AN ASSESSMENT OF ROAD SAFETY EQUATIONS FOR BRITISH COLUMBIA

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Abstract

Five quantitative relationships relating some geometric features of two-lane rural highways to accident rates were reported in Special Report 214 of the Transportation Research Board. In this study, three of these models were applied to data from several two-lane sections of two rural highways in the province of British Columbia. The models were used to predict accident rates in the road sections for the five-year period covering 1981 to 1985. The R^2 values resulting from linear regression analyses of the predicted accident rates on the actual accident rates were used as a measure of the applicability of the models to the study area.

The results of this study are valuable for conducting an extensive road safety study on two-lane rural highways in British Columbia, primarily, and other regions of the world.

Table of Contents

List of Tables

List of Figures

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Praise be to the Father, the Son, and the Holy Spirit.

Chapter 1

INTRODUCTION

Accidents are thought by many to be the result of chance or luck. Their occurrence is considered to be "perhaps the last folklore subscribed to by rational men, including well-educated professionals who have been trained to identify and reject folklore in their own areas of competence" (1). It is worth-noting that the word 'disaster', applied to the more catastrophic accidents, comes from the Latin for 'evil star'.

Accidents are not entirely uncontrollable nor do they defy systematic study. The problem of systematic study is that researchers or investigators often seek a 'primary cause' or 'proximate cause' but there is usually no such single cause. A solution to the overall highway safety problem (which increases in scope and complexity almost daily) requires knowledge of the many causative factors that come together in combination, apparently coincidentally yet with traceable logic, to create the rare situatiuon that leads to an accident.

If accidents are viewed as true incidents, which result from a combination of circumstances or a chain of related events, then they lend themselves to engineering study and systematic analysis. Engineering solutions require knowledge of the characteristics of the occurrence of accidents generally, and in each transportation mode of the major incident-producing conditions, and of the analytical tools and methods for both preventive activities and for accident investigation and analysis.

In general, transportation accidents are caused by failure of one of the three major elements of the transportation system : the human (driver, pilot, engineer), the vehicle, or the guideway/environment. Improvements in each of these areas can be expected to improve safety generally, and to reduce the potential for failure.

In highway transportation, few accidents have been traced directly to mechanical failure of vehicles. Fewer still have been attributed to 'failure' of the roadway, except for occasional direct involvement of potholes and other pavement conditions or defective traffic control devices, even though transportation engineers know there is a clear relationship between poor or substandard design or control measures and accident causation (2). The attribution of a large percentage of highway accidents to driver failure does not serve the cause of safety. All too often blaming the driver simply hides the real cause.

The basic causes of highway transportation safety problems are those forces or situations that bring about congestion on the roadway; a decline in maintenance of roadways and bridges; reduced enforcement of necessary regulations; lack of attention to clear and apparent hazards; and ineffective inspections, training, or motivation of operating and maintenance personnel.

1.1 Lane and Shoulder Width

Zegeer et al (3) found that run-off-road and opposite direction accidents were the primary accidents associated with narrow lanes. Accident rates of these two types of accidents combined tended to decrease as pavement width increased. But little reduction in accidents was gained by widening a 22 ft wide road, thus widening beyond 22 ft was not cost-effective.

Zegeer et al (3) also analysed accident rates on two-lane rural roads for various ranges of shoulder widths as no shoulder, 1 to 3 ft, 4 to 6 ft, 7 to 9 ft, and 10 to 12 ft. It was found that run-off-road and opposite direction accident rates decreased as shoulder width increased up to 9 ft. There was a slight increase in accident rate for shoulders 10 to 12 ft wide. In another aspect of the effects of shoulders on highway operation, Jorol (4) noted that narrow shoulders resulted in drivers positioning their vehicles closer to the centerline of the roadway. This action increases the probability of occurrence of head-on accidents as it increases the chance of conflict between opposing vehicles. Several other researchers (3,4,5) agreed that roadways with shoulders are safer than those without shoulders.

1.2 Horizontal Curvature

Kurimoto (7) conducted a survey of 31,800 traffic accidents in Japan. He found that 15 percent of these accidents were head-on and that for this type of accident the accident rate increased as the horizontal radius decreased. The trend in the association of head-on accident rate to curvature was noticeable for curves with a curvature of 2.2 degrees or more. Shepherd and Lowe (8), in a study of 44 road sections in England, found that head-on accidents per section increased as the degree of horizontal curvature increased, with a sharp increase beyond 3.5 degrees.

1.3 Vertical Curvature

The primary effect of vertical curvature on highway safety is related to possible restrictions on sight distance that adversely affect emergency avoidance manoeuvres (9). Sight distance is the length of road ahead visible to the driver. To enhance safety on highways, designers must provide sight distances of sufficient length so that drivers can avoid striking unexpected objects in the highway lanes (10). Steep grades also affect the operation of the vehicle and the roadway capacity (6). However, their direct effect on accident occurrence has been shown to be inconsistent (11 - 13). Although intuition suggests a relationship between head-on accidents and grade steepness, the literature lacks information in this area.

1.4 Roadside Features

Traditionally, roadside features have been considered in association with run-off-road accidents (9,14). An out-of control vehicle that leaves the travel lanes and encroaches on the roadside is a potential mate for an impact with a roadside hazard. The probability of such an impact generally decreases with increasing distance of the hazard from the edge of the travel lanes. Zegeer (15) reports that, in a study of vehicle - tree accidents in Michigan, 85 percent of the trees involved in vehicle - tree crashes were within 30 ft of the road edge, although some trees involved in accidents have been as far from the pavement edge as 90 ft. Although run-off-road accidents are the main recipients of attention when roadside hazards are mentioned, there is an indication (16) that lateral obstructions located closer than 6 ft from the edge of pavement reduce the effective width of the pavement, a fact that forces drivers to travel closer to the centerline (4). This factor can be thought of as a cause for the occurrence of head-on accidents because it increases the chance of collision between opposing vehicles.

1.5 Bridges

In 1983, 44 percent of the 550,000 highway bridges in the United States were reported to be deficient in one or more ways. Structural condition and deck geometry were considered the most pervasive deficiencies (17); many of the bridges had widths narrower than the approach roadways. Mak and Calcote (18) found that the number of bridge-related fatal accidents per 100 million vehicle miles of travel was significantly higher than the average for all road types.

Bridge width, both absolute and relative, has long been considered a major factor affecting safety at bridge sites (19,20 - 22). All of these studies pointed to the hazard of narrow bridges, but they were mostly descriptive in nature and did not provide sufficient data to establish the relationships between bridge width and accidents. In 1955, Williams and Fritts (20) reported that, based on an analysis of accident data from 10 U.S. States, the accident rate was 1.0 accident per million vehicles for bridge structures with widths of 1 ft or more narrower than the approach roadway width, 0.58 for widths of between 1 ft narrower and 5 ft wider, and only 0.12 for widths of 5 ft or more wider. Similar findings were reported in an Australian study by Brown and Foster (20). Nearly 70 percent of bridge accidents (single-vehicle accidents that occurred on bridges and their approaches) during 1961 - 1962 occurred on bridges where the bridge - to - approach roadway width ratio was less than 0.8 ft; only 14 percent occurred on bridges with full approach width.

The roads that exist now in many parts of the world (especially in the rural areas) were built many years ago, at a time when the demands put on the road, vehicle, and driver were fewer and of lesser magnitudes. As technology improved, more vehicles of greater weights and higher speeds entered the highways, subjecting the transportation system to stresses and dangers beyond the limits originally anticipation. The result today in many countries is a network of deteriorating roads. Naturally, safety oganizations have become concerned as highway accidents have increased to unprecedented heights, claiming thousands of human lives and property damage worth millions of dollars.

In the United States, as in many other countries, resurfacing, restoration, and rehabilitation (RRR) projects have been applied to older highways to address critical pavement repair needs. A new controversy has centered on which minimum geometric design standards should be applied to RRR projects to upgrade the road. Some highway organizations have contended that pavement repairs alone enhance safety and that additional safety improvements would greatly increase project costs and delay improvements to many miles of deteriorating highways. Safety organizations, on the other hand, have viewed the federal RRR program as an opportunity to make long-needed safety improvements to older highways at the time of pavement repairs.

A committee of 16 experts in highway safety, design, and administration conducted studies of current RRR design practices, reviewed current knowledge about relationships between geometric design and safety, and analysed the cost and safety trade-offs of geometric improvements to existing highways. The results were published in the Transportation Research Board's Special Report 214 in 1987.

This thesis focuses on the relationships between geometric design and safety as reported in the Special Report 214. It's main objective is to find out if the relationships developed using data from the United States apply to data from the rural roads of British Columbia.

In Chapter 2, the Special Report 214 is reviewed with a view to understanding the basis of the quantitative relationships between safety and certain highway features. Data collection and analysis of data form the basis of Chapter 3 while discussions of results and limitations of the study appear in the fourth Chapter. Finally, conclusions and recommendations are given in Chapter 5.

Chapter 2

REVIEW OF SPECIAL REPORT 214

In response to a provision in the U.S. Surface Transportation Assistance Act of 1982, the Secretary of Transportation of the United States of America requested the U.S. National Academy of Sciences to study the safety cost effectiveness of highway geometric design standards and recommend minimum standards for resurfacing, restoration, and rehabilitation (RRR) projects on existing U.S. federal-aid highways, except freeways.

To carry out the study, a committee of 16 experts in the various disciplines needed to develop and apply geometric design standards and assess their impact on safety, highway serviceability, cost, environment, and system administration was assembled. The study committee sponsored critical reviews of prior research on the safety effects of key highway features and special research projects on pavement edge drops and roadside safety. The critical reviews and findings from the special research projects were used to make judgements about relationships between safety and key highway features. For several design features, the committee found sufficient evidence to support quantitative relationships between safety and design improvements.

In addition, the study committee developed relationships between cost and key highway features. These relationships were based on an examination of published cost data, cost records, and cost-estimating procedures for a sample of highway agencies throughout the United States.

The safety and cost relationships were used to assess the safety cost-effectiveness of geometric design standards. The added cost per accident eliminated that can be expected for improvements to highway geometry was estimated for illustrative projects. This review is, however, limited to the relationships between safety and geometric design as that is the focus of this study.

2.1 Relationships Between Safety And Geometric Design

The relationships described in the Special Report 214 pertain primarily to two-lane rural roads. The following questions are of primary importance in the treatment of the safety effects of highway improvements:

- What changes in accident rates can be expected if different types of geometric improvements are made?
- Will accident rates increase if highways are resurfaced without correcting existing geometric deficiencies?
- What are the safety benefits of low-cost alternatives, such as warning signs and markings, compared with more expensive geometric improvements?

Despite the widely acknowledged importance of safety in highway design, the scientific and engineering research necessary to answer these questions is quite limited and often insufficient to establish firm and scientifically defensible numerical relationships.

In addition to geometric features, a variety of other factors affect highway safety, including other elements of the overall road environment (e.g. pavement condition, weather and lighting, traffic, and traffic regulations), driver characteristics (intoxication, age), and vehicle characteristics (braking capability, size). The effect of highway design is obscured by the presence of these factors. Infact most accidents result from a combination of factors interacting in ways that preclude determining a single accident cause. Even when a vehicle runs off the road because of driver error or equipment failure, the design of the road may affect accident severity. This complex interaction between road, driver, and vehicle characteristics complicates attempts to estimate the accident reduction that can be expected from a particular safety improvement. Almost invariably, single parameter models cannot be developed.

Highway features affect safety by:

- Influencing the ability of the driver to maintain vehicle control and identify hazards. Significant features include lane width, alignment, sight distance, superelevation, and pavement surface characteristics;

- Influencing the number and types of opportunities that exist for conflicts between vehicles. Significant features include access control, intersection design, number of lanes, and medians;

- Affecting the consequences of an out-of-control vehicle leaving the travel lanes. Significant features include shoulder width and type, edge drop, roadside conditions, sideslopes, and guardrails.

- Affecting the behaviour and attentiveness of driver, particularly the choice of travel speed. Driver behaviour is affected by virtually all elements of the roadway environment.

For nearly 50 years, researchers have tried to measure the effects of various road features on safety. Accident rates have, in general, been estimated by using actual accident records and travel data. Despite these long-term efforts, explicit, widely accepted, quantitative relationships have not emerged. In part, this can be attributed to inherent difficulties in accident research:

- Accidents are relatively infrequent so that sound statistical studies require consistent data collected over long periods of time for many miles of highway.

- Many factors interactively contribute to the occurrence and severity of accidents, and researchers are often unable to sort out the effects attributable to the specific roadway feature of interest. Controlled experiments are difficult to design and conduct.

- Reporting practices for non-fatal accidents differ from state to state. Thus, estimates of accident rates developed using data from one area might not be appropriate elsewhere.

- Some factors, such as vehicle performance and crashworthiness, that underlie relationships between safety and road design, change over time so that relationships developed at one time may no longer be representative in later years.

Fully aware of these difficulties, the study committee commissioned two special research projects and several critical reviews of the existing highway safety literature in order to assess the most likely relationships between safety and the following highway design features:

1. Lane and shoulder width and shoulder type,
2. Roadsides and sideslopes,
3. Bridge width,
4. Horizontal alignment,
5. Sight distance,
6. Intersections,
7. Pavement surface condition,
8. Pavement edge drops.

In the committee's judgment, improvements to these design features on RRR projects are most likely to have significant and measurable safety effects. Judgments were made about the most probable relationships between safety and each of the highway design features. For each feature, the study assessed

- whether a relationship between safety and the design feature exists (e.g. is lane width related to safety?);
- direction of any relationship (e.g. whether increasing shoulder width improves or degrades safety);
- where possible, the magnitude of the safety impact most likely over the range of improvements being considered in RRR projects (e.g. the reduction in accidents expected if lanes are widened from 9 to 12 feet).

