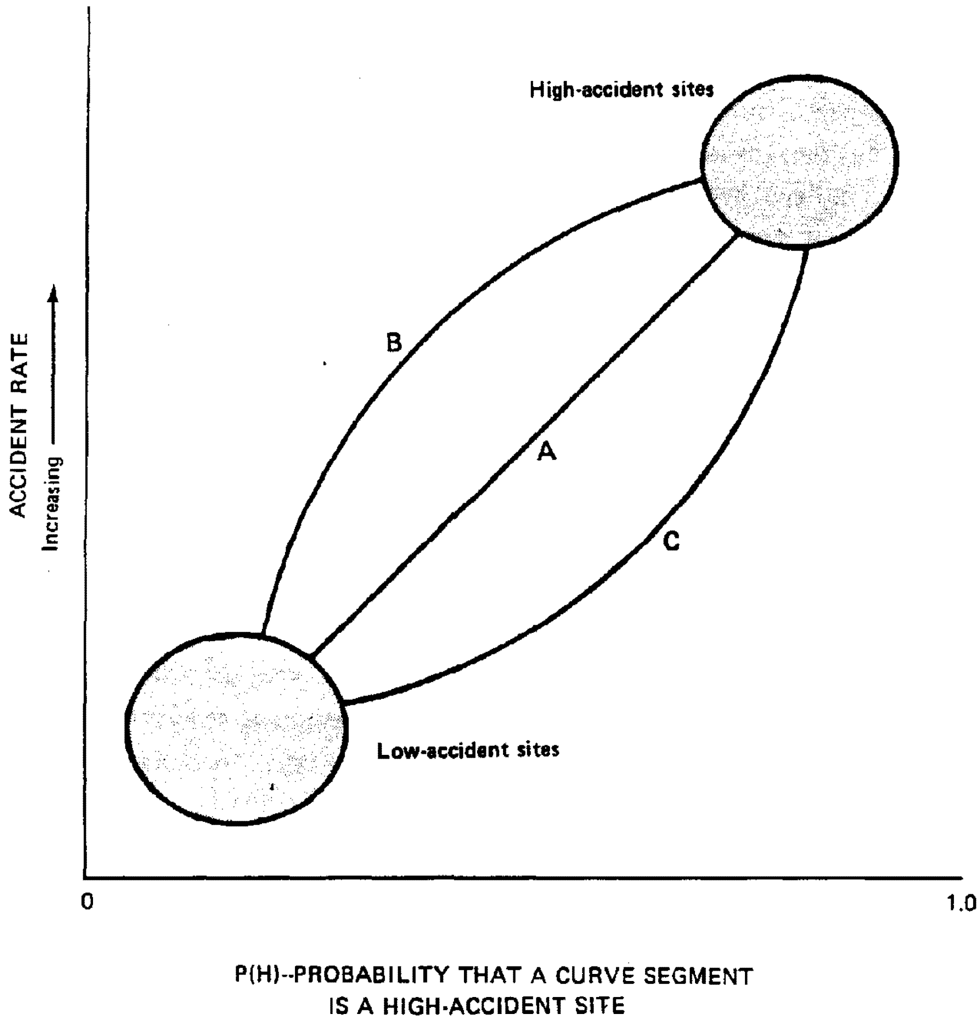


Figure 42. POSSIBLE RELATIONSHIPS BETWEEN ACCIDENT RATE AND P(H)

10 percent range of P(H), an overall accident rate was calculated as the arithmetic mean of the rates of all sites within the range. Table 40 shows the results of this analysis.

Table 40 depicts a logical, continuous relationship between average accident rate and D score. Furthermore, inspection of the distribution of D scores produces reasonable conclusions. A majority of sites (65.6 percent) could be characterized as average with respect to P(H) -- with values between 0.40 and 0.60. The results given in Table 40 verify the reasonableness of extending the discriminant analysis findings to analysis of the universe of curve sites.

Figure 43, a plot of average accident rate vs. P(H), is quite revealing. There appears to be little difference in average accident experience between curves with very good geometry (P(H) less than 0.30), and those with average geometry. Furthermore, it appears that only when a curve's geometry produces P(H) of at least 0.70 would an accident rate significantly greater than average be expected.

These conclusions are consistent with earlier findings on the relationship between accidents and geometry. They provide direct focus to a study of cost-effectiveness of two-lane rural highway safety. The following points are significant:

(1) General knowledge about the random nature of accidents is consistent with the relationship shown in Figure 43. No matter how "good" the curve geometry is (flat curve, wide shoulders) a minimum number of reported accidents should be expected. The analysis indicates this minimum number is about 1.0 accidents per million vehicle-miles (0.62 accidents per million vehicle-kilometres) on rural highway curve sections.

(2) Significant accident rate increases are not expected until geometric conditions produce a P(H) of at least 0.70. This probability level represents a meaningful threshold for selection of possible conditions for treatment. It also identifies a level below which the marginal effectiveness of additional treatments is minimal.

TABLE 40

AVERAGE ACCIDENT RATES FOR PERCENTILES
OF P(H) FOR EQUATION 9.2

P(H)	'D' Score Range For Range of P(H)	Sites No.	Sites Percent of Total		Average Accident Rate (Mean of Sites Within P(H) Range) (Accidents per Million Vehicle-Miles)
<10%	<-2.90	0	-		-
10% - 19.9%	-2.90 to -2.01	3	0.1		0.61
20% - 29.9%	-2.00 to -1.35	63	2.5		1.10
30% - 39.9%	-1.34 to -0.84	333	13.4		1.05
40% - 49.9%	-0.83 to -0.36	542	21.8		1.06
50% - 59.9%	-0.35 to 0.15	643	26.0		1.20
60% - 69.9%	0.16 to 0.68	443	17.8		1.26
70% - 79.9%	0.69 to 1.32	263	10.6		1.53
80% - 89.9%	1.33 to 2.24	144	5.8		1.88
>90%	>2.24	50	2.0		2.62
		2484	100%	Average	1.26

1 mi = 1.609 km

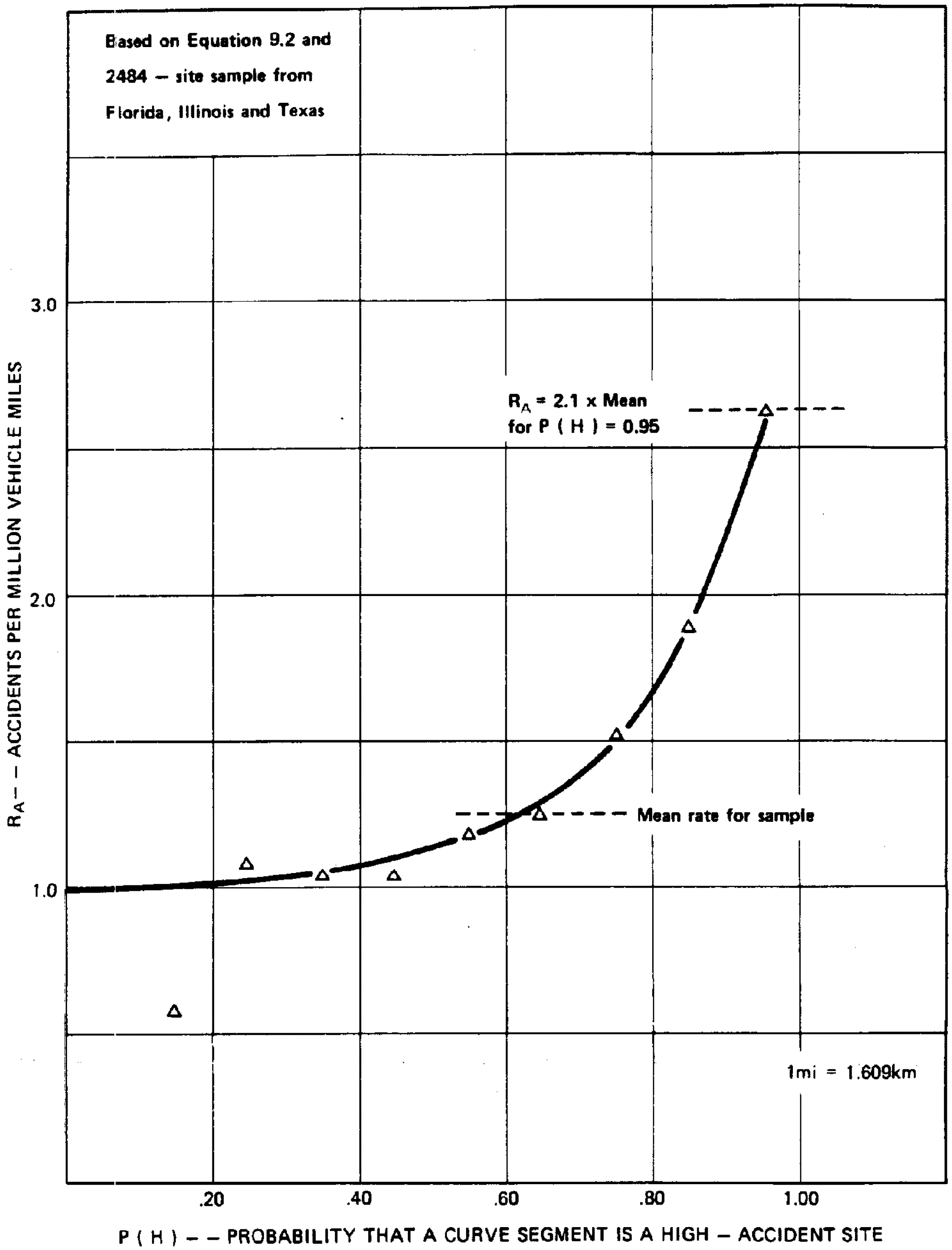


Figure 43. RELATIONSHIP BETWEEN ACCIDENT RATE AND P(H) FOR EQUATION 9.2

(3) The largest percentage effectiveness would be achieved by treating curve locations with P(H) of at least 0.80.

(4) Table 40 reveals that a relatively small percentage of rural highway curves deserves consideration for treatment. This has both positive and negative implications. First, it appears that treatment of only 2 to 8 percent of all curves would improve safety on rural highways. Specific programs designed to treat such curves may not require large expenditures. However, because so few highway curves are truly deserving of treatment (i.e., treatment could be expected to produce meaningful accident reductions), it is obviously necessary to correctly identify those sites.

Use of 5-Variable Discriminant Score Relationship

The previous discussion concerned the concept of relating the high- and low-accident analysis to all rural highway curves. By necessity, it utilized the D score equation which included only degree of curve, length of curve, and shoulder width. The 5-variable equation, which included roadside and pavement ratings as well, is believed more useful in expressing cause/effect relationships. It is therefore desirable to adapt the findings from the 5-variable equation to cost-effectiveness analysis of all highway curves, using the basic concepts developed with the 3-variable equation.

First, the link between the 3- and 5-variable equations (Equations 9.1 and 9.2) must be established. Table 44 compares the coefficients of the variables studied in both sets of analyses. In the 3-variable equation, degree of curve has a large coefficient and shoulder width a moderate coefficient. Once roadside and pavement ratings are added, the degree of curve coefficient drops sharply, and the shoulder width coefficient also is reduced. Table 42 explains why this occurs. Shoulder width is related to roadside rating. This should be expected, as one element of the roadside rating is proximity of objects to the edge of pavement. Curvature is also related to roadside rating. Sharper curves tend to have poorer roadsides. This is also logical, as lower class roads tend to have more sharp curves and uniformly poorer roadsides. Thus, the higher coefficients for degree of curve and shoulder width in the 3-variable equation appear partly to reflect their having acted as surrogates for roadside rating.

TABLE 41

COMPARISON OF 3- AND 5-VARIABLE DISCRIMINANT SCORE EQUATIONS

Variable	Coefficient 3-Variable Equation	5-Variable Equation
DC--Degree of Curve	0.378	0.071
LC--Length of Curve	3.209	2.961
SW--Shoulder Width	-0.220	-0.145
RR--Roadside Rating	-	0.107
PR--Pavement Rating	-	-0.035

TABLE 42

CORRELATION BETWEEN VARIABLES IN 5-VARIABLE DISCRIMINANT SCORE EQUATION (EQUATION 9.1)

Variable Pairs	Correlation Coefficient
DC vs. RR	0.25
DC vs. PR	-0.12
SW vs. RR	-0.36

It thus appears that degree of curve and shoulder width do not exclusively explain accident causation on curves. Furthermore, the sensitivity of degree of curve and shoulder width is not as great as the 3-variable equation predicts. Rather, roadside conditions and pavement friction also contribute to accident causation, thereby reducing the relative importance of curvature and shoulder width to a level indicated by the 5-variable equation.

In summary, both the 3- and 5-variable equations predict the relationship of accidents to geometrics of curves. The 5-variable equation better describes causative relationships, and thus better predicts the relative importance of degree of curve and shoulder width. It does so by including measures of roadside and pavement conditions.

Because the two equations are comparable it is believed that there is a relationship similar to that shown in Figure 43 which describes the 5-variable equation. It is not possible to directly generate such a relationship, however, as pavement and roadside rating data are not available for the entire population of sites. It _is_ possible to hypothesize an accident rate relationship if certain assumptions are made:

(1) Although P(H) is defined in terms of a different discriminant function, the relationship of P(H) to accident rate for the 5-variable equation is essentially the same as for the 3-variable equation. In other words, accident rate is relatively constant until P(H) exceeds 0.70, whereupon it increases rapidly with increasing P(H).

(2) Accident reporting levels in the four States are relatively consistent, enabling development of an accident rate function that is meaningful and useful.

The first assumption is reasonable given the previously discussed commonality of the data comprising each equation. Further analysis is necessary to accept the second assumption.

Effect of Accident Reporting Levels.--Difficulties arise whenever accident data from more than one jurisdiction are used. There is no way to guarantee that reporting levels among the four States are consistent. Indeed, initial inspection of the data would indicate that reporting levels are quite different. Table 43 shows a wide range of mean accident rates for the sites selected in the four States, with Ohio's mean rate particularly high. However, careful inspection of each State's geometry data base leads to explanations for the differences in reporting levels. Table 44 shows mean values for the five variables in the discriminant analysis, categorized by State. Clearly, the data base from Ohio includes much worse conditions than the other States (recall that Ohio data were restricted to curves of at least 3°). Further analysis shows Florida and Texas to have better geometry and conditions than Illinois. The roadside is better, shoulders are wider, and curvature is milder in these two States. It was concluded that much of the difference in accident rates among the States is attributable to variable geometry.

TABLE 43

MEAN ACCIDENT RATES FOR SITES IN ACCIDENT DATA BASE BY STATE

State	Number of Curve Sites in Sample	Mean Accident Rate (Accidents per Million Vehicle Miles)
Florida	839	1.34
Illinois	466	1.81
Ohio	765	3.47
Texas	1227	1.13
Overall	3297	1.82

1 mi = 1.609 km

TABLE 44

COMPARISON OF DISCRIMINANT ANALYSIS
GEOMETRY DATA BASES BY INDIVIDUAL STATE

State	Degree of Curve	Length of Curve (mi)	Pavement Rating	Shoulder Width (ft)	Roadside Rating
	Mean Values for High Accident Sites				
Florida	2.4	0.25	36.1	7.5	29.0
Illinois	3.2	0.17	36.0	5.8	32.8
Ohio	11.8	0.05	32.4	5.9	34.1
Texas	2.7	0.18	37.0	7.6	32.0
	Mean Values for Low Accident Sites				
Florida	1.4	0.21	40.7	8.9	26.3
Illinois	1.7	0.15	36.4	8.6	30.1
Ohio	6.5	0.08	35.1	8.2	30.1
Texas	1.3	0.16	38.9	8.4	26.8

1 mi = 1.609 km
1 ft = 0.305 m

It is not possible to quantify in terms of accident experience the differences in geometry among the four States. The question thus remains as to what overall accident experience should be attributed to various values for P(H). One possible solution would be to use the values shown in Figure 43, which was based on data from Florida, Illinois and Texas. However, data from Ohio were input to the 5-variable relationship, which is the intended basis for the cost-effectiveness evaluation. It seemed reasonable, therefore, to develop a curve which also reflects accident experience observed in Ohio.

Effectiveness Relationship for Accidents Versus P(H).--Given that the geometric conditions and accident histories of all four States were included in the 5-variable discriminant analysis, the effectiveness curve should reflect overall accident values. It was thus believed appropriate to adjust the curve of Figure 43 to account for the effect of Ohio accident experience. This adjustment was accomplished using the following process:

(1) Ohio site data were studied. It was determined that, primarily because Ohio curves were much sharper than the other States' highway curves, the distribution of P(H) for Ohio curve sites was greatly skewed toward the range 0.50 to 0.90. The curve of Figure 43 was therefore believed to be representative of the entire data base for values of P(H) less than 0.50.

(2) Inspection of Figure 43 shows the smoothed accident rate for P(H) of 95 percent to be 2.1 times the mean rate. This is believed to be a reasonable, conservative value given (i) all high-accident sites had rates at least twice a State's mean rate; (ii) not all sites with P(H) of 95 percent are guaranteed to have rates twice the mean; (iii) but many sites with P(H) of 95 percent have rates three or more times the mean rate.

(3) The mean rate of 1.26 for the data in Figure 43 occurs at about P(H) of 65 percent. This was also believed reasonable.

(4) The overall mean accident rate for all 3297 curves in the data base is 1.82 accidents per million vehicle-miles (1.13 accidents per million vehicle-kilometres). This is the best available estimate of average accident experience for the data base employed in the research.

(5) A curve taking the shape of that curve in Figure 43 was generated, with an accident rate of 2.1 times the overall mean rate of 1.82 assigned to P(H) of 95 percent. This curve was graphically transitioned back to the original curve for P(H) less than 50 percent.

Figure 44 shows this adjusted curve, which was adopted for the cost-effectiveness analysis.

Cost Effectiveness Analysis -- Some Caveats

The analysis of alternative treatments to rural highway curve problems is at best of a general or advisory nature. The researchers emphasize the limitations of the safety relationships and the many problems inherent in any analysis of this type. Review of the limitations and problems is appropriate here:

(1) Development of incremental accident rate changes for the full range of conditions is not a typical use of discriminant analysis. That the research uncovered any safety/geometric relationships at all is mainly attributed to the initial, purposeful divergence in accident rates between the two groups of sites evaluated. It is thus conceded that extension of these relationships in the manner presented here is tenuous. However, the choice was made to proceed with the model for accident rates for what are believed to be valid and important reasons. First, it is clear that relating accident rate to probability levels "works" in the sense that it produces logical, continuous results when applied to all curve conditions (not just high- and low-accident sites). Second, for better or worse, it is the best available tool for investigating cost-effectiveness within the research framework. Third, the intention in presenting and using it is extremely limited. If, by applying the relationship in a conservative manner, the list of potential countermeasures (or site conditions) can be narrowed, the exercise will be of value.

(2) The Discriminant Analysis suffers from a statistical limitation common to multiple linear regression. There is no ability to evaluate effects of interactions among variables in the relationship. The discriminant coefficients explain a relationship between accidents and each variable only in terms of an average contribution of each variable. For example, there is no real way, to estimate the true effectiveness of an 8-foot vs. a 4-foot (2.4 m vs. 1.2 m) shoulder for various curve designs and/or roadside conditions.

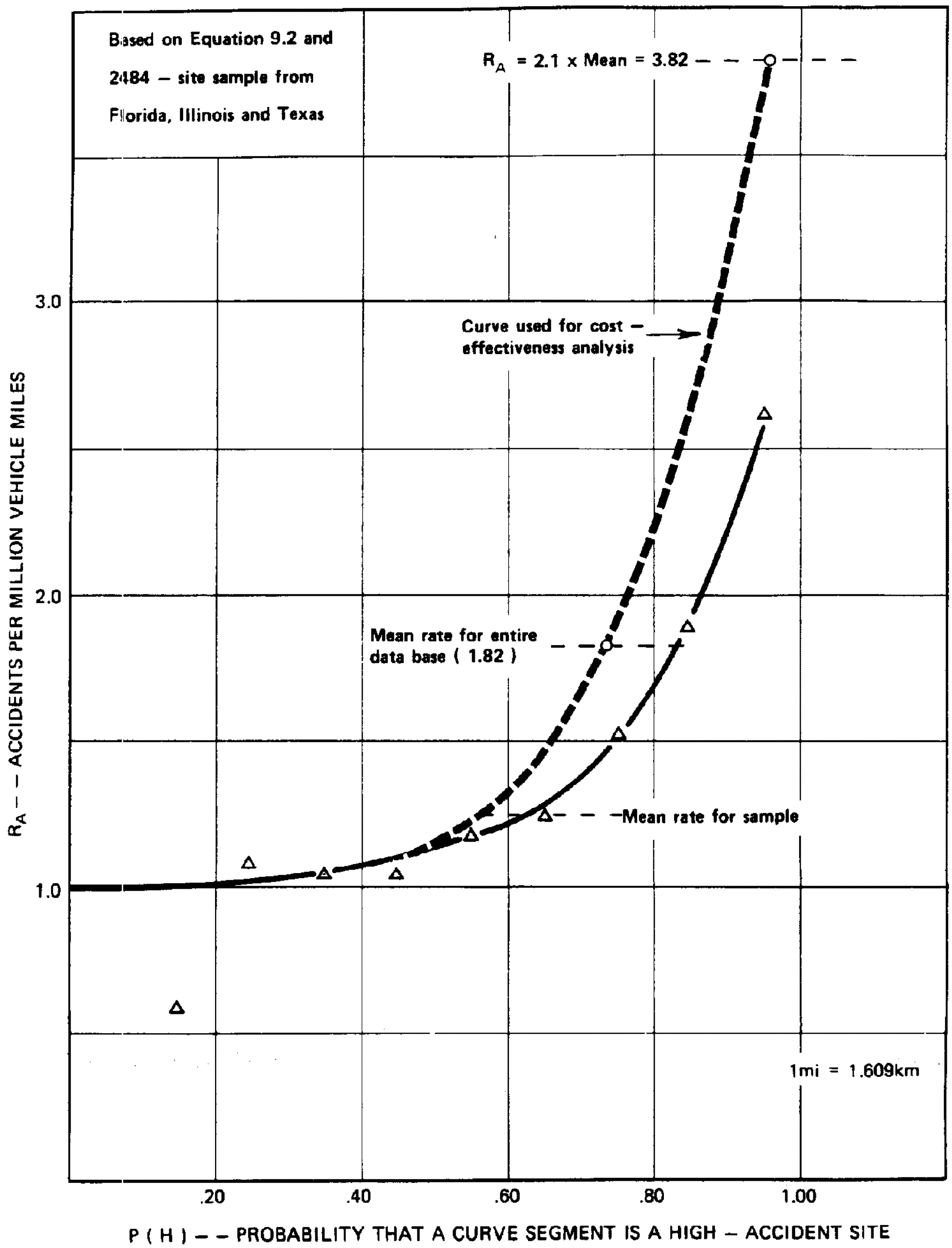


Figure 44. RELATIONSHIP BETWEEN ACCIDENT RATE AND P(H) FOR COST-EFFECTIVENESS ANALYSIS

(3) Cost-effectiveness analysis must reflect local or regional cost elements to be totally useful. The scope of this task does not allow, nor does the accuracy of the safety relationships warrant, investigation of a range of cost values.

(4) A number of factors, including a dollar value for accidents, analysis period assumptions, and interest rates all influence the results. Again, while such variables would normally be handled with sensitivity analyses, the scope of this study does not justify such investigation.

The cost-effectiveness analysis was carried out with these caveats in mind. The analysis presented here reflects general consideration of basic types of countermeasures to safety problems on rural highway curves. The results are not intended to be used as a site-specific indicator of cost-effective improvements. Rather, the results offer guidance toward development and application of system-wide programs of countermeasures.

Analysis Parameters

The cost-effectiveness analysis technique employed was a benefit/cost method. Highway user benefits (expressed in terms of dollar savings resulting from accident reductions, operating cost savings, and reduced maintenance costs) were compared to capitalized costs of construction. The results of the analysis indicated general levels of traffic volume at which alternative countermeasures would be cost-effective, i.e., at which the benefit/cost ratio would be 1.0.

Application of any form of economic analysis requires the selection of an appropriate discount rate (interest rate) and an assumed project life. Guidelines for these factors are provided in the 1977 AASHTO Manual on User Benefit Analysis of Highway and Bus-Transit Improvements (40).

Discount Rate

The selection of an appropriate discount or interest rate is based on two factors: (1) the method of valuing benefits and costs, and (2) the relative risk involved in the project being evaluated. In the analysis presented here all costs have been calculated based on 1982 unit costs, with no factor for future inflation. The risk involved in evaluating the cost-effectiveness of countermeasures to rural safety problems is in two basic areas. The first concerns the safety relationships which form the basis for user benefit calculations. Any uncertainties and inaccuracies in the development and application of the safety relationships results in an element of risk. The second element of risk relates to how the actual results are to be used. The research team, with knowledge of the shortcomings inherent in the process applied, intend the results to be advisory and illustrative. It is recognized that adoption of the cost-effectiveness findings by a State must be preceded by analysis of conditions and costs experienced by that State.

Based on the above considerations, a discount rate of 7 percent was chosen. This represents a moderate assessment of risk.

Average Cost of Accidents

Average accident costs were derived from a number of sources, including AASHTO Manual on User Benefit Analysis of Highway and Bus-Transit Improvements (40), 1975 Societal Costs of Motor Vehicle Accidents (41), and Fatal and Injury Accident Rates on Federal-aid and Other Highway Systems/1975 (42). Costs of accidents based on 1975 figures were updated to 1982 assuming a 5 percent compounded inflation rate. This resulted in base costs as given in Table 45. These costs were further adjusted to reflect distributions of fatalities and injuries in serious accidents. Data for the four States in the large data base were used. Table 46 shows summaries for fatalities and injuries per serious accident, which were derived from the 4-State data base and other sources.

The data from Tables 45 and 46 were used to calculate average accident costs as follows:

$$\begin{array}{c}\text{Average Cost}\\ \text{of a 1982}\\ \text{Fatal Accident}\end{array} = \left[\begin{array}{c}\text{Value}\\ \text{of a}\\ \text{Fatality}\end{array} \times \begin{array}{c}\text{Fatalities}\\ \text{Per Fatal}\\ \text{Accident}\end{array}\right] + \left[\begin{array}{c}\text{Value}\\ \text{of an}\\ \text{Injury}\end{array} \times \begin{array}{c}\text{Injuries}\\ \text{Per Fatal}\\ \text{Accident}\end{array}\right]$$

$$= \$404{,}084\ (1.205) + 4{,}482(2.03) = \$496{,}019$$

$$\text{Average Cost of a 1982 Injury Accident} = \left[\text{Value of an Injury} \times \text{Injuries Per Injury Accident}\right]$$

$$= \$4{,}482\ (1.596) = \$7{,}153$$

$$\text{Average Cost of a 1982 Severe (Fatal or Injury) Accident} = \left[\text{Cost of Fatal Accident} \times \text{Percent Fatal Accidents of Severe Accidents}\right]$$

$$+ \left[\text{Cost of Injury Accident} \times \text{Percent Injury Accidents of Severe Accidents}\right]$$

$$= \$496{,}019\ (.0559) + \$7{,}153\ (.9441) = \$34{,}481$$

$$\text{Average Cost of a 1982 Accident on Rural Highway Curve Sections} = \left[\text{Cost of Severe Accident (34,481)} \times \text{Percent Severe Accidents of Total Accidents (0.415)}\right]$$

$$+ \left[\text{Cost of Property Damage Only Accidents (732)} \times \text{Percent Property Damage Only Accidents of Total Accidents (0.585)}\right]$$

$$= \$14{,}738 \quad \text{say } \$\underline{14{,}700}$$

TABLE 45

BASE COSTS FOR ACCIDENTS

Item	1975 Dollar Value[1]	1982 Adjusted Value[2]
Motor Vehicle Fatality	$ 287,175	$ 404,084
Motor Vehicle Injury	$ 3,185	$ 4,482
Property Damage Only Accident	$ 520	$ 732

[1]Source: Reference (41)

[2]Inflated by a factor of 1.4071 which represents 5 percent inflation compounded over 7 years.

TABLE 46

DISTRIBUTION OF INJURIES AND FATALITIES IN
RURAL HIGHWAY ACCIDENTS

State	Fatal Accidents (1)	Fatalities (2)	(2)÷(1) (a)	Injury Accidents (3)	Injuries (4)	(1)÷ [(1)+(3)] (b)	Injuries in Fatal Accidents* (5)	[(4)-(5)] ÷(3) (c)	Number of Curves in Sample
Florida	528	634	1.201	11,299	20,274	0.0446	1072	1.699	839 (25.5%)
Illinois	570	682	1.196	12,060	20.734	0.0451	1157	1.623	466 (14.1%)
Ohio	522	605	1.159	16,561	27,212	0.0305	1060	1.579	765 (23.2%)
Texas	1437	1783	1.241	15,705	26,902	0.0838	2917	1.527	1227 (37.2%)

Weighted Averages Used in Analysis

(a)	Fatalities per fatal accident	1.205
(b)	Fatal accidents as percent of severe (fatal plus injury) accidents	5.59%
(c)	Injuries per injury accident	1.596
	Injuries per fatal accident*	2.03

Sources: References (40, 42)

Reductions in accident rates given by effectiveness of countermeasures were converted to annual accident cost savings by valuing an accident at $14,700.

Operating Costs and Travel Time

Certain countermeasures produce benefits based on reduced motor vehicle operating costs. Analysis of operating costs was based on the nomographs and procedures described in the 1977 AASHTO Manual (40). As the costs given in the manual represent 1975 costs, they were inflated to 1982 assuming 5 percent compounded rate of inflation. Because section lengths were short and curve speed differentials small, travel time was valued at $0.25 per vehicle-hour.

Maintenance Costs

One element of cost-effectiveness analysis is differences among alternatives in annual maintenance costs. While it may be expected that certain roadside countermeasures would produce lower or higher maintenance costs, it was not believed the differences would be significant. Hence, maintenance costs were not considered in the analysis.

Traffic Growth

A modest, long-term increase in traffic was believed reasonable to assume for the alternatives studied. A growth rate of 1.5 percent compounded annually was selected as a conservative figure. This factor was based on reported nationwide growth on rural highways from 1976 to 1980 (43).

Project Life

A project life of 20 years was selected for evaluation of all alternatives. This produces a conservative (i.e., high) annual cost associated with treatment of rural highway curves. It also represents the maximum time period over which traffic can be reasonably predicted. All costs of construction were amortized over 20 years. The useful life of pavements was assumed to be 10 years. Alternatives involving new pavement surfaces included resurfacing costs every 10 years.

Uniform Annual Cost Factors

The analysis findings were based on current annual costs. The following factors apply:

Annual Share of Present Amount over 20 years at 7% -- 0.094393

Present Worth of Amount 10 years hence at 7% -- 0.508349

By assuming a 1.5 percent annual traffic growth, it follows that increased accident benefits occur over time. Assuming that the benefits are costed in current dollars, and are directly a function of accident rates (which remain constant for analysis purposes), the stream of benefits increases by 1.5 percent each year. A conversion factor which annualizes this constantly increasing stream of benefits is given by the following:

$$\text{Uniform Annual Benefits} = b_1 \left[1 + \frac{\frac{b_n}{b_1} - 1}{n - 1} \ (\text{GUS @ } i,n) \right] \qquad [9.3]$$

Where b_1 = Benefits at year 1

b_n = Benefits at year n

i = Annual interest rate

n = Project life in years

(GUS @ i,n) = Gradient uniform series factor for i% at n years

The quantity b_n/b_1 is the ratio of last year to first year benefits. This is equal to the ratio of traffic at year n divided by traffic at year one, which is equivalent to the compound interest factor given by 1.5 percent and n years.

For the situation here, b_n/b_1 is 1.347. For 20 years at 7 percent, then, uniform annual benefits are equal to 1.14 times first year's benefits.

Unit Construction Cost

Data from Illinois, Louisiana, Mississippi and Wisconsin were input to the estimates of construction costs. The unit costs obtained from these States were compared with each other and with other sources (44). Table 47 lists the unit costs of construction that were developed from all data sources.

