AN ABSTRACT OF THE THESIS OF

Aram Kornsombut for the degree of Master of Science in Civil Engineering presented on April 18, 1978

Title: Trade-offs between Geometric Design Standards and Safety for Low Volume Roads

Abstract approved: Dr. R. D. Layton

Geometric design standards have not been developed specifically for low volume roads. Design standards and criteria are extrapolated from higher standard roadways that are often not relevant to low volume roads. Safety is a primary criterion used for selection of design standards along with economic, financial, social and environment factors.

This study attempts to determine trade-offs of various design standards and designs based on safety and economic criteria. It also summarizes existing geometric design standards that are employed by federal, state and county governments. A methodology for selection of design standards for low volume roads employing safety criteria is developed. The study approach is based on data and relationships from previous research. Accident costs cited by National Safety Council are adopted for analysis. Procedures are developed to determine the balance between design standards and accident costs.

Ten-foot lane width and two-foot shoulder width are found to be reasonable for low volume situations. Horizontal and vertical alignments affect safety, vehicle running costs and travel times. Guardrail warrants of low volume roads differ from that of high volume roads. The balance and trade-offs between guardrail installation cost and accident
cost expected for a given embankment geometry are evaluated. The balance between a clear zone from the edge of pavement and the accident cost savings has also been developed.

It is recommended that a comparison between the accident savings due to roadway improvement and the incremental construction cost must be made to evaluate safety and economy effectively. An approach to evaluate the level of safety relative to design is developed for decision making.
Trade-offs between Geometric Design Standards and Safety for Low Volume Roads

by

Aram Kornsombut

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TRADE-OFFS BETWEEN GEOMETRIC DESIGN STANDARDS
AND SAFETY FOR LOW VOLUME ROADS

CHAPTER I INTRODUCTION

1.1 Problem Definition

Need

There are over two million miles of roads in the United States carrying less than 400 ADT, these include the roads under the responsibilities of United States Forest Service, County and other governmental agencies (75). There are over 30,000 agencies in the United States that are involved with low volume roads (75). Each year a significant portion of the country's gross national product is committed to the design, construction and administration of low volume roads. It is alarming to think of the continuing development of a low volume road transportation system which is based on extrapolated and sometimes irrelevant design criteria and standards.

The design standards for low volume roads are frequently inappropriate, quite subjective and can not be defended scientifically. Most of these standards are extrapolated, projected and rationalized from information developed for higher type roads. There is an urgent need to develop standards and criteria that specifically relate to the design and safety of low volume roads. Appropriate design standards should fully recognize the extreme cost sensitivity and safety criteria of low volume roads. The standards must be not only technically sound, but also cost effective and safe.

Practically, geometric design standards are selected based on considerations such as economy, environment and safety. The design standards of local road agencies are often governed by financial conditions.
Due to the lack of data and research on low volume roads, highways are currently being designed and constructed that contain safety hazards. To analyze this problem for low volume roads, it is essential to understand how geometric design standards correlate with safety.

Presently, in the United States, the safety criteria for low volume, low speed roads have evolved from four publications:

1) A Policy on Geometric Design of Rural Highways, AASHTO, 1965 (2),
3) Manual of Uniform Traffic Control Devices, FHWA, 1971 (76), and

The safety criteria contained in these publications may not be applicable or relevant for low volume, low speed roads because these roads are extremely cost sensitive. The trade-offs between design standards and safety require in depth research directed at them specifically.

To impose safety requirements on low volume roads that may be applicable to higher type roads is often wasteful of financial resources. However, ignoring safety requirements for low volume roads may waste the greatest resource of all, human lives.

1.2 Purpose

The purpose of this study is to investigate the trade-offs between design standards and safety. The specific objectives are:

1) To summarize the existing design standards for low volume roads.
2) To identify safety problems associated with low design standards.

3) To develop methodology for evaluating trade-offs between design standards and safety.

1.3 Scope

This study relies on data available from the literature and previous research studies. Roadway elements have been specified excluding the intersections. Design standard information is obtained from a variety of engineering and statistical sources. Accident costs are based on those determined by the National Safety Council. Vehicle running costs are calculated from relationships developed by Winfrey (66) and Oglesby (50). In Chapter V, updated costs are developed by multiplying with an inflation factor assumed as 2 percent per year. The accident data that are available in Oregon provides little useful information toward geometric design standards due to the way it is categorized. The statistical data available from other research on low volume roads is used for analysis in this study.

Chapter III outlines the concept of trade-off analysis. Existing geometric design standards have been summarized and then evaluated in Chapter IV. Chapter V and VI include the procedures of trade-off analyses and criteria for optimizing level of safety. Conclusions and recommendations have been made in Chapter VII.
1.4 Important Variables and Interrelationships

A number of important variables for the analysis of safety design trade-offs can be isolated and defined. Those variables of the greatest importance for low volume roads include:

(1) **Traffic**; Defined as the volume of traffic that would use a new or improved highway as soon as it is opened to traffic. A low volume road is defined as a road carrying current traffic under 400 vehicles per day (51).

(2) **Pavement**; Although the total depth of new or improved materials comprise the pavement section, the wearing surface of the roadway is of primary concern for safety analysis and design. Crown is the cross slope to provide good drainage of the surface, usually in percent. The crown slope affects the ability of a driver to steer his vehicle on a particular crown slope and maintain his lane position. Although the crown slope is introduced for drainage purpose, the design practices must consider safety of vehicles, especially for automobiles or trucks with a high center of gravity which would be more easily overturned.

(3) **Lane width**; Lane width must be adequate to accommodate the vehicles that use it. The width of lane must provide adequate room for the driver to locate himself comfortably within the lane. The stress caused by driving on a narrow roadway results in fatigue and may lead to loss of confidence or alertness. However, with the possibility of wider commercial vehicles, lane width may become a primary safety consideration and should not be ignored.

(4) **Shoulder width**; The portion of the roadway contiguous with the traveled way used to accommodate stopped vehicles for emergency
uses and for lateral support of base and surface courses. Shoulder width has been found to influence accidents on the roadway (13, 10).

(5) Sideslope; A term denoting the slope area adjoining the outer edge of the shoulder. Sideslope is of concern for safety when vehicle runs off the roadway. Accident severity depends upon the steepness and conditions of the sideslope. Steep, unprotected, uncleared and short-clear area sideslopes result in serious accidents, especially for high embankment.

(6) Curvature; The degree of central angle subtended by 100 feet of curve segment. Degree of curvature is primarily concerned with speed and sight distance, particularly in the cut section. The driver must have sufficient distance to perceive and react to unsafe conditions that are upcoming. Degree of curvature, stopping sight distance and speed are strongly related to each other. Sharp curves have higher accident rates than flatter ones (54). The number or frequency of curves also affects safety (9).