For several of the more important features, such as lane width, horizontal curvature, and bridge width, evidence was judged to be sufficient to generalize quantitatively the safety effects of design improvements. For features such as pavement edge drops and sideslopes, development of quantitative models proved to be impossible even though considerable safety-related information was collected. A description of the quantitative relationships developed are presented next.

2.2 Lane Width, Shoulder Width, and Shoulder Type

Wide lanes and shoulders provide motorists with increased opportunity for safe recovery when their vehicles run off the road and increased lateral separation between overtaking and meeting vehicles.

Prior research indicates that

- Accident rates decrease with increases in lane and shoulder width
- Widening lanes has a bigger effect than widening shoulders, in terms of accidents eliminated per foot of added width
- Roads with stabilized shoulder surfaces, such as asphalt or portland cement concrete, have lower accident rates than nearly identical roads with unstabilized earth, turf, or gravel shoulders.

Commissioned research produced the following relationship between cross-section features and accident rate consistent with the findings outlined above:

$$\mathrm{A} = 0.0019(\mathrm{ADT})^{0.882}(0.879)^{W}(0.919)^{PA}(0.932)^{UP}(1.236)^{H}(0.882)^{TER1}(1.322)^{TER2} \qquad \text{.....................} \quad (2.1)$$

where

A = the number of run-off-road, head-on, opposite-direction sideswipe, and same-direction sideswipe accidents per mile per year;

ADT = two-directional average daily traffic volume;

W = lane width in feet;

PA = width of paved shoulder in feet;

UP = width of unpaved (gravel, turf, earth) shoulder in feet;

H = median roadside hazard rating for the highway segment, measured subjectively on a scale from 1 (least hazardous) to 7 (most hazardous);

TER1 = 1 for flat terrain, 0 otherwise;

TER2 = 1 for mountainous terrain, 0 otherwise.

Model Limitations

- Applicable to lane widths of 8 to 12 ft. and shoulder widths of 0 to 10 ft. Combinations of lane widths and shoulder widths that can be reasonably modelled are described by the graphs below.

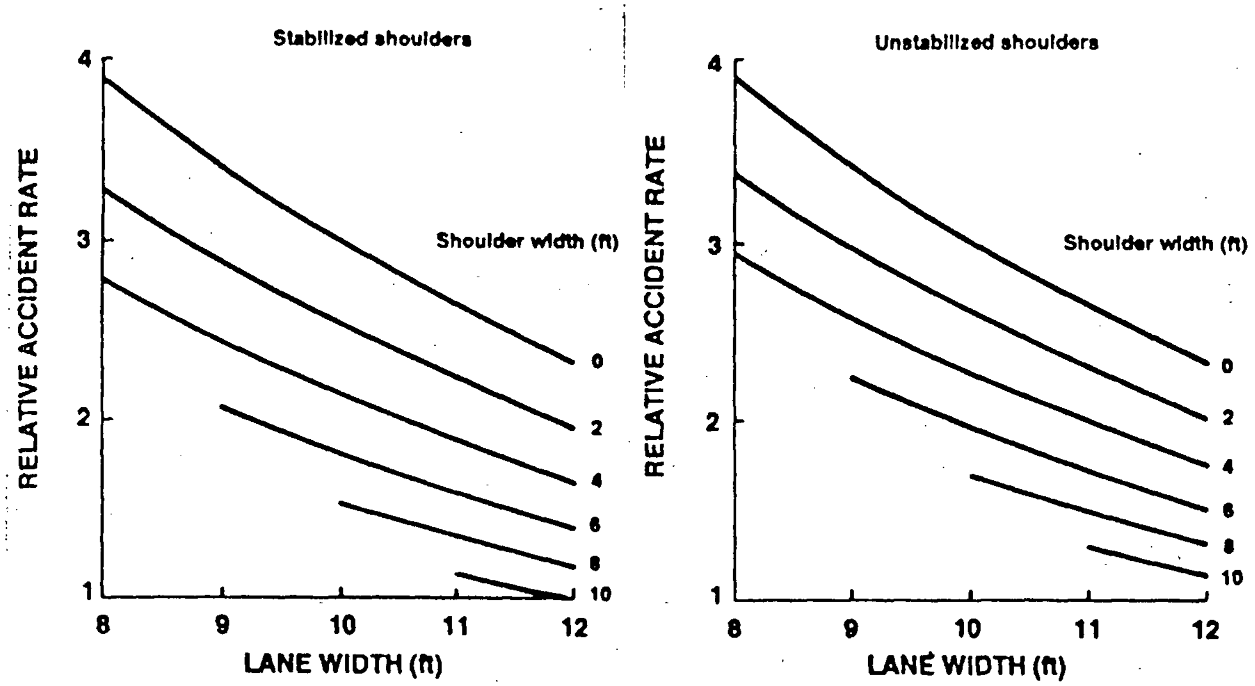

Figure 2.1: Normalized Relationship Between Accidents And Lane And Shoulder Conditions

- Accident relationship covers single-vehicle, sideswipe, and opposite-direction accidents on two-lane rural highways.

Relative accident rate is defined as a multiple of the accidents per million vehicle miles for 12-ft lanes and 10-ft stabilized shoulders.

2.3 Bridge Width And Safety

Roadway constriction at narrow bridges reduces the opportunity for safe recovery by out-of-control vehicles and can result in end-of-bridge collisions. Furthermore, bridge approaches are often on a downward grade resulting in increasing approaching speeds. When coupled with other factors such as premature icing in winter and substandard bridge rail, the special hazards associated with bridges can be significant.

Although the hazard associated with narrow bridges has been known for many years, efforts to quantitatively establish the influence of bridge cross-section and geometry on accident frequency and severity have met with limited success. The more acceptable of these efforts have found the difference between the clear bridge width and the width of the approach lanes as a better indicator of hazard than the bridge width itself. As this difference increases, observed bridge accident rates, expressed in terms of total accidents per million vehicles, markedly decrease.

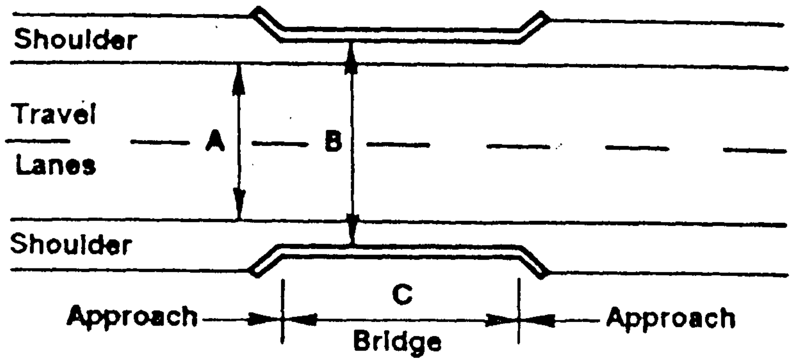

Figure 2.2: Explaining Relative Bridge Width

$B - A$ = relative bridge width, (RW)

Quantitatively, the rate of bridge-related accidents on two-lane highways can be estimated as

$$AR = 0.50 - 0.061(RW) + 0.0022(RW)^2 \tag{2.1}$$

for $0 \leq RW \leq 14$

where

- AR = number of accidents per million vehicles
- RW = relative bridge width in feet

The equation does not apply in situations where the width of the approach traffic lanes exceeds the clear bridge width : in this case, the accident rate is greatly increased by further constriction in the traffic lanes on the bridge. The equation is also not applicable to relative bridge widths in excess of 14 feet as , in this region, the upturn in computed accident rates is more likely a result of the model-building process than a valid indication of impaired safety.

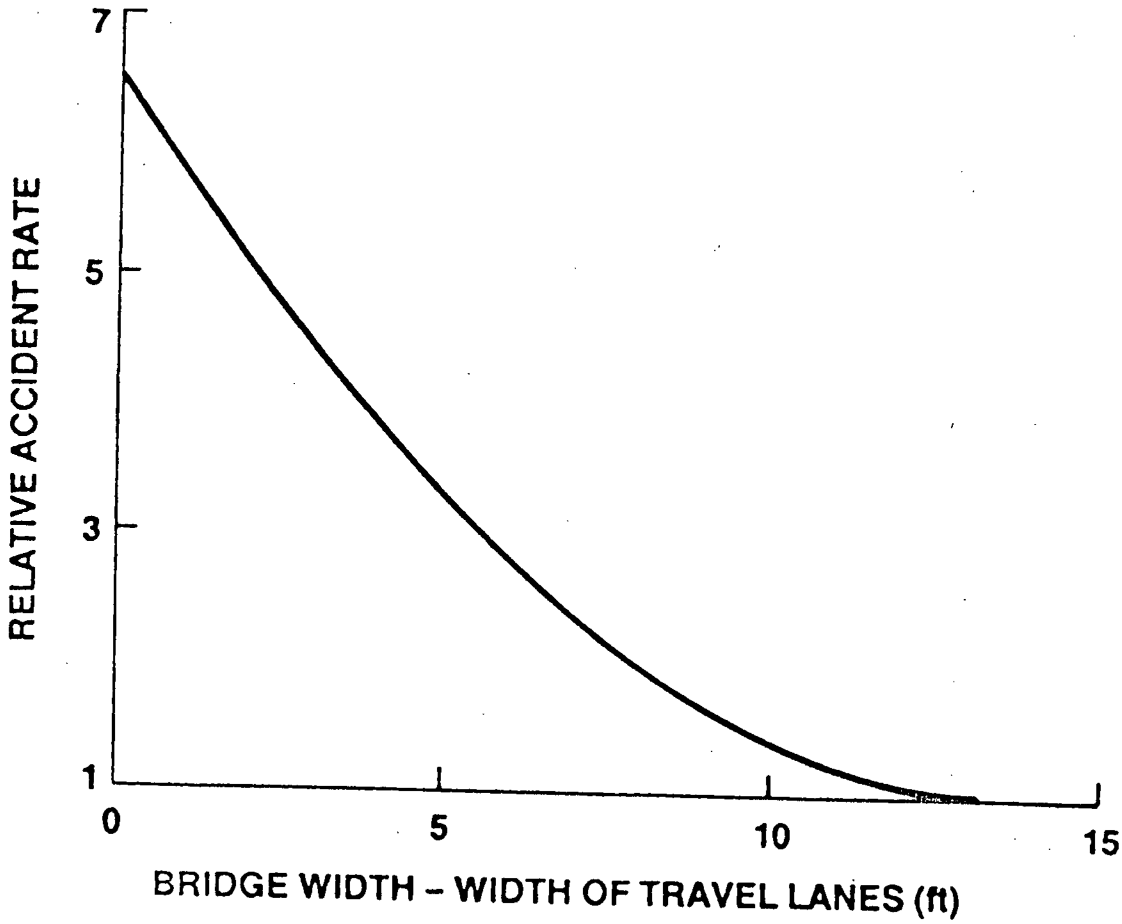

Figure 2.3: Normalized Relationship Between Accidents And Bridge Width

2.4 Relationship Between Accidents And Horizontal Curvature

Accidents are more likely to occur on horizontal curves than on straight segments of the roadway because of increased demands placed on the driver and vehicle and also because of increased friction between tires and pavement. The safety effects of an individual curve is influenced not only by the geometric characteristics of that curve, but also by the geometry of adjacent highway segments. The hazard is particularly great when the curve is unexpected, such as when it follows a long straight approach or when it is hidden from view by a hill crest. The safety effect of flattening sharp horizontal curves is of particular interest on RRR projects. When a sharp curve is improved, transitions from the straight to curved portions of the highway are smoother; the length of the curved portion of the highway is increased; and the overall length of the highway is slightly reduced.

Numerous researchers have attempted to relate changes in accident rate to specific characteristics of curve geometry, usually concentrating on degree of curve. The degree of a horizontal curve is defined as the angle subtended at the centre of the curve by a 100-ft long arc of the curve. Past studies differ in estimates of accidents per vehicle mile as a function of degree of curve, partly because of differences in techniques used for calculating the amount of travel and identifying accidents considered to be curve related. Also, the influences of other geometric and traffic characteristics on curve-related accidents were not properly treated in some of the analyses.

Concerned with the possible confounding effects of other variables (length of curve, lane width, shoulder width), Glennon et al (23) used analysis of covariance techniques to isolate the incremental effects of changes in degree of curvature on accidents. The estimated reduction in the number of accidents per million vehicles for a one-degree change in curvature was found to be 0.0336, and this number served as the basis for development of a complete accident model.

Model Development

The number of accidents at the combined straight - curved sites, A, can be represented as the sum of two components:

$$A = A_s + A_c$$

where

A_s = the number of accidents on the straight segments; and

A_c = the number of accidents on the curved segments.

$A = AR_s(L_s)(V) + A_c$

or

$$A = AR_s(L)(V) + [A_c - AR_s(L_c)(V)]$$

where

AR_s = the accident rate per million vehicle miles on the straight roadway,

L = length of site in miles

L_s = length of straight segments in miles

L_c = length of curved segment in miles

V = traffic volume in millions

$$A = AR_s(L)(V) + \Delta A_c$$

$$\Delta A_c = 0.0336(D)(V)$$

where D is the degree of curvature.

Hence the complete accident model can be expressed as

$$A = AR_s(L)(V) + 0.0336(D)(V) \tag{2.2}$$

for $L \geq L_c$

On the curved segment alone

$$A_c = AR_s(L_c)(V) + 0.0336(D)(V) \tag{2.3}$$

For the U.S. data base used by Glennon et al, $AR_s = 0.902$.

Equation 2.3 has two components: the first represents a steady-state turning effect and the second represents transitional (entry and exit) effects. The steady-state turning component is directly proportional to the vehicle miles of travel on the curve but is insensitive to degree of curvature. The transitional component is directly proportional to both degree of curvature and traffic volume.