Initial Conditions and Assumptions

Conservative assumptions were made in applying the accident rate curve of Figure 44. It was assumed that only highway curves with P(H) of at least 80 percent would be considered as candidates for improvement. In addition, improvements were not considered unless they reduced P(H) to 65 percent or less.

Hypothetical combinations of the five geometric and environmental variables were tested for the required P(H) of 80 percent. To simplify the analysis, categories of each variable (characterized by a representative value) were assumed for both initial conditions and improvements. Table 48 shows the categories used.

Cross Section.--A number of assumptions were necessary in computing costs of countermeasures. The average height of fill was assumed to be 5 feet (1.5 m). Minimum shoulder width of 3 feet (0.9 m) was selected based on a review of the high-accident geometry base. Roadside objects such as trees, utility poles, etc. were assumed to produce a 50 percent coverage factor.

Decision Rules.--Many potential combinations of the variables included poor pavements (i.e., pavement rating of 20). For such cases, it was decided that the pavement must be treated before all other countermeasures were tried. A pavement rating of 20 was believed to represent a condition which would not reasonably be ignored at the expense of other actions.

TABLE 47

UNIT COSTS OF CONSTRUCTION
FOR COST-EFFECTIVENESS ANALYSES

(1982 Price Levels)

Item	Unit Cost
Earthwork (Excavate, load and haul 5 miles; spread and compact)	$5.00 per cu yd
Pavement Removal (Cut, excavate, load and haul bituminous concrete)	$2.50 per sq yd
New Pavement	
Bituminous Concrete	$24.00 per sq yd
Base Course	$ 4.00 per sq yd
Resurfacing	$10.00 per sq yd
New Shoulder (Widening)	
3-foot Shoulder	$ 8.00 per lin ft
8-foot Shoulder	$11.00 per lin ft
Clear and Grub	$2000 per acre
Tree Removal	$150 per tree
Topsoil, Seeding and Fertilizing	$2.20 per sq yd
Drainage	10% (1)
Engineering and Contingencies	12% (2)

[1] As % of earthwork and pavement costs

[2] As % of total cost

1 mi = 1.609 km
1 cu yd = 0.766 m^3
1 sq yd = 0.837 m^2
1 lin ft = 0.305 lin m
1 acre = 0.4047 hectare

1 mi = 1.609 km
1 ft = 0.305 m
\$1.00/cu yd = \$1.31/m^3
\$1.00/sq yd = \$1.20/m^2
\$1.00/lin ft = \$3.28/lin m
\$1.00/ac = \$2.47/ha

TABLE 48

EFFECTIVENESS VALUES USED FOR
GEOMETRIC VARIABLES
IN COST-EFFECTIVENESS ANALYSIS

Roadside Rating:	Poor (50), Moderate (35), Good (25)
Pavement Rating:	Poor (20), Moderate (35), Good (50)
Shoulder Width:	Moderate (3 ft), Wide (8 ft)
Degree of Curve:	15°, 10°, 6°, 3°, 1°
Length of Curve:	0.05 mi, 0.10 mi, 0.15 mi, 0.25 mi

1 ft = 0.305 m
1 mi = 1.609 km

Other initial assumptions were made regarding the extent of the improvement. Because the accident relationships apply over the entire 0.6 mi (1 km) section, improvements were costed over the entire section. Also, roadside treatment countermeasures were applied equally to both sides of the curve.

Analysis Procedure

A set of initial conditions based on the categorical values in Table 48 was evaluated. The initial discriminant score was computed, which had to result in a P(H) of 80 percent or greater for further consideration of that set of conditions. The following steps were then followed:

(1) If pavement rating was 20, the pavement was first considered to be in need of treatment. Resurfacing was assumed to produce a new pavement rating of 50. The new P(H) was determined and compared with the objective of a P(H) of 65 percent or less.

(2) If the initial pavement rating was 35 or 50, or if P(H) was still too high following treatment of the pavement, other countermeasures were tried in sequence. Reduction in roadside rating from 50 to 35 was tried. If this did not sufficiently reduce P(H), other countermeasures were added. These included widening the shoulder (if appropriate) or further improving the roadside to a rating of 25.

(3) The analysis of a given initial set of conditions was considered complete once P(H) was no greater than 65 percent.

(4) Based on the initial P(H) and final P(H), an effectiveness measure of the countermeasure(s) was obtained from Figure 44. This was in the form of a change in accident rate attributable to the countermeasure(s).

(5) Annual accident savings per 1000 ADT (at year one) were computed based on the accident rate reduction applied to the full section length, and the average cost of an accident of $14,700.

(6) Annualized costs of the countermeasures were compared to the annual accident savings. A "break-even" ADT (at year one) was computed which represented the traffic level at which B/C = 1.

Results

The analysis results should be viewed as general indications of the cost-effectiveness for programs of highway curve safety countermeasures. The results are not applicable to site-specific curve safety problems. They are useful in providing guidance toward (1) identifying types of curve locations with potential for improvement; and (2) types of safety countermeasures that, in general, offer the greatest potential for improving safety at reasonable costs.

The analysis was intentionally conservative in nature. Countermeasures were assumed to be applied over the full curve segment. In some cases, such as shoulder widening, this is probably a reasonable assumption. In other cases, such as pavement resurfacing or roadside treatments, actual application of the countermeasure might reasonably focus on the curve and its transitions.

Figure 45 shows the results of the analysis. Under the initial requirement that P(H) be at least 80 percent, it was found that, for the variables other than degree and length of curve, countermeasure effectiveness was relatively

EXISTING GEOMETRIC CONDITIONS				
All Curves	**High Roadside Hazard (RR=50)**		**Moderate Roadside Hazard (RR=35)**	
	Shoulder Width		**Shoulder Width**	
Pavement Rating	**Medium (3 ft)**	**Wide (8 ft)**	**Medium (3 ft)**	**Wide (8 ft)**
Low (PR=20)	Repave and reduce roadside hazard to 25 2300 ADT	Repave and reduce roadside hazard to 35 2000 ADT - - - - - Reduce roadside hazard to 25 1700 ADT	Repave and reduce roadside hazard to 25 Initially 4:1 2700 ADT - - - - - Initially 6:1 1800 ADT	Repave (*) 2800 ADT - - - - - Reduce roadside hazard to 25 2400 ADT
Moderate (PR=35)	Reduce roadside hazard to 25 1600 ADT	Reduce roadside hazard to 25 1800 ADT	Reduce roadside hazard to 25 Initially 4:1 2300 ADT - - - - - Initially 6:1 700 ADT	No treatment considered------- P(H) $\leq$ 0.80
High (PR=50)	Reduce roadside hazard to 25 1700 ADT	Reduce roadside hazard to 35 1800 ADT	No treatment considered------ P(H) $\leq$ 0.80	No treatment considered------- P(H) $\leq$ 0.80

Note: 1ft = 0.305m

* Short, mild and moderate curves do not require improvement

Figure 45. MINIMUM ADT LEVELS FOR CONSIDERATION OF COUNTERMEASURES TO EXISTING HIGH-ACCIDENT RURAL HIGHWAY CURVES

consistent across the range of curvature. Therefore, Figure 45 depicts estimates of break-even ADT levels for countermeasures applied to existing conditions defined in terms of the roadsides, pavement and shoulders.

The significant finding shown in Figure 45 is the predominance of roadside treatment as cost-effective for reasonable traffic volumes. The following specific conclusions were drawn from the cost-effectiveness analysis.

Shoulder Widening.--Shoulder widening by itself does not reduce P(H) to 65 percent. Furthermore, reducing the roadside rating to equal effectiveness is much less expensive. Shoulder widening as a safety countermeasure for highway curves does not appear to be a cost-effective alternative.

Pavement Resurfacing.--The cost-effectiveness of pavement resurfacing is more difficult to assess. The analysis assumed that pavements with a rating of 20 would always be resurfaced. For site conditions with ratings of 20, Figure 45 shows that in most cases, treatment of the roadsides is also required to reduce P(H) to 65 percent. The calculated cost-effectiveness of resurfacing is obviously dependent on the assumption that the entire segment length is resurfaced. Treatment of the curve alone would be much less expensive, although estimates of the safety effectiveness of such treatment are not possible. It is evident that, in some instances, pavement resurfacing is an appropriate countermeasure. However, total effectiveness is greatly enhanced with the addition of roadside countermeasures.

Roadside Treatment.--The costs and effectiveness of treating the roadside make this the single best countermeasure. Under the conservative cost assumptions, for improving roadsides from ratings of 50 to 35 or 25, break-even ADT levels are low -- on the order of 2000 ADT.

Table 49 illustrates the meaning of the variable roadside ratings that were analyzed. The table shows that a roadside rating of 50 is essentially a 2:1 slope. Ratings of 35 are equivalent to 4:1 roadside slopes with 30-foot (9.1 m) clear zones, or to 15-foot (4.6 m) clear zones with 10 percent coverage of objects. Roadside ratings of 25 are associated with 6:1 roadside slopes with 30-foot clear zones.

Cost-effectiveness analyses assumed 50 percent coverage, which in effect fixed the possible ways of developing roadside ratings of 35 or 25. Analysis was performed of the number of objects which would need removal on a curve section given variable coverage factors. The difference between costs of removal of objects given 90 percent and 10 percent coverage is about 2.5 times. Treatment of 90 percent coverage is about 1.7 times the cost of treating 50 percent coverage.

TABLE 49

ROADSIDE HAZARD RATING

Roadside Slope	Coverage Factor	Lateral Clear Width (ft) 30	25	20	15	10	5	0
6:1 or Flatter	90	24	28	32	34	42	46	47
	60	24	27	29	30	35	38	39
	40	24	27	27	27	32	34	34
	10	24	24	24	24	25	26	26
4:1	90	35	37	39	41	44	48	49
	60	35	36	38	39	40	43	44
	40	35	36	37	37	39	41	41
	10	35	35	35	35	36	37	37
3:1	90	41	42	42	43	44	48	49
	60	41	42	42	42	43	45	46
	40	41	42	42	41	41	44	45
	10	41	42	42	41	41	42	42
2:1 or Steeper	90	53	53	53	53	45	49	50
	60	53	53	53	53	46	49	50
	40	53	53	53	53	48	50	50
	10	53	53	53	53	50	50	50

1 ft = 0.305 m

Analysis of the alternative costs of improving roadside slopes vs. removing objects shows the latter to be clearly less costly. However, as Table 49 indicates, clearing roadside slopes as a means of reducing roadside hazard is of limited value unless the slope is mild (say, 6:1).

Sensitivity of Costs.--The analysis assumed that construction costs applied over the entire section length. If, however, the countermeasures actually have their primary effectiveness at the curve and its approaches, their application might be appropriate for only a portion of the section. Resurfacing only the curve and clearing the roadside only at the curve are logical applications of the research findings discussed in Chapter VI through VIII. If the construction costs are lowered to reflect such application, break-even ADT levels would be considerably lower than indicated by Figure 45. In most cases (for typical curve lengths) break-even ADT traffic volumes would be 500 to 900 ADT. Break-even costs for roadside treatment countermeasures would be even lower if, as was suggested in Chapter VIII, the treatment is restricted to one side of the curve.

Flattening Curves

The discriminant score equations indicate one additional countermeasure may be effective. Reducing the curvature can result in a lower D score and lower P(H). This particular countermeasure was evaluated separately, as it represents a special case in treating high-accident locations. Rebuilding a curve is a major, costly undertaking. Besides the high cost of construction, there are costs of additional right-of-way, and maintenance or detouring of traffic. The following discussion illustrates the potential cost-effectiveness of curve flattening.

Trade-off Between Degree and Length of Curve.--The accident relationships expressed by Equation 9.1 indicate that both curve length and degree of curve contribute to D score. Stated differently, there is a trade-off between degree and length of curve. Very long curves as well as very sharp curves are undesirable. This safety trade-off between degree and length of curve has important implications when considering flattening an existing curve. Because the rest of the horizontal alinement remains fixed, the central angle of the curve is also fixed. For every degree the curve is flattened, thereby reducing D score, the curve must be lengthened, thereby increasing D score. Figure 46 illustrates this relationship between curvature and D score for various central angles.

Figure 46 is valuable in that it shows the sensitivities of the degree vs. length trade-off, thereby providing a basis for judging the potential effectiveness of flattening a curve. Only very long curves, or very sharp curves can be altered to significantly reduce the discriminant score and P(H). Furthermore, highway curves with smaller central angles (less than 40°) have greater potential for significant reductions in D.

Effectiveness in Reducing P(H).--Figure 46 shows that, for the most part, the maximum effectiveness in reducing D score is in the range of 0.5 to 0.8. This has a variable effect on reductions in P(H) and accident rate. From previously developed relationships it appears that the greatest net accident effectiveness would be expected with a very high initial P(H).

Analysis Assumptions.--Potential cost-effectiveness of flattening existing sharp highway curves appears limited to extreme cases. In addition, the other geometric elements (roadside, pavement, shoulders) would influence any effectiveness in reducing accident rates. Furthermore, the maximum effectiveness of curve flattening appears associated with curves with very high P(H) (say, 0.90 or greater). These points formed the basis for an example study of curve flattening cost-effectiveness, which assumed the following:

(1) Initial conditions on the curve create a P(H) of 0.90 or greater.

(2) Flattening the curve is accompanied by treatment of other conditions such as poor pavement, roadsides and shoulder width.

In effect, both assumptions characterize a typical case in which curve flattening is considered. Because of the major expense of such a countermeasure, only very severe problem locations would be considered. Furthermore, in rebuilding a section of highway, it would not be logical to put back in place a substandard cross section. Thus, the following analysis actually represents a study of complete geometric reconstruction, rather than just curve flattening.

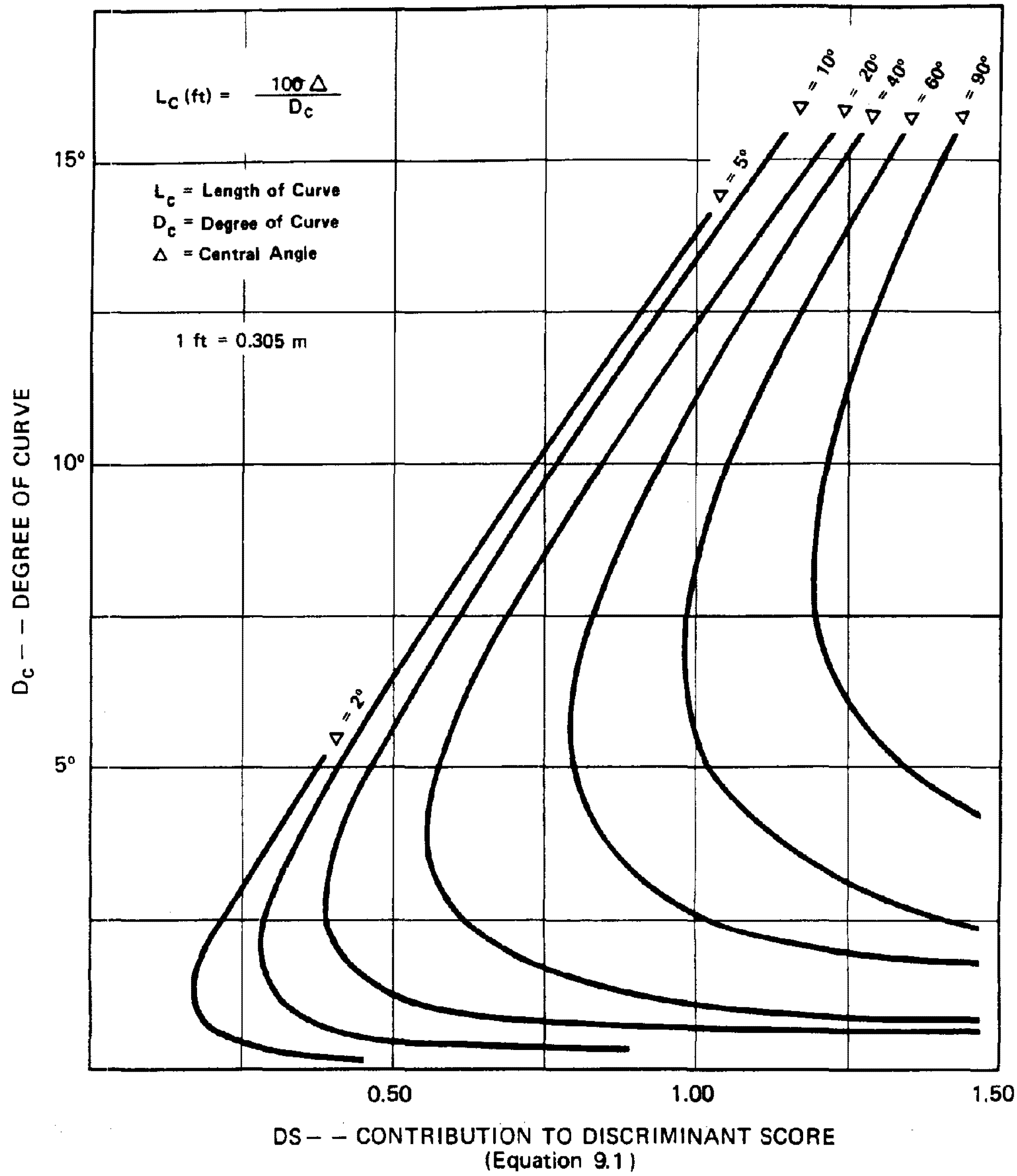


Figure 46. RELATIONSHIP BETWEEN DEGREE AND LENGTH OF CURVE IN TOTAL CONTRIBUTION TO DISCRIMINANT SCORE

Analysis.--Appendix G details the results of an analysis of a 20° highway curve with a 50° central angle, reconstructed to a 6° highway curve. Other improvements to the roadside, shoulders and pavement were also made. The analysis indicated a break-even ADT (in the initial year) of approximately 3400. No costs of right-of-way or maintenance of traffic were included, which would tend to increase this ADT level. Given these costs, and the relatively high break-even level, curve-flattening does not favorably compare with roadside treatment programs in terms of cost-effectiveness.

Summary of Cost-effectiveness Analysis

The previous analyses reveal that limited cost-effective countermeasures are available to treat high-accident locations. The large cost and relatively low effectiveness of widening shoulders, rebuilding curves or repaving indicate these countermeasures have limited applicability in programs to treat rural highway curves. On the other hand, treatment of roadsides, and particularly the clearing of objects from the roadside, holds promise for cost-effectiveness. Flattening slopes may also be cost-effective for moderate traffic volume levels.

Caveats

The analyses presented here are of a general nature, and should be treated as such. It is not possible to present a procedure or recommendations concerning site-specific improvements to safety problems. There is no substitute for evaluating the particular conditions, costs and accident history of a site before proceeding with consideration of countermeasures.

It should also be repeated that the traffic volume levels shown in Figure 45 represent general or approximate regions of cost-effectiveness. The following points are significant:

(1) The reader can, using different cost assumptions, generate different ADT levels than are shown in Figure 45.

(2) A "break-even" ADT level represents a benefit/cost ratio of 1.0. Although this level generally identifies a region of economic feasibility, it does not always indicate project or program desirability. Selection of projects or programs for implementation depends on other factors, such as available funding levels and the economic returns provided by competing uses for funds.

Other Countermeasures

The thrust of the research, and the major accident study findings, were focused on geometric elements of highway curves. Thus, the cost-effectiveness analyses presented here were restricted to geometric variables. There is evidence from other research (6,45,46) that other countermeasures such as signing, marking or delineation are effective under certain conditions. Such countermeasures may be particularly desirable for given situations because of their low implementation costs.

New Construction Versus Reconstruction

The analyses were also limited to a study of countermeasures applied to existing high-accident locations. The high cost of treating such locations contributes to the limited cost-effectiveness of the countermeasures studied. It is clear, however, that a number of geometric improvements would be very inexpensive, and hence "cost-effective" when implemented during new construction; or when considered during a planned highway rehabilitation project. These improvements include the following:

(1) The use of spiral transitions
(2) Increased superelevation
(3) Greater stopping sight distance (see reference (38))

The "costs" of improving operations on curves through careful consideration of the curvature, length and superelevation trade-offs are minimal if such consideration occurs in the route location, planning and preliminary design stages.

X. SUMMARY AND APPLICATION OF RESULTS

This section of the report attempts to draw all of the study results into a cohesive set of recommendations on design of highway curves. The discussion is functional in that it speaks to the general direction that highway design should take, rather than addresses specific dimensions for given design criteria. The reader should consult earlier chapters of the report for detail on the study results discussed here.

The following discussion treats highway curve design in terms of three basic areas--the geometry of the curve and its approach alinement, the highway cross section, and the other geometric and environmental elements that affect driver operations and safety of highway curves.

Geometric Design of Highway Curves

All of the study findings indicate that highway curves are particularly important features of the highway. The complexity of vehicle operations is evidenced by the widely varying path and speed behavior observed on a range of highway curves. The consequences of this variable behavior are demonstrated by studies of accidents on highway curves, which show that highway curves have much higher rates than highway tangents.

Highway curve design involves the geometry of the curve itself (including the degree or radius of curve, length of curve, and superelevation in the curve); as well as the design of the alinement in advance of the curve (transition design, distribution of superelevation runoff, and length of runoff).

Highway Curve Geometry

In relation to safety, the radius or degree of curve is among the more important aspects of design. The accident studies indicate that, in general, as curve radius decreases, accident rate increases. However, radius of curve is not the sole geometric element affecting safety. Indeed, the accident and field studies showed that the design of highway curves must consider a series of trade-offs among the basic elements of curves--radius, superelevation and curve length.

Observations and analyses of vehicles on highway curves produced highly significant findings about the relationship between drivers' paths and highway curve radius. These studies show a tendency by drivers to produce a path curvature that is measurably sharper than the curvature of the highway. This behavior, termed "path overshoot" occurs over a short time period, after which the driver corrects the vehicle's path to more closely match the highway alinement. Because this overshoot behavior (1) occurs on a wide range of highway curve radii; (2) occurs to varying degrees to over half the observed driving population; and (3) is independent of vehicle speed; its implications for curve design policy are considered highly significant.

Figure 47 shows the extent of observed overshoot behavior. Observe that a significant number of drivers traveling at design speed or above will greatly exceed the lateral tire acceleration implied by the AASHTO design friction factors. For example, the 95th percentile lateral tire accelerations at design speed are generally in the range of 0.20 to 0.24 g's on various AASHTO controlling curves (minimum radius for a given design speed and superelevation). This, of course, suggests that many drivers have considerably lower safety margins than implied by the AASHTO design friction factor. However, as Figure 47 shows, most existing pavements even when wet would provide some safety margin for the 95th percentile path at design speed.

In judging the adequacy of the AASHTO highway curve design procedure, it seems more appropriate to consider the needs of a nominally critical driver (say, one generating a 95th percentile path) than to assume that all drivers exactly follow the designed path of the highway curve. But, if the design criteria were changed to provide curvature and superelevation so that a nominally critical driver would only produce AASHTO friction factors at design speed, the procedure might become too restrictive.

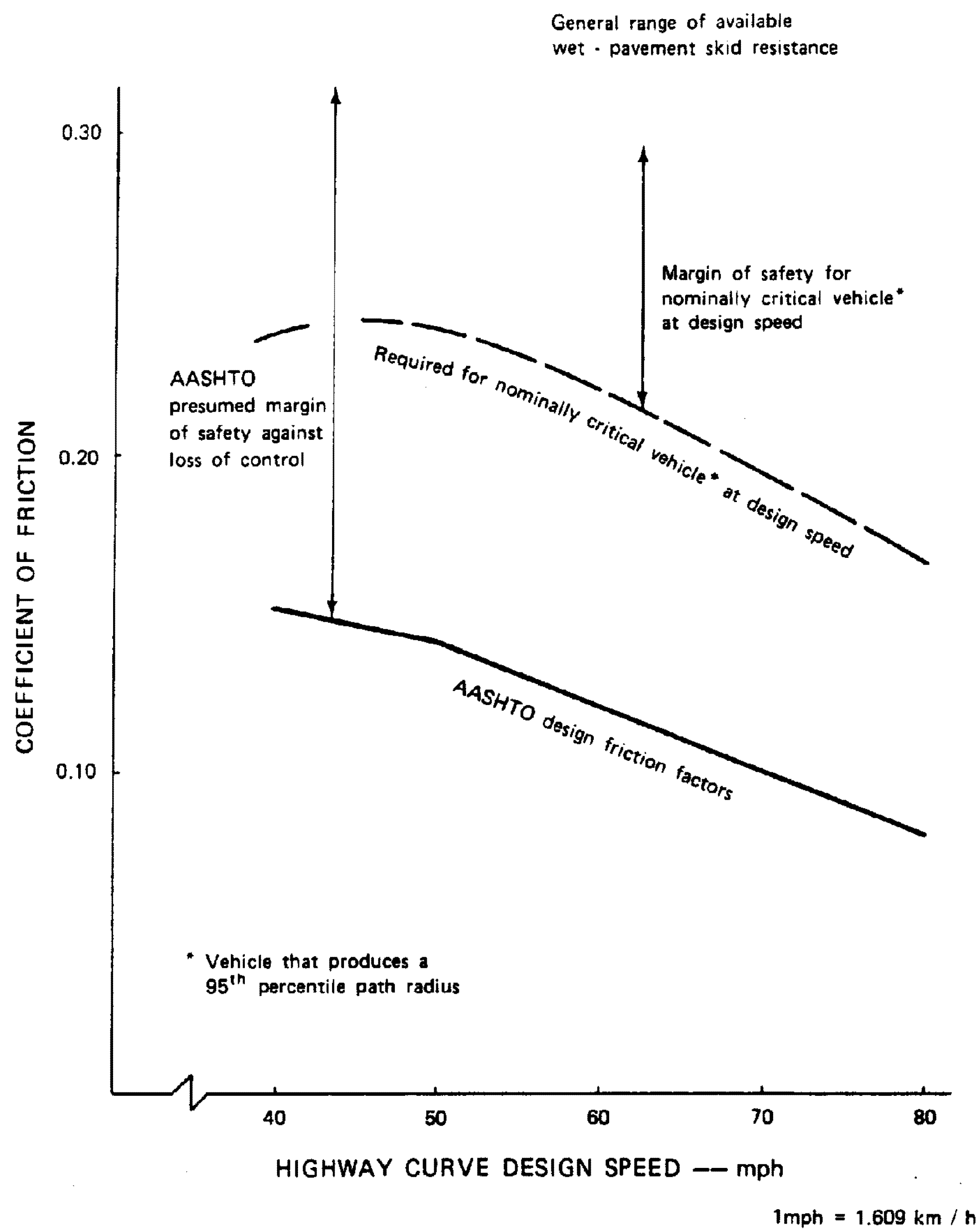


Figure 47. COMPARISON OF OBSERVED SAFETY MARGIN VERSUS AASHTO DESIGN SAFETY MARGIN FOR VEHICLES ON HIGHWAY CURVES

In any event, considering that the nominally critical driver probably has some minimal safety margin for most highway curve conditions, two conclusions seem appropriate:

(1) The AASHTO design procedure is adequate considering that controlling highway curves are minimum designs for safety, and all other AASHTO curves provide greater safety margins. The AASHTO policy should point this out more clearly and strongly suggest minimizing the use of controlling highway curves.

(2) The AASHTO policy should recognize that the provision and maintenance of adequate skid resistance on highway curves is an integral part of their design and operation. Highway agencies should be encouraged to resurface those locations that only provide minimal safety margins for critical drivers.

Further analyses of the overshoot driving behavior indicate an important design trade-off. Consider the operating characteristics of two different AASHTO controlling curves with the same design speed (i.e., curves for two different maximum superelevation design policies). Nominally critical drivers at design speed will generate lower lateral accelerations on the curve with the larger radius and lower superelevation than on the curve with the smaller radius and higher superelevation. Although this difference is small (a range of 0.02 from maximum superelevation policies of 6 to 10 percent), it illustrates an important point regarding the actual operational relationship between curve radius and superelevation. In its present form, AASHTO policy overemphasizes the dynamic effects of superelevation relative to curve radius. This is because the policy establishes superelevation rates assuming all vehicles track the highway curve. Instead, the field studies show vehicle path curvature is significantly sharper than that of the highway curve for a meaningful proportion of the driver population. Therefore, to produce the intended lateral tire accelerations at design speed for a nominally critical driver on an AASHTO highway curve, more superelevation is required than is called for by AASHTO policy.

The safety trade-off between highway curve radius and superelevation also supports the earlier conclusion of minimizing the use of AASHTO controlling curves. All other AASHTO curve designs for a given maximum superelevation policy increase radius disproportionately more than they decrease superelevation relative to the controlling curve.

The accident studies also indicate a second safety trade-off between curve radius and length. These studies show that either very sharp or very long highway curves tend to produce higher accident rates. Figure 48 shows this apparent safety trade-off between highway curve radius and length. The figure, derived from the discriminant analysis of high- and low-accident curve sites, shows optimum combinations of curve radius and length which minimize their net accident contribution for any given central angle. Although this figure should be recognized as a statistical artifact, and therefore should not be regarded as a precise representation of causal relationships, it does point out two logical conclusions. First, in preliminary design and route location, the highway designer should attempt to minimize central angles. Large central angles (say, greater than 45°) require either sharp curvature, or long curvature. Second, the designer should provide for a proper balance between curvature and length for the central angle. Highway curves that are too sharp or too long relative to the central angle should be avoided.

In applying the safety trade-off between highway curve radius and length within AASHTO policy, the designer should recognize that the effect of this principle works against the safety trade-off between curve radius and superelevation, most particularly for large central angles. This inconsistency adds additional support for the conclusion that suggests avoiding large central angles.

<u>Alinement Design in Advance of Highway Curves</u>

The operational studies of vehicle behavior, and HVOSM studies of vehicle dynamics both reveal the importance of proper design of the approach alinement to a highway curve. Because the driver neither desires, nor is physically able to effect an instantaneous transition from tangent path to curve path, the vehicle transition must be initiated on the approach to the highway curve. The manner in which the highway accommodates vehicle transition behavior greatly affects the onset of lateral acceleration on the driver, and subsequent responses to the highway curve itself.

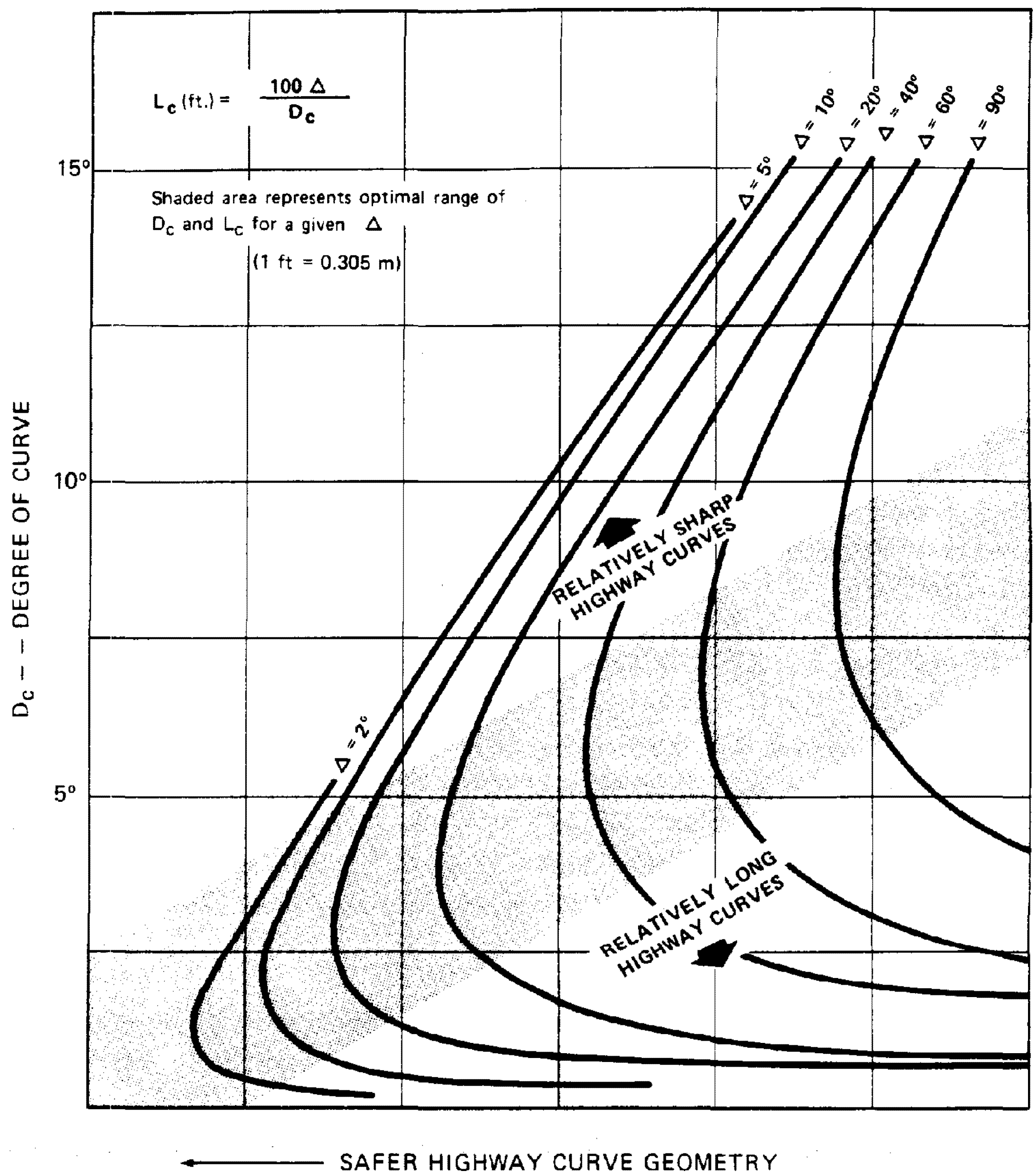


Figure 48. SAFETY TRADE-OFF BETWEEN HIGHWAY CURVATURE AND LENGTH OF CURVE

Effectiveness of Spiral Transitions.--The field studies verify AASHTO design assumptions that vehicles initiate a transitional path on the tangent approach to a circular curve. The actual observed paths simulate, for all practical purposes, a true "spiral" (clothoid) curve of the variety used for design of highway alinements. While this finding alone represents a strong argument in support of spiral curves as a necessary design feature, other study findings demonstrate the significant dynamic advantages of spiral transitions.