(7) Sight distance; The length of highway ahead that is visible to the driver. The ability to see ahead is of the utmost importance for both safety and efficient operation of a highway. The minimum stopping sight distance may be defined as the length of highway required to bring a vehicle to a stop from various design speeds where the eye of the operator is 3.75 feet above the pavement and the object causing the stop is 0.5 feet above the pavement (2).

(8) Gradient; Gradient of the vertical alignment has less effect than horizontal alignment on safety (54). Although modern highways typically are designed with flat grades, grades still contribute to the occurrence of collision accidents (62). The combination of downgrade and a
sharp horizontal curve is deemed to create a very hazardous situation (17). Likewise, vertical curves have been cited historically to be prime collision locations due to the lack of sight distance (62). But, the accidents on vertical curves have been reduced because improved design standards have been adopted.

1.5 Study Framework

Figure 1 represents the study framework. This study is an attempt to determine the trade-offs between various design standards and accidents. Horizontal alignment, vertical alignment, cross-section elements, embankment geometry and speed are the conditions taken into account. Primarily, selection of design standards must be based on design speed, terrain, traffic volume and road class.

Considerations of construction and maintenance costs are not included in the scope of this study. The influence of horizontal and vertical alignment on vehicle operating costs and travel times are analyzed. Engineers must analyze both highway costs and accident costs to obtain the optimum design standards for any terrain condition. The general criteria for selection of design standards is to minimize total costs and minimize total accidents.

1.6 Relationships between Design Standards and Accidents

Figure 2 gives an example of the relationship and trade-offs between design standards and safety. The combined vertical and horizontal alignment conditions must be evaluated. A steep upgrade would result
Figure 1  Study Framework
Figure 2  Relationship Between Combined Vertical and Horizontal Alignment and Accident Level
in low speeds for trucks and reduced speeds for passenger cars. Horizontal curvature that accommodates much higher speeds is not justified based on economic considerations since speeds are limited by grades. So a balance between vertical and horizontal alignment is suggested, implying that horizontal curvature that is compatible with the speeds dictated by upgrades be used. Steep upgrades combined with a high degree of curvature result in low construction cost but high operating and maintenance costs for trucks and passenger cars. The operation of vehicles on the downgrade must also be considered. The most hazardous conditions for horizontal and vertical alignment occur with steep downgrades and severe horizontal alignment. Therefore, the accident costs would also be high for these conditions. Flatter horizontal alignment, even though steep grades are retained, would reduce accidents significantly. The relationships of speed and roadway geometrics to accident involvements and horizontal alignment are included in Chapter II and V.

As shown in Table 1, if grade is lower and the curve is flatten to provide higher standards, it results in high construction cost but low maintenance and operating costs. Accident rates are also lower. These trade-offs between design standards, safety and highway cost must be evaluated and optimized to obtain compatible design standards for low volume roads.

As the alignment standards are modified, concommitant changes in the cross-section occur. For example, higher design standards of flatter grades and flatter curves typically results in deeper cuts and fills. Consequently, accident costs related to cross-section elements are also affected, as shown in Table 2. Speeds are increased with higher standards so the severity of accidents involving roadside hazards is also
Table 1  Relationships between Combined Horizontal and Vertical Alignment, Highway Costs and Safety Costs.

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<th>Maintenance Cost</th>
<th>Operating Cost</th>
<th>Safety Costs Related to Alignment</th>
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* Mod. = Moderate

Table 2  Relationships between Cross-section and Accident Severity.

<table>
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<th>Fill Height: Accident Severity</th>
<th>Need for Guardrail</th>
<th>Obstacle Severity with respect to Speed</th>
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1.7 Procedure of Analysis

The approach taken in this study includes:

(1) User evaluation of current design and safety practices.

(2) Estimation of probability of accidents for low volume roads.

(3) Determination of the effect of curvature on safety due to sight distance restrictions.

(4) Analysis of effect of horizontal and vertical alignments on vehicle running costs and travel time.

(5) Determination of effects of embankment height and slope on safety and guardrail installation for low volume roads.

(6) Determination of the trade-off between right-of-way and accidents.

(7) Development of a decision framework.

Relationships between various design standards and accident rates have been obtained from a literature review. These accident rates are employed as measures of safety for each situation analyzed.
2.1 Effects of Combined Geometric Standards on Accidents

Research that attempts to define the relationships between the geometric design standards of the roadway and safety by predicting the accidents per mile or per million vehicle-miles is extensive\(^{(18,27,35)}\). Limited research has attempted to determine the standards that should be used in design and the corresponding safety criteria. This is partly due to lack of available data for analysis. Very little was done on low volume roads because limited accident data are available. Accidents on low volume roads are the rare events. Important research efforts that relate accident rates to roadway design elements are summarized in this section.

Dart and Mann \(^{(18)}\) concluded in their analysis of relationships between rural highway geometrics and accident rates in Louisiana that only 46 percent of the variation in accident rates is explained by the geometric factors studied. A multiple regression analysis was performed on data from Louisiana rural highways. The geometric factors included in the study are percentage of trucks, traffic volume ratio, lane width, shoulder width, pavement cross slope, horizontal alignment, vertical alignment, percentage of continuous obstructions, marginal obstruction per mile and traffic access points per mile. This study indicates that 54 percent of the variation in accidents is caused by the driver, the vehicle or other variables that were not included in their study. The two variables appearing to have the most effect on accident rates are pavement cross slope and the number of traffic conflicts per mile. Their
research produced the following hierarchy of relative importance of geometric variables to accident rates: traffic volume and pavement cross slope, traffic conflicts and traffic volume, lane width and traffic conflicts, traffic volume and horizontal alignment, shoulder width and horizontal alignment, and traffic volume and percentage of trucks. They developed a mathematical model that attempts to predict the accidents likely to be experienced on a roadway. The following is the model for total accidents per 100 million vehicle miles. The coefficient of multiple determination ($R^2$) is 0.46. This value gives the impression of how great the association between a dependent ($Y$) and independent variables ($X_n$). The higher $R^2$ value, the closer they relate ($0 < R^2 < 1$).

$$Y = 41.32 - 1.23X_1 - 0.54X_2 - 0.67X_6 + 0.03X_1X_2 + 0.03X_2X_6 + 0.026X_2X_11 - 0.12X_4X_11 + 0.009X_5X_9$$ \hspace{1cm} (1)

where:

$Y =$ Total accidents per 100 million vehicle miles,

$X_1 =$ Percentage of trucks,

$X_2 =$ Traffic volume/capacity ratio,

$X_6 =$ Cross slope,

$X_1X_2 =$ (Percentage of trucks) (Volume/capacity ratio),

$X_2X_6 =$ (Volume/capacity ratio) (Cross slope),

$X_2X_9 =$ (Volume/capacity ratio) (Horizontal alignment),

$X_2X_{11} =$ (Volume/capacity ratio) (Traffic conflicts),

$X_4X_{11} =$ (Lane width) (Traffic conflicts) and

$X_5X_9 =$ (Shoulder width) (Horizontal alignment).