Equation 2.2 can be used to estimate the net reduction in the number of accidents, ΔA, as follows:

$$\Delta A = A_o - A_n$$

$$= AR_s(L_0 - L_n)V + 0.0336(D_0 - D_n)V$$
$$= AR_s(\Delta L)V + 0.0336(\Delta D)V$$

where the subscript, 0, refers to the original alignment and n, the new alignment.

$$\Delta L = [(2.170 \tan I/2) - (I/52.8)][1/D_n - 1/D_0]$$

For $I \leq 90\,\text{deg}$, ΔL is sufficiently small that it can be ignored. Then

$$\Delta A = 0.0336(\Delta D)V$$

2.5 Accidents And Sight Distance At Crest Vertical Curve

Sight distance is the length of road ahead visible to the driver. Sight distance restrictions result from obstructions on the inside of horizontal curves, at intersections, or at sharp hill crests. Although obstructions at horizontal curves and intersections can sometimes be eliminated without changes to highway geometry (e.g. by cutting brush or trees), obstructions at hill crests can only be corrected by changes in vertical alignment – by lengthening the existing vertical crest curve.

The safety effect of a sight-distance restriction is influenced not only by the sight-distance restriction itself, but also by the nature and location of any potential hazards hidden from view. Thus, a heavily used but hidden intersection greatly increases the likelihood of accidents at crest curves. Without the heavy use, however, the necessity for

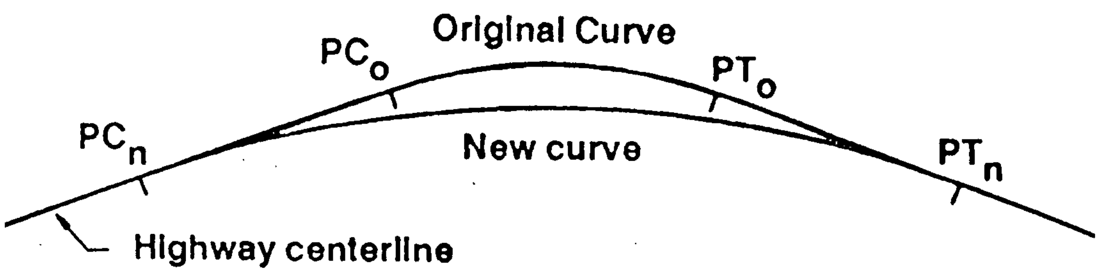

Figure 2.4: Horizontal curve before and after improvement

emergency stopping would be greatly reduced and, as a result, the likelihood of stopping-related accidents would also be reduced.

The critical review of restricted-sight effects revealed no empirically based quantitative relationship describing the influence of sight restrictions at hill crests on highway accidents. In the absence of such a relationship, a theoretical model was developed to estimate the effects of restricted sight distance at crest vertical curves on accident rates.

For a highway segment containing an isolated vertical curve, the accident model can be expressed as:

$$N = AR_h(L)(V) + AR_h(L_r)(V)(F_{ar}) \tag{2.4}$$

where

N = number of accidents on a segment of highway containing a crest curve

AR_h = average accident rate for the specific highway in accidents per million vehicle miles

L = length of highway segment in miles

V = traffic volume in millions of vehicles

L_r = length of restricted sight distance in miles (the distance over which sight distance is below or equal to the value specified by AASHTO)

F_{ar} = a hypothetical accident rate factor that varies according to the nature of the sight restriction and the nature of the hidden hazard.

To apply equation 2.4,

- The average accident rate, AR_h, should preferably be based on historical data collected for a substantial length of the highway under consideration. In the absence of such a data, the use of statewide averages for the particular highway type is recommended as alternative.
- The length of sight restriction is a complex function of the highway operating speed and curve goemetrics. It can be estimated, by approximation, as

$$L_r = (a_0 + a_1 A)(1/5280) \tag{2.5}$$

where the $a's$ are constants identified in Table 2.1 below and A is the absolute value of the grade difference in percent.

The use of Table 2.1 demands prior determination of the highway design speed which also requires computing the stopping sight distance (SSD) as follows:

$$SSD = [7.017 \times 10^6 (L_{vc})/A]^{0.5}$$

for $SSD < L_{vc}$

$$SSD = 2640(L_{vc}) + 664.5/A$$

for $SSD > L_{vc}$

L_{vc} = length of vertical curve in miles

The design speed is then found by interpolation from Table 2.2.

Table 2.1: Constants Used for Determining Length of Restricted Sight Distance (L_r) by Equation 2.5

Highway Operating Speed on Vertiacal Curve (mph)	Highway Design Speed (mph) 60	55	50	45	40	35	30	25
Values of a_0								
60	-524	-138	-25	113	202	256	305	382
55		-452	-163	11	111	172	221	301
50			-405	-65	45	115	169	248
45				-332	-76	21	82	167
40					-272	-55	15	110
35		No Sight Restriction				-231	-74	51
30							-193	19
25								-130
Values of a_1								
	207.3	152.6	120.9	80.2	56.6	38.6	29.4	15.3

Table 2.2: AASHTO Stopping Sight Distance as a Function of Design Speed

Design Speed (mph)	Stopping Sight Distance (ft)
25	150
30	200
35	225
40	275
45	325
50	400
55	450
60	525

Table 2.3: Accident Rate Factors (F_{ar})

Severity of Sight Restriction (mph)	Degree of Hazard in Sight-Restricted Area		
	Minor	Significant	Major
0	0	0.4	1.0
5	0.3	0.8	1.4
10	0.5	1.1	1.8
15	1.2	2.0	2.8
20	2.0	3.0	4.0

The accident rate factor, F_{ar}, is selected from Table 2.3. From equation (2.6), the accidents attributable to a specific curve (excluding its straight approaches) can be estimated as:

$$N_c = AR_h(L_{vc})V + AR_h(L_r)(V)(F_{ar})$$

By making comparisons with an identical length of highway, the change in accidents expected from improving the stopping sight distance is given by

$$\Delta N = AR_h(V)[\Delta(L_r F_{ar})] \quad (2.6)$$

Explanation

$\mathrm{N}_{c0} = AR_h(L_{vc})V + AR_h(L_{r0})(V)(F_{ar0})$

$$\mathrm{N}_{cn} = AR_h(L_{vc})(V) + AR_h(L_{rn})(V)(F_{arn})$$

$$\Delta N = N_{c0} - N_{cn}$$

$$= (\mathrm{AR}_h)(V)[L_{r0}F_{ar0} - L_{rn}F_{arn}]$$

$$\Delta N = AR_h(V)[\Delta(L_r F_{ar})]$$

$$\Delta N/N_c = \frac{AR_h(V)[\Delta L_r F_{ar}]}{AR_h(L_{vc})V + AR_h(L_r)(V)F_{ar}}$$

$$\Delta N/N_c = \frac{\Delta(L_r F_{ar})}{L_{vc} + L_r F_{ar}}$$

(2.7)

Equation 2.7 is useful primarily for evaluating the safety benefits of incremental improvements in sight distance over practical ranges.

2.6 Relationship Between Accidents and Specific Roadside Features

Roadside encroachments begin when the vehicle inadvertently leaves the travel lanes, veering toward the roadside. Most encroachments are quite harmless as the driver is able to regain control of the vehicle on the shoulder and return to the travel lanes. With the presence of nearby roadside hazards, however, encroachments can result in roadside accidents. More than 30 percent of all accidents on two-lane rural roads in the U.S. involve single vehicles running off the road.

Entry of an errant vehicle onto the roadside border does not in itself constitute an accident nor does it mean that an accident is inevitable. There are very great chances of recovery if the border is reasonably smooth, flat and devoid of fixed objects and other non-traversable hazards. Safety researchers generally agree that at speeds of about 55 mph, safe clear zones should have sideslopes no steeper than about 6:1 and should extend outward at least 30 ft from the edge of the travel lanes. Research commissioned for this study revealed a significant relationship between the roadside recovery distance and accident rates on two-lane rural roads (figure 2.4).

- Relative Accident Rate is defined as the ratio of the number of accidents for a recovery distance of xft to the number of accidents for a recovery distance of 20 ft.
- Only single-vehicle, sideswipe, and opposite direction accidents are considered.
- Clear recovery area is measured from outside shoulder edge to the nearest roadside hazard or obstacle.

From the figure, increasing the clear recovery area from 5 to 20 ft reduces the number of single vehicle, head-on, and sideswipe accidents by about 35 percent.

Roadside encroachment models have been used to examine the safety effects of specific roadside features. This special study calibrated a roadside encroachment model using

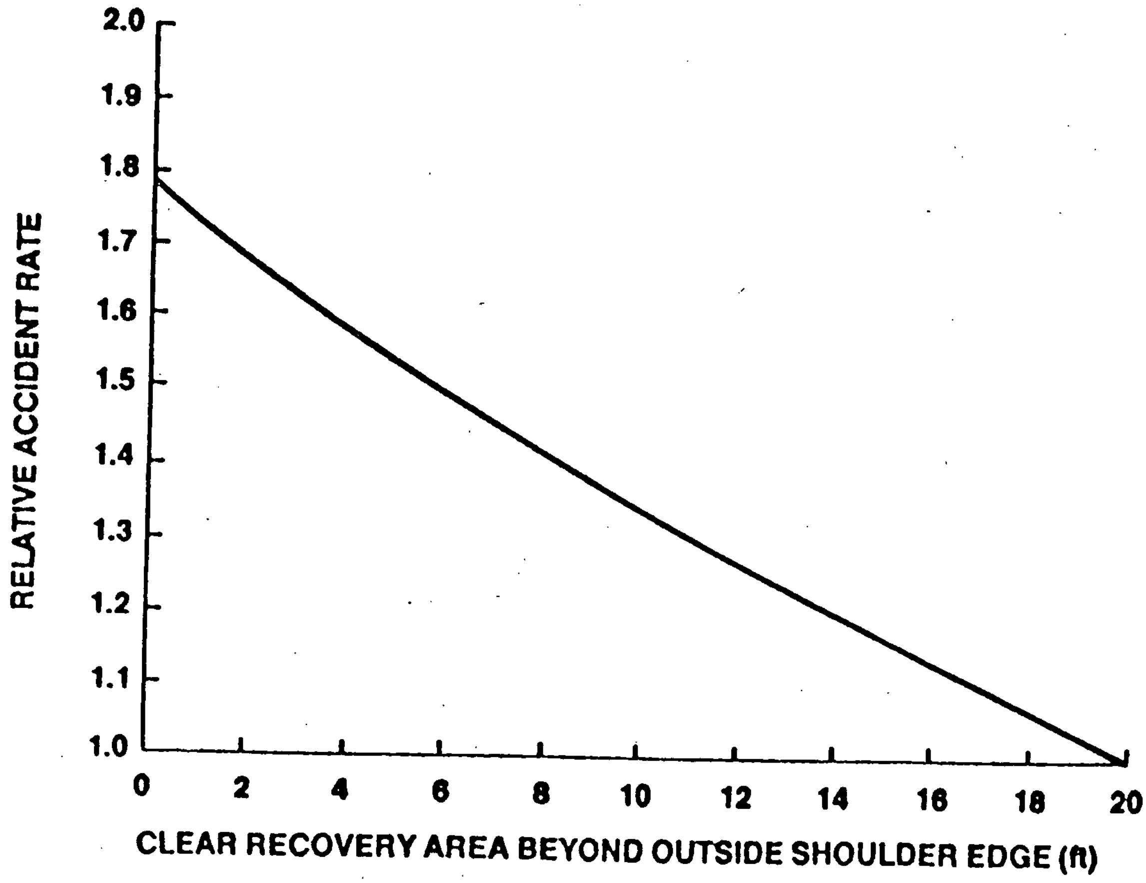

Figure 2.5: Normalized Relationship Between Accidents and Width of Clear Recovery Area

data from a study of utility pole accidents. Conceptually, roadside encroachment models are developed based on the intuitive formulation of the sequence of events that culminate in a roadside accident:

1. An out-of-control vehicle leaves the travel lanes and encroaches on the roadside,
2. The location of the encroachment is such that the path of travel is directed toward a potentially hazardous object or slope,
3. The hazardous object is sufficiently close to the travel lanes that control of the vehicle is not regained before encounter or collision between vehicle and object occurs,
4. The collision is sufficiently severe as to result in an accident

In the language of mathematical probability, the encroachment model takes the following form:

$$E_x(A_h) = E_x(E)P_r(E_h|E)P_r(C_h|E_h)P_r(A_h|C_h)$$

where

$E_x(A_h)$ = expected annual number of roadside accidents involving a specific hazard (h);

$E_x(E)$ = expected annual number of encroachments on the highway segment encompassing the hazard (typically 1 mile long);

$P_r(E_h|E)$ = conditional probability that, given an encroachment, its location is such that an impact with the hazard is possible;

$P_r(C_h|E_h)$ = conditional probability that, given an encroachment in the potential impact area, a collision between vehicle and object will occur

$P_r(A_h|C_h)$ = conditional probability that, given a collision, its severity will be so great as to result in an accident.

The expected number of casualty accidents could be determined as follows:

$$E_x(CA_h) = E_x(A_h)P_r(CA_h|A_h)$$

where

$E_x(CA_h)$ is the expected annual number of casualty (injury or fatality) accidents involving the hazard and $P_r(CA_h|A_h)$ is the conditional probability that, given an accident, an injury or fatality will occur.