The HVOSM studies indicate a dramatic reduction in lateral tire acceleration when a spiral transition is added to an unspiraled highway curve. These studies demonstrate that at design speed, a nominally critical driver will generate a maximum lateral tire acceleration less than the AASHTO design friction factor if a spiral transition is added to the highway curve. Conversely, for the same operating conditions on the unspiraled highway curve, the driver will generate considerably higher maximum lateral tire acceleration than the AASHTO design friction factor.

A more in-depth evaluation of vehicle path behavior observed in the field of unspiraled highway curves tends to support the HVOSM findings. The amount of vehicular path overshoot is significantly greater for those drivers with more severe spiraling behavior. Furthermore, severity of vehicular spiraling behavior is independent of vehicle speed. Therefore, provision for spirals on approaches to highway curves should enable drivers of all speeds to naturally perform lower spiraling rates, thereby producing less path overshoot and lower maximum lateral tire acceleration.

Although spiral transitions are highly supported by AASHTO, their use has not gained wide acceptance in the U.S. Common objections to using spiral transitions include their perceived complexity of calculation and difficulty of field stake-out. However, as is demonstrated in Appendix G, such objections are no longer valid, considering recent advances in computer technology. The merits for using spiral transitions cited by AASHTO are as follows:

(1) An easy to follow path that minimizes encroachment on adjoining lanes;
(2) A convenient arrangement for superelevation runoff;
(3) A means of facilitating pavement widening;
(4) A means of improving the appearance of the highway.

For these reasons and the safety benefits indicated by this study, spiral transitions are considered a very important and necessary element in design of most highway curves.

Unspiraled Superelevation Runoff on Curves.--While spiraled transitions are clearly preferred, most existing highway curves have tangent-to-curve alinement. In 3R design, where the highway alinement remains essentially unchanged, it is important that unspiraled curves provide optimal vehicle dynamics. The research provides guidance on the design of the length and distribution of superelevation runoff.

The field studies of full-width, unspiraled curves show that drivers generally produce path transitions of 200 to 300 feet (60 to 90 m) in length, centered on the point of curvature (PC). Design values for length of superelevation runoff should be consistent with these lengths, which represent natural driver behavior.

Appropriate superelevation runoff distribution should accommodate reasonable driver path behavior to insure a gradual, steady build-up of lateral acceleration on the driver. Superelevation that is developed too late can produce uneven generation of lateral acceleration. In this extreme, the driver experiences a rapid increase in lateral acceleration near the point of maximum vehicle curvature, about 100 to 150 feet (30 to 45 m) past the PC. Runoff lengths that are too long may result in the vehicle reaching maximum curvature without full superelevation. This increases transient peaking in lateral acceleration, which can be uncomfortable and, in the extreme, lead to driver control problems.

AASHTO policy calls for superelevation runoff lengths of 150 to 300 feet (30 to 45 m) and 60 to 80 percent of superelevation runoff to be located on the tangent approach. While the field studies indicate a 50 percent distribution would closely match average driver behavior, the AASHTO recommendations appear reasonable. This policy insures that most drivers have full superelevation by the time they reach their maximum path curvature.

Consideration of Speed

The previous discussion has focused on the geometric elements of the curve and their relationship to actual driver behavior and curve design policy. No discussion of highway curve design is complete without reference to speed.

Vehicle speed is a critical factor in designing for safe operations on highways. With respect to highway curves, speed is important in two ways. First, lateral acceleration on curves is highly sensitive to vehicle speed. Second, highway curves act as restrictive elements on drivers operating at their full or desired speeds.

Studies of vehicle speeds on approaches to and through highway curves show that, regardless of the curve and approach conditions, drivers do not adjust their speeds before about 200 to 300 feet (60 to 90 m) in advance of the PC. The amount of speed reduction from initial high speeds is minor--on the order of 5 mph (8 km/h)--and more gradual for curves flatter than 6°. More significant, rapid speed reductions occur on sharper curves, with much of the reduction occurring past the PC. The implications of these findings are important in light of the earlier discussion on highway curve design. First, the curve radius/superelevation trade-off indicates an operational dynamic benefit of flatter curves in terms of vehicle path. However, the speed studies show that vehicle speeds tend to be slightly greater on flatter curves. Higher speeds might partially negate the benefits of flatter curves by slightly increasing lateral tire acceleration. Second, and more important, drivers' speed change characteristics clearly are focused around the transition area of the curve. Thus, drivers are simultaneously decelerating and steering. This creates widely varying profiles of tire acceleration as vehicles transition into the curve, particularly on sharper highway curves. The use of spirals as transition curves should not only promote more gradual, uniform path behavior, but should also promote more gradual speed change behavior as well. This is because the driver is afforded the opportunity to perceive flatter curvature, and begin gradual speed reduction well before the final curvature is reached.

All of the previous discussion on highway curves and driver behavior points to one serious problem on existing highways. Underdesigned highway curves (i.e., curves with apparent safe operating speeds well below that of the open highway) should be viewed as important elements of any highway safety improvement program. Drivers do not totally decrease their open highway speeds to match a safe operating speed in advance of such sharp curves, and often apply their brakes within the curve. That portion of higher speed drivers with critical path behavior and/or braking will therefore generate very high tire acceleration, which in combination with wet or poor pavement surface, can lead to loss of control.

Cross Sectional Elements

A general research finding is that the primary elements of the cross section--roadway width, shoulder width and roadside character--all influence the safety and operations of highway curves. In particular, the roadside character is a critical aspect of highway curve safety.

Roadside Character

The accident studies indicate that roadside character (roadside slope, clear-zone width, coverge of fixed objects) is the most dominant contributor to the probability that a highway curve is a high-accident location. Because a high incidence of run-off-road accidents is characteristic of highway curves, close attention to producing a relatively flat and clear roadside is as important as the basic design of the highway curve. Analysis of highway curve countermeasures indicates that roadside safety improvements are the most cost-effective solutions for existing highway curves with high-accident histories.

Analytical studies using assumptions about the characteristics of vehicular encroachments on the roadsides of highway curves provide insights about appropriate levels of roadside safety design for curves. In general, it appears that different roadside slope and clear-zone width requirements may be needed for highway curves than for highway tangents. For comparable safety levels, the outside of sharper highway curves (greater than 4°) may require flatter roadside slopes and more clear-zone width than highway tangents. Conversely, the outside of milder curves may not require as flat a slope nor as wide a clear zone as highway tangents. For the inside of highway curves, the reverse appears generally true. That is, flatter curves may require more clear-zone width, and sharper curves less clear-zone width, compared with requirements for highway tangents.

Shoulder Design

The accident studies show that the shoulder width on highway curves is a safety consideration regardless of the shoulder type. As shoulder width increases, the probability that the highway curve will be a high-accident location decreases. Although this result indicates that full-width shoulders are desirable for new construction or major reconstruction, shoulder width additions as a spot improvement to existing highway curve locations are not generally cost-effective. However, it is clear that the safety effect of improved shoulder width is related to the improved clear-zone width of roadsides, and therefore cannot be totally separated from the general benefits of roadside improvements.

The HVOSM studies of the cross-slope break on highway curves show that driver control is sensitive to the shoulder slope and not to the cross-slope break between the superelevated pavement and the shoulder. However, the tolerable shoulder slope is interrelated with the design speed, curvature and superelevation. Therefore, cross-slope break is a practical design criterion, with a recommended maximum value of 8 percent for full-width shoulders. For superelevation rates between 2 to 6 percent, therefore, this criterion allows maximum (negative) shoulder slopes ranging from 6 to 2 percent, respectively. For superelevation rates exceeding 6 percent, a different kind of shoulder cross-slope design is necessary.

With acceptable shoulder slopes, these HVOSM studies imply the benefits of full-width shoulders which would give the errant driver a full four-wheel recovery traversal on a milder slope rather than having two wheels on the shoulder and two wheels on the generally steeper roadside slope. For more complete design recommendations, the reader is referred to the companion study report HVOSM Studies of Cross-Slope Breaks on Highway Curves (31).

Lane Width

The accident studies did not conclusively establish a meaningful effect of lane width on accident rates of highway curves. This lack of sensitivity probably resulted because very few roads less than 20 feet (6 m) wide were observed in the accident study data base.

The operational effects of very narrow roadways were not directly observed in the field operational studies. However, analysis of the findings leads to reasonable deductions about the effects of narrow lanes on vehicles traversing highway curves.

On unspiraled approaches to highway curves, drivers accomplish a transitional path by utilizing the full lane width of 11 to 12 feet (3.4 to 3.7 m). They tend to position their vehicles wide, off-center in the lane on the approach, and then move to the inside edge of the lane as they increase their vehicles' curvature. If less lane width, say 9 or 10 feet (2.7 to 3.0 m) were available to perform this transition, drivers would be forced to make one of two possible adjustments to their transitional behavior. The narrower lane would force either more severe path transitions, or a tendency to encroach on the opposing traffic lane of left curves, or shoulder of right curves. The latter possibility is clearly undesirable. The former possibility is also undesirable considering that the field studies show a relationship between the severity of vehicular transitioning rate and the amount of path overshoot generated by the vehicle. It thus appears that, for optimal operational behavior, full lane widths are desirable on highway curves.

Other Features

Other elements considered in the research include those highway features that occur in conjunction with the highway curve and influence highway curve operations and safety. These include pavement skid resistance, stopping sight distance, approach conditions, and grades.

Pavement Surface

The accident studies indicate that pavement skid resistance is a safety consideration. As pavement skid resistance decreases, the probability that a highway curve will be a high-accident location increases. This finding supports the recommendation that AASHTO policy should more clearly delineate the need for providing and maintaining adequate pavement skid resistance on highway curves.

As an additional caution in pavement construction and maintenance, analytical studies show that pavement washboard and short pavement humps can contribute to loss of control on highway curves.

Stopping Sight Distance

A complete functional analysis of stopping sight distance is presented in a separate project report entitled, Stopping Sight Distance -- An Operational and Cost-effectiveness Analysis (38). In this report, two aspects of highway curve operations are identified that may require special consideration of stopping sight distance on highway curves. These are (1) the increased friction demand of a vehicle that is both cornering and braking; and (2) the loss of the eye-height advantage for truck drivers on highway curves when the horizontal sight restriction is either a row of trees, a wall, or a vertical rock cut.

For AASHTO policy to be consistent in terms of allowable friction demand for both highway tangents and curves, greater sight distances are needed as a function of radius and design speed on highway curves. Then too, with horizontal sight restrictions on curves, sight distance design should consider the stopping requirements of both trucks and automobiles. Although these considerations may be infeasible at some highway locations, the implications are clear. Care should be taken to provide more than AASHTO minimim stopping sight distance on highway curves where possible.

Given that the greatest sight restictions on two-lane highways will occur because of trees along the inside of sharp highway curves, a double benefit can be gained by clearing the trees further from the highway. Not only will greater sight distance be provided, but a safer roadside clear zone will result.

Approach Conditions

Approach conditions to highway curves include such elements as approach sight distance, preceding horizontal alinement, preceding vertical alinement, distance from last intersection, etc. Because these elements could affect an individual driver's speed or readiness to negotiate a highway curve, they were included as study variables in both the accident and speed studies.

The accident studies did not indicate a measurable effect of approach conditions on the accident experience of highway curves. This result, however, may have been influenced by the somewhat general definition of approach alinement and the very limited number of severe sight distance restrictions observed in the sample of highway curve sites.

Studies of speed reduction behavior of drivers at highway curves reveal that, in general, much of the reduction is accomplished past the point of curvature, with the amount of the reduction related to the curve radius. Highway curves with restricted approach sight distance exhibit only slightly lower average approach speeds.

Although these studies generally do not reveal any safety sensitivity of approach conditions to highway curves, they are unable to address the effect of severe sight restrictions on drivers' speed behavior at night or the general ability of drivers to negotiate the highway curve alinement. Regardless of the overall conclusions, the general application of AASHTO minimum stopping sight distance requirements or greater is desirable on approaches to highway curves.

Grades

The HVOSM studies indicate no vehicle dynamic sensitivity to downgrades as steep as 5 percent in traversing highway curves. However, this finding does not account for the effect of grade on the driver's ability to either control maximum speed on the approach or properly reduce speed, if necessary, in negotiating the highway curve.

XI. CONCLUSIONS

Although highway curves are a necessary feature of the two-lane rural highway system, they clearly pose an important safety problem. Not only is the average highway curve about three times as hazardous as the average highway tangent, but certain combinations of design elements on highway curves can create extraordinarily high accident rates.

Because highway curves are very complex features of the highway system, no single research method can be expected to explain all relationships between highway curve design elements and safety. The integrated research approach applied in this study, however, has succeeded in gaining insights into many of these relationships.

Within the total AASHTO design framework, the safety of highway curves can be improved by minimizing the use of controlling (minimum radius for a given design speed and superelevation) curvature, using spiral transitions, and avoiding large central angles. The safety benefits of these suggested applications, however, may be considerably reduced if the roadside design is ignored. Because a high incidence of run-off-road accidents is characteristic of highway curves, close attention to producing a relatively flat and clear roadside is as important as the basic design of the highway curve.

The following is a list of the major conclusions of this study. These conclusions concern (1) the safety of highway curves, (2) the operational characteristics of highway curves, and (3) some general observations about research methodologies. For more details about each conclusion and its potential application to the design and operation of highway curves, the reader is referred to Chapter X, "Summary and Application of Results."

Accident Characteristics of Two-lane Rural Highway Curves

(1) Average Accident Rate - The average accident rate for highway curves is about three times the average accident rate for highway tangents.

(2) Single Vehicle Run-off-Road Accident Rate - The average single vehicle run-off-road accident rate for curves is about four times the average single vehicle run-off-road accident rate for highway tangents.

(3) Proportion of Accidents on Wet Pavement - Highway curves experience a higher proportion of wet pavement accidents than do highway tangents.

(4) Proportion of Severe Accidents - Highway curves have a higher proportion of severe (fatal and injury) accidents than do highway tangents.

(5) Relationship of Accident Types to Traffic Volume - The proportion of accidents that are single vehicle run-off-road increases substantially as average daily traffic decreases.

(6) Roadside Character as Major Accident Factor - Roadside character (roadside slope, clear-zone width, coverage of fixed objects) appears to be the most dominant contributor to the probability that a highway curve has a high reported accident rate.

(7) Other Major Accident Factors - Other measurable contributors to the probability of high reported accident rate are highway curve radius, highway curve length, shoulder width, and pavement skid resistance. No identifiable contributions were found for roadway width, superelevation rate, shoulder type, approach alinement and sight distance, superelevation runoff length, or superelevation runoff distribution.

(8) Combination of Accident Factors - Although roadside character is the dominant accident factor on highway curves, most curves with a high probability of being a high-accident location usually have one or more other factors in combination with roadside hazard that contribute to the total hazard (i.e., sharper curves or longer curves, narrower shoulders and lower pavement skid numbers).

Operational Characteristics of Two-lane Rural Highway Curves

(1) Maximum Lateral Acceleration - In traversing a highway curve, a significant proportion of drivers produce path radii less than the highway curve radius, regardless of their speed. (This behavior is termed "path overshoot".) Therefore, many drivers traveling at design speed or greater will exceed the lateral tire acceleration implied by the AASHTO design friction factor. Considering the high lateral tire friction generated by the 95th percentile path at design speed, the effective safety margin is considerably less than that implied by AASHTO criteria.

(2) Driver/Vehicle Curve Transition Behavior - All vehicles effect a spiral path transition in proceeding from tangent to circular curve alinement. This path behavior generally occurs over the full lane width and is centered about the PC (point of curvature). Although the severity or length of spiraling path behavior varies among drivers, it is independent of vehicle speed. Drivers with more severe spiraling rates tend to produce greater path overshoot, and therefore higher levels of lateral acceleration.

(3) Spiral Transitions - The addition of spiral transitions to the design of highway curves appears to dramatically reduce the severity of path behavior and associated lateral tire acceleration. Because the path overshoot increases with the severity of spiraling behavior on unspiraled highway curves, the addition of a spiral transition to the highway curve should lessen both the severity of the spiraling behavior and the amount of path overshoot. These conclusions about the effectiveness of spirals are supported by the HVOSM simulations, which showed a significant reduction in lateral tire acceleration when a spiral transition was added to the highway curve.

(4) Trade-off Between Highway Curve Radius and Superelevation - The first conclusion indicates that there is a driver control trade-off between highway curve radius and superelevation rate. In comparing two different controlling highway curves with the same design speed, the highway curve with the larger radius and lower superelevation rate may provide a slightly greater safety margin against loss of control than the highway curve with the smaller radius and higher superelevation rate.

(5) Trade-off Between Highway Curve Radius and Length - The research also reveals an apparent safety trade-off between highway curve radius and length. For a given curve radius, the tendency toward high-accident rate production increases with length of highway curve. Conversely, when comparing highway curves of a given length, this tendency toward high-accident production decreases as the radius of highway curve increases. For any central angle, therefore, the benefit of choosing a larger radius may be partially offset by the disbenefit of a longer curve. This conclusion seems diametric to the conclusion about the trade-off between radius and superelevation rate. However, the trade-off between radius and length appears significant only for the extremes of very long or very sharp curves.

(6) Driver/Vehicle Speed Behavior - Higher-speed drivers approaching sharper highway curves do not adjust their open highway speeds to match a safe or comfortable speed for the curve until the curve is imminent. Speed reduction begins about 200 to 300 feet (60 to 90 m) in advance of the PC, and continues in the initial portion of the curve. Mean speeds reached in the curve are strongly related to the highway curvature.

(7) Underdesigned Highway Curves - Existing highway curves that are significantly underdesigned for the prevailing highway speeds may pose considerable safety problems. Because drivers do not totally decrease their open highway speeds to match the safe speed of an underdesigned highway curve, that portion of high-speed drivers with extreme path behavior will tend to generate very high lateral tire accelerations.

(8) Short Highway Curves - The amount of path overshoot on highway curves of 300 feet (90 m) length is considerably less than on longer curves. On curves of all lengths, drivers effect a spiral path transition roughly centered about the point of highway curvature. Drivers do not appear to significantly adjust either the location or length of their spiral paths on short curves. Therefore, when traversing very short highway curves, most drivers spiral in and out of the curve without generating a large path overshoot.

(9) Superelevation Runoff - AASHTO design policy for superelevation runoff length and distribution appears reasonable. The research findings demonstrate the need to provide full superelevation on the curve within 150 feet (45 m) of the PC, by which point most drivers are tracking their maximum path curvature.

(10) Highway Grade - Vehicle dynamics are not sensitive to downgrades as high as 5 percent in traversing highway curves. This conclusion, however, does not consider the effect of downgrade on drivers' ability to properly control their speed.

(11) Roadside Slopes - Roadside slope traversals on highway curves appear more severe than on highway tangents. Severity is defined by the effective path angle to the slope, which is a function of highway curvature. More severe traversals lead both to generally higher vertical accelerations and higher potential for rollover. These results suggest that, for comparable safety levels, roadside slopes on highway curves may need to be flatter than those on highway tangents. Also, some further investigation is indicated toward determining variable guardrail warrants for roadside slopes on highway curves.

(12) Roadside Clear Zones - For comparable safety levels, roadside clear-zone requirements for highway curves may need to differ from requirements for highway tangents. Roadsides on the outside of flat highway curves may require less clear-zone width than highway tangents, and roadsides on the outside of sharp highway curves may require more. The converse is apparently true for the roadsides on the inside of highway curves.

(13) Roadside Safety Improvements - A limited cost-effectiveness analysis used discriminant analysis results to generate broad effectiveness measures. This analysis indicated that roadside safety countermeasures are the most (and, at some locations, the only) cost-effective means of altering the roadway to reduce accidents at existing high-accident highway curves.

(14) Pavement Irregularities - Vehicular control stability on highway curves is very sensitive to pavement washboard and short pavement humps.

(15) Stopping Sight Distance - AASHTO stopping sight distance requirements appear to be inconsistent when applied to highway curves because of higher resultant pavement friction demands created when a vehicle is both cornering and braking. Also, when the sight restriction is a vertical rock cut, wall, or line of trees, truck drivers lose their eye height advantage, which in AASHTO policy is assumed to always compensate for the longer braking distances of trucks.

(16) Cross-slope Break - For vehicles that wander onto outside shoulders of highway curves, the driver's control is sensitive to the shoulder slope and not the cross-slope break (difference between superelevation rate and shoulder cross-slope).

Research Methodologies and Techniques

(1) Determining the Accident Effects of Individual Elements - This study demonstrated the potential futility of using rigorous multivariate statistical procedures for determining the incremental accident effects of variable dimensions for individual highway elements. Not only is this endeavor sensitive to varying accident reporting levels and accuracy, but it requires an almost limitless study design and sample size to adequately represent all values of every geometric, operational and environmental element that create some variance in the accident experience.

(2) Usefulness of General Statistical Techniques - The study demonstrated the usefulness of statistical techniques such as discriminant analysis. This technique successfully isolated those highway elements and their combinations which best distinguish high-accident locations from low-accident locations.

(3) Usefulness of the HVOSM Techniques - The HVOSM simulation technique, using a 0.25 second driver preview of the highway ahead, was successful in replicating the maximum dynamic responses of extreme vehicle behavior on highway curves. This driver modeling, however, did not accurately replicate the way in which the maximum dynamic response was generated; i.e., the rate of vehicle spiraling was more severe than that observed in the field studies. This finding suggests a more complex model for driver preview may be appropriate in applying HVOSM to a study of highway curve traversal behavior. The driver's preview is apparently longer on the approach to the curve, and diminishes as the vehicle actually negotiates the highway curve.

(4) Usefulness of Field Studies - The field observations of driver behavior at a limited number of highway curve sites demonstrated an effective means for identifying both general and critical driver behavior. With a broader range of sites, a more comprehensive study could include the operational effects of roadway width, shoulder width, advanced sight distance, and other elements.

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APPENDIX A

CURVATURE EQUIVALENCY TABLES

The terms "degree of curve" and "radius of curve" are used to refer to or define the sharpness of highway curvature. Degree of curve is commonly used in North American design practice, and is related to radius of curve in the following manner:

$$D_c = [360°/2\pi R)] \text{ (Any Defined Arc Length)}$$

where D_c = Degree of Curve
R = Radius of Curve (ft)

Standard practice assumes an arc length of 100 feet, resulting in the following definition of degree of curve:

$$D_c = 5729.578/R$$

Design practice in countries that use SI units typically involves curvature defined in terms of radius of curve expressed in metres. Although some use is made of degree of curve defined in terms of alternative arc-length definitions in SI units, the radius of curve is more commonly used and understood.

The following tables show SI-equivalent radii of curve for a range of highway curvature defined by degree of curve. Also shown are curvature tables for alternative definitions of degree of curve in SI units.

TABLE 50

CURVE RADII EQUIVALENTS FOR
DEGREE OF CURVE (100 - FT ARC DEFINITION)

Degree of Curve (100-ft Arc Definition)	Radius of Curve (Feet)	Radius of Curve (Metres)
0.5	11,459.16	3,492.75
1.0	5,729.58	1,746.38
2.0	2,864.79	873.19
3.0	1,909.86	582.12
4.0	1,432.40	436.59
5.0	1,145.92	349.28
6.0	954.92	291.06
7.0	818.51	249.48
8.0	716.20	218.30
9.0	636.62	194.04
10.0	572.96	174.64
15.0	381.97	116.42
20.0	286.48	87.32

TABLE 51

CURVE RADII EQUIVALENTS
FOR DEGREE OF CURVE (30-METRE ARC DEFINITION)

Degree of Curve (30-Metre Arc Definition)	Radius of Curve (metres)	Radius of Curve (feet)	Degree of Curve (100-ft Arc Definition)
0.5	3,473.75	11,279.26	0.51
1.0	1,718.87	5,639.62	1.02
2.0	859.44	2,819.81	2.03
3.0	572.96	1,879.87	3.05
4.0	429.72	1,409.90	4.06
5.0	343.77	1,127.92	5.08
10.0	171.89	563.96	10.16
15.0	114.59	375.97	15.24
20.0	85.94	281.98	20.32

TABLE 52

CURVE RADII EQUIVALENTS
FOR DEGREE OF CURVE (100-METRE ARC DEFINITION)

Degree of Curve (100-Metre Arc Definition)	Radius of Curve (metres)	Radius of Curve (feet)	Degree of Curve (100-ft Arc Definition)
0.5	11,459.16	37,597.49	0.15
1.0	5,729.58	18,798.74	0.30
2.0	2,864.79	9,399.37	0.61
3.0	1,909.86	6,266.25	0.91
4.0	1,432.39	4,699.69	1.22
5.0	1,145.92	3,759.75	1.52
10.0	572.96	1,879.87	3.05
15.0	381.97	1,253.25	4.57
20.0	286.48	939.94	6.10
30.0	190.98	626.62	9.14
40.0	143.24	469.97	12.19
50.0	114.59	375.97	15.24

APPENDIX B

This Appendix describes the field procedures that were developed for the detailed field studies of high- and low-accident sites. The material in this Appendix includes instructions to the field crews and sample survey forms.

Field Procedures

The purpose of the field studies is to learn more about highway curves than can be obtained from state geometry files. We are interested in describing the environment around the curve (e.g., its approach conditions, roadside conditions) as well as certain geometric characteristics (e.g., superelevation, grade). A number of special field procedures and field forms have been developed for these studies. The following discussion relates each aspect of the studies and the equipment to be used.

Locating the Curve

County maps, data from straight-line diagrams and computer output will be available to aid in locating the curve. Prior to each survey, a sketch diagram on the field forms should be drawn showing the orientation of the curve, its location with respect to nearby towns, major intersections, structures, etc., and any other information which might assist in locating the curve.

Determine Approach Conditions

The first characteristic of the curve to be studied is its approach condition. This includes the character of the horizontal and vertical alinement for 2-3 miles on each side of the curve, as well as the location of any intersection, speed zone or city limit which would have an effect on speed approaching the curve. The field crew should drive through the curve, classifying the alinement on the first approach; continue downstream for a distance before turning around to classify the alinement on the other approach. A minimum of one picture per approach should be taken approximately 500 feet ahead of the curve. This can be taken in the car through the windshield. The picture number should be noted on the field form.

On curves which have limited sight distance on one or both approaches, the crew should attempt to determine the extent of that restriction. This can be done by driving the approach at a constant speed (say 40 mph) and timing the travel time from the moment the curve becomes visible to the onset of the vehicle entering the curve. The approach speed and time should be recorded on the field sheet.

The final in-car measurement to be made is that of the degree of curvature using the ball bank indicator. At least two runs should be made through the curve at different speeds. Record the speed and ball bank indication at about midpoint of the curve for each run.

When the superelevation angle has been determined, the degree of curve can be calculated using the chart on the field form. Note that in instances where the calculated degree of curve from each of the two runs differs significantly, a third run should be made.

The "recorded degree of curve" is that indicated on the State's inventory and should be reasonably close to degree of curve calculated using the ball bank indicator. If not, curvature must be measured using the chord-offset procedure.

Investigate Curve Characteristics

Find a safe, nearby roadside spot to pull over and park the car. One of the crew members should be responsible for the inventory of all signs and pavement markings. Using the sample field form showing sign types, note the location (approximate) and type of the sign on the inventory form. If possible, determine whether or not the sign is reflectorized, or if its condition is poor. The same crew member should note the presence and condition of all pavement markings, including no-passing zones and edge markings. All signs and pavement marking information should be recorded for each approach on the field form.

The second crew member is responsible for the collection of roadside information. The roadsides will be classified in the office on the basis of pictures and a description by the field crew. Therefore, the crew member responsible for this aspect of the study should concentrate on taking a series of pictures which indicate the character of the roadside on each approach and through the curve. The pictures should show proximity and extent of roadside features such as trees, telephone poles, fences; and continuous features such as side slopes, and ditch sections. At least four pictures should be taken (two on each side). The crew member taking the pictures should note his impression of the roadside (e.g., "free of obstacles, 4:1 side slope predominates," or "line of trees just outside of ditch section") to assist in classifying the roadside condition.

All information from the roadside inventory, including picture numbers and their locations, and comments on the roadside, should be recorded on the field forms.

Determine Curve Geometry

The next step in the field study is the collection of data on the roadway and shoulder width, and superelevation. A separate field form is provided which indicates the measurements to be taken.

The first step is to locate the PC of one approach. (Only one curve approach will be studied. The crew should select the approach which appears to be safest in terms of working with traffic.) The location of the PC is to be determined visually; the expected accuracy is $\pm$ 50 feet. The PC should be marked on the pavement edge and 50--foot increments to 300 feet from the PC on tangent and 200 feet from the PC into the curve should be marked. At these points of reference the roadway width, shoulder width and roadbed width should be measured where indicated on the field form. Superelevation measurements of the normal crown section and transition to full superelevation should also be made. On very sharp curves (generally 4° and over) the crew should also make measurements of the adverse shoulder cross slope at full superelevation on the curve.

Determine Pavement and Shoulder Characteristics

The last step in the basic field procedure is a classification of pavement and shoulder types and condition. The pavement should be examined in the inside wheel path approximately 100 feet downstream from the PC of the curve, and classified using the list of descriptions provided with these instructions. We are interested in determining the friction capabilities of the pavement, and are therefore in need of a classification of that pavement in terms of the size and roughness of the aggregate, number and depth of asperities, and drainage capabilities.

The shoulder type (paved, lawn, gravel) should also be noted.

Any unusual characteristics such as washboarding, rutting, worn wheel paths contributing to poor drainage across the pavement, pavement dropoff, or shoulder softness should also be noted.

The final step in this phase is to take a picture of the pavement from about 2 feet above the pavement, shooting at on oblique angle.

Completion of Basic Field Studies

After all phases of the basic study are complete, the crew should return to the car to check each phase. All forms should be filled out, including a plot of the superelevation transition. All pictures should be properly referenced on the appropriate field form. The crew should make sure they have all equipment before leaving the site. At the completion of each curve survey, the complete set of field forms for each curve studied should be placed into a large envelope, with the date and section codes recorded on the outside of the envelope.

Speed Studies

A limited number of speed studies will be undertaken at sites with certain specified approach and curve conditions. Speeds of free-moving passenger cars will be observed on the approach to, at the PC of, and in the middle of the curve. Radar guns will be used, with observers placed near the roadway in an unobtrusive manner. A minimum of 25 observations should be made for each approach studied.