Gupta and Jain (27) studied the effect of certain roadway characteristics on accident rates for two-lane, two-way roads in Connecticut
using multiple linear regression analysis. The resulting coefficient of determination were quite small (between 0.003 and 0.073). Of the four geometric characteristics considered; i.e., pavement width, shoulder width, horizontal curvature, vertical clearance and sight distance restrictions, accident rate is better correlated with the restricted sight distance rating than the others. As restricted sight distance rating increases, accident rate decreases. That is, accident rate is higher on roadway sections that have inadequate sight distances. If safety is to be built into highway, the design must provide sight distances of sufficient length to give drivers enough time and distance to make the speed and distance judgments required for vehicle control.

The next best correlation with accident rate in their study was obtained for the horizontal curvature rating. The lower the degree of curvature of a curve was, the higher the rating was, and hence, the lower the accident rate was. However, the statistical regression equations obtained from their analysis are not significant. For example, only 9.75 percent of the variation in accident rate is explained by the geometric characteristics included in their study for the section with 0 to 1,400 ADT. Therefore, they suggested that sections with poor sight distance and sharp horizontal curvature should be given top priority in highway safety improvement programs.

Kihlberg and Tharp (35) of the Cornell Aeronautical Laboratory, investigated rates of accidents related to specific geometric features, such as number of lanes, access control, median presence, highway curvature, gradient, ADT and the presence of intersections or structures. From these data a mathematical model was developed. The model is valid
only for the data considered; i.e., Connecticut, Florida and Ohio. They found that (a) curvature, gradient, intersections were strongly correlated with safety with the least significant being gradient and (b) the combination of these elements results in accident rates higher than those of individual elements.

Spark (59) studied the influence of highway characteristics on accident rates on the rural state highway system of the State of Oklahoma, excluding intersection accidents. Statistical analysis indicated that roadway characteristics are responsible for at least 45 of the variation in the accident rate and that the remaining 55 percent of the variability are attributable to driver and vehicle factors. This supports the analysis of Dart and Mann (18).

Many research efforts that have used statistical regression analysis to develop the accident-geometric features relationships have shown poor coefficient of determination. This might be due to the randomness of the available data. However, individual geometric factors, such as sight distance, curvature, pavement cross slope, lane width, shoulder width and gradient, are found to be strongly related to accidents.

2.2 Effects of Individual Variables on Accidents

Speed

Moore (45) found in his research that the accident severity rate is relatively unchanged up to 50 MPH, and then increases rapidly above 50 MPH. This relationship is shown on Figure 3, including the 95 percent
Figure 3  Accident Severity versus Speed

Source: Reference 45

Figure 4  Involvement and Injury Rate versus Travel Speed (Daytime)
Source: Reference 58
Figure 5  Variation of Involvement Rate from Average Speed (Daytime)

Source: Reference 58.
Solomon (58) attempted to relate daytime rates of accident involvements to travel speed on two and four-lane rural road sections. Accident rates were found to be highest at very low speeds, lowest at about the average speed, and increase again at the very high speeds, forming a V-shaped distribution. Those results for involvements and injuries are shown on Figure 4. Figure 5 demonstrates that the minimum accident involvement rate occurs for those travelling from the average speed to 10 MPH above it. This supports the speed zoning concept that the least variation in speeds within the traffic stream provides the safest situation (25, 39).

Researchers at Texas A&M developed a curve using Solomon's findings, relating the accident involvement rate of truck to the speed reduction from the average of all vehicles on a highway (57). This curve is shown on Figure 6. The graph exhibits the sharp increase in involvement rate of trucks when the speed reduction exceeds 10 MPH. The Texas A&M study suggests that a differential of 10 MPH be used as a maximum acceptable level of safety. The AASHTO design policy uses a 15 MPH speed reduction as the criterion for the "critical length" of grade for alignment design to establish need for a truck climbing lane.

The relative accident involvement rates as determined from Figure 6 have been superimposed in Figure 7 on the AASHTO criteria for design of critical lengths of grade. The two sets of data on the same chart indicate the need for designs with small variation from the average speed of traffic on the road.
Figure 6  Accident Involvement Rate versus Speed Reduction from the Average of all vehicles on a Highway
Source: Reference 57.

Figure 7  Accident Involvement due to Speed Reduction as related to AASHTO Critical Lengths of Grade for Design
Source: Reference 2, 57
Speed is an important factor influencing the safety of a roadway. Design speed is based on the terrain and type of highways, but the selection of design speed should also consider safety. The concepts of speed zoning have been introduced for higher type highways. These concepts, speeds on both horizontal and vertical alignments are plotted on the same speed profile. The section which does not meet the requirement of the 10 MPH maximum speed difference between speeds on horizontal and vertical alignments must be improved or re-constructed. The principle employed in this updated design speed approach is the 10 MPH (15 KPH) maximum speed reduction rule suggested by Leisch (39) and Glennon (25). Application of these concepts lowers the accident rate, but the suitability and economy for low volume roads are questionable.

Lane Width

Dart and Mann (18) indicated in their study of accidents on rural roads that lane width is a significant factor in injury and fatal accidents and is relevant in wet-weather accidents. As illustrated on Figure 8, study findings verify present design practice in that as lane width decreases below 11 feet, the accident rate increases uniformly. Figure 8 also includes the data obtained by Charlesworth (18) and Raff (54). These studies also illustrate that accident rates decrease with increasing lane width on two-lane tangent sections. Figure 9 from a study by Raff (54) indicates the highest accidents between 9 and 10 feet lane width on two-lane curves.

Petty and Michael (53) analyzed accidents on county roads in
Figure 8  Accident Rate versus Lane Width on Two-Lane Tangent
Source: Reference 18, 54

Figure 9  Accident Rate versus Lane Width on Two-Lane Curve
Source: Reference 54
Indiana. They recommended that a major cause of accidents on county roads is narrow roadway, narrow shoulder and the absence of centerlines.

Research has shown that accident rate is affected by lane width. The lane widths of various classes of highways have been recommended in AASHTO "A Policy on Geometric Design of Rural Highways 1965" (2). A 12 feet lane width is generally recommended. However, for economy, the lane width should be lower than 12 feet for low volume roads. This decrease in lane width would not reduce safety appreciably.

**Crown Slope**

The crown on low volume roads is extremely important to provide surface drainage. For example, the road construction for low cost timber access roads (26) is primarily in steep terrain, so drainage and erosion are significant problems. For safety purpose as reported by Dart and Mann (18), Figure 10 shows a relationship indicating that roadways with relatively flat cross slopes are more accident prone than those with better slopes.