Model Development

1. The first required element of the model is $E_x(E)$, the expected annual number of encroachments per mile of the highway (figure 2.5). For encroachment on both sides of the road, the model is:

$$E_x(E) = \frac{E_x(EXC)}{2} + \frac{E_x(EXC)P_r(Y \geq L)}{2}$$

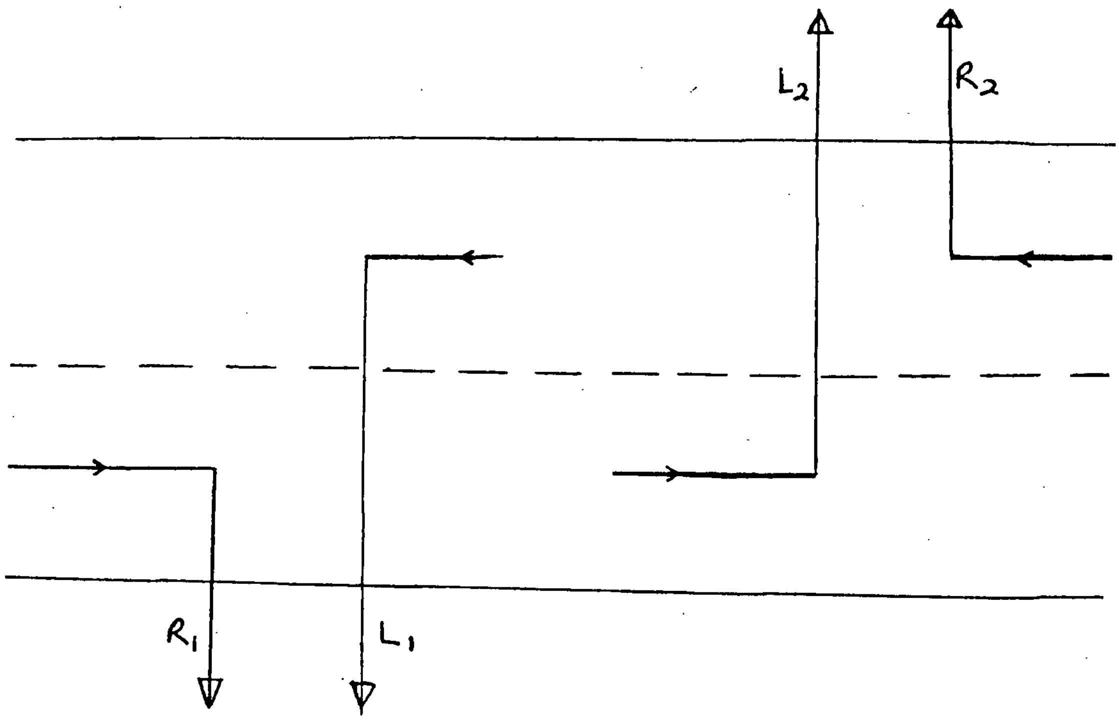

Figure 2.6: Encroachments on the left and right sides of the roadway

$$\sum E(L_i) = \frac{E_x(EXC)P_r(Y \geq L)}{2}$$

$$\sum E(R_i) = \frac{E_x(EXC)}{2}$$

For encroachment on one side of the road

$$\mathrm{E}_x(E) = \frac{E_x(EXC)}{4}[1 + P_r(Y \geq L)]$$

where $E_x(EXC)$ is the expected annual number of lane encroachments per mile and $P_r(Y \geq L)$ is the probability that an errant vehicle veering to the left will cross the adjacent lane of width, L, and encroach on the roadside. It is assumed in the model that out-of-control movements are equally likely for right and left directions. The expected annual number of encroachments was assumed to be related only to traffic flow as follows:

$$\mathrm{E}_x(EXC) = a(ADT)^b$$

where ADT is the two-directional average daily traffic volume in vehicles per day and a and b are calibration constants.

2. The next element to be modeled is $P_r(E_h|E)$, the conditional probability that, given an encroachment, its location is such that an impact with the roadside hazard is

possible. If X represents the distance, in feet, along the roadway within which an encroachment, if continued sufficiently far, will result in a collision with the hazard, then:

$$\mathrm{P}_r(E_h|E) = X/5280$$

where $E_x(E)$ represents encroachments on only a single roadside. To compute X, the following impact envelope was used (figure 2.7).

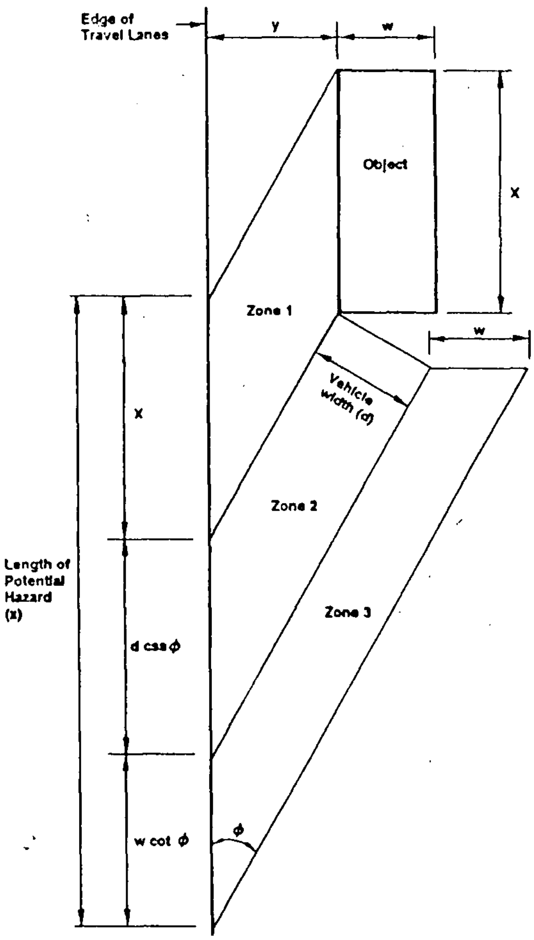

Figure 2.7: Impact Envelope

- Zone 1 :- collision on parallel side of object
- Zone 2 :- collision on near corner of object
- Zone 3 :- collision on perpendicular side of object.

The model assumes that the path of the errant vehicle is straight and so X is a function only of the angle of departure, the cross-sectional dimensions of the object and the width of the colliding vehicle. For the calibration using the utility pole data, each utility pole was assumed to have a square cross section of side 8 in; the departure angle, ϕ, was taken as 6.1 degrees for right-side departures and 11.5 degrees for left-side departures. The width of the colliding vehicle, d, was taken as 6 ft. Using these values, the projected length, along the roadway, of potential hazard from a single utility pole was obtained as 63.4 ft for a right-side departure and 34.0 ft for a left-hand departure.

Thus, for traffic distributed equally in both directions, the expected annual number of encroachments, $E_x(E_p)$, in the impact zone of a utility pole is given by:

$$E_x(E_p) = a(ADT)^b[\frac{1}{4}(63.4/5280) + \frac{1}{4}(34.0/5280)P_r(Y \geq L)]$$

3. The next element to be modeled is $P_r(C_h|E_h)$, the conditional probability that, given an encroachment in the potential impact area, a collision between vehicle and object will occur. With the assumption that the path of the travel is straight, this probability reflects the likelihood that control of the vehicle will be regained before the vehicle reaches the object. The relevant mathematical expression is:

$$P_r(C_h|E_h) = P_r(Y \geq y)$$

where $P_r(Y \geq y)$ is the probability that the outer front fender of the vehicle will continue beyond a lateral distance, y, from the lane boundary if its travel is not impeded by a prior collision or overturn and if control is not regained. To describe a function for $P_r(Y \geq y)$, three, one-parameter distributions were considered:

<u>**Linear**</u>

$$P_r(Y \geq y) = (1 - y/c)$$

for $y \leq c$

$$P_r(Y \geq y) = 0$$

for $y > c$

<u>**Exponential**</u>

$$P_r(Y \geq y) = e^{c(y)}$$

<u>**Sinusoidal**</u>

$$P_r(Y \geq y) = (1 + \cos cy)/2 \tag{2.7}$$

for $y \leq 180/c$

$$P_r(Y \geq y) = 0$$

for $y > 180/c$

where c is a calibration constant.

The expected annual number of collisions with the utility pole, $E_x(C_p)$, reflects both the number of encroachments in the impact zone and the lateral offset of the pole from the travel lanes. Increasing the offset reduces the number of collisions as there is greater chance of regaining control before vehicles reach the pole. The offset to the near-most front fender of colliding vehicles is constant for an impact at any location within Zone 1 but it varies with the specific location of impact in

Zones 2 and 3. For utility poles, 'w' is small and so the offset for Zone 3 impacts is assumed to occur at the midpoint location. Zone 2 is divided into six, 1-ft strips and the midpoint offset of each strip is used. Thus, the length of potential hazard, X, along the roadway could be divided into eight strips. The expected annual number of collisions with a pole can, therefore, be approximated as:

$$E_x(C_p) = \frac{a(ADT)^b}{5280}(0.25)[\sum_{i=1}^{8} x_i P_r(Y \geq y_i) + \sum_{j=1}^{8} x_j P_r(Y \geq y_j)]$$

where the values of x_i, x_j, y_i, y_j are given in Table 2.4, and the i and j subscripts refer to near-side and far-side encroachments, respectively.

4. The conditional probability of an accident given a collision, $P_r(A_h|C_h)$, has not been extensively addressed in the literature. Zegeer and Parker (24) have presented estimates for various hazards with an average of 0.9 accidents occuring for each collision. Using this value, the expected annual number of accidents with the utility pole, $E_x(A_p)$, is given by:

$$E_x(A_p) = \frac{a(ADT)^b}{23467}[\sum_{i=1}^{8} x_i P_r(Y \geq y_i) + \sum_{j=1}^{8} x_j P_r(Y \geq y_j)]$$

Actual utility pole accident data were used to evaluate the three lateral travel distribution models and to calibrate the unknown constants a, b, and c, for each. For each type of lateral travel distribution, the best calibration is one that:

Table 2.4: Length Of and Offset To Utility Pole

Zone (Figure 2.7)	Segment Number	Near-Side Encroachments Hazard Length (ft)	Near-Side Encroachments Offset (ft)	Far-Side Encroachments Hazard Length (ft)	Far-Side Encroachments Offset (ft)
1	1	0.67	y	0.67	y+12.00
2	2	9.41	y+0.50	5.02	y+12.49
2	3	9.41	y+1.49	5.02	y+13.47
2	4	9.41	y+2.48	5.02	y+14.45
2	5	9.41	y+3.48	5.02	y+15.43
2	6	9.41	y+4.47	5.02	y+16.41
2	7	9.41	y+5.47	5.02	y+17.39
3	8	6.27	y+6.30	3.29	y+18.21

- yields a predicted mean accident frequency that equals the actual frequency,
- most accurately predicts accident frequency as measured by the correlation coefficient between the actual and predicted accident frequencies.

It came out that the three lateral travel distributions offer approximately the same accuracy but the exponential model was recommended both for its ease of use and for its greater sensitivity to lateral offsets in regions near the travel lanes.(Fig. 2.8).

For the exponential model,

$$P_r(Y \geq y) = e^{-0.08224y}$$

i.e $c = -0.08224$

and

$$Z = 0.07285(ADT)^{0.5935}$$

Thus, the expected annual number of accidents, $E_x(A_h)$, involving any roadside hazard is given by

$$\mathrm{E}_x(A_h) = \frac{0.07285}{21120}(ADT)^{0.5935}P_r(A_h|C_h)[\textstyle\sum_{i=1}^{8} x_i e^{-0.08224y_i} + \sum_{j=1}^{8} x_j e^{-0.08224y_j}]$$

For utility poles, using a value of 0.9 for $P_r(A_h|C_h)$, we have:

$$\mathrm{E}_x(A_h) = \frac{0.07285}{23467}(ADT)^{0.5935}[\textstyle\sum_{i=1}^{8} x_i e^{-0.08224y_i} + \sum_{j=1}^{8} x_j e^{-0.08224y_j}]$$

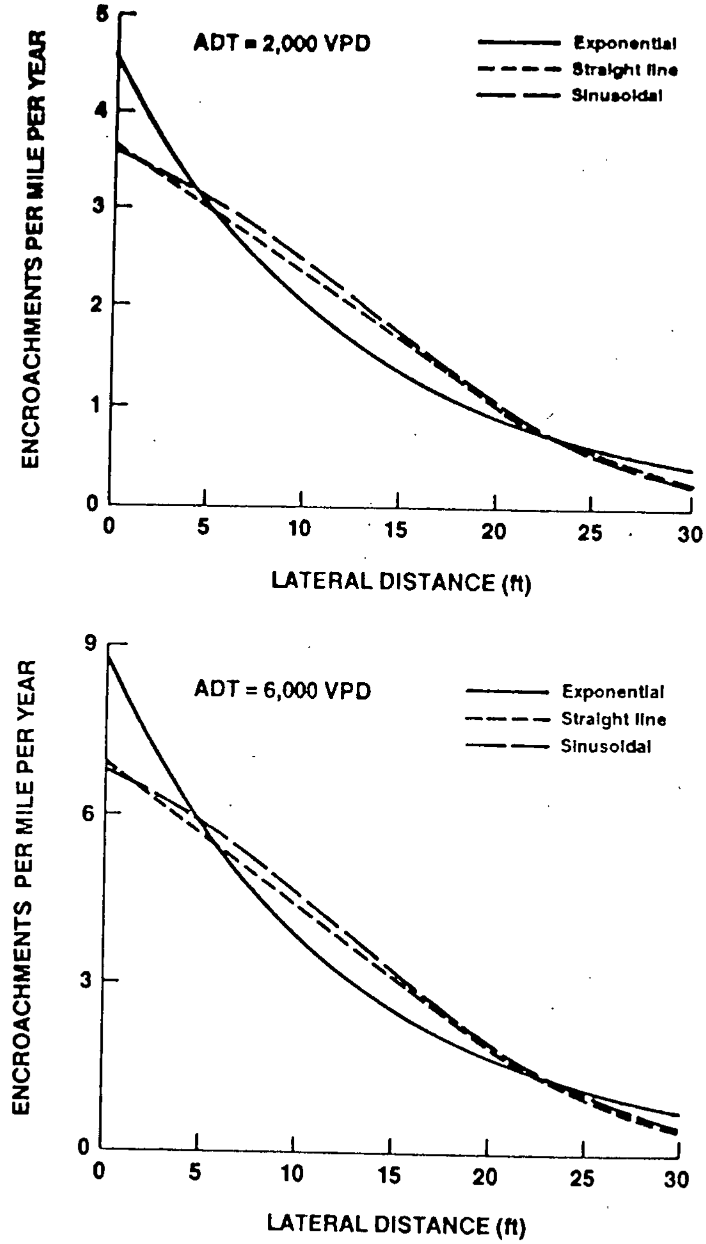

Figure 2.8: Comparison of lateral travel distribution models on the basis of the frequency of roadside encroachments.