The following table describes the conditions and number of observations of each type for each states studies. If it is apparent that only one curve with a given set of conditions can be found in a state, the field crew should take a minimum of 50 observations at that site.

FORM 1

APPROACH AND PAVEMENT CONDITION

STATE ____________ COUNTY ____________

HIGHWAY ____________ MILEPOSTS ____________

SERIAL NO. ____________

PAVEMENT CONDITION

PAVEMENT TYPE ____________ SHOULDER TYPE ____________

PAVEMENT RATING ____________ PICTURE NO. ____________

PAVEMENT CONDITION ____________

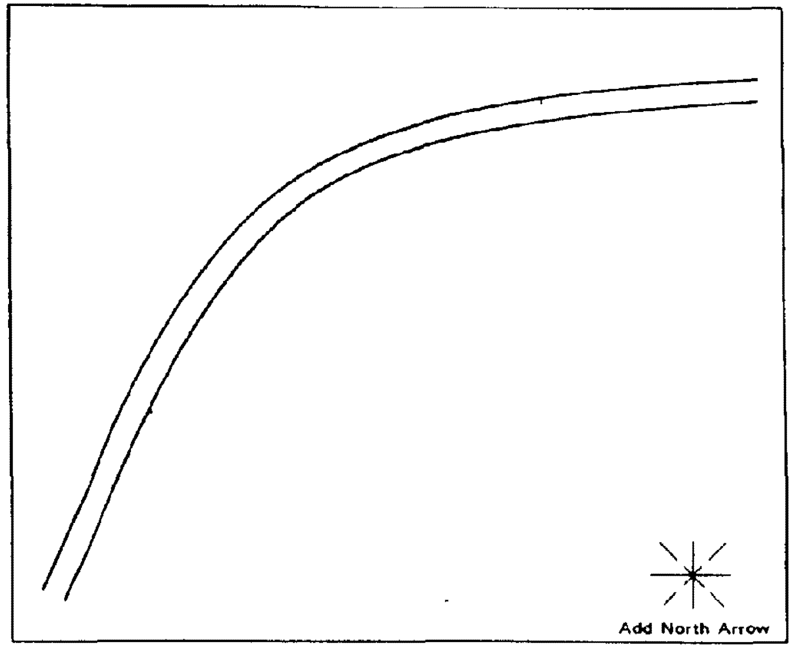
COMMENTS: NOTE ANY WASHBOARD, RUTTING, SHOULDER DROP-OFF, BROKEN PAVEMENT, ETC. ____________

____

____

____________ PICTURE NOS. ____________

Sketch nearby features and indicate approach picture numbers at right.



APPROACH FROM ____________

SIGHT DISTANCE TO CURVE:

☐ UNRESTRICTED ☐ RESTRICTED

IF RESTRICTED, RECORD TIME THAT CURVE IS VISIBLE TO DRIVER.

________ sec. AT ________ mph

CHARACTERISTIC OF ALINEMENT PRECEDING CURVE

(CHECK ONE)
- ☐ PRIMARILY TANGENT
- ☐ MODERATE, MILD CURVATURE
- ☐ INTERMITTENT, SHARP CURVATURE
- ☐ PREDOMINATELY CURVILINEAR

(CHECK ONE)
- ☐ PRIMARILY LEVEL OR MILD GRADES
- ☐ SOME MODERATE GRADES
- ☐ HILLY, MULTIPLE GRADE CHANGES

PROXIMITY TO NEARBY FEATURES

CITY LIMIT ____________ MILES ______

INTERSECTION (State or U.S. Hwy.) ____________ MILES ______

APPROACH FROM ____________

SIGHT DISTANCE TO CURVE:

☐ UNRESTRICTED ☐ RESTRICTED

IF RESTRICTED, RECORD TIME THAT CURVE IS VISIBLE TO DRIVER.

________ sec. AT ________ mph.

CHARACTERISTIC OF ALINEMENT PRECEDING CURVE

(CHECK ONE)
- ☐ PRIMARILY TANGENT
- ☐ MODERATE, MILD CURVATURE
- ☐ INTERMITTENT, SHARP CURVATURE
- ☐ PREDOMINATELY CURVILINEAR

(CHECK ONE)
- ☐ PRIMARILY LEVEL OR MILD GRADES
- ☐ SOME MODERATE GRADES
- ☐ HILLY, MULTIPLE GRADE CHANGES

PROXIMITY TO NEARBY FEATURES

CITY LIMIT ____________ MILES ______

INTERSECTION (State or U.S. Hwy.) ____________ MILES ______

FORM 3

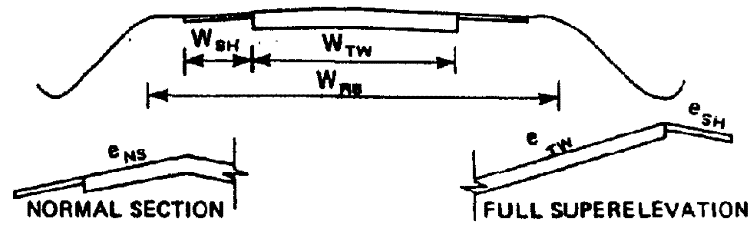
ROADWAY GEOMETRY

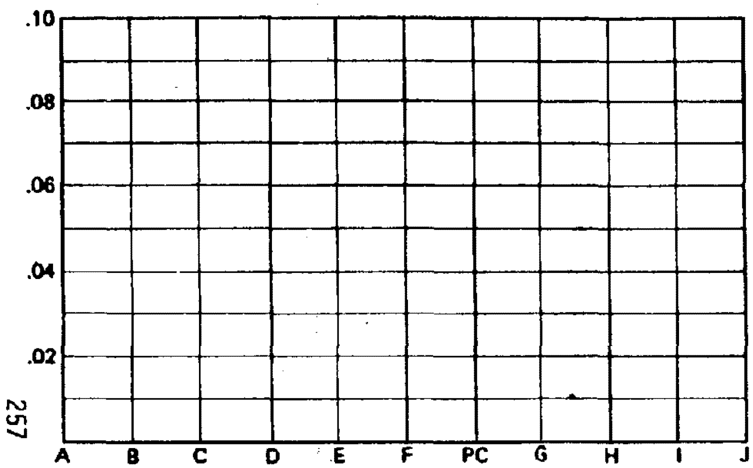
SERIAL NO. ____________

STATE ____________ COUNTY ____________

HIGHWAY ____________ MILEPOSTS ____________

DEGREE OF CURVE* ____________ LENGTH OF CURVE ____________





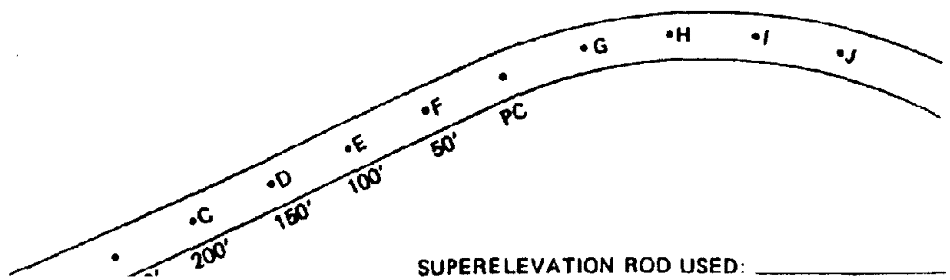
Location	W_{TW}	W_{SH}	W_{RS}	e_{TW}	e_{NS}**	e_{SH}**
A						
B						
C						
D						
E						
F						
PC						
G						
H						
I						
J						

**Record twice the reading

*If Degree of Curve does not appear as recorded and determined from Ball-Bank Indicator, note Mid-Ordinate Measurements below.

Observation	Chord Length	Mid-Ordinate
1		
2		
3		
4		

COMMENTS: ____________



SUPERELEVATION ROD USED: ____________

FORM 2

SIGNS AND MARKINGS

STATE ______________ COUNTY ______________

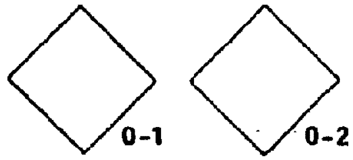
HIGHWAY ______________ MILEPOSTS ______________

RECORDERS ______________ DATE ______________

SERIAL NO. ______________

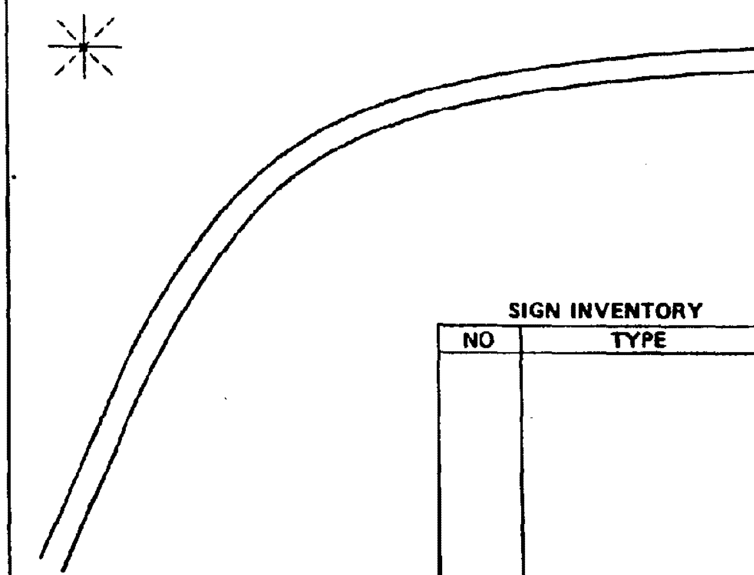
ON DIAGRAM:

- INDICATE ROLL AND PICTURE NUMBERS.
- INDICATE NORTH ARROW.
- SKETCH LOCATIONS OF INTERSECTIONS, DRIVEWAYS, BRIDGES AND OTHER UNUSUAL FEATURES.
- LOCATE AND INVENTORY ALL SIGNS AND REFERENCE IN SIGN INVENTORY TABLE.



0-1 0-2

Add North Arrow



SIGN INVENTORY

NO	TYPE

COMMENTS ON ROADSIDE CONDITION ______________

____

____

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APPROACH FROM ______________

PAVEMENT MARKINGS

EDGE LINES ______________ CONDITION ______________

CENTERLINE ______________ CONDITION ______________

NO PASSING STRIPE ______________ CONDITION ______________

Does it begin before Superelevation Transition? ______________

Does it end before the end of the curve? ______________

SIGNS

ANY SUBSTANDARD SIGNS? ______________

SIGN CONDITIONS ______________

ANY UNREFLECTORIZED SIGNS? ______________

ARE THERE DELINEATORS? ______________

TYPE: ______________

APPROACH FROM ______________

PAVEMENT MARKINGS

EDGE LINES ______________ CONDITION ______________

CENTERLINE ______________ CONDITION ______________

NO PASSING STRIPE ______________ CONDITION ______________

Does it begin before Superelevation Transition? ______________

Does it end before the end of the curve? ______________

SIGNS

ANY SUBSTANDARD SIGNS? ______________

SIGN CONDITION ______________

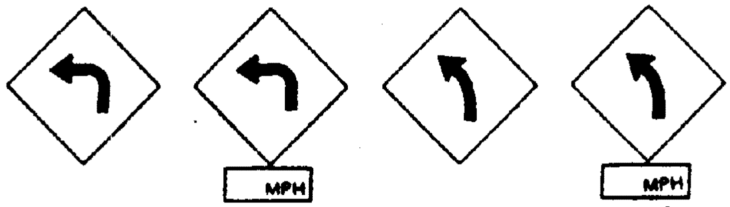
ANY UNREFLECTORIZED SIGNS? ______________

ARE THERE DELINEATORS? ______________

TYPE: ______________

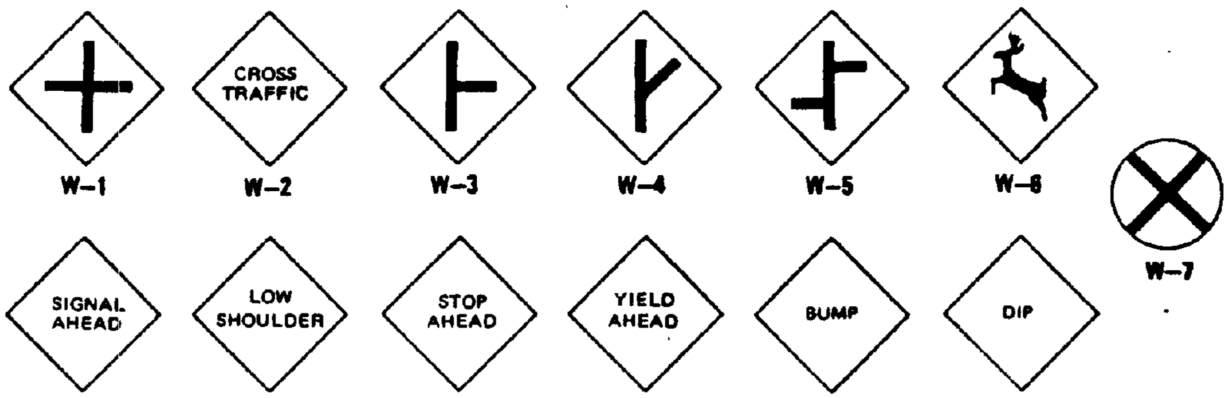
SIGN INVENTORY KEY

CURVE SIGNS



C-1 C-2 C-3 C-4

WARNING SIGNS



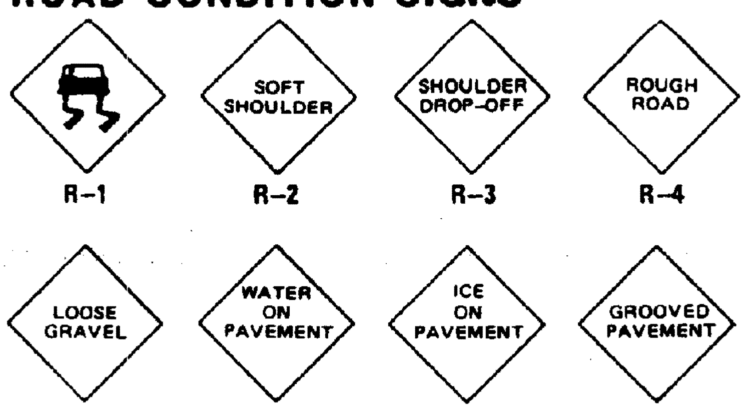
W-1 W-2 W-3 W-4 W-5 W-6 W-7

W-8 W-9 W-10 W-11 W-12 W-13

PASSING SIGNS

P-1 P-2 P-3

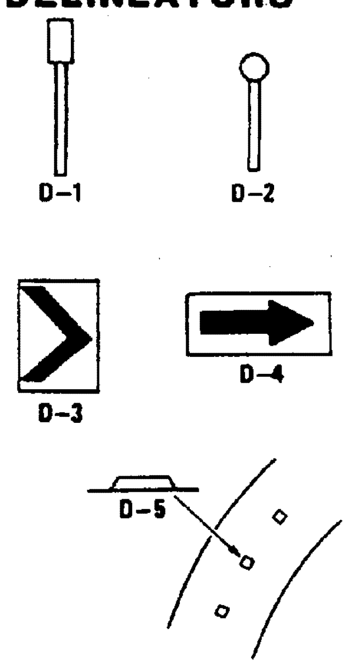
ROAD CONDITION SIGNS



R-1 R-2 R-3 R-4

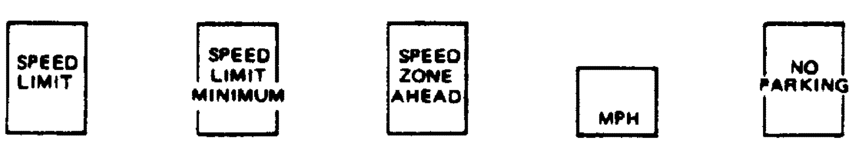
R-5 R-6 R-7 R-8

DELINEATORS



D-1 D-2 D-3 D-4 D-5

SPEED AND PARKING RESTRICTION SIGNS



S-1 S-2 S-3 S-4 S-5

BALL BANK INDICATOR SURVEY RECORD

STATE ____________ COUNTY ____________ SERIAL NO. ____________

HIGHWAY ____________ MILEPOST ____________

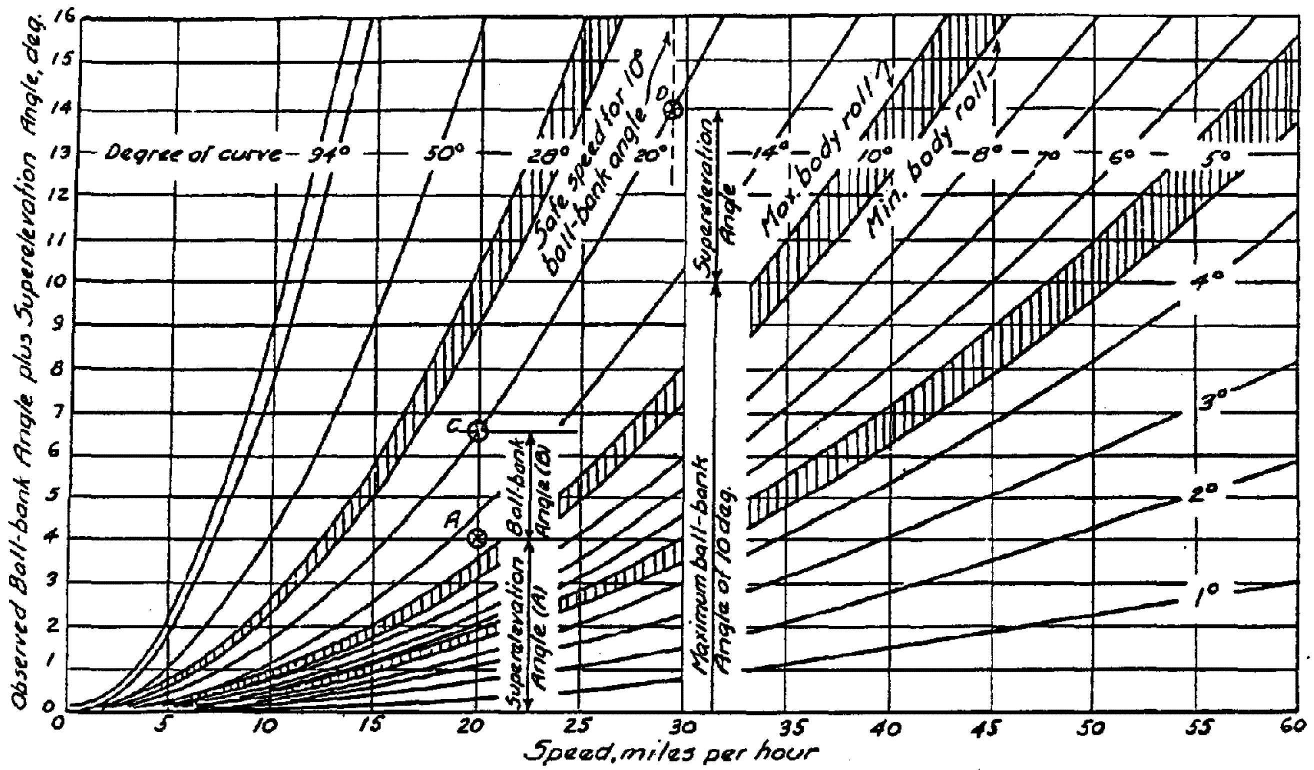


Figure 10. Chart to determine safe speed indications for highway curve signs. Patterned after chart by R. F. Riegelmeier, Traffic Engineer, Pennsylvania Department of Highways

Rule for Safe Speed

Add superelevation angle (A) to ball bank angle (B) observed at any given speed to obtain the total angle (C). Follow curve on which (C) is located to point (D) which is sum of superelevation angle and 10 degree ball bank angle. Maximum safe speed for reflectorized sign is the closest speed value for point (D) to the nearest 5 miles per hour.

	RUN 1	RUN 2	RUN 3*
SPEED (mph)			
BALL BANK ANGLE (B)			
SUPERELEVATION ANGLE** (A)			
TOTAL ANGLE (C)			
DEGREE OF CURVE			
RECORDED DEGREE OF CURVE			

* OPTIONAL

** SEE CONVERSION CHART

SPEED SURVEY

STATE ________________ COUNTY ________________ HIGHWAY ______________ MILEPOST ______________

SURVEY PERIOD SERIAL NO. ____________ SHEET ______ OF ______

DATE ________________ DAY ____________ WEATHER ______________ RECORDER ________________

TIME OF SPEED CHECK FROM ___:___ TO ___:___ POINTS RECORDED ________________

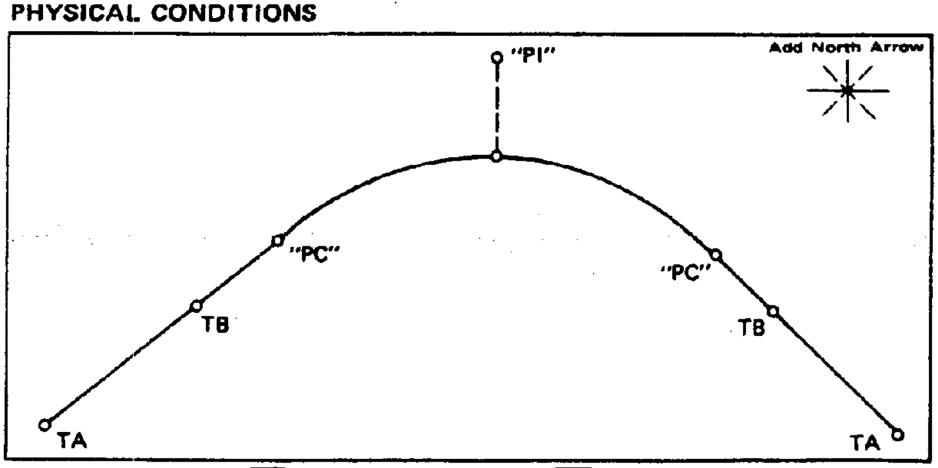
OBS	IDENTIFICATION	SPEED AT TA	TB	"PC"	"PI"

COLOR/TYPE
- C - CAR
- V - VAN
- P - PICKUP

DO NOT RECORD TRUCKS

NOTE: CROSS OUT COLUMNS NOT USED

PHYSICAL CONDITIONS



ON DIAGRAM INDICATE:
- North Arrow
- Direction of Travel Surveyed
- Surveyor Locations and Speed Points Covered
- Roadside Conditions Affecting
  - Sight Distance
  - Surveyor Location
  - Ground Covering

SIGHT DISTANCE ☐ 600' OR GREATER ☐ LESS THAN 600' ____________________
SPECIFY

MEASURING DISTANCES IF SD = 600' OR GREATER

TA TO "PC" (700' - 1000') ____________ TB TO "PC" (200' - 300') ______________ "PC" TO "PI" ______________

IF SD = LESS THAN 600'

TA TO "PC" (Approx 600') ____________ TB TO "PC" (Skip TB Point) "PC" TO "PI" ______________

PAVEMENT RATINGS FOR ASPHALTIC-CONCRETE SURFACES

(Measured in Wheel Path @ 100' Downstream of P.C. of Curve)

DEPTH OF ASPERITIES	SURFACE ROUGHNESS: Very Rough	Rough	Moderately Rough	Moderately Smooth	Smooth	Slick
Very Deep	56	52	48	-	-	-
Deep	52	48	44	-	-	-
Moderate	48	44	40	36	-	-
Shallow	-	40	36	32	28	-
Very Shallow	-	-	32	28	24	-
None	-	-	-	-	-	20

DEFINITION: DEPTH OF ASPERITIES

Class	Description
Very Deep	1/2"
Deep	3/8"
Moderate	1/4"
Shallow	1/8"
Very Shallow	1/16"
None	0

DEFINITION: SURFACE ROUGHNESS

Class	Description
Very Rough	Jagged Corners Protruding
Rough	Protruding Corners With Some Rounding
Moderately Rough	Round or Rounded Aggregate With Good Drainage Paths Through Asperities
Moderately Smooth	Aggregate Highly Polished With Good Drainage Paths Through Asperities
Smooth	Aggregate Highly Polished With Irregular Drainage Paths Through Asperities
Slick	Bleeding Asphalt

PAVEMENT RATINGS FOR PORTLAND-CEMENT CONCRETE AND SAND-ASPHALT SURFACES

(Measured in Wheel-Path @ 100' Downstream of Curve P.C.)

SURFACE CLASSIFICATION	DESCRIPTION	PAVEMENT RATING
1. Very Rough	Noticeably Heavy Textured Finish (PCC only)	56
2. Rough to Very Rough		52
3. Rough	Very Rough Sandpaper Reel (Some Textured Finish on PCC)	48
4. Moderately Rough to Rough		44
5. Moderately Rough	Rough Sandpaper Feel	40
6. Moderately Smooth to Moderately Rough		36
7. Moderately Smooth	Medium Sandpaper Feel	32
8. Smooth to Moderately Smooth		28
9. Smooth	Fine Sandpaper Feel	24
10. Slick	Highly Polished, No Sandpaper Feel (Bleeding Asphalt for Sand-Asphalt Surface)	20

SPEED STUDY MATRIX

(Curves of 500' length or more)

Type of Site	No. of Samples					
	S.D. < 600			S.D. > 600		
	≥ 6°	3-4°	1-2°	≥ 6°	3-4°	1-2°
Class 1 Primarily Tangent Primarily Level No Closeby Intersection or City	2	2	2	2	2	2
Class 2 Moderate Mild Curvature Some Moderate Grades	2	2	0	2	2	0
Class 3 Predominantly Curvilinear Hilly, Multiple Grade Change	2	0	0	2	0	0

APPENDIX C

DEVELOPMENT OF ROADSIDE SEVERITY RATING

OBJECTIVE: To develop a roadside hazard rating irrespective of roadway configuration, or in other words, a severity rating for various roadside configuration.

HAZARD MEASURE: The effectiveness measure used is the probability of an injury accident given a roadside encroachment.

THEORETICAL BASIS: Roadside Hazard Model from NCHRP Report 148, (29).

PARAMETERS:

(1) Roadside slope at 10 ft (3.1 m) from edge of pavement

(2) Lateral clear-zone width

(3) Percent coverage of severe fixed objects, such as trees. (This is expressed as the proportion of the highway "shadowed" by fixed objects, or conversely, the probability that a vehicle reaching the clear-zone width will hit a fixed object.)

For a given length of highway, the NCHRP Report 148 Model can be simplified for a noncontiguous roadside obstacle (say a constant roadside slope with no fixed objects) to:

$$H = E_f\ (S)\ P\ [y \geq s]$$

where:

H = Hazard Index, number of fatal plus nonfatal injury accidents per year

E_f = Encroachment Frequency, number of encroachments per section length per year

S = Severity Index, percent of impacts with fixed objects resulting in fatal or injury accidents

$P[y \geq s]$ = probability of a vehicle lateral displacement y, greater than some value, s. These values taken from Figure 4 in NCHRP Report 148.

The probability, R_h, of a fatal or nonfatal injury accident given an encroachment is:

$$R_h = (S)\ P\ [y \geq s]$$

Development of Rating Equations

The hazard rating equations for roadside configurations must consider:

(1) a roadside slope break at a given distance, L_s;
(2) a clear-zone width, L_c;
(3) an obstacle coverage factor, C;
(4) a slope severity S_s, and
(5) an obstacle severity index, S_o.

The severity indices for 2-lane rural highways are taken from the 1974 FHWA report "Effectiveness of Roadside Safety Improvements" (20) as follows:

Obstacle	S_o -- Obstacle Severity Index
Large Trees, Utility Poles Culverts, etc. (Ave.)	0.50
2:1 or steeper fill	0.60
3:1 fill	0.45
4:1 fill	0.35
5:1 fill	0.25
6:1 or flatter fill	0.15

The development of the appropriate equations must consider whether the clear zone is less than or greater than the distance to the roadside slope break point.

Clear Zone Behind Slope Break

If the roadside slope is 2:1 or steeper, the severity of the slope is greater than that of the fixed objects and the appropriate equation is:

$$R_h = 0.6\ P(y \geq L_s)$$

The more general equation relating to roadside slopes of 3:1 or flatter must consider the incremental effects of the roadside slope and the fixed objects as follows:

$$
\begin{aligned}
R_h &= S_s\ P[L_s \leq y \leq L_c] + 0.5C\ P[y \geq L_c] \\
&\quad + S_s(1-C)\ P[L_c \leq y \leq L_q] \\
&\quad + 0.5(1-C)\ P[y \geq L_q]
\end{aligned}
$$

The last term of this equation makes the simplifying assumption that all vehicles that travel 30 ft (9.2 m) from the road edge will experience conditions with a severity index of 0.50.

Clear Zone In Front of Slope Break

The general equation for fixed objects in front of the slope break point must consider the incremental effects of both the fixed objects and the slope as follows:

$$
\begin{aligned}
R_h &= 0.5(C)\ P[y \geq L_c] + S_s(1-C)\ P[L_s \leq y \leq L_q] \\
&\quad + 0.5(1-C)\ P[y \geq L_q]
\end{aligned}
$$

Again, the last term in the equation makes the simplifying assumption that all vehicles that travel 30 ft (9.2 m) from the edge of pavement will experience a roadside condition with a severity index of 0.50.

Example of Hazard Ratings

The following table shows the hazard rating for various roadside configurations assuming the average roadside slope hinge point is 10 ft (3.1 m) from the edge of road pavement. (Note: the rating is not too sensitive to this distance except for a 2:1 slope).

TABLE 53

ROADSIDE HAZARD RATING

Side Slope	Coverage Factor	Lateral Clear Width (ft) 30	25	20	15	10	5	0
6:1 or Flatter	90	.24	.28	.32	.34	.42	.46	.47
	60	.24	.27	.29	.30	.35	.38	.39
	40	.24	.27	.27	.27	.32	.34	.34
	10	.24	.24	.24	.24	.25	.26	.26
4:1	90	.35	.37	.39	.41	.44	.48	.49
	60	.35	.36	.38	.39	.40	.43	.44
	40	.35	.36	.37	.37	.39	.41	.41
	10	.35	.35	.35	.35	.36	.37	.37
3:1	90	.41	.42	.42	.43	.44	.48	.49
	60	.41	.42	.42	.42	.43	.45	.46
	40	.41	.42	.42	.41	.41	.44	.45
	10	.41	.42	.42	.41	.41	.42	.42
2:1 or Steeper	90	.53	.53	.53	.53	.45	.49	.50
	60	.53	.53	.53	.53	.46	.49	.50
	40	.53	.53	.53	.53	.48	.50	.50
	10	.53	.53	.53	.53	.50	.50	.50

1 ft = 0.305 m

APPENDIX D

HVOSM CURVE RUN DOCUMENTATION

HVOSM Input Parameters

The Roadside Design (RD2) version of HVOSM, as documented in Reference (49) was used for the present research. Some modifications of the simulation program were incorporated for this application as discussed later in this Appendix.

The specific vehicle that was simulated in the curve studies was a 1971 Dodge Coronet 4-door sedan. The inputs for the simulated vehicle were obtained from Appendix D of Reference (48). An input data deck listing and a corresponding parameter list of the inputs are presented in Figures 49 and 50.

HVOSM Curve Study Setup Procedure

The procedure to set up an HVOSM curve run for the present research effort was as follows:

(1) Analytically determine the extent of roadway required to meet the requirements of the particular run (i.e., roadway radius and length).

(2) Set up and run a Terrain Table Generator (TTG) run based on roadway specifications.

(3) Insert TTG run output "cards" into HVOSM data deck.

(4) Set up and insert HVOSM Driver Model Input cards per run specification into HVOSM data deck.

(5) Perform the simulation run.