Figure 10 does not show the effect of cross slope that is greater than 2.5 percent since the crown normally lies between 2 to 3 percent for paved surface. An unpaved surface requires 4 - 5 percent to provide adequate surface drainage. The design cross slope should be related to the design vehicle for a roadway. For example, the high center of gravity vehicles for logging trucks may result in overturning if a steep cross slope is used.
Figure 10 Accident Rate Versus Pavement Crown Slope

Source: Reference 18
Belmont (10) investigated the relationship between injury accident rate and width of paved shoulders on California two-lane tangents. The accident rate tended to increase with shoulder width, for traffic volume above 2,000 vehicles per day. These are represented by the following equation and Figure 11.

\[ A = 0.234 + 0.00644 V + 0.00225 SV - 0.0286 S \]  \hspace{1cm} (2)

where, 

- \( A \) = The expected number of accidents,
- \( V \) = The average traffic volume, in hundreds of vehicle per day, and
- \( S \) = Shoulder width.

The coefficient of determination for the equation (2) is 0.946. Belmont concluded that shoulders 6-foot wide are safer than wider shoulders. This is similar to his previous findings (11). Also, shoulders 6-foot wide are safer than narrower shoulders. Wider shoulders might cause accidents due to unnecessary passing on shoulders. Figure 12 and 13 also illustrate the previous works of Belmont indicating the same results as mentioned above.

However, Belmont did not investigate the effects of unpaved or gravel shoulders which are extensively employed and suitable for low volume roads. Bellis (10) explained that low volume two-lane roads can serve satisfactorily without a shoulder because the function of the shoulder is closely related to the roadway capacity.

Oglesby and Altenhofen (51) found that accident costs on low volume roads are low compared to construction and operating costs. This
Figure 11  Accident Rate by Shoulder Width and Traffic Volume

Source: Reference 10
Figure 12  Observed Relation between Accident Rate and Shoulder Widths
Source: Reference 11

Figure 13  Theoretical Relation between Accident Rates and Shoulder Widths
Source: Reference 11
suggests that wide shoulder may not offer a particularly attractive economic gain.

Much research has been done on accidents on high-type highways with wide shoulders. Little research has been done on low volume roads. According to Figure 11, shoulder width between 2 and 4 feet seems to be reasonable for low volume roads because they make very little difference on accident rates.

**Alignment**

Low volume roads frequently require sharp curves and/or steep grades. In considering alignment design criteria versus cost, the designer must have sufficient information upon which to base design refinements for improved quality and safety.

Much research has been undertaken on horizontal and vertical elements in relation to highway safety (7,9,18,35,38,54). Researchers have taken two approaches; the correlation of alignment elements with accident experience and a review of basic assumptions used to derive alignment design standards.

**Horizontal Alignment:** Horizontal alignment is created by combining circular curves, tangents and transition spirals. Strongly related to the selection of curves is the rate of superelevation used. The choice of minimum radius of curvature is based on assumptions concerning drivers, vehicle and roadway characteristics, including friction factors and sight distance requirements.
Horizontal alignment features generally have been found to have a strong influence on accident experience. Figure 14 illustrates results of research studies of the relationship between accident rate and degree of curvature (38).

Investigations by Raff (54) on all types of highways showed a pronounced rise in incidents of accidents as sharpness of curvature increases. For two-lane road, the number of accidents per million vehicle miles increases by about 0.15 for each additional degree of curvature. Increase in traffic volume or ADT directly cause the increase in accident rates (18, 35, 38, 54). Other researchers showed similar results except Mullins and Keese (46) who concluded that the influence of horizontal curvature on accident frequency on freeways seemed to be negligible. Low volume roads normally may be designed for operation at speeds less than 50 MPH and have curvatures ranging from 2 degrees to more than 16 degrees, based on terrain. The relationships in Figure 14 indicate that an increase from 2 to 10 or more accidents per million vehicle miles may be experienced over this range of curvature.

Another attempt to relate the effect of highway curvature to accidents was carried out by Babkov (7). His results are shown on Figure 15. This indicates the sharp increase in relative number of road accidents if the degree of curvature is greater than 3°.

The relationship found by Raff (54) between degree of curvature and accident rates on two-lane rural highways is shown in Figure 16. Baldwin (9) found strong evidence, as shown in Figure 17, that accident rate declined with increases in curve frequency. The introduction of a relatively sharp curve in a section on which curves are
Figure 14 Accident rate related to Horizontal Curvature
Source: Ref. 17, 38, 46, 54

Figure 15 Relative number of road accidents versus radius of Horizontal Curves
Source: Ref. 7
Figure 16  Accident Rates on Curves for Two-Lane Highways
Source: Reference 54

Figure 17  Accident Rate Related to Horizontal Curvature and Frequency of Curves
Source: Reference 9
infrequent was found to present a hazard several times that of the same curve in a section where curves are frequent.

Horizontal curvature has a major influence on the safety of roadways. The effect of horizontal alignment on low volume roads on accidents must be understood to be able to evaluate accident costs and design trade-offs.

**Vertical Alignment:** Vertical alignment is created by combinations of tangent sections of a particular slope and vertical curves, usually of parabolic form at both crests and sags. The selection of rates of grade and length of curve depends upon assumptions concerning driver, vehicle and roadway characteristics and sight distance requirements. A number of studies have been made of the elements of vertical alignment alone and combinations of vertical and horizontal alignments to determine whether the above effects create hazardous conditions.

Raff (54) concluded in his study that

"On tangent highway sections there does not appear to be any relation between grade and accident rates. In these analyses the roads have been classified only by grade may have some effects on the accident rate when the appropriate other features are held constant."

Almost all of the studies have been done for high-type highways, little has been done for low-type highways. Some findings on high-type may be applicable to low-type roads. As part of the Crosstown Associates study (17), accident statistics for 126 sections of Chicago expressway containing grade were divided into nine groups and analyzed. Table 3 compares the means for these groups against that of the control sections,
TABLE 3 Comparison of mean accident rates for various management groupings of roadway sections containing grades.

( After Crosstown Associates (17) )

<table>
<thead>
<tr>
<th>Type of Road Section</th>
<th>Horizontal Condition</th>
<th>Vertical Condition</th>
<th>Mean Accident Rates (Accidents/MVM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control Section</td>
<td>Tangent</td>
<td>Level (less than 0.5 %)</td>
<td>1.10</td>
</tr>
<tr>
<td>Composite Section</td>
<td>All alignments (Tangented and curved)</td>
<td>All grades (Up and Down)</td>
<td>2.26</td>
</tr>
<tr>
<td>Straight Roadways</td>
<td>Tangent</td>
<td>Up grades</td>
<td>1.87</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Down grades</td>
<td>1.49</td>
</tr>
<tr>
<td></td>
<td></td>
<td>All grades (Up and Down)</td>
<td>2.21</td>
</tr>
<tr>
<td>Curved Roadways</td>
<td>Curved</td>
<td>Up grades</td>
<td>2.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Down grades</td>
<td>2.56</td>
</tr>
<tr>
<td></td>
<td></td>
<td>All grades (Up and Down)</td>
<td>2.40</td>
</tr>
</tbody>
</table>
of nearly level straight segments. The mean accident rates of those groupings of sections containing grade are approximately twice that of the straight, near-level sections. The difference in the means between each group and that of the control sections was found to be statistically significant, demonstrating the increased accident potential of both curvature and grades, especially downgrades. A positive correlation was found in this study between one important classification of grade condition and accidents, for which the resulting equation is:

\[ Y = 0.10 + 1.5734 \times X \]  

where,

\( Y = \) Mean accident rate on constant uniform upgrades on tangent section, and
\( X = \) Grade in percent.