Chapter 3

DATA COLLECTION AND ANALYSIS OF DATA

3.1 Data Collection

The routes selected for this study were Route 3 (from Hope to Osoyoos) and Route 99 (from Horseshoe Bay to Pemberton). The selection was based on the availability of reasonable lengths of continuous two-lane sections and the availability of accident, roadway, and traffic data. For each of the Routes, the necessary traffic, accident, and roadway geometry data were obtained from the Department of Highways.

3.1.1 Accident Data

The accident data were in two main forms. In both forms, the entire length of each Route is divided into 100 metre segments. One representation has the accidents that occurred in each 100 metre segment, within the period 1981 to 1985, related to location with respect to highway features (Appendix A). This representation makes it possible to obtain the number of bridge-related accidents that occurred within the 100 metre segment; the number of accidents that occurred at an intersection within the same 100 metre segment; the number of curve-related accidents, and so on.

The other form of the accident data is accident by severity. The total number of accidents in each 100 metre section is broken down into fatal, injury, and property damage only accidents. These are represented symbollically by F, I, and P respectively, and the representation is in the form of a pseudo-histogram (Appendix B). These forms of accident data presentation were not available by month nor season of year.

3.1.2 Traffic Volumes

Three sources provided the traffic volumes used in the analysis.

- Average Daily Traffic Volumes for the months of July and August are compiled in two publications covering the periods 1981 to 1984 and 1982 to 1985 (25,26). Tabulations of the vehicular traffic volumes taken at Temporary (or Short) and Permanent Counters located at strategic points on the Province's highways are presented in the publications. The tabulation is by numbered Routes.

- The locations of the Counters on the various Routes appear in the Systems Planning Branch Technical Road Inventory Report (27). From this inventory, the locations of the Counters are obtained, to the nearest 10 m, and so, it is possible to estimate the stretch of road over which each traffic volume is applicable.

- Conversion or expansion factors for traffic volumes obtained at the Permanent Counters, from 1981 to 1985, are tabulated for the entire road network of British Columbia (Appendix C). These factors are the values of the ratio of July - August traffic volume to annual traffic volume.

- To provide conversion factors for traffic volumes obtained at Short Counters, R^2 values resulting from linear regression analyses of traffic volumes at Short Counters and those at Permanent Counters have been provided by the Department of Highways (Appendix D). A Short Counter assumes, for its conversion factor, the conversion factor of the Permanent Counter with which it best correlates (i.e. for which the R^2 value is greatest). The assumption here is that traffic pattern at the location of the Short Counter is similar to that at the Permanent Counter.

3.1.3 Lane Width, Shoulder Width, and Shoulder Type

Values for the lane width, shoulder width, and shoulder type at various sections of each Route are direct entries in the Systems Planning Branch Technical Road Inventory (27).

3.1.4 Bridge Width

Bridge widths were obtained from plans of as-constructed bridge drawings from the Bridges Division of the Ministry of Highways. The difference between the clear bridge width and the width of approach travel lanes, excluding the shoulder, if any, gives the relative bridge width. Each bridge has a code number and these same code numbers are the identifying numbers for bridges in the accident data and the Road Inventory Report. Thus, it was possible to match bridge-related accidents in each 100 metre section to a particular bridge.

3.1.5 Horizontal Curves

As-constructed drawings of the Routes used in this study were obtained from the Constructions Division of the Ministry of Highways and Transportation. The chainages of the beginning and end of each horizontal curve are provided in these drawings, as well as the radii and curvatures of the curves. The chainages are in imperial units of measurement so they were converted to metric units to obtain the location of each horizontal curve selected on each Route to the nearest 100 metres (0.1 km). This was necessary because distances on the accident histogram are presented to the nearest 100 metres.

3.1.6 Utility Poles

An inventory of utility poles on the Routes and their dispositions from the edge of the travel lanes could not be obtained. Also, there was no entry for utility-pole-related accidents in the tabulations of accident data. The tabulations simply gave the number of accidents involved with a roadside hazard - be it a tree, a utility pole, or a rock face. No provision was made for the distance of the hazard from the edge of the travel lanes. It, therefore, was futile to attempt to relate accident rate to the distance of the nearest hazard from the edge of the road without detailed site investigation.

3.2 Analysis Of Data

3.2.1 Relationship Between Accident Rate and Lane Width, Shoulder Width, and Shoulder Type

Equation (2.1), reproduced below, is used to predict accident rates on various segments of the highways.

$$A_c = 0.0019(ADT)^{0.882}(0.879)^{W}(0.919)^{PA}(0.932)^{UP}(1.236)^{H}(0.882)^{TER1}(1.322)^{TER2}$$

In this equation, the following values were assumed:

$TER1 = 1$

$TER2 = 0$

$H = 5$ for Route 3

$H = 6$ for Route 99

$PA = 0$ since no segment has a paved shoulder section.

Route 3

Route 3 was subdivided into two sections : Hope to Princeton and Princeton to Osoyoos. Table (3.5) gives the characteristics of the segments taken from the two-lane portion of the Hope to Princeton section. The total length of two-lane highway included in this section of length $135.2km$ was $41.4km$. The remaining segments of the section were either 3-lane, 4-lane, or too close to urban centers to be considered rural.

In Table (3.6), the values in column 3 are the accident rates for the segments calculated using the quantitative model developed in the Special Report 214 (equation 2.1) while column 4 gives the accident rates calculated from actual accident data. Accident rates are in accidents per mile per year. Figure (3.9) is a plot of the predicted accident rate using the model (A_c) against the actual accident rate.

The geometrical properties of the two-lane segments in the Princeton to Osoyoos section of the highway are presented in Table (3.7). The total length of two-lane highway involved in this section of length 113.3 km was 76 km. The entire length of Princeton to Keremeos is a two-lane rural highway but the section from Keremeos to Osoyoos is interspersed with 3-lane segments. The entries in columns 3 and 4 in Table (3.8) are the values of the predicted accident rates and the actual accident rates respectively. The graphical representation of the correlation between the two sets of values is given in figure (3.10).

Table 3.5: Lane Width, Shoulder Width, and Shoulder Type of Segments – Route 3 : Hope to Princeton

Road Segment (km)	Length of Segment (km)	Lane Width W (ft)	Shoulder Width UP (ft)	Shoulder Type
7.9 - 9.7	1.8	12	4	gravel
11.3 - 11.6	0.3	12	2	gravel
16.3 - 19.7	3.4	12	1	gravel
19.7 - 31.9	12.2	12	3	gravel
32 - 36	4	12	2	gravel
39.5 - 44.6	5.1	12	2	gravel
47.2 - 48.2	1.0	12	3	gravel
87.8 - 94.5	6.7	12	3	gravel
95.8 - 100	4.2	12	3	gravel
102.6 - 105.3	2.7	12	4	gravel

Table 3.6: Predicted Versus Actual Accident Rates – Route 3 : Hope to Princeton (Lane and Shoulder Conditions' Model)

Road Segment km	Length of Segment miles	AADT vpd	Number of Accidents	Predicted Accident Rate (A_c)	Actual Accident Rate (A_a)
7.9 - 9.7	1.12	5600	24	2.91	2.58
11.3 - 11.6	0.19	5600	4	3.34	2.58
16.3 - 19.7	2.11	5600	35	3.58	1.98
19.7 - 31.9	7.58	5600	153	3.12	2.42
32 - 36	2.49	5600	103	3.35	4.98
39.5 - 44.6	3.17	3200	59	2.04	2.22
47.2 - 48.2	0.62	3200	19	1.92	3.66
87.8 - 94.5	4.16	3200	93	1.91	2.70
95.8 - 100	2.61	3200	49	1.91	2.28
102.6 - 105.3	1.68	3200	19	1.77	1.38

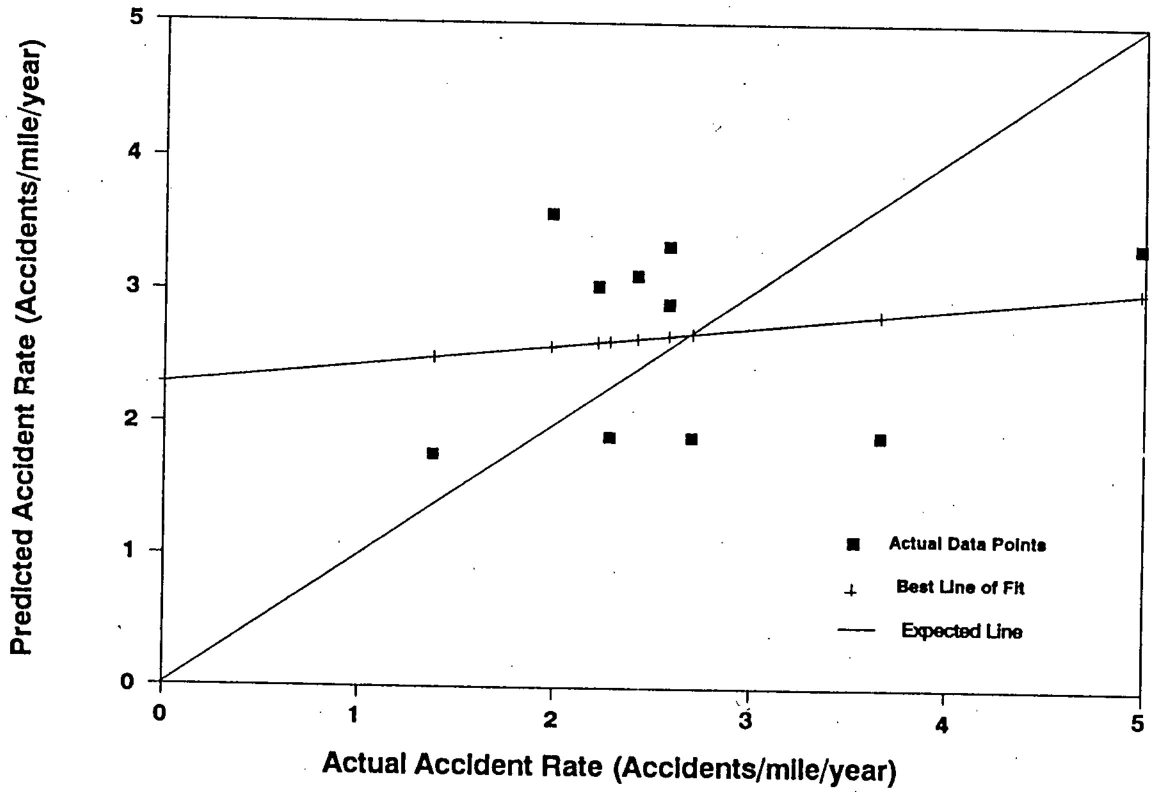

Figure 3.9: Predicted Versus Actual Accident Rate for the Lane and Shoulder Conditions' Model – Route 3 : Hope to Princeton

Table 3.7: Lane Width, Shoulder Width, and Shoulder Type of Segments – Route 3 : Princeton to Osoyoos

Road Segment km	Length of Segment km	Lane Width W (ft)	Shoulder Width UP (ft)	Shoulder Type
Princeton	to	Keremeos		
1 - 30	29	12	3	gravel
34 - 60	26	12	3	gravel
Keremeos	to	Osoyoos		
2 - 8	6	12	3	gravel
8 - 12	4	12	4	gravel
18 - 29	11	12	2	gravel

Route 99

Horseshoe Bay to Pemberton is the section of Route 99 selected for the study. The widths of the lanes and shoulders, the type of shoulder, and the lengths of the various segments are presented in Table (3.9). The distance from Horseshoe Bay to Pemberton is 137.8 km. From this length of highway, a total of 121.2 km of two-lane portions was obtained. columns 3 and 4 in Table (3.10) are plotted in figure (3.11).

Refinement

In this step, only the segments with a length of 5 miles or more were considered. Tables (3.11) and (3.12) summarize the results of this action in tabular form. The graphical representation is given in figure (3.12). The total length of two-lane highway in this summary analysis is 116.5 miles (187.4 km).