The "cards" referred to were actually disk files and all insertions and manipulations of "card" decks were actually done interactively on disk files. The use of disk files enabled the rapid manipulation of "card" decks for each simulation run, as well as retention of the card deck for each run in a single partitional disk data set.

```
MCI-JEL HVOSM CURVE STUDIES:  RUN:HCS#18                                                   0 100
0.0      4.87     0.010    0.010    70.0     0.0      0.0                                  0 101
0        0                                                                                 0 102
       1                                                                                   0 103
       1        1        1        1        1        1        1                             0 104
1971 DODGE CORONET 4-DOOR SEDAN                                                            0 200
8.43     0.51     0.82     3760.0   23000.0  23300.0  530.0    550.0                       0 201
49.3     68.7     59.8     61.8     0.0      47.0                                          0 202
0.0      -14.0    0.0      -68.7    -30.9    10.1     10.82    10.68                       0 203
106.0    189.0    600.0    588.0    600.0    0.50     -2.40    2.1                         0 204
120.0    324.0    600.0    864.0    600.0    0.50     -4.40    3.6                         0 205
6.85     40.0     0.10     7.48     38.0     0.10                                          0 206
40400.0  -5100.   0.02                                                                     0 207
                  0.559                                                                    0 208
-3.0     3.0      1.0                                                                      0 209
-0.43    -0.95    -1.22    -1.26    -0.98    -0.41    0.0                                  1 209
FIRESTONE RADIAL VI                                                                        0 300
1.0      1.0      1.0      1.0      6.0      0.25                                          0 301
1450.0   3.0      10.0     -37.0    13.2     3043.    .58      91435.   1.0                1 301
.78                                 13.2                                                   0 302
210 M PATH,5% BRAKING,PROBE 25%                                                            0 400
0.0      5.0      1.0      0.0      0.0      1.0                                           0 401
-95.     -95.     -95.     -95.     -95.                                                   1 401
1.0      1.0      1.0      0.05     .00905   0.000    0.0                                  0 402
4.0      100.     0.0      0.0      1.5708   120.                                          0 403
0.0      0.0      0.0      600.     -.6892   720.     -.6892   12000.                      0 404
0.0      0.1      264.     0.0      0.5      400.     0.00380  0.000380                    0 405
210 M RADIUS,10% SE,5% GRADE,80 M RUNOFF,20/80% DIST                                       0 500
 -600.00   600.00    60.00      0.0  1200.00    60.00   0.0                                0 501
     6.00     9.00    12.00    13.73    15.46    17.19    18.92    20.65    22.38           1 501
    24.12    25.85    27.58    29.18    30.91    32.61    34.28    35.75    37.33           2 501
    38.85    40.39    41.88      0.0      0.0      0.0      0.0      0.0      0.0           3 501
     5.40     8.40    11.40    13.26    15.12    16.97    18.83    20.69    22.55           4 501
    24.40    26.26    28.12    29.86    31.71    33.54    35.33    36.94    38.65           5 501
    40.30    41.97    43.59      0.0      0.0      0.0      0.0      0.0      0.0           6 501
     4.80     7.80    10.80    12.78    14.77    16.75    18.74    20.72    22.71           7 501
    24.69    26.68    28.66    30.53    32.51    34.47    36.39    38.13    39.97           8 501
    41.75    43.55    45.31      0.0      0.0      0.0      0.0      0.0      0.0           9 501
     4.20     7.20    10.20    12.31    14.42    16.53    18.65    20.76    22.87          10 501
    24.98    27.09    29.20    31.21    33.31    35.40    37.45    39.33    41.31          11 501
    43.22    45.15    47.04      0.0      0.0      0.0      0.0      0.0      0.0          12 501
     3.60     6.60     9.60    11.84    14.08    16.32    18.55    20.79    23.03          13 501
    25.27    27.51    29.75    31.89    34.12    36.33    38.52    40.54    42.65          14 501
    44.70    46.76    48.79      0.0      0.0      0.0      0.0      0.0      0.0          15 501
     3.00     6.00     9.00    11.37    13.73    16.10    18.46    20.83    23.19          16 501
    25.56    27.92    30.29    32.57    34.93    37.27    39.59    41.75    44.00          17 501
    46.19    48.39    50.55      0.0      0.0      0.0      0.0      0.0      0.0          18 501
     2.40     5.40     8.40    10.89    13.38    15.88    18.37    20.86    23.35          19 501
    25.85    28.34    30.83    33.25    35.74    38.21    40.66    42.98    45.35          20 501
    47.69    50.02    52.33      0.0      0.0      0.0      0.0      0.0      0.0          21 501
     1.80     4.80     7.80    10.42    13.04    15.66    18.28    20.90    23.52          22 501
    26.13    28.75    31.37    33.94    36.55    39.15    41.73    44.20    46.72          23 501
    49.20    51.67    54.12      0.0      0.0      0.0      0.0      0.0      0.0          24 501
     1.20     4.20     7.20     9.95    12.69    15.44    18.18    20.93    23.68          25 501
    26.42    29.17    31.92    34.62    37.36    40.09    42.81    45.44    48.10          26 501
    50.72    53.34    55.92      0.0      0.0      0.0      0.0      0.0      0.0          27 501
     0.60     3.60     6.60     9.47    12.35    15.22    18.09    20.97    23.84          28 501
    26.71    29.58    32.46    35.31    38.18    41.04    43.89    46.68    49.48          29 501
    52.26    55.02    57.75      0.0      0.0      0.0      0.0      0.0      0.0          30 501
      0.0     3.00     6.00     9.00    12.00    15.00    18.00    21.00    24.00          31 501
```

FIGURE 49. TYPICAL CARD IMAGE OF HVOSM INPUTS FOR HVOSM CURVE STUDY

80.70	83.02	85.28	87.47	89.61	91.68	93.69	95.62	97.49	29	502
99.27	100.98	102.61	0.0	0.0	0.0	0.0	0.0	0.0	30	502
59.59	62.44	65.25	68.03	70.78	73.48	76.14	78.75	81.31	31	502
83.82	86.28	88.67	91.01	93.28	95.49	97.63	99.69	101.68	32	502
103.60	105.44	107.19	0.0	0.0	0.0	0.0	0.0	0.0	33	502
61.44	64.44	67.40	70.33	73.22	76.07	78.87	81.62	84.33	34	502
86.98	89.57	92.11	94.58	96.99	99.33	101.61	103.81	105.93	35	502
107.97	109.94	111.82	0.0	0.0	0.0	0.0	0.0	0.0	36	502
63.31	66.46	69.57	72.65	75.69	78.68	81.63	84.53	87.38	37	502
90.17	92.91	95.59	98.20	100.75	103.23	105.63	107.97	110.22	38	502
112.40	114.50	116.51	0.0	0.0	0.0	0.0	0.0	0.0	39	502
65.20	68.51	71.76	75.00	78.18	81.33	84.42	87.47	90.46	40	502
93.40	96.28	99.10	101.86	104.55	107.17	109.71	112.18	114.57	41	502
116.89	119.11	121.26	0.0	0.0	0.0	0.0	0.0	0.0	42	502
67.11	70.57	73.98	77.37	80.70	84.00	87.24	90.44	93.58	43	502
96.67	99.70	102.66	105.56	108.39	111.15	113.84	116.45	118.97	44	502
121.42	123.78	126.06	0.0	0.0	0.0	0.0	0.0	0.0	45	502
69.03	72.65	76.22	79.76	83.25	86.70	90.10	93.45	96.74	46	502
99.97	103.15	106.26	109.31	112.28	115.19	118.01	120.76	123.43	47	502
126.01	128.51	130.92	0.0	0.0	0.0	0.0	0.0	0.0	48	502
70.97	74.76	78.48	82.18	85.82	89.43	92.98	96.48	99.93	49	502
103.32	106.65	109.90	113.10	116.22	119.27	122.24	125.13	127.94	50	502
130.66	133.29	135.84	0.0	0.0	0.0	0.0	0.0	0.0	51	502
72.94	76.88	80.76	84.62	88.43	92.19	95.90	99.56	103.17	52	502
106.70	110.19	113.59	116.94	120.20	123.41	126.51	129.56	132.50	53	502
135.37	138.14	140.82	0.0	0.0	0.0	0.0	0.0	0.0	54	502
74.92	79.03	83.08	87.09	91.07	94.98	98.86	102.67	106.44	55	502
110.12	113.77	117.32	120.83	124.24	127.59	130.85	134.03	137.12	56	502
140.13	143.05	145.87	0.0	0.0	0.0	0.0	0.0	0.0	57	502
76.92	81.20	85.41	89.59	93.73	97.81	101.86	105.81	109.75	58	502
113.59	117.39	121.10	124.76	128.32	131.82	135.23	138.57	141.80	59	502
144.96	148.02	150.98	0.0	0.0	0.0	0.0	0.0	0.0	60	502
78.95	83.38	87.78	92.12	96.43	100.66	104.89	109.00	113.10	61	502
117.10	121.06	124.92	128.74	132.46	136.11	139.68	143.16	146.55	62	502
149.84	153.05	156.15	0.0	0.0	0.0	0.0	0.0	0.0	63	502
-240.00	1440.00	120.00	2280.00	4680.00	120.00	0.0			0	503
86.59	89.19	91.48	93.43	95.04	96.28	97.15	97.62	97.68	1	503
97.33	98.30	99.81	101.13	102.28	103.23	103.98	104.55	104.92	2	503
105.13	105.18	105.02	0.0	0.0	0.0	0.0	0.0	0.0	3	503
94.99	98.08	100.85	103.27	105.34	107.02	108.32	109.21	109.68	4	503
109.75	111.44	112.96	114.30	115.44	116.39	117.14	117.72	118.09	5	503
118.30	118.30	118.14	0.0	0.0	0.0	0.0	0.0	0.0	6	503
103.60	107.19	110.44	113.35	115.88	118.02	119.75	121.06	121.95	7	503
122.90	124.61	126.15	127.49	128.64	129.58	130.34	130.91	131.27	8	503
131.48	131.47	131.28	0.0	0.0	0.0	0.0	0.0	0.0	9	503
112.40	116.51	120.27	123.66	126.67	129.28	131.46	133.21	134.51	10	503
136.07	137.81	139.35	140.71	141.86	142.81	143.57	144.11	144.49	11	503
144.66	144.66	144.46	0.0	0.0	0.0	0.0	0.0	0.0	12	503
121.42	126.06	130.33	134.23	137.73	140.81	143.44	145.64	147.37	13	503
149.28	151.04	152.61	153.96	155.11	156.06	156.82	157.36	157.74	14	503
157.90	157.87	157.68	0.0	0.0	0.0	0.0	0.0	0.0	15	503
130.66	135.84	140.65	145.06	149.05	152.62	155.73	158.38	160.57	16	503
162.53	164.31	165.88	167.24	168.39	169.35	170.09	170.65	171.01	17	503
171.18	171.12	170.92	0.0	0.0	0.0	0.0	0.0	0.0	18	503
140.13	145.87	151.22	156.15	160.66	164.72	168.32	171.42	173.82	19	503
175.82	177.60	179.19	180.56	181.71	182.67	183.41	183.97	184.30	20	503
184.44	184.42	184.17	0.0	0.0	0.0	0.0	0.0	0.0	21	503
149.84	156.15	162.05	167.53	172.57	177.13	181.22	184.80	187.12	22	503
189.13	190.94	192.53	193.90	195.07	196.01	196.76	197.31	197.64	23	503
197.77	197.73	197.49	0.0	0.0	0.0	0.0	0.0	0.0	24	503
159.80	166.70	173.18	179.21	184.78	189.87	194.46	198.21	200.45	25	503

FIGURE 49. TYPICAL CARD IMAGE OF HVOSM INPUTS FOR HVOSM CURVE STUDY (Continued)

27.00	30.00	33.00	36.00	38.99	41.99	44.97	47.93	50.88	32	501
53.80	56.71	59.59	0.0	0.0	0.0	0.0	0.0	0.0	33	501
-0.60	2.40	5.40	8.53	11.65	14.78	17.91	21.03	24.16	34	501
27.29	30.42	33.54	36.69	39.81	42.94	46.08	49.19	52.29	35	501
55.36	58.42	61.44	0.0	0.0	0.0	0.0	0.0	0.0	36	501
-1.20	1.80	4.80	8.05	11.31	14.56	17.82	21.07	24.32	37	501
27.58	30.83	34.08	37.39	40.63	43.89	47.20	50.45	53.71	38	501
56.93	60.14	63.31	0.0	0.0	0.0	0.0	0.0	0.0	39	501
-1.80	1.20	4.20	7.58	10.96	14.34	17.72	21.10	24.48	40	501
27.87	31.25	34.63	38.08	41.46	44.84	48.33	51.72	55.15	41	501
58.51	61.88	65.20	0.0	0.0	0.0	0.0	0.0	0.0	42	501
-2.40	0.60	3.60	7.11	10.62	14.12	17.63	21.14	24.65	43	501
28.15	31.66	35.17	38.78	42.28	45.80	49.46	53.00	56.59	44	501
60.11	63.64	67.11	0.0	0.0	0.0	0.0	0.0	0.0	45	501
-3.00	0.00	3.00	6.63	10.27	13.90	17.54	21.17	24.81	46	501
28.44	32.08	35.71	39.48	43.11	46.76	50.61	54.29	58.05	47	501
61.71	65.41	69.03	0.0	0.0	0.0	0.0	0.0	0.0	48	501
-3.60	-0.60	2.40	6.16	9.92	13.68	17.45	21.21	24.97	49	501
28.73	32.49	36.25	40.18	43.93	47.72	51.75	55.58	59.52	50	501
63.34	67.20	70.97	0.0	0.0	0.0	0.0	0.0	0.0	51	501
-4.20	-1.20	1.80	5.69	9.58	13.47	17.35	21.24	25.13	52	501
29.02	32.91	36.80	40.88	44.76	48.69	52.91	56.88	61.00	53	501
64.97	69.00	72.94	0.0	0.0	0.0	0.0	0.0	0.0	54	501
-4.80	-1.80	1.20	5.22	9.23	13.25	17.26	21.28	25.28	55	501
29.31	33.32	37.34	41.59	45.59	49.65	54.07	58.20	62.48	56	501
66.62	70.82	74.92	0.0	0.0	0.0	0.0	0.0	0.0	57	501
-5.40	-2.40	0.60	4.74	8.88	13.03	17.17	21.31	25.45	58	501
29.60	33.74	37.88	42.29	46.43	50.62	55.23	59.52	63.98	59	501
68.28	72.65	76.92	0.0	0.0	0.0	0.0	0.0	0.0	60	501
-6.00	-3.00	0.00	4.27	8.54	12.81	17.08	21.35	25.62	61	501
29.88	34.15	38.42	43.00	47.26	51.59	56.40	60.84	65.50	62	501
69.96	74.50	78.95	0.0	0.0	0.0	0.0	0.0	0.0	63	501
-600.00	600.00	60.00	1200.00	2400.00	60.00	0.0			0	502
41.88	43.38	44.81	46.26	47.63	49.01	50.31	51.61	52.85	1	502
54.05	55.20	56.31	57.36	58.36	59.30	60.18	61.01	61.77	2	502
62.48	63.10	63.67	0.0	0.0	0.0	0.0	0.0	0.0	3	502
43.59	45.21	46.78	48.34	49.85	51.35	52.78	54.21	55:57	4	502
56.89	58.16	59.39	60.56	61.68	62.75	63.75	64.69	65.57	5	502
66.39	67.12	67.81	0.0	0.0	0.0	0.0	0.0	0.0	6	502
45.31	47.06	48.76	50.45	52.09	53.71	55.28	56.82	58.30	7	502
59.76	61.15	62.51	63.80	65.04	66.23	67.35	68.41	69.40	8	502
70.34	71.19	71.99	0.0	0.0	0.0	0.0	0.0	0.0	9	502
47.04	48.92	50.75	52.58	54.35	56.10	57.79	59.47	61.08	10	502
62.66	64.17	65.66	67.07	68.44	69.74	70.99	72.17	73.28	11	502
74.33	75.31	76.22	0.0	0.0	0.0	0.0	0.0	0.0	12	502
48.79	50.80	52.77	54.72	56.63	58.51	60.34	62.14	63.88	13	502
65.59	67.23	68:84	70.38	71.87	73.30	74.67	75.97	77.21	14	502
78.37	79.47	80.49	0.0	0.0	0.0	0.0	0.0	0.0	15	502
50.55	52.69	54.80	56.89	58.93	60.94	62.90	64.83	66.71	16	502
68.55	70.32	72.06	73.72	75.34	76.89	78.39	79.81	81.17	17	502
82.46	83.68	84.82	0.0	0.0	0.0	0.0	0.0	0.0	18	502
52.33	54.61	56.85	59.07	61.25	63.40	65.50	67.56	69.57	19	502
71.54	73.44	75.31	77.10	78.85	80.53	82.15	83.69	85.18	20	502
86.59	87.93	89.19	0.0	0.0	0.0	0.0	0.0	0.0	21	502
54.12	56.54	58.92	61.28	63.60	65.88	68.12	70.31	72.46	22	502
74.56	76.60	78.59	80.52	82.40	84.20	85.95	87.63	89.24	23	502
90.77	92.23	93.61	0.0	0.0	0.0	0.0	0.0	0.0	24	502
55.92	58.49	61.01	63.51	65.97	68.39	70.76	73.09	75.38	25	502
77.61	79.79	81.92	83.98	85.99	87.92	89.80	91.60	93.34	26	502
94.99	96.58	98.08	0.0	0.0	0.0	0.0	0.0	0.0	27	502
57.75	60.45	63.12	65.76	68.36	70.92	73.44	75.91	78.33	28	502

FIGURE 49. TYPICAL CARD IMAGE OF HVOSM INPUTS FOR HVOSM CURVE STUDY (Continued)

```
 202.49  204.31  205.91  207.29  208.46  209.40  210.15  210.68  211.00  26 503
 211.14  211.06  210.81    0.0     0.0     0.0     0.0     0.0     0.0   27 503
 170.04  177.54  184.60  191.21  197.33  202.95  208.04  211.55  213.83  28 503
 215.89  217.72  219.33  220.71  221.89  222.83  223.58  224.10  224.40  29 503
 224.53  224.46  224.15    0.0     0.0     0.0     0.0     0.0     0.0   30 503
 180.56  188.67  196.33  203.51  210.19  216.38  221.98  224.94  227.24  31 503
 229.32  231.16  232.78  234.17  235.34  236.30  237.03  237.54  237.86  32 503
 237.95  237.86  237.53    0.0     0.0     0.0     0.0     0.0     0.0   33 503
 191.34  200.09  208.37  216.15  223.41  230.14  235.83  238.37  240.69  34 503
 242.79  244.65  246.28  247.68  248.84  249.80  250.52  251.02  251.33  35 503
 251.44  251.29  250.88    0.0     0.0     0.0     0.0     0.0     0.0   36 503
 202.42  211.83  220.75  229.15  237.02  244.31  249.28  251.85  254.20  37 503
 256.31  258.18  259.82  261.21  262.40  263.34  264.06  264.57  264.85  38 503
 264.93  264.76  264.32    0.0     0.0     0.0     0.0     0.0     0.0   39 503
 213.84  223.93  233.51  242.54  250.99  258.86  262.75  265.36  267.73  40 503
 269.86  271.75  273.40  274.80  275.99  276.93  277.63  278.14  278.42  41 503
 278.45  278.21  277.78    0.0     0.0     0.0     0.0     0.0     0.0   42 503
 225.62  236.38  246.60  256.27 ·265.35  273.40  276.28  278.92  281.32  43 503
 283.47  285.37  287.03  288.45  289.62  290.55  291.26  291.75  292.02  44 503
 292.03  291.76  291.30    0.0     0.0     0.0     0.0     0.0     0.0   45 503
 240.00 2400.00  127.06 4560.00 6000.00  120.00  0.0                      0 504
 157.87  157.68  157.29  156.74  155.97  154.95  153.80  152.48  151.15   1 504
 149.34  147.65  145.67  143.25    0.0     0.0     0.0     0.0     0.0    2 504
 171.90  171.70  171.31  170.69  169.90  168.85  167.68  166.41  164.79   3 504
 163.10  161.38  159.08  157.16    0.0     0.0     0.0     0.0     0.0    4 504
 185.98  185.73  185.33  184.69  183.79  182.75  181.69 '180.30  178.62   5 504
 177.03  174.87  172.79  170.82    0.0     0.0     0.0     0.0     0.0    6 504
 200.08  199.84  199.37  198.64  197.79  196.72  195.62  194.10  192.61   7 504
 190.84  188.64  186.78  184.21    0.0     0.0     0.0     0.0     0.0    8 504
 214.23  213.95  213.45  212.70  211.78  210.83  209.43  208.05  206.48   9 504
 204.42  202.69  200.52  197.90    0.0     0.0     0.0     0.0     0.0   10 504
 228.40  228.08  227.50  226.75  225.84  224.85  223.42  222.05  220.15  11 504
 218.28  216.50  214.03  211.54    0.0     0.0     0.0     0.0     0.0   12 504
 242.59  242.26  241.65  240.87  240.04  238.77  237.55  236.04  234.08  13 504
 232.42  230.06  227.72  225.26    0.0     0.0     0.0     0.0     0.0   14 504
 256.84  256.41  255.81  255.14  254.03  252.85  251.58  249.72  248.27  15 504
 246.06  243.91  241.52  239.31    0.0     0.0     0.0     0.0     0.0   16 504
 271.06  270.67  270.03  269.33  268.19  267.07  265.45  264.03  262.24  17 504
 259.98  257.70  255.62  252.63    0.0     0.0     0.0     0.0     0.0   18 504
 285.38  284.93  284.39  283.43  282.48  281.22  279.55  278.08  275.98  19 504
 273.87  271.89  269.46  266.43    0.0     0.0     0.0     0.0     0.0   20 504
 299.72  299.36  298.70  297.70  296.70  295.21  293.90  291.94  289.98  21 504
 287.85  285.80  282.84  280.59    0.0     0.0     0.0     0.0     0.0   22 504
 314.13  313.73  312.94  312.09  310.78  309.62  307.84  306.04  304.03  23 504
 302.13  299.33  297.15  294.39    0.0     0.0     0.0     0.0     0.0   24 504
 328.66  328.05  327.40  326.27  325.07  323.87  322.03  320.10  318.36  25 504
 315.74  313.30  311.04  308.14    0.0     0.0     0.0     0.0     0.0   26 504
 343.17  342.52  341.82  340.65  339.60  337.93  336.21  334.55  332.47  27 504
 329.79  327.65  324.86  321.90    0.0     0.0     0.0     0.0     0.0   28 504
 357.62  357.11  356.14  355.25  353.77  352.22  350.48  348.75  346.25  29 504
 344.22  341.55  338.72  335.76    0.0     0.0     0.0     0.0     0.0   30 504
 372.30  371.54  370.66  369.72  368.19  366.58  364.98  362.56  360.75  31 504
 358.21  355.58  352.67  350.13    0.0     0.0     0.0     0.0     0.0   32 504
 386.95  386.15  385.38  384.05  382.65  381.16  378.97  377.21  374.84  33 504
 372.33  369.56  367.00  363.85    0.0     0.0     0.0     0.0     0.0   34 504
 401.54  400.96  399.82  398.57  397.27  395.28  393.68  391.53  389.06  35 504
 386.43  384.02  380.94  377.76    0.0     0.0     0.0     0.0     0.0   36 504
1.0     1.0     1.0     1.0                                               0 506
   100 KPH                                                                0 600
0.57    -2.86   90.     0.0     0.0     0.0                               0 601
0.0     120.    -17.3   1056.                                             0 602
0.0     0.0     0.0     0.0                                               0 603
                                                                          09999
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FIGURE 49. TYPICAL CARD IMAGE OF HVOSM INPUTS FOR HVOSM CURVE STUDY (Continued)

MCI-JEL HVOSM CURVE STUDIES: RUN:HCS#18 10/05/81
1971 DODGE CORONET 4-DOOR SEDAN FIRESTONE RADIAL VI 210 M PATH,5% BRAKING,PROBE 25%
210 M RADIUS,10% SE,5% GRADE,80 M RUNOFF 100 KPH

P R O G R A M C O N T R O L D A T A

START TIME	TO	=	0.0	SEC	
END TIME	T1	=	4.9700	SEC	
INTEGRATION INCREMENT	DTCOMP	=	0.0100	SEC	
INTEGRATION MODE	MODE	=	1		(0=VARIABLE STEP ADAMS-MOULTON -)1= RUNGA-KUTTA (2= FIXED STEP ADAMS-MOULTON
PRINT INTERVAL	DTPRNT	=	0.0100	SEC	
SUSPENSION OPTION	ISUS	=	0		(0= INDEPENDENT FRONT SUSPENSION, SOLID REAR AXLE -)1= INDEPENDENT FRONT AND REAR SUSPENSION (2= SOLID FRONT AND REAR AXLES
CURB/STEER OPTION	INDCRB	=	0		(0= NO CURB, NO STEER DEGREE OF FREEDOM -)1= CURB (-1=STEER DEGREE OF FREEDOM, NO CURB
CURB INTEGRATION INCR.	DELTC	=	0.0	SEC	
BARRIER OPTION	INDB	=	0		(0= NO BARRIER |1= RIGID BARRIER , FINITE VERT. DIM. -)2= '' '' ,INFINITE '' '' |3= DEFORM. '' , FINITE '' '' (4= '' '' ,INFINITE '' ''
BARRIER INTEGRATION INCR.	DELTB	=	0.0	SEC	

I N I T I A L C O N D I T I O N S

	XCOP	=	0.0	INCHES		UO	=	1056.00	IN/SEC
SPRUNG MASS C.G. POSITION	YCOP	=	120.00	INCHES	SPRUNG MASS LINEAR VELOCITY	VO	=	0.0	IN/SEC
	ZCOP	=	-17.30	INCHES		WO	=	0.0	IN/SEC
	PHIO	=	0.57	DEGREES		PO	=	0.0	DEG/SEC
SPRUNG MASS ORIENTATION	THETAO	=	-2.86	DEGREES	SPRUNG MASS ANGULAR VELOCITY	QO	=	0.0	DEG/SEC
	PSIO	=	90.00	DEGREES		RO	=	0.0	DEG/SEC
	DEL10	=	0.0	INCHES		DEL100	=	0.0	IN/SEC
UNSPRUNG MASS POSITIONS	DEL20	=	0.0	INCHES	UNSPRUNG MASS VELOCITIES	DEL200	=	0.0	IN/SEC
	DEL30	=	0.0	INCHES		DEL300	=	0.0	IN/SEC
	PHIRO	=	0.0	DEGREES		PHIROO	=	0.0	DEG/SEC
STEER ANGLE	PSIFIO	=	0.0	DEGREES	STEER VELOCITY	PSIFDO	=	0.0	DEG/SEC

FIGURE 50. INPUT PARAMETER LISTING FOR "TYPICAL" HVOSM CURVE RUN

MCI-JEL HVOSM CURVE STUDIES: RUN:HCS#18 10/05/81
1971 DODGE CORONET 4-DOOR SEDAN FIRESTONE RADIAL VI 210 M PATH,5% BRAKING,PROBE 25%
210 M RADIUS,10% SE,5% GRADE,80 M RUNOFF 100 KPH

SPRUNG MASS	XMS =	8.430	LB-SEC**2/IN	FRONT WHEEL X LOCATION	A =	49.300	INCHES
FRONT UNSPRUNG MASS	XMUF =	0.510	LB-SEC**2/IN	REAR WHEEL X LOCATION	B =	68.700	INCHES
REAR UNSPRUNG MASS	XMUR =	0.820	LB-SEC**2/IN	FRONT WHEEL Z LOCATION	ZF =	10.820	INCHES
X MOMENT OF INERTIA	XIX =	3760.000	LB-SEC**2-IN	REAR WHEEL Z LOCATION	ZR =	10.680	INCHES
Y MOMENT OF INERTIA	XIY =	23000.000	LB-SEC**2-IN	FRONT WHEEL TRACK	TF =	59.800	INCHES
Z MOMENT OF INERTIA	XIZ =	23300.000	LB-SEC**2-IN	REAR WHEEL TRACK	TR =	61.800	INCHES
XZ PRODUCT OF INERTIA	XIXZ =	530.000	LB-SEC**2-IN	FRONT ROLL AXIS	RHOF =	0.0	NOT USED
FRONT AXLE MOMENT OF INERTIA	XIF =	0.0	NOT USED	REAR ROLL AXIS	RHO =	0.0	INCHES
REAR AXLE MOMENT OF INERTIA	XIR =	550.000	LB-SEC**2-IN	FRONT SPRING TRACK	TSF =	0.0	NOT USED
GRAVITY	G =	386.400	IN/SEC**2	REAR SPRING TRACK	TS =	47.000	INCHES
	X1 =	0.0	INCHES	FRONT AUX ROLL STIFFNESS	RF =	40400.00	LB-IN/RAD
ACCELEROMETER 1 POSITION	Y1 =	-14.00	INCHES	REAR AUX ROLL STIFFNESS	RR =	-5100.00	LB-IN/RAD
	Z1 =	0.0	INCHES	REAR ROLL-STEER COEF.	AKRS =	0.0200	RAD/RAD
	X2 =	-68.70	INCHES		AKDS =	0.0	NOT USED
ACCELEROMETER 2 POSITION	Y2 =	-30.90	INCHES	REAR DEFL-STEER COEFS.	AKDS1=	0.0	NOT USED
	Z2 =	10.10	INCHES		AKDS2=	0.0	NOT USED
					AKDS3=	0.0	NOT USED

STEERING SYSTEM

MOMENT OF INERTIA	XIPS =	0.0	LB-SEC**2-IN
COULOMB FRICTION TORQUE	CPSP =	0.0	LB-IN
FRICTION LAG	EPSP =	0.0	RAD/SEC
ANGULAR STOP RATE	AKPS =	0.0	LB-IN/RAD
ANGULAR STOP POSITION	OMGPS =	0.559	RADIANS
PNEUMATIC TRAIL	XPS =	0.0	INCHES

	FRONT SUSPENSION			REAR SUSPENSION		
SUSPENSION RATE	AKF =	105.000	LB/IN	AKR =	120.000	LB/IN
COMPRESSION STOP COEFS.	AKFC =	189.000	LB/IN	AKRC =	324.000	LB/IN
	AKFCP =	600.000	LB/IN**3	AKRCP =	600.000	LB/IN**3
EXTENSION STOP COEFS.	AKFE =	588.000	LB/IN	AKRE =	864.000	LB/IN
	AKFEP =	600.000	LB/IN**3	AKREP =	600.000	LB/IN**3
COMPRESSION STOP LOCATION	OMEGFC =	-2.400	INCHES	OMEGRC =	-4.400	INCHES
EXTENSION STOP LOCATION	OMEGFE =	2.100	INCHES	OMEGRE =	3.600	INCHES
STOP ENERGY DISSIPATION FACTOR	XLAMF =	0.500		XLAMR =	0.500	
VISCOUS DAMPING COEF.	CF =	6.850	LB-SEC/IN	CR =	7.480	LB-SEC/IN
COULOMB FRICTION	CFP =	40.000	LB	CRP =	38.000	LB
FRICTION LAG	EPSF =	0.100	IN/SEC	EPSR =	0.100	IN/SEC

FIGURE 50. INPUT PARAMETER LISTING FOR "TYPICAL" HVOSM CURVE RUN (Continued)

MCI-JEL HVOSM CURVE STUDIES: RUN:HCS#18 10/05/81
1971 DODGE CORONET 4-DOOR SEDAN FIRESTONE RADIAL VI 210 M PATH,5% BRAKING,PROBE 25%
210 M RADIUS,10% SE,5% GRADE,80 M RUNOFF 100 KPH

FRONT WHEEL CAMBER VS SUSPENSION DEFLECTION		REAR WHEEL CAMBER VS SUSPENSION DEFLECTION		FRONT HALF-TRACK CHANGE VS SUSPENSION DEFLECTION		REAR HALF-TRACK CHANGE VS SUSPENSION DEFLECTION	
DELTAF INCHES	PHIC DEGREES	DELTAR NOT USED	PHIRC NOT USED	DELTAF INCHES	DTHF INCHES	DELTAR NOT USED	DTHR NOT USED
-3.00	-0.43	-3.00	0.0	-3.00	0.0	-3.00	0.0
-2.00	-0.95	-2.00	0.0	-2.00	0.0	-2.00	0.0
-1.00	-1.22	-1.00	0.0	-1.00	0.0	-1.00	0.0
0.0	-1.26	0.0	0.0	0.0	0.0	0.0	0.0
1.00	-0.98	1.00	0.0	1.00	0.0	1.00	0.0
2.00	-0.41	2.00	0.0	2.00	0.0	2.00	0.0
3.00	0.0	3.00	0.0	3.00	0.0	3.00	0.0

DRIVER CONTROL TABLES

T SEC	PSIF DEG	TQF LB-FT	TQR LB-FT	T SEC	PSIF DEG	TQF LB-FT	TQR LB-FT	T SEC	PSIF DEG	TQF LB-FT	TQR LB-FT	T SEC	PSIF DEG	TQF LB-FT	TQR LB-FT
0.0	0.0	0.0	-95.0	2.000	0.0	0.0	-95.0	4.000	0.0	0.0	-95.0				
1.000	0.0	0.0	-95.0	3.000	0.0	0.0	-95.0	5.000	0.0	0.0	0.0				

T I R E   D A T A

			RF	LF	RR	LR	
TIRE LINEAR SPRING RATE	AKT	=	1450.000	1450.000	1450.000	1450.000	LB/IN
DEFL. FOR INCREASED RATE	SIGT	=	3.000	3.000	3.000	3.000	INCHES
SPRING RATE INCREASING FACTOR	XLAMT	=	10.000	10.000	10.000	10.000	
	AO	=	-37.000	-37.000	-37.000	-37.000	
	A1	=	13.200	13.200	13.200	13.200	
SIDE FORCE COEFFICIENTS	A2	=	3043.000	3043.000	3043.000	3043.000	
	A3	=	0.580	0.580	0.580	0.580	
	A4	=	91435.000	91435.000	91435.000	91435.000	
TIRE OVERLOAD FACTOR	OMEGT	=	1.000	1.000	1.000	1.000	
TIRE UNDEFLECTED RADIUS	RW	=	13.200	13.200	13.200	13.200	INCHES
TIRE / GROUND FRICTION COEF.	AMU	=	0.780	0.780	0.780	0.780	

NO ANTI-PITCH TABLES

FIGURE 50. INPUT PARAMETER LISTING FOR "TYPICAL" HVOSM CURVE RUN (Continued)

MCI-JEL HVOSM CURVE STUDIES: RUN:HCS#18
1971 DODGE CORONET 4-DOOR SEDAN FIRESTONE RADIAL VI 10/05/81
210 M RADIUS,10% SE,5% GRADE,80 M RUNOFF 100 KPH 210 M PATH,5% BRAKING,PROBE 25%

PATH DESCRIPTORS	IPATH	=	1
NUMBER OF PATH DESCRIPTORS	KLI	=	4
NUMBER OF POINTS ON PATH	NPTS	=	100
DISTANCE BETWEEN POINTS	DELL	=	120.000 INCHES
COORDINATES OF 1ST PATH POINTS:	XINIT	=	0.0 INCHES
	YINIT	=	0.0 INCHES
INITIAL ROADWAY HEADING	PSA	=	90.00 DEGREES

PATH CURVATURE DESCRIPTORS:

DEGREE OF CURVATURE	DI(1)	=	0.0 DEGREES
DISTANCE ALONG PATH	RLI(1)	=	0.0 INCHES
DEGREE OF CURVATURE	DI(1)	=	0.0 DEGREES
DISTANCE ALONG PATH	RLI(1)	=	600.00 INCHES
DEGREE OF CURVATURE	DI(1)	=	-8.2704 DEGREES
DISTANCE ALONG PATH	RLI(1)	=	720.00 INCHES
DEGREE OF CURVATURE	DI(1)	=	-8.2704 DEGREES
DISTANCE ALONG PATH	RLI(1)	=	12000.00 INCHES

WAGON TONGUE STEER DESCRIPTORS	IWAGN	=	1
INITIAL PROBE SAMPLE TIME	TPRB	=	0.0 SECONDS
TIME INCREMENT BETWEEN SAMPLES	DPRB	=	0.100 SECONDS
LENGTH OF PROBE	PLGTH	=	264.00 INCHES
MINIMUM ACCEPTABLE ERROR	PMIN	=	0.0 INCHES
MAXIMUM OCCUPANT ACCELERATION	PMAX	=	0.500 G-UNITS
STEER CORRECTION FACTOR	PGAIN	=	.0038000 RAD/IN
STEER CORRECTION DAMPING FACTOR	QGAIN	=	.0003800 RAD-SEC/IN
MAXIMUM STEERING WHEEL RATE	PSIFD	=	400.000 DEG/SEC

FILTER DESCRIPTORS	IFILT	=	1
TIME LAG OF FILTER	TIL	=	0.050000 SECONDS
TIME LEAD OF FILTER	TI	=	0.009050 SECONDS
TIME DELAY OF FILTER	TAUF	=	0.0 SECONDS

FIGURE 50. INPUT PARAMETER LISTING FOR "TYPICAL" HVOSM CURVE RUN (Continued)

MCI-JEL HVOSM CURVE STUDIES: RUN:HCS#18
1971 DODGE CORONET 4-DOOR SEDAN FIRESTONE RADIAL VI
210 M RADIUS, 10% SE, 5% GRADE, 80 M RUNOFF 100 KPH

210 M PATH, 5% BRAKING, PROBE 25% 10/05/81

N	PATH COORDINATES X(N) (FT)	Y(N) (FT)	TANGENT VECTORS DX(N) (DEG)	DY(N) (DEG)	DEGREE OF CURVATURE D(N) (DEG)
1	0.0	0.0	90.000	90.000	0.0
2	-0.000	10.000	90.000	90.000	0.0
3	-0.000	20.000	90.000	90.000	0.0
4	-0.000	30.000	90.000	90.000	0.0
5	-0.000	40.000	90.000	90.000	0.0
6	-0.000	50.000	90.000	90.000	0.0
7	-0.000	60.000	90.000	90.000	-8.270
8	0.072	70.000	89.173	89.173	-8.270
9	0.288	79.997	88.346	88.346	-8.270
10	0.649	89.991	87.519	87.519	-8.270
11	1.154	99.978	86.692	86.692	-8.270
12	1.803	109.957	85.865	85.865	-8.270
13	2.598	119.925	85.038	85.038	-8.270
14	3.533	129.881	84.211	84.211	-8.270
15	4.613	139.823	83.384	83.384	-8.270
16	5.837	149.747	82.557	82.557	-8.270
17	7.204	159.653	81.730	81.730	-8.270
18	8.714	169.539	80.903	80.903	-8.270
19	10.366	179.401	80.076	80.076	-8.270
20	12.161	189.239	79.249	79.249	-8.270
21	14.097	199.049	78.422	78.421	-8.270
22	16.175	208.831	77.594	77.594	-8.270
23	18.394	218.581	76.767	76.767	-8.270
24	20.753	228.299	75.940	75.940	-8.270
25	23.252	237.981	75.113	75.113	-8.270
26	25.891	247.627	74.286	74.286	-8.270
27	28.668	257.233	73.459	73.459	-8.270
28	31.584	266.798	72.632	72.632	-8.270
29	34.638	276.320	71.805	71.805	-8.270
30	37.829	285.797	70.978	70.978	-8.270
31	41.156	295.227	70.151	70.151	-8.270
32	44.619	304.608	69.324	69.324	-8.270
33	48.218	313.938	68.497	68.497	-8.270
34	51.950	323.215	67.670	67.670	-8.270
35	55.816	332.437	66.843	66.843	-8.270
36	59.815	341.603	66.016	66.016	-8.270
37	63.945	350.709	65.189	65.189	-8.270
38	68.207	359.755	64.362	64.362	-8.270
39	72.598	368.739	63.535	63.535	-8.270
40	77.119	377.658	62.708	62.708	-8.270
41	81.768	386.511	61.881	61.881	-8.270
42	86.545	395.295	61.054	61.054	-8.270
43	91.448	404.011	60.227	60.227	-8.270
44	96.476	412.654	59.400	59.399	-8.270
45	101.628	421.224	58.573	58.572	-8.270
46	106.903	429.719	57.745	57.745	-8.270
47	112.301	438.137	56.918	56.918	-8.270
48	117.819	446.476	56.091	56.091	-8.270

FIGURE 50. INPUT PARAMETER LISTING FOR "TYPICAL" HVOSM CURVE RUN (Continued)

MCI-JEL HVOSM CURVE STUDIES: RUN:HCS#18
1971 DODGE CORONET 4-DOOR SEDAN FIRESTONE RADIAL VI
210 M RADIUS, 10% SE, 5% GRADE, 80 M RUNOFF 100 KPH

10/05/81
210 M PATH, 5% BRAKING, PROBE 25%

N	PATH COORDINATES X(N) (FT)	Y(N) (FT)	TANGENT VECTORS DX(N) (DEG)	DY(N) (DEG)	DEGREE OF CURVATURE D(N) (DEG)
51	135.088	471.002	53.610	53.610	-8.270
52	141.079	479.009	52.783	52.783	-8.270
53	147.184	486.928	51.956	51.956	-8.270
54	153.403	494.758	51.129	51.129	-8.270
55	159.735	502.498	50.302	50.302	-8.270
56	166.177	510.145	49.475	49.475	-8.270
57	172.729	517.699	48.648	48.648	-8.270
58	179.390	525.157	47.821	47.821	-8.270
59	186.157	532.519	46.994	46.994	-8.270
60	193.030	539.781	46.167	46.167	-8.270
61	200.007	546.944	45.340	45.340	-8.270
62	207.087	554.005	44.513	44.513	-8.270
63	214.268	560.964	43.686	43.686	-8.270
64	221.549	567.818	42.859	42.859	-8.270
65	228.927	574.566	42.032	42.032	-8.270
66	236.403	581.207	41.205	41.205	-8.270
67	243.973	587.740	40.378	40.377	-8.270
68	251.637	594.162	39.551	39.550	-8.270
69	259.393	600.474	38.724	38.723	-8.270
70	267.239	606.672	37.896	37.896	-8.270
71	275.174	612.757	37.069	37.069	-8.270
72	283.196	618.726	36.242	36.242	-8.270
73	291.303	624.579	35.415	35.415	-8.270
74	299.494	630.315	34.588	34.588	-8.270
75	307.767	635.931	33.761	33.761	-8.270
76	316.120	641.428	32.934	32.934	-8.270
77	324.551	646.803	32.107	32.107	-8.270
78	333.059	652.057	31.280	31.280	-8.270
79	341.642	657.187	30.453	30.453	-8.270
80	350.298	662.192	29.626	29.626	-8.270
81	359.025	667.072	28.799	28.799	-8.270
82	367.822	671.825	27.972	27.972	-8.270
83	376.687	676.451	27.145	27.145	-8.270
84	385.617	680.949	26.318	26.318	-8.270
85	394.611	685.317	25.491	25.491	-8.270
86	403.668	689.555	24.664	24.664	-8.270
87	412.785	693.662	23.837	23.837	-8.270
88	421.960	697.636	23.010	23.009	-8.270
89	431.191	701.478	22.183	22.182	-8.270
90	440.477	705.187	21.356	21.355	-8.270
91	449.816	708.760	20.529	20.528	-8.270
92	459.205	712.199	19.702	19.701	-8.270
93	468.643	715.501	18.875	18.874	-8.270
94	478.127	718.667	18.048	18.047	-8.270
95	487.656	721.697	17.221	17.220	-8.270
96	497.228	724.587	16.394	16.393	-8.270
97	506.840	727.340	15.566	15.566	-8.270
98	516.492	729.954	14.739	14.739	-8.270

FIGURE 50. INPUT PARAMETER LISTING FOR "TYPICAL" HVOSM CURVE RUN (Continued)

The actual HVOSM simulation runs were performed in batch by use of the interactive remote job entry (RJE) commands.

HVOSM Modifications

A number of refinements and revisions to the HVOSM program were required, including additional outputs of vehicle responses, revision of the path-following driver model, and development of a preprocessing program to simplify the interface between highway definition and HVOSM card inputs. These revisions are described below.

Additional Outputs

Additional calculations and outputs of the existing HVOSM RD2 program were found to be required to enable the evaluation of the curve study. The revisions were as follows:

"Discomfort Factor".--The lateral acceleration output of HVOSM corresponds to measurements made with a "hard-mounted," or body-fixed accelerometer oriented laterally on the vehicle. During cornering, the lateral acceleration of the vehicle is directed toward the center of the turn. On a superelevated turn, the component of gravity that acts laterally on the vehicle is also directed toward the turn center. Thus, the lateral acceleration output is increased by superelevation.

Since the vehicle occupants respond to centrifugal force, their inertial reaction is toward the outside of the turn and therefore the component of gravity that acts laterally on them in a superelevated turn reduces the magnitude of the disturbance produced by cornering. A corresponding program output has been defined to evaluate occupant discomfort in turns.

The effects of a vehicle's roll angle and lateral acceleration on occupants are combined in a "discomfort factor" relationship which represents the net lateral disturbance felt by the occupants (i.e., the occupants' reaction to the combined effects of the lateral acceleration and roll angle).

The "discomfort factor" is coded in the following form:

$$\text{DISCOMFORT FACTOR} = - \text{YLAT} + 1.0 * \text{SIN}\ \Theta$$

Where: DISCOMFORT FACTOR is in G-units

YLAT = Vehicle Lateral Acceleration in vehicle-fixed coordinate system, G units

Θ = Vehicle roll angle, radians.

Calculations related to the discomfort factor and corresponding outputs were incorporated into the HVOSM.

Friction Demand.--The friction demand is defined to be the ratio of the side force to the normal load of an individual tire. It is indicative of the friction being utilized by each individual tire. The standard outputs of HVOSM include the side force and normal force for each tire. Coding changes were incorporated to calculate and print out the friction demand for each tire at each interval of time.

Driver Model

A recognized problem in the use of either simulation models or full-scale testing in relation to investigations of automobile dynamics is the manner of guiding and controlling the vehicle. Repeatability is essential, and the control inputs must be either representative of an average driver or optimized to achieve a selected maneuver without "hunting" or oscillation. In this investigation of geometric features of highways, the transient portions of the vehicle responses constituted justification for applying a complex computer simulation. The steady-state portions of the vehicle responses can be predicted by means of straightforward hand calculations. Thus, it is essential that the transient responses should not be contaminated by oscillatory steering control inputs.

The Driver model contained in the distributed version of the HVOSM Vehicle Dynamics program was intended to be incorporated into the HVOSM Roadside Design version, but it proved to be inadequate for the present research effort. Therefore, new routines were written for the HVOSM Roadside Design program as described below.

"Wagon-Tongue" Algorithm.--The "wagon-tongue" type of steering control incorporated into the HVOSM Roadside Design Version is one in which the front wheel steer angle is directly proportional to the error of a point on a forward extension of the vehicle X-axis relative to the desired path.

The basic inputs to the "wagon-tongue" algorithm are described in Table 54.

Table 54

INPUTS FOR "WAGON-TONGUE" DRIVER MODEL

Input	Description	Units
TPRB	Time at which driver model is to begin	sec
DPRB	Time between driver model samples	sec
PLGTH	Probe length measured from the center of gravity of the vehicle along the vehicle-fixed X axis	in
PMIN	Null band, minimum acceptable error	in
PMAX	Maximum allowable discomfort factor above which driver model will only reduce steer angle	g-units
PGAIN	Steer correction multiplier--error of probe from desired path multiplied by PGAIN to determine steer correction	rad/in

1 in = 25.4 mm

Desired Path Definition.--The revision to the HVOSM driver model included the incorporation of a "path generating" routine to create a desired path of X,Y data pairs from standard roadway geometric descriptors. Figure 51 lists the path generating routine.