**Combined vertical and horizontal alignments:**

Billion and Stohner (13) described that on two-lane highways with 20-foot pavement width, curve over 5° had 4.35 times as many accidents as the average section of highways. Sections with curves over 5° and grade over 5 percent had 19.27 times as many accidents. A possible explanation for part of this may be drawn from the work of Giles (21). Giles calculated the relative likelihood of certain features being associated with skidding accidents (see Figure 18). According to his analysis, curves over 5° and grades over 5 percent would have a relative liability of a skidding accident 23 times that of straight and level highway sections.

Raff (54) concludes that at low volumes the accident rate increases with increasing curvature while the effect of grade is not statistically significant.
Figure 18 Alignment Features and Skidding Accidents
Source: Reference 21
It is found that the combination of downgrade and curve created a higher accident rates than other sections. Limitations of grade conforming to design vehicle, terrain and costs should be carefully considered.

Pavement

For low volume roads, use of low-type pavements such as gravel surface, oil mat, penetration macadam and surface treatment, is deemed acceptable. Most automobile accidents involving skidding are due to the combination of a wet pavement and an attempt on the part of the driver to perform some maneuver, such as braking, cornering, accelerating, at a speed too high for the conditions.

Reliable data to support a comprehensive assessment of the role of slippery pavements and skidding in low volume road accidents are scarce. This is due partially to inadequate or confusing accident reporting systems (62). As indicated in Figure 19, a survey of 150 miles of unrestricted rural roads in England revealed a decreasing skidding accident rate for an increasing sideway force coefficient. The researchers concluded that as the skidding resistance of a road decreases, the risk of skidding in accidents increases. This situation is important in the Pacific Northwest due to the normal rainfall received. The skidding resistance of wet pavements is suggested as design criteria.

Pavement type is a factor affecting accidents on the roadways. Selection of the wearing surface is based on the relative economic benefits and costs with variations of traffic volume.
Figure 19  Skidding Accident Rate and Skidding Resistance (England)
Source: Reference 19.
Oglesby and Altenhofen (51) determined that the potential change in annual accident costs caused by improving a gravel surface to a paved surface is inconsequential in relation to other annual costs.
3.1 Basic Concept of the Trade-off Analysis

Highway geometric design standards must be set with traffic safety in mind. Horizontal alignment, vertical alignment, their combinations, cross-section elements and speed differential must be analyzed relative to the safety provided to determine the trade-offs. Trade-offs can be in the form of accidents or accident costs. Highway design standards are directly associated with highway cost; higher standards result in higher highway cost and a better level of service than for lower standard roads. Figure 20 presents the concept of the trade-off analysis for cost, safety and service. This figure illustrates the relationship between highway cost \( C \) and highway service \( F \). The level of service, \( F \), is defined in the standard engineering sense by speed and delay. The parameter, \( S \), is given by the attributable costs incurred in establishing the level of safety. The highway costs increase with an increase in service or speed of operation. In addition, highway costs increase when a higher level of safety is provided under the same service condition.

Conceptually, the trade-off problem is one of simultaneously satisfying multiple objectives for \( S, C \) and \( F \). However, practically the problem can be reduced into a single objective problem with constraints.

Suppose the objective is to maintain the cost within the bounds \( C_1 \) and \( C_2 \) with the constraint that the level of service should be
Figure 20  The Concept of Trade-off Analysis  
Source: Reference 70.
maintained between \( S_1 \) and \( S_2 \) and the level of service should be between \( F_1 \) and \( F_2 \). In addition, it is postulated that there is a bonding relationship between cost and service given by \( C = f(F) \). Under these circumstances the net benefits, in the form of accident savings from the level of safety \( S_2 \) to level \( S_1 \), which satisfy these constraints are shown by the shaded region, \( B_1B_2B_3B_4B_5 \). The constraint, \( C_1 \) is the maximum budget that can be provided for the particular bonding \( C = f(F) \). \( S_1 \) is the maximum safety justified or possible within reason. \( S_2 \) is the lowest safety acceptable. Several other interesting points can be noted. Point \( B_2 \) is the solution for highest level of safety; point \( B_5 \) is the solution of lowest cost and point \( B_4 \) is the point of diminishing returns. The point of diminishing returns indicates when less proportional increase in safety occurs with the increment of highway cost. Point \( B_3 \) indicates the highest level of service provided by the bonding relationship \( C = f(F) \), representing the cost to provide the service, and restricted by the maximum budget constraint, \( C_1 \).

3.2 Decision Framework

The selection of design standards for low volume road is a complex program for decision making. In addition to the safety considerations, economic, financial, environmental, social and political factors must be taken into account in the selection of design standards.

The decision framework on Figure 21 indicates the element needed in selection of design standards. According to this framework, changes of accident costs, due to changes in design standards, and changes of vehicle running costs, due to the effects of horizontal and vertical
Figure 21 A Decision Framework for Selection of Design Standards for Low Volume Roads
alignments, are used to calculate the expected loss in safety and operation.

In this study, safety and economy have been considered as the primary elements of the criteria to the trade-offs between standards and safety. Other factors rather than safety and economy, such as the road function must also be considered to determine the optimum alternative. The net benefits of safety and service to road users, landowner and the total community must be analyzed and evaluated for effective standards to be selected.
CHAPTER IV EVALUATION OF CURRENT DESIGN AND SAFETY PRACTICES.

4.1 Questionnaire

The county road system was selected for study since many of the roads supervised and constructed by local governments are low volume. Questionnaires were distributed to counties in the State of Oregon requesting the existing geometric design standards that they use on their county road system. No accident data was requested in the questionnaire since little accident data are collected by local jurisdictions that can be related to engineering roadway characteristics.

According to the questionnaire, only a few counties have their own design standards. Some use standards for design and construction that are based on the higher standards of the state or highway associations, such as AASHTO, NACE and Oregon State Highway Departments. Table 4 contains the design standards for various states and highway associations. The design standards identified by this questionnaire are in Table 5.

4.1.1 Geometric Design Standards for Local Counties

Design Speed. The returned questionnaires indicate a lower design speed on county road networks than for state or federal government standards. The average design speed is 41.3 MPH for flat terrain compared to 47.3 MPH for the state or federal level. One county implied
that design speed is not a factor in the road design.

**Degree of Curvature.** A maximum value of 38.2 degrees is used by Deschutes County, Oregon. Their standards do not specify the type of terrain. Many counties have arbitrarily selected the degree of curvature as determined by topography, so the standards of both state and county levels are comparable.