Table 3.8: Predicted Versus Actual Accident Rates – Route 3 : Princeton to Osoyoos (Lane and Shoulder Conditions' Model)

Road Segment km	Length of Segment miles	Traffic Volume (AADT) V	Number of Accidents in Segment	Predicted Accident Rate (A_c)	Actual Accident Rate (A_a)
Princeton	to	Keremeos			
1 - 30	18	4000	182	1.25	1.21
34 - 60	16	3700	180	1.17	1.34
Keremeos	to	Osoyoos			
2 - 8	3.73	4100	40	1.28	1.29
8 - 12	2.48	1400	5	0.50	0.24
18 - 29	6.84	1700	33	0.59	0.58

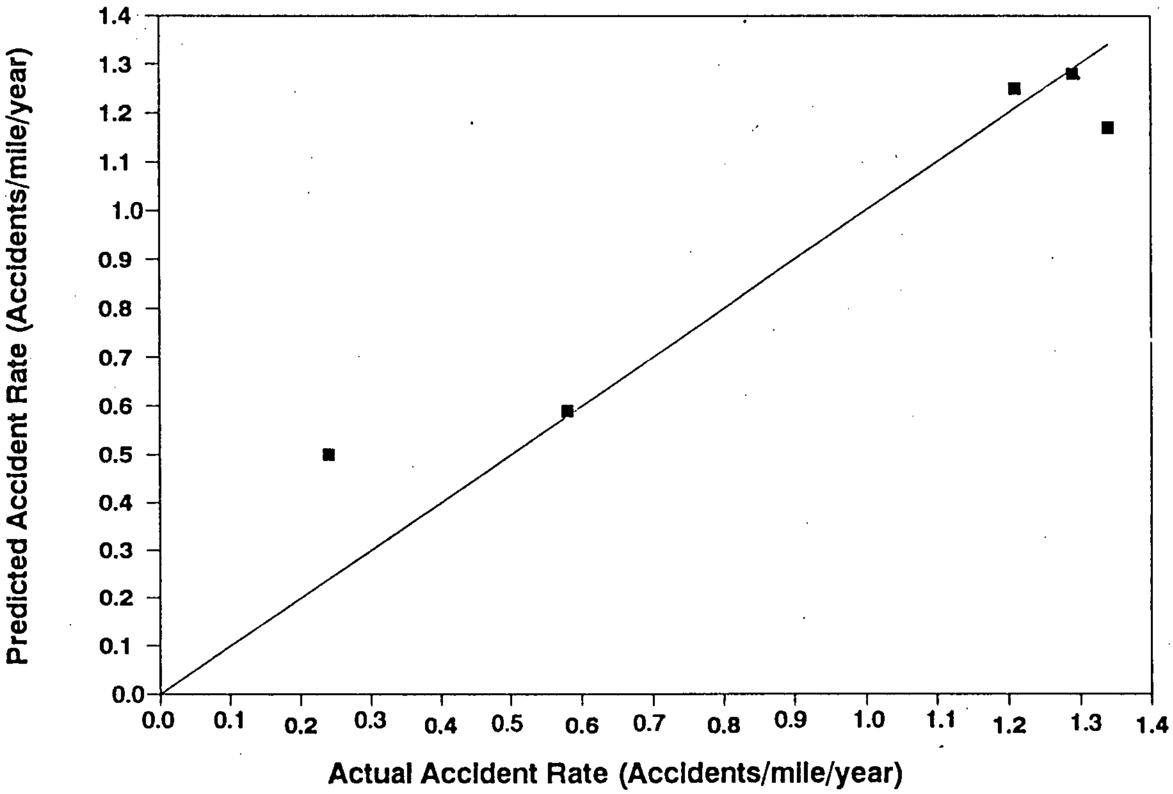

Figure 3.10: Predicted Versus Actual Accident Rate for the Lane and Shoulder Conditions' Model – Route 3 : Princeton to Osoyoos

Table 3.9: Lane Width, Shoulder Width, and Shoulder Type of Segments – Route 99 : Horseshoe Bay to Pemberton

Road Segment (km)	Length of Segment (km)	Lane Width (ft)	Shoulder Width (ft)	Shoulder Type
Horseshoe	Bay	to	Squamish	
5 - 30	25	13.6	2	gravel
31.3 - 33	1.7	12	2	gravel
35.3 - 36.1	0.8	12	0.5	gravel
37.5 - 38.6	1.1	12	0.5	gravel
39.1 - 41.1	2.0	12	0.5	gravel
42 - 43.7	1.7	12	1.5	gravel
Squamish	to	Pemberton		
1 - 2.8	1.8	12	2	gravel
2.9 - 11.4	8.5	12	1	gravel
14.7 - 56.2	41.5	12	1	gravel
56.3 - 91.1	34.2	12	1	gravel
91.1 - 94	2.9	12	1	gravel

Table 3.10: Predicted Versus Actual Accident Rates – Route 99 : Horseshoe Bay to Pemberton (Lane and Shoulder Conditions' Model)

Road Segment km	Length of Segment miles	AADT vpd	Number of Accidents in Segment	Predicted Accident Rate (A_c)	Actual Accident Rate (A_a)
Horseshoe	Bay to	Squamish			
5 - 30	15.5	6700	432	3.19	3.34
31.3 - 33	1.06	6700	45	3.92	5.12
35.3 - 36.1	0.5	6700	11	4.36	2.66
37.5 - 38.6	0.68	6700	19	4.36	3.34
39.1 - 41.1	1.24	5660	23	3.76	2.23
42 - 43.7	1.06	5660	50	3.5	5.69
Squamish	to	Pemberton			
1 - 2.8	1.12	2682	32	1.75	3.44
2.9 - 11.4	5.28	2682	121	1.88	2.76
14.7 - 56.2	25.8	2682	546	1.88	2.55
56.3 - 91.1	21.3	2500	192	1.76	1.07
91.1 - 94	1.8	1060	3	0.88	0.2

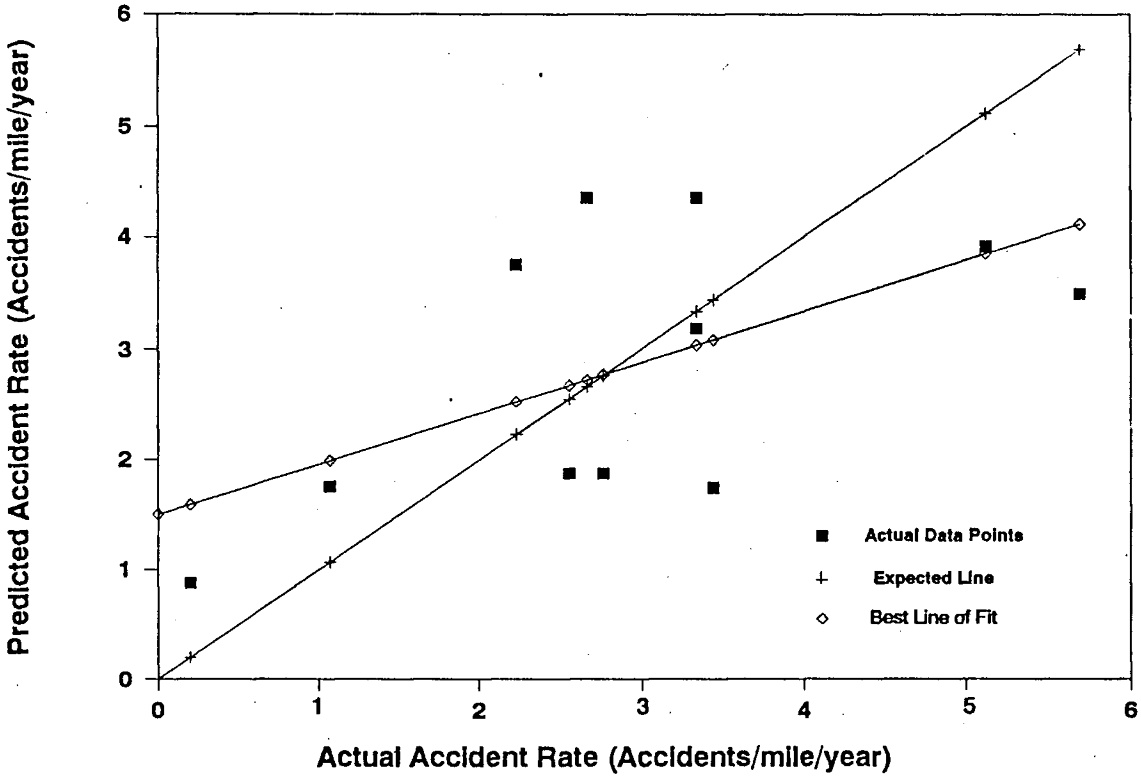

Figure 3.11: Predicted Versus Actual Accident Rate for the Lane and Shoulder Conditions' Model – Route 99 : Horseshoe Bay to Pemberton

Table 3.11: Lane Width, Shoulder Width, and Shoulder Type - All Segments of Length 5 miles or more

Segment Taken From Route	Segment Length (miles)	Lane Width (ft)	Shoulder Width (ft)	Shoulder Type
99	52.38	12	1	gravel
99	15.5	13.6	2	gravel
3	6.8	12	2	gravel
3	34.2	12	3	gravel
3	7.58	12	3	gravel

Table 3.12: Predicted Versus Actual Accident Rates - All Segments of Length 5 miles or more (Lane and Shoulder Conditions' Model)

Segment Taken From Route	Segment Length (miles)	AADT vpd	Number of Accidents in Segment	Predicted Accident Rate(A_c)	Actual Accident Rate (A_a)
99	52.38	2600	859	1.82	1.97
99	15.5	6700	432	3.19	3.34
3	6.8	1700	33	0.59	0.58
3	34.2	3850	362	1.22	1.27
3	7.58	5600	153	3.12	2.42

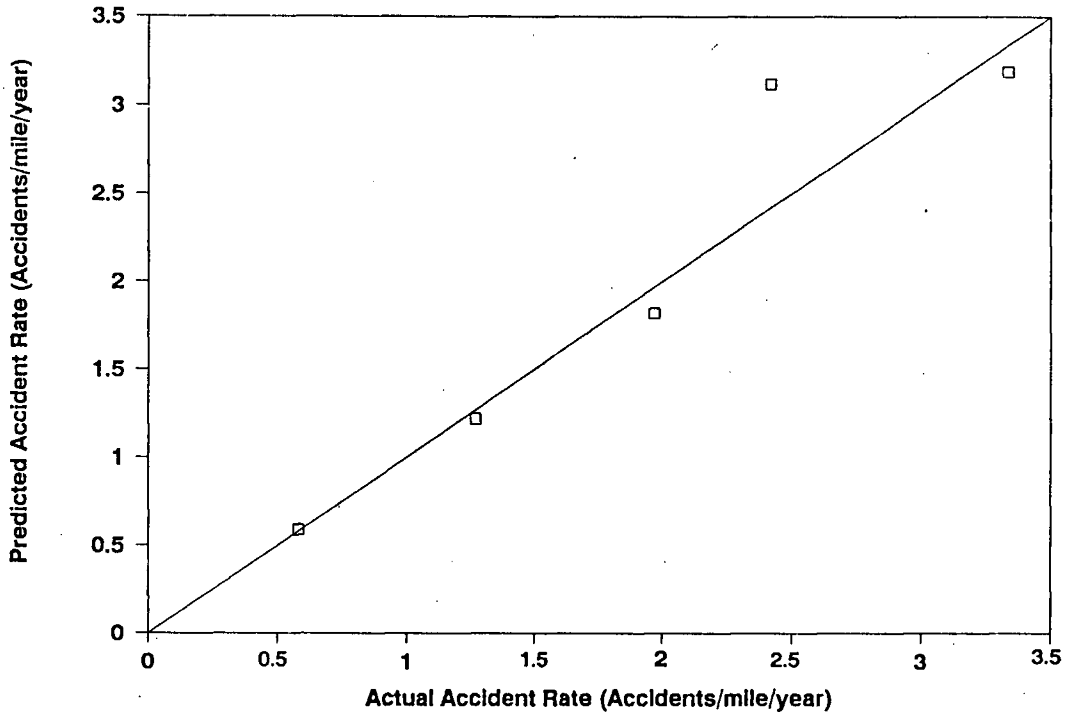

Figure 3.12: Predicted Versus Actual Accident Rate for the Lane and Shoulder Conditions' Model - All Segments 5 miles or more in length

3.2.2 Relationship Between Horizontal Curve Geometry And Safety

The quantitative model developed in the Special Report 214 to predict the incremental effect of horizontal curvature on accident rate is equation (2.2) reproduced below:

$$A = AR_s(L)(V) + 0.0336(D)(V)$$

The only property of the curve that appears in the model is the degree of the curve, D. The model is ideally applicable to segments of the roadway $1km$ ($0.61mile$) long or longer containing a single horizontal curve. However, due to the difficulty of obtaining a reasonably large sample of such isolated curves on the selected routes, highway segments of length $0.5km$ or more were selected for initial analysis. Also, all the curves were chosen from route 3, due to the difficulty of determining exact locations on route 99. Table (3.13) gives the degrees of curvature of the curves in the segments on route 3 (from Hope to Princeton) that satisfy the length criterion.

In table (3.14), average daily traffic volumes were used to calculate column 5, predicted number of accidents in the respective highway segments. A value of 1.4 was assumed for AR_s, the accident rate on straight portions of the roadway. A plot of the actual number of accidents in a segment against the predicted number of accidents in the same segment, using average daily traffic volumes in the horizontal curve model, is shown in figure (3.13). A linear regression analysis of the actual accident rate on the predicted accident rate produced an R^2 value of 0.52. A separate analysis was made for the segments that had a length of 1 km or more, that is, the segments that strictly satisfied the model's requirement for the length of a segment. Table (3.14) refers to these segments and figure (3.14) is the corresponding plot of the actual versus the predicted number of accidents. The R^2 value in this case is 0.89.