```
C  PATHT.FOR  F12                30 DECEMBER 1980        J T FLECK
C   PATH GENERATOR
C   ROUTINE TO TEST PATH GENERATION SUBROUTINES SETD AND PATHG
C           MAY BE USED TO GENERATE DATA SETS FOR TERRAIN GENERATOR
C           OR HVOSM
C
C   INPUTS:
C       NPTS            NUMBER OF POINTS DESIRED
C       XINIT           X COORDINATE OF FIRST POINT
C       YINIT           Y COORDINATE OF FIRST POINT
C       DELL            SPACING BETWEEN POINTS (ALONG STRAIGHT LINE)
C       PSA             INITIAL HEADING (TANGENT TO PATH)
C       KLI             NUMBER OF SECTIONS (CURVATURES)
C             IF  = 0   PROGRAM DEFAULTS TO POINTS IN DATA STATEMENT
C             IF  > 0   REQUIRES THE FOLLOWING INPUT    L = 1, KLI
C                       DI(L)   CURVATURE     > 0 RIGHT TURN
C                                             = 0 STRAIGHT
C                                             < 0 LEFT TURN
C                       RLI(L)  DISTANCE FROM INITIAL POINT WHERE DI(L)
C                               IS EFFECTIVE.
C                               DISTANCE IS MEASURED IN STAIGHT LINE
C                               SEGMENTS BETWEEN POINTS. IF DISTANCE
C                             ' ALONG ARC IS REQUIRED SUBROUTINE SETD
C                               MUST BE MODIFIED.
C              NOTE: KLI MAY BE 1 OR GREATER
C                       E.G. TO GENERATE A STAIGHT PATH N*DELL UNITS
C                       LONG AND THEN A RIGHT TURN WITH A CURVATURE OF 20
C                       INPUT KLI = 1, DI(1) = 20., RLI(1) = N*DELL
C                       THE ANGLE OF TURN IS GIVEN BY
C                       ANGLE = 2*ARCSIN[(DELL/2)*(PI/180)*(DI(L)/100)]
C
C   OUTPUT
C         X(I), Y(I)    COORDINATES OF POINT I  I = 1 TO NPTS
C        DX(I),DY(I)    TANGENT AT POINT I (DIRECTION OF PATH)
C         D(I)          CURVATURE DEFINING PATH FROM POINT I TO POINT I+1
C
C         THESE ARE WRITTEN ON A DATA SET (SY1:PTH.DAT) FOR USE BY OTHER
C         ROUTINES
C
C
      INTEGER PLOT
      DIMENSION X(100),Y(100),DX(100),DY(100),D(100),DI(100),RLI(100)
      DIMENSION PLOT(70,70)
      DATA RAD/0.01745329/, D /10*0.0,9*20.0,9*-20.0,9*20.0,63*0.0/
      DATA KLI/0/, DI/100*0.0/, RLI/100*0.0/
C
      CALL OPEN(6,'SY1:PTH.DAT ')
C ENTER INITIAL DATA
    1 WRITE(1,5)
    5 FORMAT(1X,' ENTER NPTS,XINIT,YINIT,DELL,PSA '/)
      READ(1,6)NPTS,XINIT,YINIT,DELL,PSA
    6 FORMAT(I4,4F9.0)
      IF(NPTS.LT.2)ENDFILE 6
      IF(NPTS.LT.2)STOP NPTS
C
C ENTER # OF CURVATURES (IF 0 ROUTINE USES D SET BY DATA STATEMENT)
C     AND OUTPUT UNIT IOUT =0 DEFAULTS TO SCREEN, IOUT =2 FOR PRINTER
      WRITE(1,7)
    7 FORMAT(' ENTER KLI,IOUT'/)
      READ(1,11)KLI,IOUT
```

FIGURE 51. PATH GENERATING ROUTINE

```
   11    FORMAT(2I4)
C
         IF(IOUT.EQ.0)IOUT = 1
CHECK IF DI'S AND RLI' ARE TO BE INPUTTED
         IF(KLI.EQ.0)GO TO 17
         DO 15 I =1,KLI
         WRITE(1,14)
   14    FORMAT(' ENTER DI, RLI'/)
   15    READ(1,16)DI(I),RLI(I)
   16    FORMAT(2F9.0)
C
CALL ROUTINE TO COMPUTE D'S FROM DI'S
         CALL SETD(KLI,DI,RLI,NPTS,DELL,D)
C
C INITIALIZE POINTS
   17    X(1)  = XINIT
         Y(1)  = YINIT
C
C INITIALIZE TANGENT
         DX(1) = COS(PSA *RAD)
         DY(1) = SIN(PSA *RAD)
C
CALL ROUTINE TO SET PATH
         CALL PATHG(NPTS,DELL,X,Y,D,DX,DY)
C
         WRITE(6)NPTS,DELL,PSA ,X,Y,DX,DY,D
         WRITE(IOUT,23)NPTS,KLI,DELL,PSA
   23    FORMAT(1X,'NPTS=',I4,', KLI=',I4,',DELL=',F10.4,',PSA =',F10.4/)
         IF(KLI.GT.0)WRITE(IOUT,24)(L,DI(L),RLI(L),L=1,KLI)
   24    FORMAT(1X,I4,2F10.4)
         WRITE(IOUT,25)
   25    FORMAT(/' POINT #      POSITION',19X,'TANGENT',10X,'CURVATURE')
         WRITE(IOUT,26)(I,X(I),Y(I),DX(I),DY(I),D(I),I=1,NPTS)
   26    FORMAT(1X,I4,2F10.2,10X,2F10.5,F10.2)
C
C PRINTER PLOT: SPECIAL ROUTINE TO TEST ABOVE DATA
         M  = NPTS
         XX = X(1)
         XM = X(1)
         YX = Y(1)
         YM = Y(1)
         DO 35 I =1,M
         IF(X(I).GT.XX)XX = X(I)
         IF(X(I).LT.XM)XM = X(I)
         IF(Y(I).GT.YX)YX = Y(I)
   35    IF(Y(I).LT.YM)YM = Y(I)
         SC = XX-XM
         IF(YX-YM.GT.SC)SC = YX-YM
         SX = 60./SC
         SY = 0.6*SX
         DO 38 I=1,70
         DO 38 J=1,70
   38    PLOT(I,J) = ' '
         IMAX = 1
         DO 40 K=1,M
         J = (X(K)-XM)*SX +1.
         I = (Y(K)-YM)*SY +1.
         IF(I.GT.IMAX)IMAX = I
   40    PLOT(I,J) = '*'
         IF(IOUT.EQ.2)WRITE(2,41)
```

FIGURE 51. PATH GENERATING ROUTINE (Continued)

```
41     FORMAT(1H1)
C
       DO 50 I=1,IMAX
       LM = 61
       DO 44 J=1,60
       IF(PLOT(I,LM).NE.' ')GO TO 45
44     LM = LM-1
45     WRITE(IOUT,47)(PLOT(I,L),L=1,LM)
47     FORMAT(5X,71A1)
50     CONTINUE
       GO TO 1
       END
```

FIGURE 51. PATH GENERATING ROUTINE (Continued)

```
C  SUBROUTINE PATH:  PATH.FOR  F12        30 DECEMBER 1980  J T FLECK
C   PATH GENERATOR  HVOSM RD-2
C   ROUTINE USED IN HVOSM RD-2 TO GENERATE PATH DATA
C
C   INPUTS:
C       NPTS              NUMBER OF POINTS DESIRED
C       XINIT             X COORDINATE OF FIRST POINT
C       YINIT             Y COORDINATE OF FIRST POINT
C       DELL              SPACING BETWEEN POINTS (ALONG STRAIGHT LINE)
C       PSA               INITIAL HEADING (TANGENT TO PATH)
C       KLI               NUMBER OF SECTIONS (CURVATURES)
C             IF  = 0     PROGRAM DEFAULTS TO POINTS IN DATA STATEMENT
C             IF  > 0     REQUIRES THE FOLLOWING INPUT    L = 1, KLI
C                         DI(L)   CURVATURE    > 0 RIGHT TURN
C                                              = 0 STRAIGHT
C                                              < 0 LEFT TURN
C                         RLI(L)  DISTANCE FROM INITIAL POINT WHERE DI(L)
C                                 IS EFFECTIVE.
C                                 DISTANCE IS MEASURED IN STAIGHT LINE
C                                 SEGMENTS BETWEEN POINTS. IF DISTANCE
C                                 ALONG ARC IS REQUIRED SUBROUTINE SETD
C                                 MUST BE MODIFIED.
C               NOTE: KLI MAY BE 1 OR GREATER
C                         E.G. TO GENERATE A STAIGHT PATH N*DELL UNITS
C                         LONG AND THEN A RIGHT TURN WITH A CURVATURE OF 20
C                         INPUT KLI = 1, DI(1) = 20., RLI(1) = N*DELL
C                         THE ANGLE OF TURN IS GIVEN BY
C                         ANGLE = 2*ARCSIN[(DELL/2)*(PI/180)*(DI(L)/100)]
C
C  OUTPUT
C       X(I), Y(I)    COORDINATES OF POINT I   I = 1 TO NPTS
C      DX(I),DY(I)    TANGENT AT POINT I (DIRECTION OF PATH)
C       D(I)          CURVATURE DEFINING PATH FROM POINT I TO POINT I+1
C
C
C
C
      SUBROUTINE PATH
      COMMON/PATHD/IPATH ,KLI ,DI(10),RLI(10),
     1            NPTS,XINIT,YINIT,PSA,DELL,
     2            X(100),Y(100),DX(100),DY(100),D(100)
C LIMIT ARRAY SIZES
      IF(KLI.GT.10)KLI = 10
      IF(NPTS.GT.100)NPTS = 100
      CALL SETD(KLI,DI,RLI,NPTS,DELL,D)
C SETD WAS MODIFIED ON 30 DEC 1980 TO PRODUCE SPIRAL
C INITIALIZE FIRST POINT AND TANGENT
      X(1)  =  XINIT
      Y(1)  =  YINIT
      DX(1) =  COS(PSA)
      DY(1) =  SIN(PSA)
C
      CALL PATHG(NPTS,DELL,X,Y,D,DX,DY)
C
      RETURN
      END
C
C
C
C
```

FIGURE 51. PATH GENERATING ROUTINE (Continued)

```
C PROBE.FOR  F12                  30 DECEMBER 1980       J T FLECK
C SUBROUTINE PROBE:  CALCULATES DISTANCE OF A POINT FROM CENTERLINE
C
C USED IN HVOSM RD-2 MOD'S
C
C INPUTS
C       XP,YP          GIVEN POINT
C       M              NUMBER OF REFERENCE POINTS (= NPTS)
C       X(I), Y(I)     REFERENCE POINTS OF PATH , I =1,NPTS
C       DX(I),DY(I)    TANGENT VECTOR AT REFERENCE POINT
C       D(I)           DEGREE OF CURVATURE AT BETWEEN POINT I AND I+1
C                         D > 0  RIGHT TURN
C                         D = 0  STRAIGHT LINE
C                         D < 0  LEFT TURN
C
C OUTPUTS
C       I              POINT IDENTIFYING SECTOR OF CLOSEST APPROACH
C       DIST           DISTANCE OF POINT FROM ARC
C                         POSITIVE IF POINT IS TO RIGHT OF ARC
C                         NEGATIVE IF POINT IS TO LEFT  OF ARC
C       XX ,YY         POINT ON ARC NEAREST GIVEN POINT
C
C NOTE: ON FIRST ENTRY ROUTINE STARTS WITH I = 1, ON SUBSEQUENT
C       ENTRIES THE PREVIOUS VALUE OF I IS USED.  THIS LOGIC SHOULD BE
C       ADEQUATE FOR THE PROPOSED USE OF THE ROUTINE.
C
C       CALCULATION OF XX AND YY MAY BE DELETED IF THIS POINT IS NOT NEEDED
C
C
      SUBROUTINE PROBE(XP,YP,M,X,Y,DX,DY,D,I,DIST,XX,YY) .
      DIMENSION X(1),Y(1),DX(1),DY(1),D(1)
      DATA RAD/0.017453292519943296/,ILAST/1/
C INITIALIZE
      I    = ILAST
      TEST = DX(I)*(XP-X(I))+DY(I)*(YP-Y(I))
      TSAV = SIGN(1.0,TEST)
      GO TO 15
C
C START SEARCH
C
    7 I = I + 1
      IF(I.LE.M)GO TO 10
      IF(TSAV.LT.0.0)GO TO 20
      I = M
      GO TO 25
   10 TEST = DX(I)*(XP-X(I))+DY(I)*(YP-Y(I))
      IF(TEST*TSAV.LE.0.0)GO TO 25
   15 IF(TEST)20,25,7
   20 I = I - 1
      IF(I.GE.1)GO TO 10
      IF(TSAV.GT.0.0)GO TO 7
      I = 1
C
C FINISH SEARCH
   25 IF((TEST.LT.0.0).AND.(I.GT.1))I=I-1
      ILAST = I
C FINISH OF DETERMINATION OF I
C
C
C
```

FIGURE 51. PATH GENERATING ROUTINE
(Continued)

```
C
CALCULATE DISTANCE
        ZDN  = -DY(I)*(XP-X(I))+DX(I)*(YP-Y(I))
        CONS =  D(I)*RAD*0.005
        ZDZ  =  ((XP-X(I))**2+(YP-Y(I))**2)*CONS
        DIST =  (ZDN-ZDZ)/(0.5+SQRT(0.25-CONS*(ZDN-ZDZ)))
C
CALCULATE POSITION OF CLOSEST APPROACH POINT ON ARC
C THE FOLLOWING CODE MAY BE DELETED AND THE REFERENCES TO XX AND YY TAKEN
C OUT OF THE CALL IF THE POINT OF CLOSEST APPROACH ON THE ARC IS NOT NEEDED
C
        DEN = 1.0-2.0*DIST*CONS
C
        IF(DEN.GT.0.0)GO TO 30
        WRITE(1,26)I,XP,YP,DIST,DEN
   26   FORMAT(' SUBROUTINE PROBE HAS NEGATIVE OR ZERO DENOMINATOR'/
     1  ' IN POSITION  FORMULA; IMPLIES POINT NOT IN SECTOR'/I6,4F10.4)
        STOP PROBE
C THIS STOP SHOULD NEVER OCCUR IN NORMAL USAGE
C
   30   XX = (XP-X(I)+DIST*DY(I))/DEN + X(I)
        YY = (YP-Y(I)-DIST*DX(I))/DEN + Y(I)
   35   RETURN
        END
C
C
C************
C       IF TANGENT VECTOR IS NOT AVAILABLE IT MAY BE REPLACED BY
C               DX = X(I+1)-X(I)   , DY = Y(I+1)-Y(I)   ,I < M
C               DX = X(M)  -X(M-1), DY = Y(M)  -Y(M-1),I = M
C
C               USE DX FOR DX(I) AND DY FOR DY(I) IN CALCULATION OF TEST
C
C       RETURN CAN BE PUT AT END OF DETERMINATION OF I AND THE
C       DISTANCE AND CALCULATION OF XX,YY DONE BY ANOTHER ROUTINE.
C       (FORMULAS FOR DIST, XX AND YY ARE ONLY VALID FOR CIRCULAR ARCS
C        OR STRAIGHT LINES)
```

FIGURE 51. PATH GENERATING ROUTINE
(Continued)

```
C PATHG.FOR  F12          30 DECEMBER 1980        J T FLECK
C   PATH GENERATOR, SUBROUTINE PATHG    HVOSM RD-2
C   INPUTS
C       NPTS           NUMBER OF DESIRED POINTS ( > 1)
C       DELL           SPACING BETWEEN POINTS
C        X(1), Y(1)    INITIAL POSITION SET BY CALLING ROUTINE
C       DX(1),DY(1)    INITIAL TANGENT  SET BY CALLING ROUTINE
C       D(I)           DEGREE OF CURVATURE, I = 1 TO NPTS
C                      D(I) > 0  TURN TO RIGHT
C                      D(I) = 0  STRAIGHT
C                      D(I) < 0  TURN TO LEFT
C             NOTE:    RADIUS OF CURVATURE IS DEFINED AS
C                      EQUAL TO  (180/PI)*(100/D) = (5729.6/D)
C                      (D HAS DIMENSION OF DEGREES PER 100 UNITS OF DELL)
C
C   OUTPUTS            I = 1 TO NPTS
C         X(I), Y(I)   COORDINATES OF POINTS
C        DX(I),DY(I)   TANGENT VECTOR (DIRECTION OF PATH AT X,Y)
C
C   NOTE:  ROUTINE PRODUCES SMOOTH CURVE SUCH THAT TANGENTS ARE CONTINUOUS
C
      SUBROUTINE PATHG(NPTS,DELL,X,Y,D,DX,DY)
      DIMENSION X(1),Y(1),DX(1),DY(1),D(1)
      DATA RAD/0.01745329251994329 6/
C INITIALIZE
      CONS  = DELL*RAD/200.0
C*
      DXX   = DELL*DX(1)
      DYY   = DELL*DY(1)
C*
      DS1   = 0.0
      DC1   = 1.0
C START LOOP
      DO 20 I = 2, NPTS
COMPUTE SINE AND COSINE OF HALF SECTOR ANGLE
      DS2   = CONS*D(I-1)
      DC2   = SQRT((1.0-DS2)*(1.0+DS2))
C**
COMPUTE SINE AND COSINE OF SECTOR ANGLE
      SP    = 2.0*DS2*DC2
      CP    = 1.0 - 2.0*DS2**2
C UPDATE TANGENT VECTOR
      DX(I) =  CP*DX(I-1) - SP*DY(I-1)
      DY(I) =  SP*DX(I-1) + CP*DY(I-1)
C**
COMPUTE SINE AND COSINE OF AVERAGE SECTOR ANGLE
      SP    =     DS1*DC2 + DC1*DS2
      CP    =     DC1*DC2 - DS1*DS2
COMPUTE NEW INCREMENTS
      DXS   =  DXX
      DXX   =  DXS*CP  - DYY*SP
      DYY   =  DXS*SP  + DYY*CP
C UPDATE POSITION
      X(I)  =  X(I-1)  + DXX
      Y(I)  =  Y(I-1)  + DYY
C SAVE SINE AND COSINE OF HALF SECTOR ANGLE FOR NEXT I
      DS1   =  DS2
   20 DC1   =  DC2
      RETURN
      END
```

FIGURE 51. PATH GENERATING ROUTINE (Continued)

Neuro-Muscular Filter.--The "neuro-muscular" filter from the HVOSM-Vehicle Dynamics Version (Ref. (47), Vol. 3, p. 166-168) was incorporated into the HVOSM Roadside Design version. The filter structure corresponds to the first-order effects of the neurological and muscular systems of a human driver. For the curve study, the following inputs were used for the filter for all runs:

TIL	Time lag of filter	0.05	seconds
TI	Time lead of filter	0.00905	seconds
TAUF	Time delay of filter	0.0	seconds

The related revisions to the Driver model were incorporated into the FHWA distributed Roadside Design version of the HVOSM. However, the revised path-following algorithm was found to produce sustained oscillations about a specified path under some operating conditions. Since the extent of oscillation is dependent on the guidance system parameters as well as the vehicle speed and path curvature, it is possible to obtain peak values of transient response predictions that reflect an artifact of the guidance system rather than a real effect of the highway geometrics under investigation. For example, in Reference (49), comparisons are made between peak transient and steady-state response values which are believed to be more reflective of effects of the guidance system than of the simulated roadway geometrics. Therefore, the following additional modifications were added to the Driver model:

(1) Damping
A damping term (QGAIN) was added to limit the extent of steering activity. Initial runs utilizing the damping term exhibited a reduction in the steering activity as expected. The value used in the curve study was QGAIN (rad-sec/m) = PGAIN/10, where PGAIN is the steering velocity term described below.

(2) Steer Velocity

In addition to the damping term, an adjustable limit on the steering angle velocity (PGAIN) was incorporated in the path-follower algorithm, enabling the user to limit the maximum instantaneous front wheel steer velocity to a selected value. The value used in the curve study was PGAIN (rad/sec) = 1/Probe Length.

(3) Steer Initialization
For runs such as those being performed in relation to the cross-slope break study, the starting point must be relatively close to the cross-slope break to achieve an economical use of computer time. Thus, the input of an initial steer angle to approximate steady-state steer was

required. Previously, the path-follower algorithm was initialized to a steer angle of 0.0 degrees, regardless of the input value for the initial steer angle. Corresponding revisions were made to Subroutine DRIVER to enable input of an initial steer angle.

A revised listing of Subroutine DRIVER, including the cited modifications, is presented in Figure 52.

Terrain Table Generator

The version of the HVOSM maintained by FHWA has the capability of accepting a 3-dimensional definition of the highway surface. The manual generation of these inputs to the HVOSM, however, is time consuming, and the nature and number of geometric configurations to be studied required automation of the procedure.

The automation of the procedure to create terrain tables for the HVOSM consisted of providing an interface between standard roadway geometric descriptions and inputs to the HVOSM. A description of the required inputs to the TTG are as follows:

Centerline Descriptors.--The basic input to the TTG for the generation of centerline points is the radius of curvature of the centerline as a function of distance along the curve. Transitions between descriptors are user controlled and may be spiral or constant. The TTG converts the centerline description into X,Y data pairs and calculates second-order polynomial coefficients for each segment between the data pairs.

Superelevation and/or Gradient Descriptors.--The inputs for the superelevation and gradient are rates as a function of distance along the curve. Transitions between rates are user-controlled and may be spiral or constant.

HVOSM Terrain Table Descriptors.--HVOSM accepts up to four constant increment terrain tables with up to 21 x 21 grid points each as input. Inputs for the TTG to create the HVOSM terrain tables include the definition of the location, size and number of grid points for up to four terrain tables to be created by the TTG.