**Gradient.** The maximum grade employed is 20 percent for Marion County. This is higher than the average value of 11.6 percent for State standards.

**Pavement Width.** According to safety criteria as concluded in the literature review section, County standards are found to provide a reasonable lane width for low volume roads. Pavement width varies from 20 feet to 24 feet for 2-lane roads.

**Pavement Type.** A paved surface is generally designed by many of the counties in Oregon. Oil mat pavement is a popular one.

**Shoulder Width.** Shoulder width seems to be a significant design element. Every county responding recognized the function of shoulders. The standard widths vary from 2 feet to 5 feet with an average of 4.0 feet. Gravel surfaces are typically used.

### 4.2 Geometric Design Standards for Federal, State, Government level and National Organizations.

#### 4.2.1 Standards with ADT under 100

**Design Speed.** Design speeds range from 20 MPH for forest development roads to 55 MPH for the Oregon State Highway Department in flat terrain. The U.S. Forest Service allows a minimum of 10 MPH and
# TABLE 4 Summaries of existing geometric design standards for low volume roads

<table>
<thead>
<tr>
<th>Design Control</th>
<th>AASHTO (1956)(5)</th>
<th>National Association of County Engineers(47)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ADT under 100</td>
<td>ADT=100-400</td>
</tr>
<tr>
<td><strong>Design Speed (mph)</strong></td>
<td>minimum F-40 R-30 M-20</td>
<td>minimum F-45 R-35 M-25</td>
</tr>
<tr>
<td></td>
<td><strong>F</strong>= Flat terrain, <strong>R</strong>= Rolling terrain, <strong>M</strong>= Mountainous terrain.</td>
<td></td>
</tr>
<tr>
<td><strong>Max. degree of curvature (degree)</strong></td>
<td>minimum F-14 R-25 M-56</td>
<td>minimum F-11 R-18 M-36</td>
</tr>
<tr>
<td></td>
<td>desirable F-8 R-12 M-15</td>
<td>desirable F-7 R-11 M-18</td>
</tr>
<tr>
<td><strong>Max. gradient (percent)</strong></td>
<td>minimum F-8 R-12 M-15</td>
<td>minimum F-8 R-10 M-12</td>
</tr>
<tr>
<td></td>
<td>desirable F-5 R-7 M-10</td>
<td>desirable F-5 R-7 M-9</td>
</tr>
<tr>
<td><strong>Stopping sight distance (feet)</strong></td>
<td>minimum F-315 R-240 M-165</td>
<td>desirable F-415 R-315 M-240</td>
</tr>
<tr>
<td><strong>Pavement width (feet)</strong></td>
<td>minimum -12 (if any)</td>
<td>minimum -16 desirable -20</td>
</tr>
<tr>
<td><strong>Surface type</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Shoulder width (feet)</strong></td>
<td>minimum -4</td>
<td>minimum -4 desirable -4</td>
</tr>
<tr>
<td><strong>Shoulder type</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Right-of-way (feet)</strong></td>
<td>minimum -40*</td>
<td>minimum -40 desirable -80</td>
</tr>
<tr>
<td>* minimum of 40-foot or as required for construction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>** minimum of 40-foot or as required for construction **</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design Control</td>
<td>Oregon State Highway Department</td>
<td>Georgia State Highway Department(77)</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>---------------------------------</td>
<td>-------------------------------------</td>
</tr>
<tr>
<td></td>
<td>ADT under 100</td>
<td>ADT = 100-400</td>
</tr>
<tr>
<td>Design Speed (mph)</td>
<td>minimum F-55 R-40 M-30</td>
<td>minimum F-40 R-30 M-20</td>
</tr>
<tr>
<td></td>
<td>F-60 R-45 M-35</td>
<td>F-50 R-40 M-25</td>
</tr>
<tr>
<td>Max. degree of curvature (degree)</td>
<td>minimum F-7 R-14 M-24</td>
<td>minimum F-5 R-10 M-18</td>
</tr>
<tr>
<td>Max. gradient (percent)</td>
<td>minimum F-6 R-8 M-9</td>
<td>minimum F-5 R-6 M-6</td>
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<tr>
<td>Stopping sight distance (feet)</td>
<td>minimum F-415 R-275 M-200</td>
<td>minimum F-475 R-315 M-240</td>
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<tr>
<td>Pavement width (feet)</td>
<td>minimum -20 or 22</td>
<td>desirable F-475 R-315 M-240</td>
</tr>
<tr>
<td>Surface type</td>
<td>crushed material</td>
<td>oil mat</td>
</tr>
<tr>
<td>Shoulder width (feet)</td>
<td>minimum -2</td>
<td>minimum -4</td>
</tr>
<tr>
<td>Shoulder type</td>
<td>crushed material</td>
<td>oil mat</td>
</tr>
<tr>
<td>Right-of-way (feet)</td>
<td></td>
<td></td>
</tr>
<tr>
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TABLE 4 (continued) Summaries of existing geometric design standards for low volume roads

<table>
<thead>
<tr>
<th>Design Control</th>
<th>Forest Development Roads(??) ADT under 100</th>
<th>ADT =100-400</th>
<th>International Standards ( I.B.R.D*) (16) ADT under 100</th>
<th>ADT = 100-400</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed (mph)</td>
<td>minimum F-20 R-15 M-10</td>
<td>minimum F-40 R-35 M-30</td>
<td>desirable F-40 R-26.7 M-20</td>
<td>desirable F-53.3 R-43.3 M-26.7</td>
</tr>
<tr>
<td>Max. degree of curvature (degree)</td>
<td>minimum F-36 R-58 M-82</td>
<td>minimum F-14 R-20 M-25</td>
<td>desirable F-15 R-35</td>
<td>desirable F-9 R-13 M-35</td>
</tr>
<tr>
<td>Max. gradient (percent)</td>
<td>minimum F-6 R-6 M-8**</td>
<td>minimum F-7 R-8 M-10**</td>
<td>desirable F-6 R-8 M-10</td>
<td>desirable F-6 R-7 M-9</td>
</tr>
<tr>
<td>Stopping sight distance (feet)</td>
<td>minimum F-225 R-85 M-50</td>
<td>minimum F-275 R-225 M-200</td>
<td>desirable F-235 R-132</td>
<td>desirable F-380 R-270 M-135</td>
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<td>Pavement width (feet)</td>
<td>minimum --12***</td>
<td>minimum --22****</td>
<td>desirable --13.2 to 16.5 depending on design speed</td>
<td>desirable --18 to 20.5 depending on design speed</td>
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<td>Surface type</td>
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<td>surface treatment</td>
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<td>gravel</td>
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<td>Shoulder width (feet)</td>
<td>minimum--F,R-3 M-2</td>
<td>desirable --3.3</td>
<td>desirable --5</td>
<td>desirable --5</td>
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<tr>
<td>Shoulder type</td>
<td></td>
<td></td>
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<tr>
<td>Right-of-way (feet)</td>
<td>minimum --60</td>
<td>minimum --60</td>
<td>desirable--85</td>
<td>desirable--120</td>
</tr>
</tbody>
</table>