Table 3.13: Curve Data From Route 3 – Hope to Princeton

Segment begins (km)	Segment ends (km)	Length of Segment (km)	Length of Curve (ft)	Radius of Curve (ft)	Degree of Curve
4.2	4.7	0.5	348.9	1335.7	3
4.7	5.7	1.0	438.3	2864.6	2
5.3	6.3	1.0	410.3	1433.6	4
7.4	7.9	0.5	347.9	1432.3	4
7.9	8.5	0.6	302.2	1910	3
16.9	17.6	0.7	217.2	1910.9	3
20	21	1.0	1366.7	5730.3	1
21	21.7	0.7	403.3	718.6	8
22.3	22.8	0.5	414.2	1433.6	4
22.8	23.3	0.5	582.7	959	6
27.6	28.6	1.0	300.6	1147.4	5
33.4	34	0.6	371.7	956.8	6
94.9	95.9	1.0	77.7	881.5	10
102.8	103.9	1.1	625.8	1432.4	4
110.1	110.7	0.6	405.9	256.8	24
114.6	115.7	1.1	1644	2298.5	2.5
125.8	127	1.2	762.5	2864.8	2

Table 3.14: Actual and Predicted Number of Accidents on Segments containing single horizontal curves

Length of Segment (km)	Degree of Curve D	Traffic Volume (AADT) V	Actual Number of Accidents in Segment	Predicted Number of Accidents in Segment
0.5	3	5600	8	5.5
1.0	2	5600	15	9.6
1.0	4	5600	14	10.2
0.5	4	5600	7	5.8
0.6	3	5600	12	6.4
0.7	3	5600	8	7.2
1.0	1	5600	10	9.2
0.7	8	5600	8	9.0
0.5	4	5600	5	5.8
0.5	6	5600	10	6.5
1.0	5	5600	15	10.6
0.6	3	5600	6	6.4
1.0	10	3200	6	7.0
1.1	4	3200	5	6.4
0.6	24	3200	5	7.7
1.1	2.5	3200	7	6.1
1.2	2	3200	5	6.5

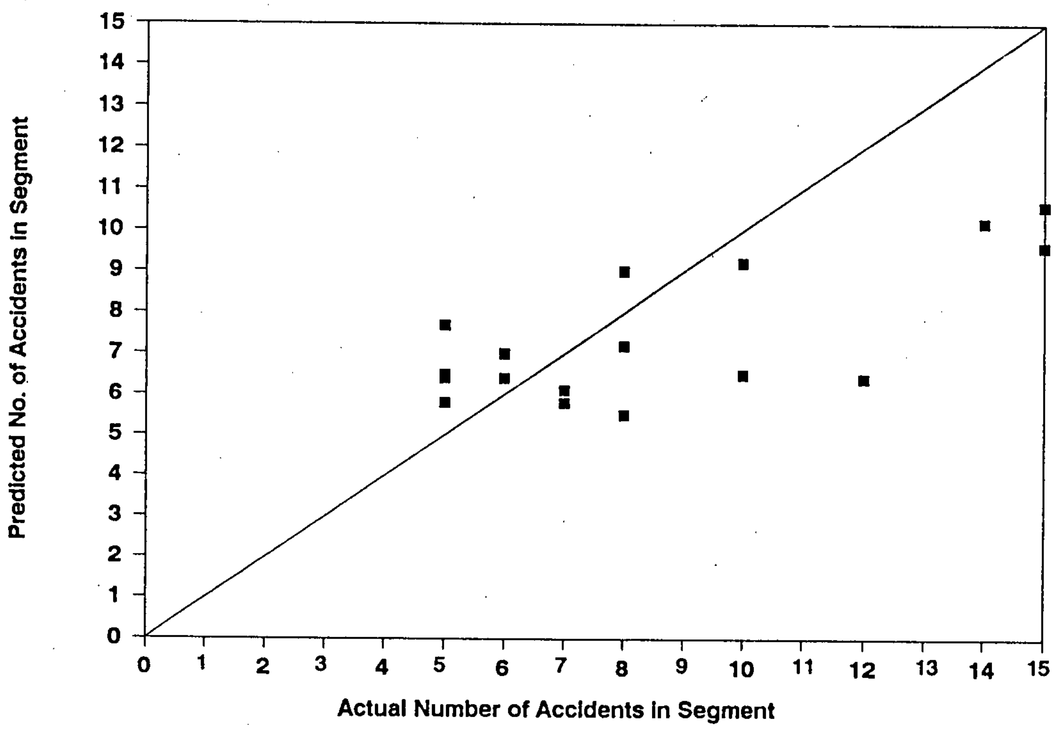

Figure 3.13: Predicted Versus Actual Number of Accidents for the Horizontal Curve Model – All Segments of Length 0.5 km or more

Table 3.15: Actual and Predicted number of accidents on Segments of length 1 km or more containing single horizontal curves

Length of Segment (km)	Degree of Curve D	Traffic Volume (AADT) V	Actual Number of Accidents in Segment	Predicted Number of Accidents in Segment
1.0	2	5600	15	9.6
1.0	4	5600	14	10.2
1.0	1	5600	10	9.2
1.0	5	5600	15	10.6
1.0	10	3200	6	7.0
1.1	4	3200	5	6.4
1.1	2.5	3200	7	6.1
1.2	2	3200	5	6.5

3.2.3 Relation Between Safety and Relative Bridge Width

Initially, geometric data on all forty two coded bridges on Routes 3 and 99, from Hope to Princeton and Horseshoe Bay to Pemberton respectively, were obtained. Half of the bridges did not reach the final candidacy stage for the analysis due to one or another of the following reasons :

- the bridge was on a multi-lane highway,
- the width of the approach travel lanes was less than the clear bridge width, or
- no accident data existed for the particular bridge.

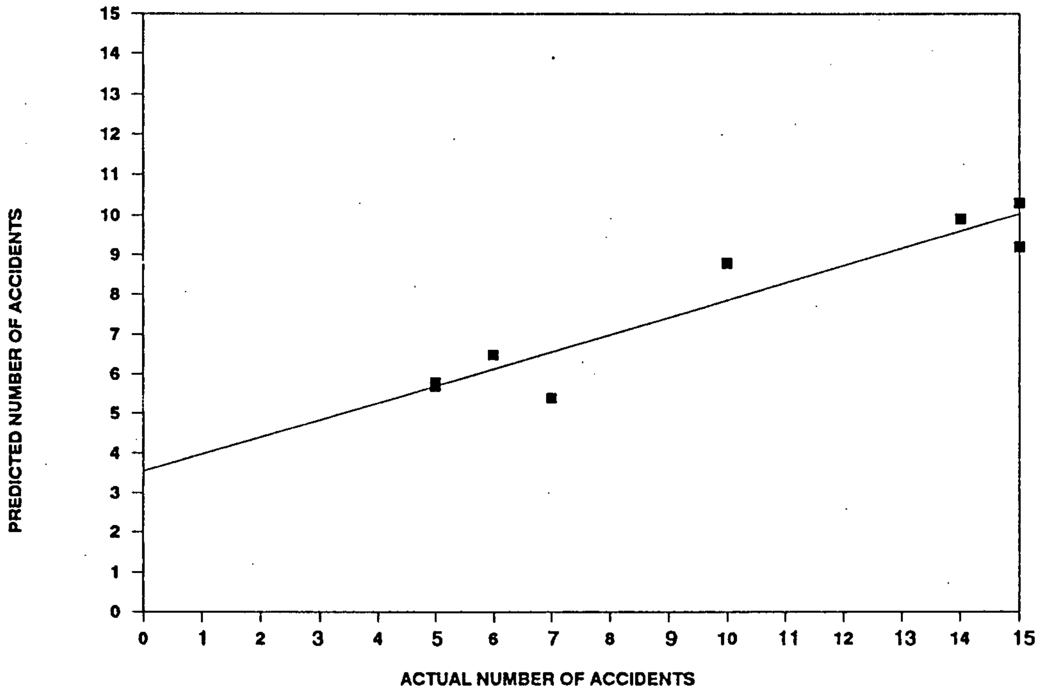

Figure 3.14: Predicted Versus Actual Number of Accidents for the Horizontal Curve Model – All Segments of Length 1 km or more

The relevant geometrical properties of the remaining bridges and their approach lanes are presented in table (3.16). The bridge model

$$AR = 0.50 - 0.061(RW) + 0.0022(RW)^2$$

was used to predict accident rates on the remaining bridges. Table (3.17) compares the predicted accident rates with the corresponding actual values measured in accidents per million vehicles. Figure (3.15) is the graph of the predicted accident rate on the bridges against the actual rates.

Table 3.16: Raw Bridge Data

Bridge Code Number	Clear Bridge Width (ft)	Lane Width (ft)	Relative Bridge Width (ft)
1208	23.67	23.42	0.25
1268	23.67	23.42	0.25
1215	23.67	23.42	0.25
1216	26.1	23.42	2.68
1217	23.67	23.42	0.25
1218	26	23.42	2.58
1219	23.67	23.42	0.25
1225	29.67	23.42	6.25
1446	30	27.3	2.7
1433	30	27.3	2.7
1457	32	27.3	4.7
1286	24	23.42	0.58
1626	36	23.42	12.58
1455	36	23.42	12.58
1011	24	23.42	0.58
2002	28	23.42	4.58
1029	32	23.42	8.58
2214	32	19.6	12.4
2519	32	19.6	12.4
747	32	23.42	8.58
2244	32	23.42	8.58

Table 3.17: Predicted and Actual Accident Rates on Bridges

Bridge Code Number	Rel. Bridge Width (ft)	Traffic Volume (AADT) V	Number of Accidents on Bridge	Predicted Accid. Rate (PMV)	Actual Accid. Rate (PMV)
1208	0.25	5600	1	0.48	0.10
1268	0.25	5600	0	0.48	0
1215	0.25	5600	0	0.48	0
1216	2.68	5600	1	0.35	0.10
1217	0.25	5600	0	0.48	0
1218	2.58	5600	0	0.36	0
1219	0.25	5600	0	0.48	0
1225	6.25	3200	3	0.22	0.52
1446	2.7	6700	0	0.35	0
1433	2.7	6700	1	0.35	0.08
1457	4.7	6700	3	0.26	0.25
1286	0.58	6700	0	0.47	0
1626	12.58	5660	1	0.08	0.1
1455	12.58	5660	1	0.08	0.1
1011	0.58	5660	1	0.47	0.1
2002	4.58	5660	3	0.27	0.29
1029	8.58	2682	6	0.14	1.23
2214	12.4	2682	3	0.08	0.61
2519	12.4	2682	0	0.08	0
747	8.58	1020	0	0.14	0
2244	8.58	1020	0	0.14	0

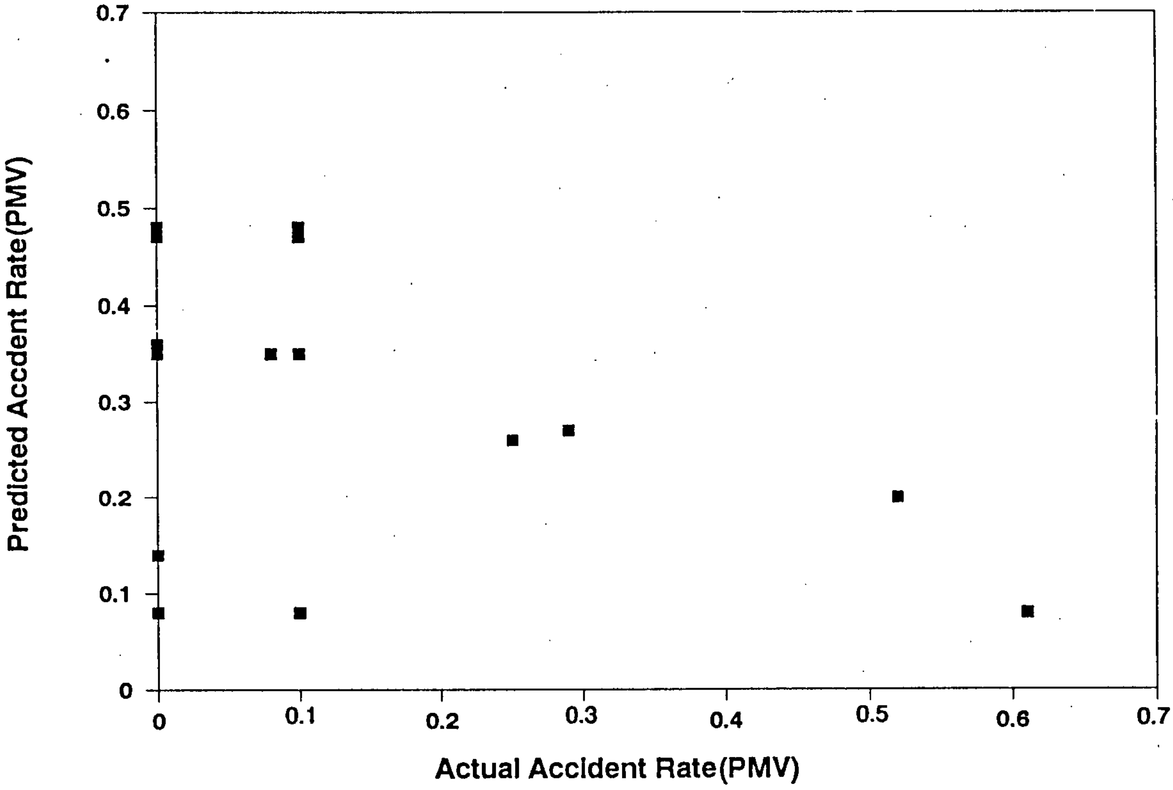

Figure 3.15: Predicted Versus Actual Accident Rate Using the Bridges' Model

Chapter 4

DISCUSSION OF RESULTS AND RESEARCH LIMITATIONS

4.1 Discussion of Results

4.1.1 Lane Width, Shoulder Width, and Shoulder Type

Route 3, Hope to Osoyoos, was split into two sections : one section from Hope to Princeton and the other from Princeton to Osoyoos. This division was necessary due to the apparent difference in the lengths and distribution of two-lane segments in the two stretches. The Hope to Princeton section of the highway had relatively short two-lane segments interspersed with multi-lane segments (the longest continuous two-lane segment was 7.58 miles long). In contrast, the Princeton to Osoyoos section had two-lane segments of over 15 miles long.

Interestingly, but not surprisingly, the accident rates predicted by the model were strikingly different, with respect to their closeness to the actual accident rates, for the two sections. Regression analyses of the predicted accident rates on the actual accident rates were performed for each section. For each section, the resulting R^2 value and the linear regression equation are recorded below.

Route 3 : Hope to Princeton

$$R^2 = 0.04$$

$$A_p = 2.30 + 0.14A_a$$

Route 3 : Princeton to Osoyoos

$$R^2 = 0.94$$

$$A_p = 0.26 + 0.75A_a$$

Route 99 : Horseshoe Bay to Pemberton

$$R^2 = 0.34$$

$$A_p = 1.5 + 0.46A_a$$

All Segments 5 miles long or longer

$$R^2 = 0.90$$

$$A_p = 0.01 + 1.03A_a$$

$$\longrightarrow \mathbf{A}_p \approx A_a$$

It must be recalled that the models being investigated in this research are only applicable to two-lane rural highways. When a 1 km long two-lane segment is sandwiched between two multi-lane segments, it is reasonable to suspect that the two-lane segment will not behave in a way similar to, as far as safety is concerned, a continuous two-lane segment of, say, 10 miles or more in length. The results of the regression analysis for the Hope to Princeton section of Route 3 and Horseshoe Bay to Pemberton confirm this suggestion of intuition. Only 4 percent of the variation in the values of the predicted accident rates is accounted for by variations in the actual values. The weakness of the model's applicability to safety on the two routes is seen in the large constant values of 2.3 and 1.5 in the regression equations for the two sections.