```
C SUBROUTINE DRIVER FOR HVOSM RD-2
C
      SUBROUTINE DRIVER(PSI,DPSI,JJ,IFLAG,A,B,AMTX,OMGPS)
      DIMENSION AMTX(3,3),PPD(50),TPD(50)
      COMMON/PATHD/IPATH,KLI,DI(10),RLI(10),NPTS,XINIT,YINIT,
     1       PSA,DELL,X(100),Y(100),DX(100),DY(100),D(100)
      COMMON/WAGON/IWAGN,TPRB,DPRB,PLGTH,PMIN,PMAX,PGAIN,QGAIN,PSIFD
      COMMON/FILT/ IFILT,TIL ,TI  ,TMT  ,TAUF
      COMMON/INTG/ NEQ  ,T   ,DT  ,VAR(50),DER(50)
      COMMON/ACC/CMFCG,CMFA1,CMFA2
      DATA NPDMAX/50/,NPD/0/,DPSL/0.0/,N/0/
      JJ  = 0
      IF(IWAGN.EQ.0)GO TO 90
      JJ  = 1
      PSIA = PSI
      DTP  = DPRB
      DPS  = 0.0
      DPSI = 0.0
      IF(IFLAG.EQ.0)GO TO 90
      IF(TPRB.GT.T + 0.1*DT)GO TO 10
C  COMPUTE NEW CHANGE IN STEER ANGLE
      TPRB  = TPRB + DPRB
      XP = VAR(18) + AMTX(1,1)*PLGTH
      YP = VAR(19) + AMTX(2,1)*PLGTH
      CALL PROBE(XP,YP,NPTS,X,Y,DX,DY,D,IPRB,DIST,XX,YY)
C  SELECTED POINT INDEX IPRB AND LOCATION OF CLOSEST POINT ON PATH XX,YY
C  ARE NOT CURRENTLY USED
      IF(DIST.EQ.0.0)GO TO 8
      SGND=DIST/ABS(DIST)
      IF(T.NE.TPRB) DDIST = (DIST-DISTA)/DPRB
    9 IF(ABS(DIST).GT.PMIN)DPS = -PGAIN*(ABS(DIST)-PMIN)*SGND
     1                            -QGAIN*DDIST
    8 IF(ABS(DIST).LE.PMIN) DPS= -QGAIN*DDIST
      IF(IFILT.EQ.0)GO TO 55
      IF(NPD.EQ.NPDMAX)GO TO 10
      NPD = NPD + 1
      PPD(NPD) = DPS - PSIA
      TPD(NPD) = T + TAUF
   10 IF(IFILT.EQ.0)GO TO 55
C
C  FILTER
C
      IF(NPD.EQ.NPDMAX) GO TO 10
      TPDTMP = TPD(N)
      DO 20 NN = 1,NPD
      N = NPD + 1 - NN
   20 IF(T.GE.TPD(N))GO TO 30
      GO TO 99
   30 IF(TPDTMP.LT.TPD(N)) DPSL = 0.0
      DPSI = PPD(N)*TMT*EXP(-(T - TPD(N))/TIL)/TIL
      DPSN = PPD(N) - TIL*DPSI
      DTP  = 0.0
      DPS  = DPSN - DPSL
      DPSL = DPSN
      IF(NPD.EQ.1)GO TO 50
C
C
```

FIGURE 52. SUBROUTINE DRIVER

```
 35   L    = 1
      DO 40 NN = N,NPD
      PPD(L) = PPD(NN)
      TPD(L) = TPD(NN)
   40 L = L + 1
      NPD = L - 1
C
   50 PSI = PSIA + DPS
      GO TO 58
   55 PSI = DPS
   58 CONTINUE
C CHECK PREVIOUS TIME INTERVAL COMFORT FACTOR (SEE SUBROUTINE OUTPUT)
C      IF GREATER THAN PMAX ALLOW ONLY REDUCTION IN STEER ANGLE
      IF((PMAX.GT.0.0).AND.(ABS(CMFA1).LT.PMAX))GO TO 60
      IF(ABS(PSI).GT.ABS(PSIA)) PSI=PSIA
   60  CONTINUE
C CHECK MAX STEER ANGLE
      IF((OMGPS.GT.0.0).AND.(ABS(PSI) .GT.  OMGPS))
     1    PSI = SIGN(OMGPS,PSI)
      IF(DTP.NE.0.0)DPSI = (PSI-PSIA)/DTP
C***   1/16/81   MCI   ********************************************
      DPSD = DPS*57.2958
      PSIAD = PSIA*57.2958
      PSID = PSI*57.2958
      DELPSI = PSID- PSIAD
      XPFT = XP/12.0
      YPFT = YP/12.0
      XXFT = XX/12.0
      YYFT = YY/12.0
C     IF(FKD.EQ.1.0) GO TO 90
      IF(KPAGE.LE.50.AND.T.NE.0.0000) GO TO 110
      WRITE(50,100)
  100 FORMAT(
     A1H1,33X,37HPROBE COORDINATES     PATH COORDINATES,5X,3HPSI,6X,
     B3HDPS,6X,4HPSIA,2X,7HDPSI    ,2X,7HDPSN    ,5HIFLAG,2X,4HIPRB/
     C31H  TIME      DELTA PSIF     ERROR ,6X,1HX,9X,1HY,10X,1HX,8X,1HY/
     D31H  (SEC)        (DEG)          (IN)   ,4X,4H(FT),6X,4H(FT),7X,
     E4H(FT),5X,4H(FT)//)
      KPAGE = 0
  110 WRITE(50,120) T,DELPSI,DIST,XPFT,YPFT,XXFT,YYFT,PSID,DPSD,
     A                 PSIAD,DPSI,DPSN,IFLAG,IPRB
  120 FORMAT(1H ,F7.3,2(4X,F7.3),2(3X,F7.1),2X,2(2X,F7.1),3(2X,F7.4),
     A           2X,F7.5,2X,F7.5,2X,I3,2X,I2)
      KPAGE = KPAGE + 1
   90 RETURN
C******************************************************************
      END
C******************************************************************
```

FIGURE 52. SUBROUTINE DRIVER (Continued)

The TTG calculates the elevation for each terrain table grid point by determining the perpendicular distance from the grid point to the centerline and using that in combination with the superelevation and gradient. The TTG then creates HVOSM card inputs for HVOSM which may be inserted directly into the main HVOSM data deck.

Typical inputs for the TTG are included in Figure 53. The outputs from the TTG consist primarily of either a card or disk data deck for use with HVOSM. Additional diagnostic dumps may also be output to insure the accuracy of the results.

A typical batch job for the TTG costs approximately $1.00 to $5.00, dependent on table size, extent of dumps, etc. The cost compares favorably with the hours of manual labor required to create a table manually and indicates that the TTG can provide a useful interface between standard geometric descriptors and HVOSM inputs.

```
HVOSM PRE-PROCESSING PROGRAM-TERRAIN TABLE GENERATOR

CONTRACT NO.DOT-FH-11-9575,PROGRAMMER-MCHENRY CONSULTANTS, INC.,CARY,N.C.

210 M RADIUS,10% SE,5% GRADE,80 M RUNOFF,20/80% DIST                              0 100
30.      10.      90.      0.0      0.0      1000.    -6.      1000.    6.0       0 101
1        10       02       00       00       00       0.00                        0 102
CENTERLINE DESCRIPTORS                                                            0 200
0.0      0.0      0.0                                                             0 201
60.      8.27     0.0                                                             1 201
1000.    8.27     0.0                                                             2 201
SUPERELEVATION                                                                    0 400
0.0      -.01     0.0                                                             0 401
10.      -.01     1.0                                                             1 401
270.     0.10     0.0                                                             2 401
1000.    0.10     0.0                                                             3 401
GRADIENT                                                                          0 500
0.0      -.05     0.0                                                             0 501
500.     -.05     0.0                                                             1 501
1000.    -.05     0.0                                                             2 501
-50.     50.      21.      0.0      100.     21.                                  0 601
-50.     50.      21.      100.     200.     21.0                                 1 601
-20.     120.     15.      190.     390.     21.0                                 2 601
20.      200.     18.      380.     500.     13.0                                 3 601
                                                                                  09999
```

FIGURE 53. TYPICAL TERRAIN TABLE INPUTS FOR HVOSM CURVE STUDY

APPENDIX E

PROGRAMMING DEVELOPED FOR ANALYSIS
OF VEHICLE PATH DATA

Program PATH was written to calculate instantaneous measures of vehicle path characteristics from data collected in the vehicle traversal studies. PATH uses (1) surveyed coordinates of centerline points of the curve, which are stored in a curve data file named CODAF: and (2) radial offsets measured of the vehicle at each point, which are stored in a data file named VODAF. The program performs the necessary geometric calculations to establish vehicle coordinates at each point, and calculates an appropriate curve radius for the vehicle path at each point. Using film speed and frame count as the vehicle passes each point, the vehicle speed is also computed at each point. Based on the above calculated items, a generated friction factor is calculated for the vehicle at each point.

The various input data and calculated values are output to the printer and saved in an output file named CCALC for further processing.

The following table describes the functions performed by various blocks of code. A program listing, input and output file formats and an example of printed output follow.

Program Lines	Functional Description
1 - 9	Program Description
10	Necessary to avoid subroutines upon execution
100 - 362	Routines entered through GOSUB statements throughout program including:
100 - 142	Routine to change numeric variable (SI) to string variable (RO) of specified number of places (F) used for formatting output
150 - 160	Routine to print page headings
200 - 216	Routine to print table headings
250 - 278	Routine to format and print table entries
300 - 346	Routine which accepts coordinates of two points (N1, E1, N2, E2) and calculates distance (L) and azimuth (AZ) between the points

Program Lines	Functional Description
350 - 362	Routine to determine the internal (smaller) angle (RO) between two azimuths (A1, A2)
500 - 1420	Program mainline including:
500 - 570	Establishes constants and dimensioned variables
575 - 615	Inputs file names for run
620 - 685	Reads curve data from file "CODAF"
690 - 840	Calculates slope of line perpendicular to each point on curve to be used later in determining coordinates of vehicle path at each point.
690 - 810	Determine points on tangent section of points in curve file. The PC is included only if less than two tangent points exists. An average slope is calculated and its perpendicular assigned to all curve points on the tangent.
815 - 840	Calculates perpendicular slope for points on curved section based on coordinates of the point on the curve and coordinates of the center of the curve approximated as a separate step and stored in the curve data file CODAF.
845 - 1400	Performs calculations and output for each point passed by an observed vehicle. This section is separated until all vehicles in a vehicle file for a given curve have been completed.
845 - 945	Reads data for an observed vehicle from the VODAF data file.
950 - 955	Prints page heading
975 - 1025	Calculates coordinates of vehicle path at each point using perpendicular slope calculated above (lines 690 - 840) and offset distance. Offset distance is calculated based on offset units stored for vehicle at point and equation of the form Offset = C_1 (offset units) + C_2. C_1 and C_2 are constants read from the CODAF file. If a data item is missing coordinates are set to 99999.

Program Lines	Functional Description
1030 - 1185	Calculates instantaneous radius of curvature of vehicle path through N points for point 3 through N-2 according to methodology shown in Figure 20. Note that curvature is approximated over a distance represented by 5 points. The following substeps are included: (References are to Figure 21)
1100 - 1115	Finds beginning azimuth (AZ(N - 2))
1120 - 1135	Finds ending azimuth (AZ(N + 2))
1140 - 1150	Finds chord distance (LC_n)
1170 - 1175	Finds curve delta (ΔN)
1180 - 1185	Calculates radius R_n
1220 - 1235	Calculates vehicle speed using film speed, difference in frame count and conversion to miles per hour
1240 - 1250	Calculates generated friction factor using formula: friction factor = $(\text{Speed})^2/15$ Radius) - superelevation
1300 - 1380	Outputs calculated volumes to CCALC file
1385 - 1395	Prints output table
1400	Returns for next vehicle
1405 - 1420	Ends program

CODAF FILE FORMAT

CODAF FILE stored as sequential file; file per curve site containing description of curve.

Item #	Description
1	Site Number
2	State Code
3	Degree of Curve Code
4	Roadway Width Code
5	Approach Code
6	Transition Code
7	Curve Direction
8	Number of Points on Curve
9	PC Point Number
10	Curve Center North Coordinate
11	Curve Center East Coordinate
12	Point 1 - Point #
13	Point 1 - North Coordinate
14	Point 1 - East Coordinate
15	Point 1 - Elevation
16	Point 1 - Superelevation
17	Point 1 - Offset Conversion Constant C_1
18	Point 1 - Offset Conversion Constant C_2
5+(N*7)	Point N - Point #
6+(N*7)	Point N - North Coordinate
7+(N*7)	Point N - East Coordinate
8+(N*7)	Point N - Elevation
9+(N*7)	Point N - Superelevation
10+(N*7)	Point N - Offset Conversion Constant C_1
11+(N*7)	Point N - Offset Conversion Constant C_2

VODAF FILE FORMAT

VODAF file stored as random access file; one file per site, one record per observed vehicle plus one trailing record.

Item #	Description
1	Film Speed
2	Vehicle Type
3	Vehicle Identification
4	Number of Points in File
5	Point 1 - Curve Point Number
6	Point 1 - Frame Count
7	Point 1 - Offset Units
2+(3*N)	Point N - Curve Point Number
2+(3*N)	Point N - Frame Count
2+(3*N)	Point N - Offset Units

Last Record in File

Item 1   "stop"

Remaining Items ----- "0"

CCALC FILE

CCALC file stored as random access file, one file per location, one record per observed vehicle.

Item #	Description
1	File Speed
2	Vehicle Type
3	Vehicle Identification
4	Number of Points in File
5	Point 1 - Curve Point Number
6	Point 1 - Vehicle North Coordinate
7	Point 1 - Vehicle East Coordinate
8	Point 1 - Vehicle Offset
9	Point 1 - Long Chord Distance N-2 to N+2
10	Point 1 - Curve Delta
11	Point 1 - Curve Radius
12	Point 1 - Vehicle Speed
13	Point 1 - Friction Factor
-4+(N*9)	Point N - Curve Point Number
-3+(N*9)	Point N - Vehicle North Coordinate
-2+(N*9)	Point N - Vehicle East Coordinate
-1+(N*9)	Point N - Vehicle Offset
0+(N*9)	Point N - Long Chord Distance N-2 to N+2
1+(N*9)	Point N - Curve Delta
2+(N*9)	Point N - Curve Radius
3+(N*9)	Point N - Vehicle Speed
4+(N*9)	Point N - Friction Factor

```
1  REM  PROGRAM 'PATH'
2  REM  PROGRAM REQUIRES
3  REM  CURVE DATA FROM 'CODAF' FILE
4  REM  VEHICLE DATA FROM 'VODAF' FILE
5  REM  PROGRAM CALCULATES VEHICLE COORDINATES, OFFSET FROM CENTERL
INE, CURVATURE PROPERTIES OF VEHICLE PATH
6  REM  VEHICLE SPEED AND FRICTION FACTOR
7  REM  FOR POINTS ON CURVE
8  REM  PROGRAM STOREES SELECTED DATA IN FILE 'CCALC' FOR FURTHER U
SE
9  REM
10  GOTO 505
100  REM
102  REM   SUBROUTINE TO TURN NUMBER INTO OUTPUT $TRING OF SPECIFIE
D FIELD LENGTH
104  REM
106 FD = ((F + .02) -  INT (F)) * 10:FD =  INT (FD) + 1
108 FI =  INT (F) - FD:SI = SI + ((5 * (10 ^  - FD)) *  SGN (SI))
110 FA =  INT (SI): IF  INT ( ABS (SI)) (  )  ABS (FA) THEN FA = FA
 + 1
112 FS$ =  STR$ (FA): IF FA = 0 AND  SGN (SI) ( 0 THEN FS$ = "-" +
FS$
114 FL =  LEN (FS$)
116  IF FL ) FI THEN  GOTO 138
118  IF  SGN (FA) ( 0 THEN FA = FA + 1
120 FB =  ABS (SI) -  ABS (FA) + 100
122 FF$ =  STR$ (FB)
124 FT$ =  MID$ (FF$,4,FD)
126  IF FL = FI THEN  GOTO 140
128 Q = FI - FL
130  FOR QQ = 1 TO Q
132 FS$ = " " + FS$
134  NEXT QQ
136  GOTO 140
138 FS$ = "*":FT$ = FS$
140 RO$ = FS$ + FT$
142  RETURN
150  REM
152  REM  SUBROUTINE FOR PAGE HEADINGS
154  REM
156  PRINT : PRINT : PRINT : PRINT  TAB( 10)"140-1  FHWA" TAB( 65)"
SITE" TAB( 35)KJ$
158  PRINT  TAB( 10)"VEHICLE OPERATING CHARACTERISTICS"
160  PRINT  TAB( 10)"RUN DATE "DA$ TAB( 65)"VEHICLE" TAB( 33)R
162  PRINT : PRINT : PRINT
164  PRINT  TAB( 10)"-------------------------------------------
------------------"
166  PRINT  TAB( 10)"DATA ERRORS"
168  RETURN
200  REM
202  REM  SUBROUTINE FOR TABLE HEADINGS
204  REM
206  PRINT  TAB( 10)"-------------------------------------------
-------------------": PRINT : PRINT
208  PRINT  TAB( 18)"CENTERLINE" TAB( 30)"VEHICLE" TAB( 39)"EFFECTI
```

FIGURE 54. PROGRAM PATH

```
VE" TAB( 11)"SUPER-"
210  PRINT  TAB( 10)"POINT" TAB( 20)"OFFSET" TAB( 31)"SPEED" TAB( 4
0)"VEHICLE" TAB( 11)"ELEV" TAB( 20)"FRICTION"
212  PRINT  TAB( 10)"NUMBER" TAB( 20)"(FEET)" TAB( 31)"(MPH)" TAB(
39)"RADIUS-FT" TAB( 10)"AT POINT" TAB( 21)"FACTOR"
214  PRINT  TAB( 10)"------" TAB( 18)"----------" TAB( 30)"-------"
 TAB( 39)"---------" TAB( 10)"--------" TAB( 20)"--------": PRINT
216  RETURN
250  REM
252  REM  SUBROUTINE TO OUTPUT TABLE VALUES
254  REM
256  FOR N = 1 TO IV(4)
258 NS = 2 + (3 * N):FO$(1) =  STR$ (IV(NS)):NS = (IV(NS) * 7) + 9
260 F = 6.3:SI = VD(N): GOSUB 102:FO$(2) = RO$
262 F = 4.1:SI = CV(7,N): GOSUB 102:FO$(3) = RO$
264 F = 7.0:SI = CV(6,N): GOSUB 102:FO$(4) = RO$
266 F = 6.3:SI = IC(NS): GOSUB 102:FO$(5) = RO$
268 F = 6.3:SI = CV(8,N): GOSUB 102:FO$(6) = RO$
270 NS = 2 + (3 * N)
272  IF IV(NS) = IC(9) THEN FO$(1) = FO$(1) + " PI"
274  PRINT  TAB( 12)FO$(1) TAB( 20)FO$(2) TAB( 33)FO$(3) TAB( 40)FO
$(4) TAB( 11)FO$(5) TAB( 21)FO$(6): PRINT
276  NEXT N
278  RETURN
300  REM
302  REM   SUBROUTINE TO PERFORM INVERSE CALCULATIONS BETWEEN TWO P
OINTS
304  REM
306 DN = N1 - N2
308 DE = E1 - E2
310  IF DN = 0 GOTO 338
312 TNBEAR =  ABS (DE / DN)
314 BEAR = ( ATN (TNBEAR)) * (180 / PI)
316  IF DN < 0 GOTO 324
318  IF DE <  = 0 GOTO 330
320  REM   'N- 'E+
322 AZ = BEAR + 180: GOTO 344
324  IF DE <  = 0 GOTO 334
326  REM   'N- 'E+
328 AZ = 360 - BEAR: GOTO 344
330  REM   'N+ 'E- OR 0
332 AZ = 180 - BEAR: GOTO 344
334  REM   'N- 'E- OR 0
336 AZ = BEAR: GOTO 344
338  IF DE > 0 THEN AZ = 270
340  IF DE < 0 THEN AZ = 90
342  REM   LENGTH CALC
344 L =  SQR (DE ^ 2 + DN ^ 2)
346  RETURN
350  REM
352  REM   SUBROUTINE TO RETURN INTERNAL ANGLE FROM TWO AZIMUTHS
354  REM
356  IF A1 > 180 THEN A1 = A1 - 180
358  IF A2 > 180 THEN A2 = A2 - 180
360 RO =  ABS (A1 - A2): IF RO > 90 THEN RO = 180 - RO
362  RETURN
500  REM
```

FIGURE 54. PROGRAM PATH (Continued)

```
505  REM  START MAIN PROGRAM
510  REM
515 D$ =  CHR$ (4)
520 PI = 3.1415927
525 RC = 3.1415927 / 180
530  DEF  FN RD(W) = W * RC
535  POKE 33,40
540  DIM IC(150),IV(80),SL(20),VE(20),VN(20),VD(20),CV(8,20),FO$(6)
545  REM   IC(150) - CURVE DATA
550  REM   IV(80) - VEHICLE DATA
555  REM   SL(20) - SLOPE PERPENDICULAR TO CENTERLINE AT EACH CURVE
 POINT
560  REM  VE,VN,VD(20) - VEHICLE COORDINATES AND OFFSET FROM CENTER
LINE
565  REM   CV(8,20) - CALCULATED VALUES FOR A VEHCILE AT EACH POINT
 - SEE BELOW
570 R = 0
575  REM
580  REM  INPUT DATA
585  REM
590  HOME : INPUT "ENTER DATE ";DA$
595  PRINT D$"PR#0": PRINT "ENTER CODAF SUFFIX AS 99 TO EXIT PROGRA
M"
600  INPUT "CODAF FILE SUFFIX ";KJ$
605  IF  VAL (KJ$) = 99 THEN  GOTO 1410
610  INPUT "VODAF FILE SUFFIX ";JK$
615  INPUT "CCALC FILE SUFFIX ";KK$
620  REM
625  REM  READ CURVE DATA
630  REM
635  PRINT D$"OPEN CODAF"KJ$",D1"
640  PRINT D$"READ CODAF"KJ$
645  FOR J = 1 TO 8
650  INPUT IC(J)
655  NEXT J
660  PRINT D$"READ CODAF"KJ$
665 SB = 11 + (IC(8) * 7)
670  FOR J = 9 TO SB
675  INPUT IC(J)
680  NEXT J
685  PRINT D$"CLOSE CODAF"KJ$
690  REM
695  REM  CALCULATE PERPENDICULAR SLOPES AT EACH POINT ON CURVE
700  REM   CALCULATE SLOPES ON TANGENT SECTION
705  REM
710  IF IC(9) = IC(12) THEN  GOTO 815: REM  IF NO TANGENT SECTION
715  REM  FIND SLOPE OF TANGENT SECTION AS AVERAGE OF SLOPES OF ADJ
ACIENT POINTS
720 PS = IC(12) - IC(9)
725  IF PS > 1 THEN PS = PS - 1
730 DE = 0:DN = 0:M = 0
735  FOR J = 1 TO PS
740 ES = (7 + (J * 7)):NS = (6 + (J * 7))
745 DE = IC(ES + 7) - IC(ES)
750 DN = IC(NS + 7) - IC(NS)
755 M = M + DN / DE
760  NEXT J
```

FIGURE 54. PROGRAM PATH (Continued)

```
765 M = M / PS
770  REM  PERPENDICULAR SLOPE
775 MP =  - 1 / M
780  REM
785  REM  ASSIGN SLOPE TO ALL TANGENT POINTS
790  REM
795  FOR J = 1 TO PS
800 SL(J) = MP
805  NEXT J
810  REM
815  REM  CALCULATE CIRCULAR SECTION SLOPES
820  REM
825  FOR J = PC TO IC(8)
830 ES = (7 + (J * 7)):NS = (6 + (J * 7))
835 SL(J) = (IC(10) - IC(NS)) / (IC(11) - IC(ES))
840  NEXT J
845  REM
850  REM  FOR EACH OBSERVED VEHICLE COUNTED BY 'R'
855  REM
860  REM  READ VEHICLE DATA
865  REM
870 R = R + 1
875  PRINT D$"OPEN VODAF"JK$",L250,D1"
880  PRINT D$"READ VODAF"JK$",R"R
885  INPUT R$
890  IF  ASC (R$) = 83 THEN  GOTO 595
895 IV(1) =  VAL (R$)
900  FOR J = 2 TO 4
905  REM
910  INPUT IV(J)
915  NEXT J
920 K = 4 + (IV(4) * 3)
925  FOR J = 5 TO K
930  REM
935  INPUT IV(J)
940  NEXT J
945  PRINT D$"CLOSE VODAF"JK$
950  PRINT D$"PR#1": HOME : PRINT  CHR$ (12)
955  GOSUB 152
960  REM
965  REM  PERFORM VEHICLE CALCULATIONS AT EACH POINT
970  REM
975  REM  CALCULATE COORDINATES OF VEHICLE AT EACH POINT USING PERP
INDICLUAR SLOPE AND OFFSET
980  REM
985  FOR N = 1 TO IV(4)
990 SO = 4 + (N * 3):RN = IV(SO - 2):SC = 11 + (RN * 7)
995  IF IV(SO) = 99999 THEN VD(N) = 99999:VE(N) = 99999:VN(N) = 999
99: GOTO 1025
1000 VD(N) = (IC(SC - 1) * IV(SO)) + IC(SC)
1005 DX =  SQR (VD(N) ^ 2 / (1 + (SL(RN) ^ 2))) *  SGN (IV(SO)):DY
= SL(RN) * DX
1010 ES = (RN * 7) + 7:NS = ES - 1
1015 VE(N) = IC(ES) + DX
1020 VN(N) = IC(NS) + DY
1025  NEXT N
1030  REM
```

FIGURE 54. PROGRAM PATH (Continued)

```
1035 REM    CALCULATE INSTANTANEOUS RADIUS OF CURVATURE O VEHICLE
AT EACH POINT
1040 REM
1045 REM   SAVE INTERMEDIATE RESULTS AS SHOWN
1050 REM   USE ARRAY CV(8- DATA,20- POINT) I, N
1055 REM    CV(1) = BEGINNING ASIMUTH
1060 REM     CV(2) = ENDING ASIMUTH
1065 REM    CV(3) = CHORD ASIMUTH
1070 REM    CV(4) = CHORD DISTANCE
1075 REM    CV(5) = CURVE DELTA
1080 REM    CV(6) = CURVE RADIUS
1085 REM
1090 FOR N = 3 TO IV(4) - 2
1095 IF VN(N) = 99999 OR VN(N - 2) = 99999 OR VN(N + 2) = 99999 TH
EN  GOTO 1260
1100 N1 = VN(N - 2):N2 = VN(N)
1105 E1 = VE(N - 2):E2 = VE(N)
1110  GOSUB 304
1115 CV(1,N) = AZ
1120 N1 = VN(N):N2 = VN(N + 2)
1125 E1 = VE(N):E2 = VE(N + 2)
1130  GOSUB 304
1135 CV(2,N) = AZ
1140 N1 = VN(N - 2):E1 = VE(N - 2):N2 = VN(N + 2):E2 = VE(N + 2)
1145  GOSUB 304
1150 CV(3,N) = AZ:CV(4,N) = L
1155  REM
1160  REM  FIND POINT DELTA
1165  REM
1170 A1 = CV(2,N):A2 = CV(1,N)
1175  GOSUB 352:CV(5,N) = RO * 2
1180 AZ =  FN RD(CV(5,N))
1185 CV(6,N) = (CV(4,N) / 2) /  SIN (AZ / 2)
1190  REM
1195  REM  CALCULATE VEHICLE DYNAMICS
1200  REM
1205  REM  CV(7) = SPEED
1210  REM  CV(8) = FRICTION FACTOR
1215  REM
1220 RN = 2 + (3 * N):RN = IV(RN)
1225 FS = 3 + (3 * N)
1230  IF IV(FS + 6) = 99999 OR IV(FS - 6) = 99999 THEN CV(7,N) = 99
999:CV(8,N) = 99999: PRINT "NO FRAMECOUNT FOR "JK$" "R" "RN:CV(7,N)
 = 99999:CV(8,N) = 99999: GOTO 1285
1235 CV(7,N) = (((IV(1) * CV(4,N)) / (IV(FS + 6) - IV(FS - 6))) * (
3600 / 5280))
1240 CV(8,N) = (CV(7,N) ^ 2) / (15 * CV(6,N))
1245 RN = IV(FS - 1):SS = 9 + (RN * 7)
1250 CV(8,N) = CV(8,N) - IC(SS)
1255  GOTO 1285
1260  FOR J = 1 TO 8
1265 CV(J,N) = 99999
1270  NEXT J
1275  PRINT "NO OFFSET FOR  "JK$"   "R"  "RN
1280  PRINT D$"PR#0"
1285  NEXT N
1290  REM
```

FIGURE 54. PROGRAM PATH (Continued)