* International Bank for Reconstruction and Development (World Bank)

** Steeper grade may be approved if soil erosion is not critical.

*** Single lane width may be increased as necessary to accommodate trucks.

**** Lane width may be increased as necessary to accommodate trucks.
### TABLE 4 (continued) Summaries of existing geometric design standards for low volume roads

<table>
<thead>
<tr>
<th>Design Control</th>
<th>Puerto Rico Highway Authority (??)</th>
<th>Minnesota State Highway Department *</th>
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<tbody>
<tr>
<td></td>
<td>ADT under 100</td>
<td>ADT = 100-400</td>
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<tr>
<td>Design Speed (mph)</td>
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<td></td>
</tr>
<tr>
<td>minimum</td>
<td>minimum</td>
<td>minimum</td>
</tr>
<tr>
<td>F-30 R-25 M-20</td>
<td>F-35 R-25 M-20</td>
<td>F-45 R-40 M-30</td>
</tr>
<tr>
<td>desirable</td>
<td>desirable</td>
<td>desirable</td>
</tr>
<tr>
<td>F-40 R-35 M-30</td>
<td>F-45 R-40 M-35</td>
<td></td>
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<tr>
<td>Max. degree of curvature (degree)</td>
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<tr>
<td>minimum</td>
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<td>minimum</td>
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<td>F-25 R-35 M-58</td>
<td>F-18 R-35 M-58</td>
<td>F-10 R-12 M-22</td>
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<tr>
<td>desirable</td>
<td>desirable</td>
<td>F-11 R-14 M-18</td>
</tr>
<tr>
<td>F-14 R-18 M-25</td>
<td>F-11 R-14 M-18</td>
<td></td>
</tr>
<tr>
<td>Max. gradient (percent)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>minimum</td>
<td>minimum</td>
<td>minimum</td>
</tr>
<tr>
<td>F-8 R-12 M-15</td>
<td>F-8 R-10 M-12</td>
<td>F-5 R-7 M-10</td>
</tr>
<tr>
<td>desirable</td>
<td>desirable</td>
<td>F-5 R-7 M-9</td>
</tr>
<tr>
<td>F-5 R-7 M-10</td>
<td>F-5 R-7 M-9</td>
<td></td>
</tr>
<tr>
<td>Stopping sight distance (feet)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>minimum</td>
<td>minimum</td>
<td>minimum</td>
</tr>
<tr>
<td>F-200 R-165 M-132</td>
<td>F-250 R-165 M-132</td>
<td>F-320 R-300 M-275</td>
</tr>
<tr>
<td>desirable</td>
<td>desirable</td>
<td>F-280 R-250 M-200</td>
</tr>
<tr>
<td>F-315 R-280 M-250</td>
<td>F-315 R-280 M-250</td>
<td></td>
</tr>
<tr>
<td>Pavement width (feet)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>minimum --13.2</td>
<td>minimum --16.2</td>
<td>minimum --22</td>
</tr>
<tr>
<td>desirable --16.2</td>
<td>desirable --18.2</td>
<td></td>
</tr>
<tr>
<td>Pavement type</td>
<td></td>
<td></td>
</tr>
<tr>
<td>asphalitic concrete</td>
<td>asphalitic concrete</td>
<td>traffic bound aggregate</td>
</tr>
<tr>
<td>Shoulder width (feet)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>minimum --3.3</td>
<td>minimum --4</td>
<td>minimum --1</td>
</tr>
<tr>
<td>desirable --4</td>
<td>desirable --5</td>
<td></td>
</tr>
<tr>
<td>Shoulder type</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Right-of-way (feet)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>minimum --40</td>
<td>minimum --42</td>
<td>minimum --83</td>
</tr>
<tr>
<td>desirable --83</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Highway Research Board bulletin no.158 p.83
<table>
<thead>
<tr>
<th>Design Control</th>
<th>Jackson County</th>
<th>Josephine County</th>
<th>Lincoln County</th>
<th>Benton County</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ADT under 100</td>
<td>ADT = 100-400</td>
<td>ADT = 0-400</td>
<td>ADT = 0-400</td>
</tr>
<tr>
<td>Design Speed (mph)</td>
<td>minimum --25</td>
<td>minimum --30</td>
<td>F-45 R,M --lower</td>
<td>minimum --30</td>
</tr>
<tr>
<td></td>
<td>desirable --30</td>
<td>desirable --35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. degree of curvature (degree)</td>
<td>minimum --30</td>
<td>minimum --21</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>desirable --21</td>
<td>desirable --15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. gradient (percent)</td>
<td>minimum --15</td>
<td>minimum --12</td>
<td>F--6 M--8</td>
<td>minimum--12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>desirable --12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stopping sight distance (feet)</td>
<td>minimum --200</td>
<td>minimum --240</td>
<td>--300</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>desirable --20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pavement width (feet)</td>
<td>minimum --20</td>
<td>minimum --22</td>
<td>--24 decrease to 22ft. with variations</td>
<td>--20</td>
</tr>
<tr>
<td></td>
<td>desirable --22</td>
<td>desirable --22</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Surface type</td>
<td>oil mat</td>
<td>oil mat</td>
<td>oil mat-AC</td>
<td>asphalt penetration macadam</td>
</tr>
<tr>
<td>Shoulder width (feet)</td>
<td>minimum --2</td>
<td>minimum --4 *</td>
<td>minimum --4</td>
<td>--5</td>
</tr>
<tr>
<td>Shoulder type</td>
<td>gravel</td>
<td>gravel</td>
<td>oil mat-AC</td>
<td>gravel</td>
</tr>
<tr>
<td>Right-of-way (feet)</td>
<td>minimum --50</td>
<td>minimum --50</td>
<td>minimum --60</td>
<td>minimum--60</td>
</tr>
</tbody>
</table>