The regression of the predicted accident rate on the actual accident rate accounts for 94 percent of the variation in the values of the predicted accident rate on Route 3, Princeton to Osoyoos, and 90 percent of the variance of the predicted accident rate on all segments of length 5 miles or more. The model has a strong predictive capability for all two-lane highway sections of at least 5 miles in length on both Routes 3 and 99, as evidenced by the very low constant value (0.01) in the regression equation and the high R^2 value. Thus, safety effects of lane and shoulder width and shoulder type on two-lane sections of Route 3 (Hope to Osoyoos) and Route 99 (Horseshoe Bay to Pemberton), can be estimated by equation (2.1) reproduced below :

$$A = 0.0019(ADT)^{0.882}(0.879)^{W}(0.919)^{PA}(0.932)^{UP}(1.236)^{H}(0.882)^{TER1}(1.322)^{TER2}$$

The two-lane sections of the highway to which the model is applicable must be at least 5 miles long. The model predicts that widening lanes from 10 ft to 12 ft reduces accidents by 23 percent while widening unpaved shoulders from 2 ft to 6 ft reduces accidents by 25 percent. Paving an existing 6 ft unstabilised shoulder reduces accidents by 8 percent.

4.1.2 Horizontal Curves

When all segments of length 500 meters or more were considered, an R^2 value of 0.52 resulted from the regression of the predicted number of accidents in a segment on the actual number. However, when the segments were screened to include only those segments which were 1 km long or longer, the R^2 value jumped to 0.89. It could, therefore, be said that the accident frequency on a two-lane segment of nominal length 1 km containing a single horizontal curve can be estimated by equation (2.3).

$$A = AR_s(L)(V) + 0.0336(D)(V)$$

This relationship predicts that, as degree of curvature decreases, the number of accidents at a curve also decreases by about 3 accidents per degree of curvature for each 100 million vehicles passing through the curve. Flattening a sharp curve on a road carrying 6000 vehicles per day eliminates about 3 accidents every 8 years for each reduction in curvature of 5 degrees.

4.1.3 Bridge Safety

As evidenced by figure (3.15), there does not seem to be any mathematically well defined relationship between the actual rate of bridge-related accidents and the rate predicted by the quantitative model reported in Special Report 214.

4.2 Research Limitations And Assumptions

The following are some of the main limitations encountered and assumptions made in this study:

1. For most sections of the study area, actual traffic volume counts were done only for the months of July and August. Average annual traffic volumes were obtained by using the ratios of July - August traffic volumes to annual traffic volumes at permanent count stations. The selection of a permanent count station for each short count station was based on regression analysis of traffic counts done in each short count station against traffic counts in each permanent count station for the same time periods. The time intervals for the traffic counts for these regression analysis were typically in the range of 100 to 400 hours. The permanent count station whose hourly traffic counts best correlated with the corresponding traffic counts of a short count station was selected.

   The R^2 values for these regression analyses were typically between 0.85 and 0.95. Whereas it could be said that these were good R^2 values, the average annual traffic volumes resulting from their usage are still *only estimates* of the actual volumes.

2. The selection of values for $TER1$, $TER2$, and H in equation 2.1 were based largely on subjective reasoning. To assume a value of 1 for $TER1$ and a value of 0 for $TER2$ is, strictly speaking, to say that every section of the study area is mountainous. This, obviously, cannot be supported. However, since a detailed site investigation could not be carried out to assign values of 0 or 1 to individual segments of the highways, the

assumption that, on the average, the study area is in a mountainous terrain is a reasonable one. Similar arguments hold for the values of 5 and 6 assumed for the roadside hazard rating for Route 3 and Route 99 respectively.

3. As-constructed drawings were used to locate horizontal curves on the routes. Some of these drawings date back to the 1950's and the measurements were in imperial units. It was difficult to locate some curves with any appreciable degree of accuracy. This arose due to the fact that some reconstruction works might have been done in some portions of the highway resulting, in some cases, in reduction or lengthening of local highway sections. It was difficult to ascertain whether all such reconstruction works had been documented or not. It was for this reason that Route 99 (Horseshoe Bay to Pemberton) was not used for the analysis of horizontal curve accidents. Several reconstruction works had been done on this route to the extent that the ability of the original drawings to represent current conditions on the highway could not be guaranteed.

4. Accident rate on straight portions of two-lane rural highways in British Columbia was not available in the literature. This value, represented in symbols by AR_s, was needed in the horizontal curve model (equation 2.3).

   Accident rate on straight portions of Route 3 (Hope to Osoyoos) for the period 1981 to 1985 was used as an estimate for AR_s. This value was determined by taking several straight sections of the route and using the accidents on these sections and the section lengths. A value of 1.4 was obtained for AR_s.

5. One of the models this study set out to work on was the utility pole model developed in the Special Report 214 of the Transportation Research Board (*equation* 2.9). At the data collection stage, it was found out that even though accidents involving utility poles and other roadside hazards were recorded, the distances of the objects of impact from the edge of the travel lanes, denoted by y_i and y_j in the model, were not recorded. Without detailed site investigation, it was not possible to apply the model to the accident data.

6. Highway accidents are rare events and very few are expected to occur within a short section of a roadway in a short period of time. This was the case with bridge-related accidents in this study. The five-year data did not fit the model in any way. An attempt to assess the long term validity of the model using accident data for a period of at least 15 *years* failed because data for such length of time was not available.

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

Five quantitative models relating some geometric properties of two-lane rural highways to highway accident rates were reviewed in the study. The models, all reported in Special Report 214 of the Transportation Research Board, are summarized below.

- Relationship between lane width, shoulder width, shoulder type, and highway safety. Mathematically, the relationship can be written as :

$$A = 0.0019(ADT)^{0.882}(0.879)^{w}(0.919)^{PA}(0.932)^{UP}(1.236)^{H}(0.882)^{TER1}(1.322)^{TER2}$$

- Relationship between horizontal curvature of roadway and accident rate. This relationship can be written in symbols as :

$$A = AR_s(L)(V) + 0.0336(D)(V)$$

- Relationship between relative bridge width and rate of bridge-related accidents :

$$AR = 0.50 - 0.061(RW) + 0.0022(RW)^2$$

- **Relationship between the offset of a utility pole from the edge of the travel lanes and the rate of utility pole-related accidents :**

$$E_x(A_h) = \frac{0.07285}{23467}(ADT)^{0.5935}[\sum_{i=1}^{8} x_i e^{-0.08224 y_i} + \sum_{j=1}^{8} x_j e^{-0.08224 y_j}]$$

- **Relationship between accidents and sight distance at crest vertical curve. For a highway segment containing an isolated vertical curve, the accident model can be expressed as :**

$$N = AR_h(L)(V) + AR_h(L_r)(V)(F_{ar})$$

The meanings of the various symbols used in the models have been explained in chapter 2.

The model developed for utility pole accidents could not be applied due to the non-availability of values for the offsets of utility poles involved in accidents from the edge of the travel lanes, an important variable in the model. The model for predicting crest vertical curve accidents was hypothetically developed and its application would involve detailed site measurements beyond the scope of this study.

The remaining three models were applied to accident and road geometry data from Route 3 (Hope to Osoyoos) and Route 99 (Horseshoe Bay to Pemberton) in British Columbia. Accident rates predicted by each model were plotted against actual accident rates in various segments of the highways. The R^2 values resulting from regression analyses of these plots were used as the statistical measures of predictability for the models. The lane

and shoulder conditions' model produced an R^2 value of 0.90 while the horizontal curve model produced an R^2 value of 0.89. With these R^2 values, it can be concluded from this study that these two models could be used to predict accident rates in the two-lane sections of Route 3 (Hope to Osoyoos) and Route 99 (Horseshoe Bay to Pemberton). The plot of the actual versus predicted accident rate using the bridges model was a pure scatter with no apparent relationship.

The following recommendations are pertinent to future research in road safety in British Columbia.

1. The two models that fitted the data from the study area should be applied to data from a much larger sample of two-lane rural highways in British Columbia.

2. Offsets of utility poles and other roadside hazards involved in accidents from the edge of the travel lanes should be recorded in the police accident report and subsequently documented in the accident records of the safety division of the Ministry of Transportation and Highways.

3. Road inventories should be updated regularly as reconstruction works invalidates existing inventories.

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Appendix A

Table A1: Crosstabulation of Kilometre Mark Versus Accident Location
Time Period: Jan. 1, 1981 to Dec. 31, 1985
Squamish to Pemberton

Kilometre Mark	Accident Location					
	Unknown	At Xing	Between Xings	Bridge	Exit Ramp	Entrance Lane
	Frequency of Accidents					
2.6	0	0	2	0	0	0
2.7	0	0	3	0	0	0
2.8	0	2	4	0	0	0
2.9	0	4	3	6	0	0
3.0	0	3	4	0	0	0
3.1	0	0	1	0	0	0
3.2	0	1	0	0	0	0
3.6	0	0	1	0	0	0
3.7	0	11	3	0	0	0
3.8	0	2	0	0	0	0
3.9	0	5	0	0	0	0
4.0	0	2	0	0	0	0

Appendix B

Table B1 – Accident Histogram
Time Period: Jan. 1, 1981 to Dec. 31, 1985
Squamish to Pemberton

Landmark Description	Kilometre Mark	Histogram of Accident Frequency
	2.6	F I
N. Jct To Squamish	2.7	I I P
Washout Bridge 1029	2.9	I I I P P P P P P P P P P
	3.0	I I I P P P P
	3.1	P
	3.2	P
	3.3	
	3.4	
	3.5	
	3.6	P
Diamond Head Road	3.7	I I I I I P P P P P P P P P
	3.8	P P
	3.9	I I P P P
	4.0	P P

Appendix C

1981 - 1985 PERMANENT COUNT STATION EXPANSION FACTORS

STATION	DESCRIPTION OF COUNTER LOCATION	RATIO OF JULY-AUG. TO ANNUAL VOLUME(1)	AVERAGED AADT(2)
P-11-1N&S	Route 1, 0.6 km. west of Helmcken Road and 1.1 km. south of Thetis Lake Interchange north of Victoria	0.188 689	35,314
P-11-2N&S	Route 17, 0.6 km. north of Quadra Street at Royal Oak in Saanich	0.201 017	30,903
P-14-1N&S	Route 19, 5.1 km. south of Englishman River Bridge south of Parksville	0.227 448	10,910
P-15-1N&S	Routes 1A Bridge in Vancouver	0.178 462	59,142
P-15-2N&S	Route 1 at south end of Second Narrows Bridge in Vancouver	0.176 567	80,820
P-15-3N&S	Route 99 on Cheekeye River Bridge, 11.5 km. north of Squamish	0.179 668	2,682
P-16-1E&W	Routes 1A & 99 at south end of Patullo Bridge and west of Scott Road	0.170 109	65,306
P-16-2E&W	Route 1, 0.4 km. west of north end of Port Mann Bridge	0.181 105	68,469
P-16-2E		0.181 946	35,946
P-16-2W		0.185 196	33,274
P-16-3N&S	Route 99 on Oak Street Bridge north of off-ramp to Sea Island Way in Richmond	0.176 473	83,492
P-16-4N&S	Route 99 at south end of Deas Slough Bridge in Delta	0.183 976	68,130

Appendix D

MINISTRY OF TRANSPORTATION & HIGHWAYS
SHORT COUNT MATCH STATISTICS

SHORT STATION: 16- 95N

	1983			1984			1985		
	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS
1	P-16-4	0.8375	109	P-16-2W	0.8917	151	P-16-2W	0.9320	152
2	P-16-2	0.8227	109	P-16-1	0.8479	151	P-16-4	0.8833	152
3	P-16-1	0.8148	109	P-16-3AN	0.8335	151	P-16-1W	0.8597	152

SHORT STATION: 16- 95S

	1983			1984			1985		
	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS
1	P-15-2N	0.8554	152	P-16-2E	0.8200	153	P-16-2E	0.8546	152
2	P-15-2	0.8481	152	P-15-2N	0.8071	153	P-15-2N	0.8539	152
3	P-15-1	0.8429	152	P-15-2	0.8024	153	P-15-2	0.8418	152

SHORT STATION: 16- 96

	1983			1984			1985		
	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS
1	P-17-2	0.9011	138			0			0
2	P-11-1	0.8083	138			0			0
3	P-16-2	0.7886	138			0			0

SHORT STATION: 16- 96E

	1983			1984			1985		
	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS
1	P-17-2E	0.9621	150			0			0
2	P-11-1N	0.9055	150			0			0
3	P-16-4S	0.8778	150			0			0

SHORT STATION: 16- 96W

	1983			1984			1985		
	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS
1	P-17-2W	0.8478	138	P-17-2W	0.9045	149			0
2	P-11-1S	0.8382	138	P-11-2S	0.7482	149			0
3	P-16-2W	0.8315	138	P-17-2	0.7394	149			0

SHORT STATION: 16- 97

	1983			1984			1985		
	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS
1			0	P-25-1	0.9148	149	P-25-1	0.9305	149
2			0	P-17-2	0.8903	149	P-11-1	0.9112	149
3			0	P-11-1	0.8804	149	P-11-2	0.9083	149

SHORT STATION: 16- 97E

	1983			1984			1985		
	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS
1			0	P-25-1S	0.8748	149	P-11-2N	0.9193	149
2			0	P-21-1E	0.8600	149	P-25-1	0.9059	149
3			0	P-17-2E	0.8527	149	P-25-1S	0.8994	149

SHORT STATION: 16- 97W

	1983			1984			1985		
	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS	PERM	R*SQRE	HRS
1			0	P-11-2	0.8736	149	P-11-2	0.8535	149
2			0	P-25-1	0.8661	149	P-11-1	0.8416	149
3			0	P-11-1	0.8548	149	P-16-3A	0.8411	149