```
1295  REM  END OF CALCULATIONS FOR CURRENT VEHICLE
1300  REM
1305  REM   OUTPUT CALCULATIONS FILE, 1 RECORD/VEHICLE,   1 FILE PE
R CURVE
1310  REM
1315  PRINT D$"PR#0"
1320  PRINT D$"OPEN CCALC"KK$",L1000,D2"
1325  PRINT D$"WRITE CCALC"KK$",R";R
1330  FOR P = 1 TO 4
1335  PRINT IV(P)
1340  NEXT P
1345  FOR P = 1 TO IV(4)
1350 RN = 2 + (P * 3):RN = IV(RN)
1355  PRINT RN: PRINT VN(P): PRINT VE(P): PRINT VD(P): REM  PRINT S
L(RN)
1360  FOR PP = 4 TO 8
1365  PRINT CV(PP,P)
1370  NEXT PP
1375  NEXT P
1380  PRINT D$"CLOSE CCALC"KK$
1385  PRINT D$"PR#1": GOSUB 202
1390  GOSUB 252
1395  PRINT D$"PR#0"
1400  GOTO 870
1405  PRINT D$"PR#1"
1410  PRINT : PRINT : PRINT : PRINT "PROCESSING COMPLETE - PATH
 V1.2"
1415  PRINT D$"PR#0"
1420  END
```

FIGURE 54. PROGRAM PATH (Continued)

APPENDIX F

ANALYSIS OF THE DYNAMIC EFFECTS OF VERTICAL IRREGULARITIES ON HIGHWAY CURVES

Derivation of Cornering Model

Referring to Figure 55, a_n is the lateral acceleration on the cornering vehicle expressed as:

$$\bar{a}_n = v^2/R$$

where a_n = lateral acceleration, ft/s^2 (m/s^2)
v = vehicle speed, ft/s (m/s)
R = vehicle path radius, ft (m)

Also shown in Figure 55 is the resultant of tire forces, P, the weight of the vehicle, W, the superelevation, (expressed as tan Θ), and the angle, α, the tangent of which represents the ratio of resultant lateral to resultant normal tire forces.

Analyzing the summation of horizontal and vertical forces on the vehicle yields the following:

$$[\Sigma F_n = m\bar{a}_n] \quad \text{or} \quad P \sin(\Theta + \alpha) = (W/g)\,(v^2/R)$$

$$[\Sigma F_y = 0] \quad \text{or} \quad P \cos(\Theta + \alpha) = W$$

where W = vehicle weight, lb (kg)
g = acceleration of gravity, ft/s^2 (m/s^2)

Dividing these two equations gives the following:

$$\tan(\Theta + \alpha) = v^2/gR$$

or $$(\tan \Theta + \tan \alpha)/(1 - \tan \Theta \tan \alpha) = v^2/gR$$

or $$(e + f)/(1 - ef) = v^2/gR$$

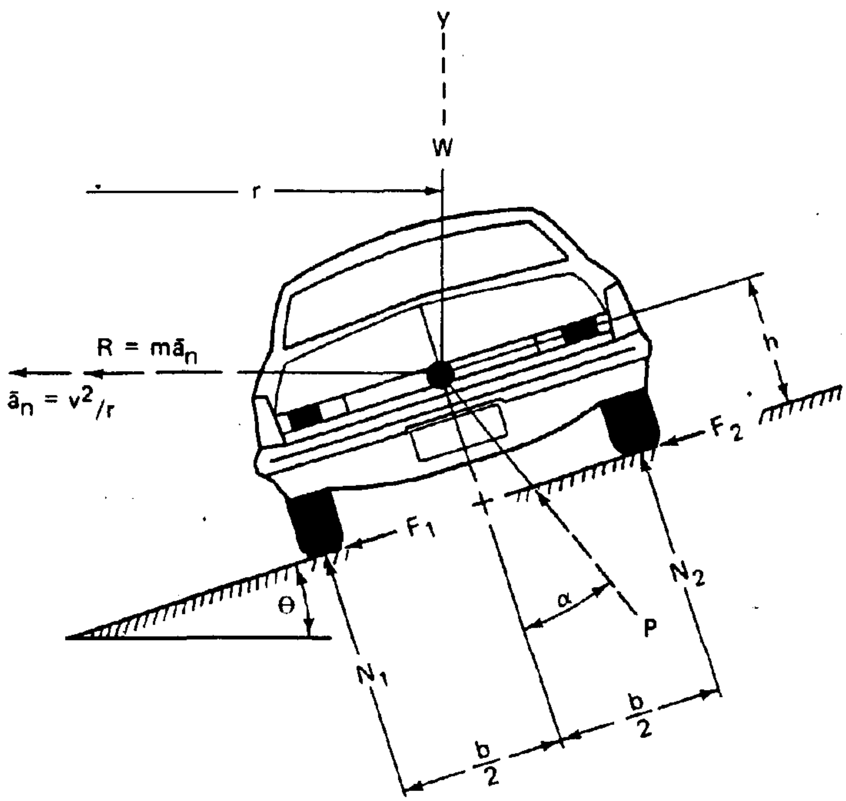


Figure 55. VEHICLE CORNERING RELATIONSHIPS

Substituting the value of 32.2 ft/s^2 (9.8 m/s^2) for the acceleration of gravity, and converting vehicle speed to V, in mph (km/h), yields

$$(e + f)/(1 - ef) = V^2/15R$$

$$1 \text{ mph} = 1.609 \text{ km/h}$$

Derivation of Cornering Model With Vertical Irregularity

This derivation is identical to the previous derivation, except the resultant of vertical forces includes a centripetal acceleration term associated with the vertical irregularity. The summation of horizontal and vertical force is:

$$[\Sigma F_n = ma_n] \quad \text{or} \quad P \sin(\Theta + \alpha) = Wv^2/gR_h$$

$$[\Sigma F_y = ma_y] \quad \text{or} \quad P \cos(\Theta + \alpha) = W \pm (Wv^2/gR_v)$$

where R_h = vehicle path radius, ft (m)
R_v = radius of vertical irregularity, ft (m)

Dividing these two equations gives the following:

$$\tan(\Theta + \alpha) = v^2/[gR_h(1 \pm (v^2/gR_v))]$$

$$\text{or} \quad (e + f)/(1 - ef) = v^2/[gR_h(1 \pm (v^2/gR_v))]$$

Substituting the value for the acceleration of gravity, and converting vehicle speed to V, in mph(km/h), yields:

$$(e + f)/(1 - ef) = V^2/[R_h(15 \pm (V^2/R_v))]$$

$$1 \text{ mph} = 1 \text{ km/h}$$

Note that when R_v is infinite (no irregularity) that the equation is the same as the basic cornering equation.

Derivation of Vertical Radius for Take-off

For any vehicle speed, the maximum vertical radius that will create vehicle take-off is derived by knowing that the vertical force on the vehicle is zero. Therefore,

$$W - (Wv_t^2/gR_t) = 0$$

or $$v_t^2 = gR_t$$

where v_t = take-off speed, ft/s (m/s)

R_t = vertical take-off radius, ft (m)

Substituting the value for the acceleration of gravity, and converting vehicle speed to V_t, in mph (km/h), yields:

$$V_t = 15 R_t$$

1 mph = 1.609 km/h

To express this equation in terms of the dimensions of a parabolic curve, the take-off radius can be expressed as:

$$R_t = \sqrt{100 L/A}$$

where L = length of parabolic curve, ft (m)

A = algebraic difference in grade, percent

Therefore, the take-off speed on a parabolic curve can be expressed as:

$$V_t = \sqrt{1500 L/A}$$

1 mph = 1.609 km/h

APPENDIX G

ANALYSIS OF CURVE RECONSTRUCTION AS A COUNTERMEASURE TO HIGH-ACCIDENT CURVE SITES

The following pages summarize cost-effectiveness analysis of a hypothetical reconstruction problem involving a sharp highway curve. The purpose of the exercise is to demonstrate the relative cost-effectiveness of programs involving complete reconstruction of sharp, high-accident curves.

The analysis uses a benefit/cost ratio format, with operational benefits associated with reduced vehicle operating costs and accident cost savings.

GEOMETRIC CONDITIONS

	Existing	Proposed
Degree of Curve	20°	6°
Length of Curve	0.05 mi	0.16 mi
Central Angle	50°	50°
Roadside Rating	50	25
Roadside Slope	2:1	6:1
Clear-zone Width	10 ft	30 ft
Coverage of Fixed Objects	50%	50%
Pavement Rating	20	50
Width of Shoulders	0 ft	4 ft

1 mi = 1.609 km

1 ft = 0.305 m

ACCIDENT CONDITIONS

	Existing	Proposed
Discriminant Score (Equation 9.1)	4.22	-0.30
P(H)	0.99	0.51
Accident Rate (per MVM)	4.35	1.15

$$\Delta \, Ra = 4.35 - 1.15 = 3.20 \text{ Accidents per MVM}$$

OPERATIONAL BENEFITS

Operational benefits consist of reductions in accidents and reductions in vehicle operating costs over the curve. The 1977 AASHTO Manual nomographs were used to calculate vehicle operating costs for existing and proposed conditions.

Vehicle Operating Costs* (dollars per 1000 vehicles)	Existing	Proposed
Tangent	$ 49.6	41.8
Transition	4.6	2.0
Curve	8.1	11.5
Travel Time	0.5	0.7
	$ 62.8	56.0
Vehicle Operating costs per 1000 ADT per segment per year	$22,922	$20,440
Accident Costs		
Accident Rate (per MVM)	4.35	1.15
Accidents per 1000 ADT per segment per year	0.98	0.26
Accident Cost per 1000 ADT per segment per year (at $14,700 per accident)	$14,406	$ 3,822

* 1975 Costs from Reference (40); nomograph figures 9, 13, 17 and 20 were used in analysis.

COSTS OF CONSTRUCTION

Initial Costs (Includes old pavement removal, earthwork, clearing and grubbing, tree/fixed object removal, new pavement, topsoil and landscaping, drainage, and engineering)	\$493,500
Costs at 10 years (Includes resurfacing of pavement)	\$ 87,500

Annualized costs of improvements =

493,500 [CRF @ 7% and 20 years]

+ 87,500 [PW @ 7% and 10 years][CRF @ 7% and 20 years]

= 46,389 + 4,178 = \$50,567

BENEFIT/COST ANALYSIS OF CURVE RECONSTRUCTION

At the initial year, per 1000 ADT, annual benefits are calculated as follows:

Benefits = Existing Costs - Proposed Costs

= (22,922 + 14,406) - (20,440 + 3,822)

= 37,328 - 24,262

= \$13,066 per 1000 ADT

Assuming 1.5% traffic growth annually for 20 years and applying an adjustment factor to produce equivalent uniform annual benefits, the annual benefit of curve reconstruction per 1000 ADT is equal to \$14,895. The annualized cost of construction is \$50,567. Therefore, the "breakeven" ADT is given by

(50,567/14,895)(1000 ADT)

or 3395 say 3400 ADT

EXISTING GEOMETRIC CONDITIONS

All Curves	High Roadside Hazard (RR=50)		Moderate Roadside Hazard (RR=35)	
Pavement Rating	Shoulder Width		Shoulder Width	
	Medium (3 ft)	Wide (8 ft)	Medium (3 ft)	Wide (8 ft)
Low (PR=20)	Repave and reduce roadside hazard to 25 2300 ADT	Repave and reduce roadside hazard to 35 2000 ADT - - - - - - - - Reduce roadside hazard to 25 1700 ADT	Repave and reduce roadside hazard to 25 Initially 4:1 2700 ADT - - - - - - - - 6:1 1800 ADT	Repave 2800 ADT - - - - - - - - - Reduce roadside hazard to 25 2400 ADT
Moderate (PR=35)	Reduce roadside hazard to 25 1600 ADT	Reduce roadside hazard to 25 1800 ADT	Reduce roadside hazard to 25 Initially 4:1 2300 ADT - - - - - - - - 6:1 700 ADT	$P(H) \le 0.80$
High (PR=50)	Reduce roadside hazard to 25 1700 ADT	Reduce roadside hazard to 35 1800 ADT*	$P(H) \le 0.80$	$P(H) \le 0.80$

1 ft = 0.305 m

* Short mild and moderate curves do not require improvement

FIGURE 56. BREAK-EVEN ADT's FOR COUNTERMEASURES TO HIGH-ACCIDENT RURAL HIGHWAY CURVES

EXISTING GEOMETRIC CONDITIONS

All Curves	High Roadside Hazard (RR=50)				Moderate Roadside Hazard (RR=35)			
Pavement Rating	Shoulder Width				Shoulder Width			
	Medium (3 ft)		Wide (8 ft)		Medium (3 ft)		Wide (8 ft)	
	Length (mi)	Break-even ADT	Length (mi)	Break-even ADT	Length (mi)	Break-even ADT	Length (mi)	Break-even ADT
Low (PR=20)	.13	500	.13	500	.13	500	.13	600
	.23	800	.23	700	.23	1000	.23	1000
	.33	1200	.33	1000	.33	1400	.33	1400
	.62	2300	.62	2000	.62	2700	.62	2800
	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			.13	300	.13	400	.13	500
			.23	600	.23	600	.23	800
			.33	800	.33	300	.33	1200
			.62	1700	.62	1800	.62	2400
	Length (mi)	Break-even ADT	Length (mi)	Break-even ADT	Length (mi)	Break-even ADT		
Moderate (PR=35)	.13	300	.13	400	.13	500		
	.23	600	.23	600	.23	800		
	.33	800	.33	900	.33	1200		
	.62	1600	.62	1800	.62	2300		
	- - -	- - -	- - -	- - -	- - -	- - -		
					.13	200		
					.23	300		
					.33	400		
					.62	700		
	Length (mi)	Break-even ADT	Length (mi)	Break-even ADT				
High (PR=50)	.13	400	.13	400			Entries above dashed line -- 4:1 side slope initially	
	.23	600	.23	600			- - - - - - - - - -	
	.33	900	.33	900			Entries below dashed line -- 6:1 side slope initially	
	.62	1700	.62	1800				

1 ft = 0.305 m
1 mi = 1.609 km

Note: "Length" refers to length of highway assumed to be improved to reduce P(H) to the desired level.

FIGURE 57. BREAK-EVEN ADT's FOR COUNTERMEASURES TO HIGH-ACCIDENT HIGHWAY CURVES, ASSUMING ONLY CURVE PLUS NOMINAL APPROACH LENGTHS REQUIRE IMPROVEMENT

APPENDIX H

STAKEOUT OF SPIRAL CURVES

In recent years, significant changes have taken place in the design, calculation and stakeout of spiral curves. This has been brought about largely through the use of electronic computers, special programming, and associated development work. Spirals can now be computed just as rapidly and directly as circular curves. The format for stakeout for spirals can be identical to that of circular curves, with computer-generated stakeout notes providing complete information for the survey crews.

The following example of a segment of alinement with circular and spiral curves (shown in Figures 58 and 59) illustrates the computer-produced stakeout notes. These indicate curve numbers 22, 23 and 24, complete with stationing, deflection angles, bearings and curve data. Curve No. 23 is a spiral, for which information is provided for the same stakeout method as for the circular curves.

Although not included here, additional tabulations for utilizing intermediate set-ups, stakeout in reverse direction, or any number of odd stations, can be part of the stakeout notes "package" for both circular curves and spirals.

Also shown in Figure 60 is another printout form which includes more closely spaced station points along with back-sight angles to set local tangents at each of the points. Then, turning right angles from the local tangents, radial lines can be set for whatever accurate offsets may be required, as on a bridge. Figure 61 is a diagram with nomenclature pertaining to this printout.

In short, notes precomputed in the office specifically for alinement stakeout greatly simplify the field work and permit spirals to be used as readily as circular curves.

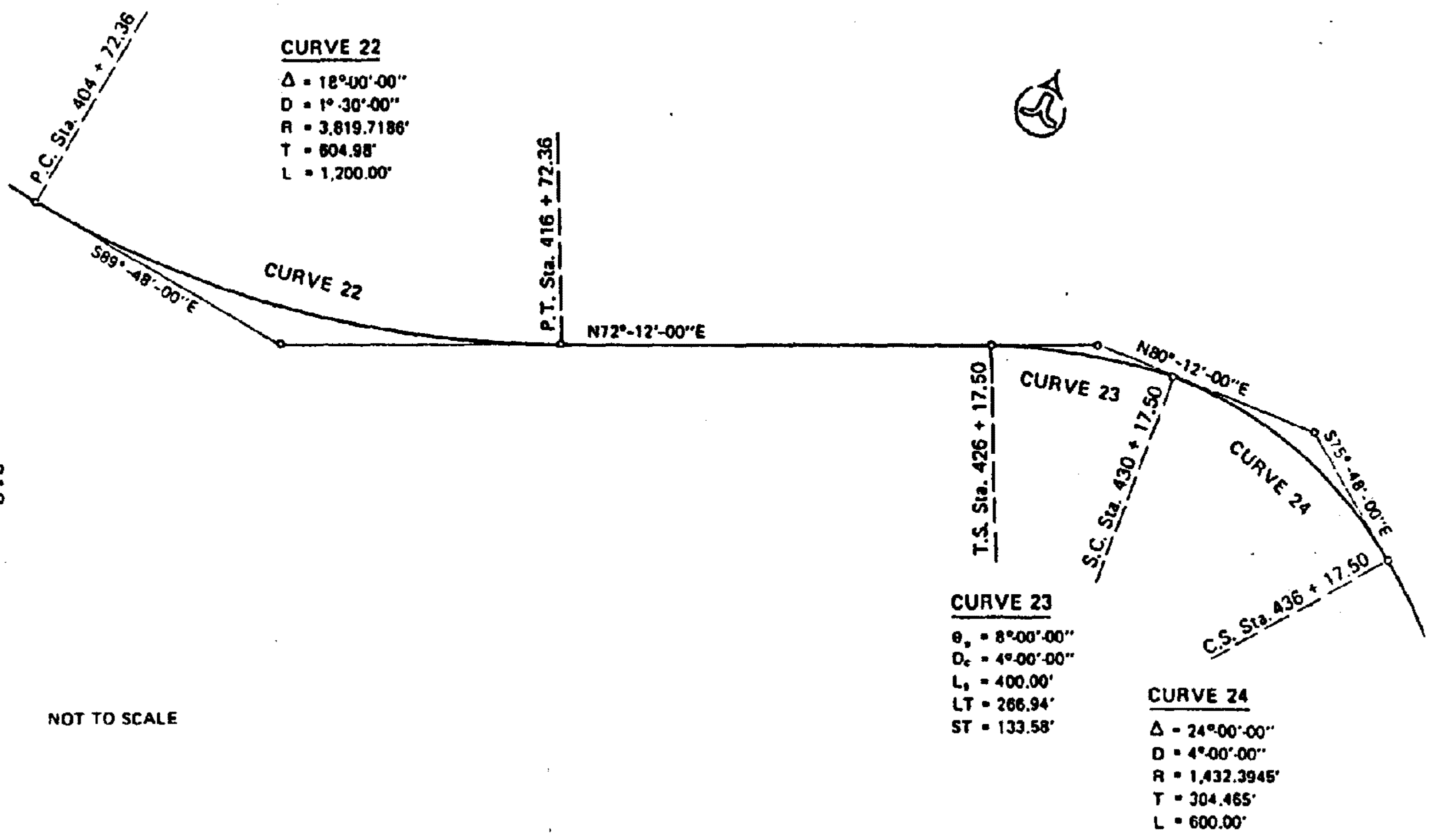


FIGURE 58. EXAMPLE HIGHWAY ALINEMENT

PROJECT: I-49, Alexandria, La.
PROJECT NO.: 700-12-01 (155-1)
BY: vas CKD BY: jo
DATE: 6-15-81

CURVE 22
Circular Curve - Left

Station	Def. Angle	Chord - Ft.	Curve Data
P.C. 404+72.36	0°	--	P.C.
405+00.00	0°-12.4'	27.64	
405+50.00	0°-34.9'	50.00	
406+00.00	0°-57.4'	50.00	Bear S89°-48'-00"
406+50.00	1°-19.9'	50.00	
407+00.00	1°-42.4'	50.00	
407+50.00	2°-04.9'	50.00	
408+00.00	2°-27.4'	50.00	
408+50.00	2°-49.9'	50.00	
409+00.00	3°-12.4'	50.00	
409+50.00	3°-34.9'	50.00	P.I. 410+77.34
410+00.00	3°-57.4'	50.00	Δ=18°-00'-00"Lt.
410+50.00	4°-19.9'	50.00	D=1°-30'-00"
			R=3,819.7186'
411+00.00	4°-42.4'	50.00	T=604.984'
411+50.00	5°-04.9'	50.00	L=1200.000'
412+00.00	5°-27.4'	50.00	
412+50.00	5°-49.9'	50.00	
413+00.00	6°-12.4'	50.00	
413+50.00	6°-34.9'	50.00	
414+00.00	6°-57.4'	50.00	
414+50.00	7°-19.9'	50.00	
415+00.00	7°-42.4'	50.00	
415+50.00	8°-04.9'	50.00	Bear N72°-12'-00"
416+00.00	8°-27.4'	50.00	
416+50.00	8°-49.9'	50.00	
P.T. 416+72.36	9°-00.0'	22.36	P.T.

FIGURE 59. EXAMPLE COMPUTER-PRODUCED ALINEMENT STAKEOUT NOTES

PROJECT: I-49, Alexandria, La.
PROJECT NO.: 700-12-01 (155-1)
BY: vas CKD BY: jo
DATE: 6-15-81

CURVE 23

Spiral Curve - Right

	Station	Def. Angle	Chord - Ft.	Curve Data
T.S.	426+17.50	0°	--	T.S.
	426+50.00	0°-01.1'	32.50	Bear N72°-12'-00"E
	427+00.00	0°-06.8'	50.00	
	427+50.00	0°-17.6'	50.00	P.I. 428+84.44
				θs=8°-00'-00" Rt.
	428+00.00	0°-33.3'	50.00	Dc=4°-00'-00"
	428+50.00	0°-54.1'	50.00	Ls=400.000'
				LT=266.939'
	429+00.00	1°-19.8'	50.00	ST=133.581'
	429+50.00	1°-50.6'	50.00	
				Bear N80°-12'-00"
	430+00.00	2°-26.3'	50.00	
S.C.	430+17.50	2°-40.0'	17.50	S.C.

FIGURE 59. EXAMPLE COMPUTER-PRODUCED ALINEMENT STAKEOUT NOTES (Continued)

PROJECT: I-49, Alexandria, La.
PROJECT NO.: 700-12-01 (155-1)
BY: vas CKD BY: jo
DATE: 6-15-81

CURVE 24

Circular Curve - Right

	Station	Def. Angle	Chord - Ft.	Curve Data
S.C.	430+17.50	0°	--	S.C.
	430+50.00	0°-39.0'	32.50	Bear N80°-12'-00"]
	431+00.00	1°-39.0'	50.00	
	431+50.00	2°-39.0'	50.00	P.I. 433+21.97
	432+00.00	3°-39.0'	50.00	
	432+50.00	4°-39.0'	50.00	Δ=24°-00'-00" Rt.
				D=4°-00'-00"
	433+00.00	5°-39.0'	50.00	R=1,432.3945'
	433+50.00	6°-39.0'	50.00	T=304.465'
				L=600.000'
	434+00.00	7°-39.0'	50.00	
	434+50.00	8°-39.0'	50.00	
	435+00.00	9°-39.0'	50.00	
	435+50.00	10°-39.0'	50.00	Bear S75°-48'-00"E
	436+00.00	11°-39.0'	50.00	
P.T.	436+17.50	12°-00.0'	17.50	P.T.

FIGURE 59. EXAMPLE COMPUTER-PRODUCED ALINEMENT STAKEOUT NOTES (Continued)

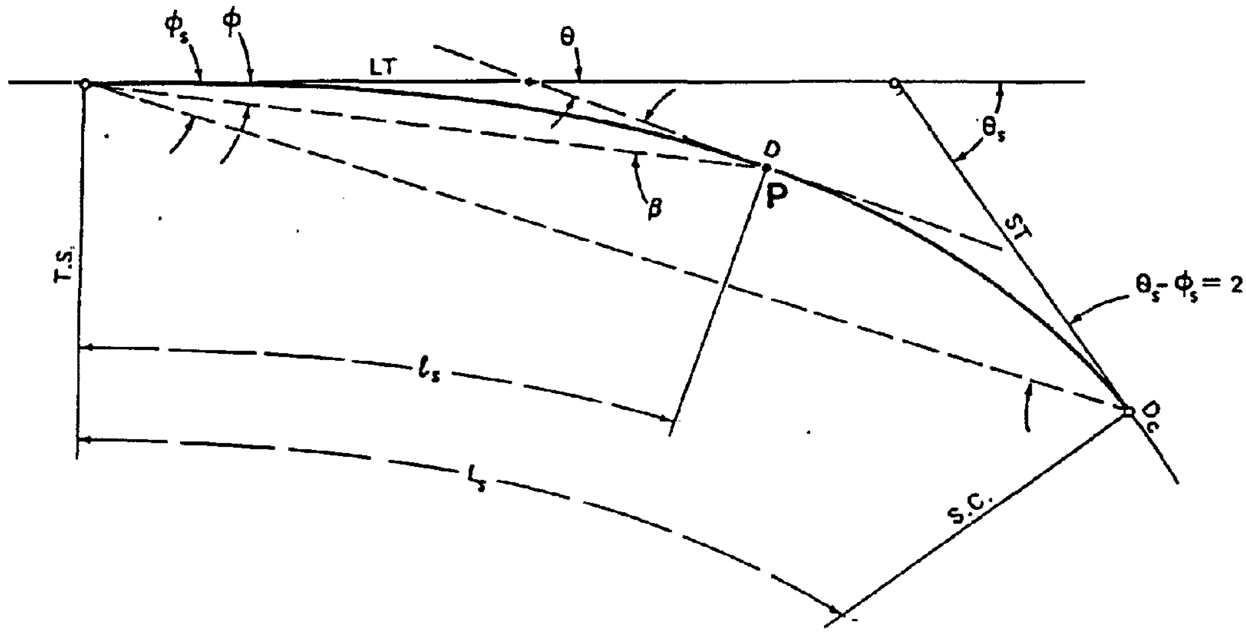
PROJECT: I-49, Alexandria, La.
PROJECT NO.: 700-12-01 (155-1)
BY: vas CKD BY: jo
DATE: 6-15-81

CURVE 23

Spiral Curve - Right

	Station	Defl. Δ ϕ	Chord - Ft. c	Bk. Sight Δ Local Tan - β	Curve Data
T.S.	426+17.50	0°	--	0°	T.S.
	426+25.00	0°-00.1'	7.50	0°-00.1'	
	426+50.00	0°-01.1'	25.00	0°-02.1'	Bear N72°-12'-00"E
	426+75.00	0°-03.3'	25.00	0°-06.6'	
	427+00.00	0°-06.8'	25.00	0°-13.6'	
	427+25.00	0°-11.6'	25.00	0°-21.5'	P.I. 428+84.44
	427+50.00	0°-17.6'	25.00	0°-35.1'	
	427+75.00	0°-24.8'	25.00	0°-49.6'	θs=8°-00'-00"Rt. Dc=4°-00'-00"
	428+00.00	0°-33.3'	25.00	1°-06.6'	Ls=400.000'
	428+25.00	0°-43.1'	25.00	1°-26.1'	LT=266.939' ST=133.581'
	428+50.00	0°-54.1'	25.00	1°-48.1'	
	428+75.00	1°-06.3'	25.00	2°-12.6'	
	429+00.00	1°-19.8'	25.00	2°-39.6'	
	429+25.00	1°-34.6'	25.00	3°-09.1'	Bear N80°-12'-00"E
	429+50.00	1°-50.6'	25.00	3°-41.1'	
	429+75.00	2°-07.8'	25.00	4°-15.6'	
	430+00.00	2°-26.3'	25.00	4°-52.6'	
S.C.	430+17.50	2°-40.0'	17.50	5°-20.0'	S.C.

FIGURE 60. EXAMPLE SPECIAL STAKEOUT NOTES FOR CLOSELY SPACED STATIONS



$L_s = \frac{200\,\theta_s}{D_c}$ $\theta_s = \frac{L_s D_c}{200}$ $\phi_s = \frac{\theta_s}{3}$ $D = \frac{l_s}{L_s} D_c$

$\theta = \frac{l_s^2}{L_s^2}\theta_s$ External angle for any point P

$\phi = \frac{\theta}{3}$ Deflection angle to point P

$\beta = \theta - \phi = 2\phi$

FIGURE 61. TERMINOLOGY AND EQUATIONS FOR CALCULATION OF DEFLECTION ANGLES

This research has identified the apparent need for and advantages in the use of spirals to improve the quality of geometrics and the natural flow of alinement of high-type facilities. In addition to the improved appearance, the spiral performs its major function in allowing the driver to operate with minimum steering effort and maximum comfort in negotiating a given section of curved roadway. When designed with appropriate length and coordinated with development of superelevation over this length, the spiral provides for improved comfort and safety as the driver negotiates a smoothly transitional path.

The accompanying tabulation of points relating to "Advantages/Disadvantages" of spirals is compiled as part of the summary discussion. Clearly, the development of recent techniques in combination with computer applications now permits the design, plan layout, and stakeout of both spirals and circular curves in a simple, direct manner.

"Advantages/Disadvantages" of Spirals Summarized

Advantages of Spirals (As Reported in AASHTO Policies)

1. Properly designed transition curves provide a natural, easy to follow path for drivers, thereby improving operations and safety.

2. The length of spiral provides a convenient and desirable arrangement for superelevation runoff.

3. Where pavement is to be widened around a circular curve, the spiral facilitates the transition in width.

4. The appearance of the highway is enhanced by the application of spirals.

Discussion of Perceived Problems with Spirals -- Disadvantages Countered or Dispelled

(1) **Spirals are complex and hard to calculate, and present problems in design.**

The use of minicomputers permits direct, easy calculations with convenient printouts, in formats similar to circular curves for every possible layout, design requirement, and plan preparation. Procedures have been so simplified that it is no more difficult to design highway alinements with spirals than without.

(2) **Surveyors are not familiar with spirals and find them difficult to stake out.**

Computer-generated field stake-out notes can be produced and packaged in a way to permit spirals to be staked in an identical manner to circular curves. Comprehensive printouts of stake out notes for entire projects are made available to the surveyors in book form.

(3) **Spirals present problems in layout and construction of bridges and other structures.**

As in (2), complete geometric design notes can be provided specifically for bridges. These are also formatted similar to notes for circular curves, with local tangents, radials for offsets, and a multitude of other points provided.

(4) **In setting of right-of-way, spirals present problems, particularly where right-of-way parallels the roadway at a uniform distance from the centerline.**

Since right-of-way is normally set by straight, angular lines, the spiral has no effect on right-of-way description. Where concentric right-of-way lines equidistant from the highway centerline are required, a procedure using two circular curves on the right-of-way line opposite the centerline spiral produces a simulated concentric right-of-way line. Thus, no spirals are needed to describe and set right-of-way lines.