* may be reduced to 2 ft with approval of the Public Work Director.
<table>
<thead>
<tr>
<th>Design Control</th>
<th>Marion County ADT =0-400</th>
<th>Hood River County ADT =0-400</th>
<th>Linn County ADT =0-400</th>
<th>Curry County ADT =0-400</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed (mph)</td>
<td>not design based on speed</td>
<td>F-53 R-45 M-34</td>
<td>varies</td>
<td>The design standards currently being employed by Curry County road department are similar to the Standards of Oregon State Highway Department</td>
</tr>
<tr>
<td>Max. degree of curvature (degree)</td>
<td>terrain is generally controlling factor</td>
<td>F-6 R-8.5 M-16</td>
<td>only as determined by terrain</td>
<td></td>
</tr>
<tr>
<td>Max. gradient (percent)</td>
<td>F--12 M--20</td>
<td>F-5 R-7 M-10</td>
<td>--12</td>
<td></td>
</tr>
<tr>
<td>Stopping sight distance (feet)</td>
<td>the same as Oregon State Highway Standards</td>
<td>F-390 R-315 M-230</td>
<td>the same as AASHTO Standards</td>
<td></td>
</tr>
<tr>
<td>Pavement width (feet)</td>
<td>--20</td>
<td>--22</td>
<td>--22-24 (min., max.)</td>
<td></td>
</tr>
<tr>
<td>Pavement type</td>
<td>oil mat</td>
<td></td>
<td>oiled or paved</td>
<td></td>
</tr>
<tr>
<td>Shoulder width (feet)</td>
<td>--5</td>
<td>--4</td>
<td>--2 to 4</td>
<td></td>
</tr>
<tr>
<td>Shoulder type</td>
<td>gravel</td>
<td></td>
<td>gravel</td>
<td></td>
</tr>
<tr>
<td>Right-of-way (feet)</td>
<td>minimum--5</td>
<td>minimum--15</td>
<td>desirable--25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>desirable--15</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### TABLE 5  (continued) Summaries of existing geometric design standards for low volume roads

<table>
<thead>
<tr>
<th>Design Control</th>
<th>Deschutes County (local roads)</th>
<th>Lane County ADT =0-400</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed (mph)</td>
<td>--60 varies with terrain</td>
<td>The road design procedures currently employed by Lane County are conforming to the Oregon State Highway Department Design Manual No.19; AASHTO Geometric Design of Rural Highways 1965 Edition; AASHTO Policy on Design of Urban Highways and Arterial Streets 1973 Edition.</td>
</tr>
<tr>
<td>Max. degree of curvature (degree)</td>
<td>--38.2</td>
<td></td>
</tr>
<tr>
<td>Max. gradient (percent)</td>
<td>--8</td>
<td></td>
</tr>
<tr>
<td>Stopping sight distance (feet)</td>
<td>--AASHTO Standards</td>
<td></td>
</tr>
<tr>
<td>Pavement width (feet)</td>
<td>--24</td>
<td></td>
</tr>
<tr>
<td>Pavement type</td>
<td>--oil mat</td>
<td></td>
</tr>
<tr>
<td>Shoulder width (feet)</td>
<td>--4</td>
<td></td>
</tr>
<tr>
<td>Shoulder type</td>
<td>cinder or gravel</td>
<td></td>
</tr>
<tr>
<td>Right-of-way (feet)</td>
<td>minimum--10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>desirable--20</td>
<td></td>
</tr>
</tbody>
</table>
National Association of County Engineers (NACE) provides a minimum of 30 MPH for mountainous topography. Low design speed has been employed on forest road systems to provide access to timber without high construction costs, rather than for high speed travel for public transportation. On the contrary, the State highway system must provide a high degree of mobility, low operating costs and high capacity requiring higher design standards.

Maximum Degree of Curvature. Oregon State highway standards allow a maximum degree of curvature of 7 degrees on flat terrain. U.S. Forest Service road standards allows the highest curvature of 36 degrees. For mountainous areas, the highest curvature permitted by U.S. Forest Service road standards is 82 degrees. The NACE and Minnesota Highway Department allow curvature of 21 degrees and 22 degrees respectively, in mountainous terrain. As with the design speed standard, Forest road horizontal alignment standards show a lower degree of safety standard than others.

Maximum Gradient. AASHTO, Georgia State and Puerto Rico standards have the same maximum allowable grade of 8 percent for flat topography, while NACE and Minnesota standards allow 5 percent. The highest for mountainous area is of Georgia State, 16 percent. AASHTO and Puerto Rico standards both permit 15 percent. Forest road standards have employed a design gradient not greater than 8 percent, which is suitable for heavy logging trucks.

Pavement Width. Standards are given for both 2-lane and single lane roadways. AASHTO, U.S.Forest Service and International Standards (IBRD) provide for single lane roadways of 12, 12 and 13.2 feet in width, respectively. For two lane roadways, Georgia permits a minimum
width of 18 feet, Oregon standard provides 20 feet, while the maximum width given by the NACE and Minnesota standards are 22 feet. As mentioned in the literature review section, pavement widths of 20 feet, or 10 lane width, is deemed to be reasonable for two-lane low volume roads. Forest roads include many miles of single lane roads with turnouts for passing because little public use is expected on those routes. The U.S. Forest Service designs the single lane roads based on economic criteria. The delay due to stopping on the turnouts and construction cost are important factors for economic analysis. Two-lane roads may be specified where traffic volume is higher than 100 vehicles per day. The International Standards (IBRD) are slightly different from that of the U.S. State Highway System due to the dimensions of the design vehicle which affects the design lane width.

Pavement Type. Gravel, oil mat and surface treatment are generally used as wearing surfaces. Only Puerto Rico standard requires asphaltic concrete for low volume roads.

Shoulder Width. The width varies from 1 foot for Minnesota to 4 feet for AASHTO and Georgia. The International Bank for Reconstruction and Development (IBRD) and Puerto Rico require a 3.3 feet width of shoulder. NACE provides 1 foot shoulder width in mountainous area and 2 feet in flat and rolling terrains. Forest roads do not specify shoulder width for roads with an ADT under 100 vehicles per day.

4.2.2 Standards with ADT of 100-400

Design Speed. For this group, design speed has been designed with an increase about 20-30 percent of that for the group ADT under
Puerto Rico's standard has the lowest design value of 35 MPH for flat terrain and 20 MPH for mountainous areas compared to that of 60 MPH for Oregon standards for flat terrain and 35 MPH for mountainous topography.

**Degree of Curvature.** Puerto Rico Highway Authority allows a maximum value of 58 degrees for mountainous areas. Georgia sets a maximum of 53 degrees. The maximum degree of curvature for flat terrain required by Puerto Rico is 18 degrees. The U.S. Forest Service sets a maximum of 14 degrees.

**Gradient.** The maximum for Georgia is 16 percent in mountainous terrain, for rolling terrain the standard sets a maximum gradient of 8 percent. The international standards set by IBRD allow a maximum of 9 percent. The maximum required for flat area is 8 percent for Puerto Rico and AASHTO standards.

**Pavement Width.** The width standard at an ADT between 100 and 400 varies from 16 feet for AASHTO standards to a high of 22 feet for NACE, Oregon, Forest Service and Minnesota standards. The average width is 19.8 feet. Available practical design standards and safety considerations suggest that it is reasonable to provide a 10 foot lane width, 20 foot pavement width for two lane roads, for low volume situations.

**Surface Type.** Most of the standards require paved surfaces of bituminous surface in various forms, such as road mix, oil mat, surface treatments and high type asphalt concrete.

**Shoulder Width.** The width varies from 2 feet to 4 feet. In addition, the standards for the six agencies previously identified prefer a 4 ft. shoulder width or wider for the group of ADT between 100 and 400 vehicles per day.
4.3 Safety Criteria and Warrants

The existing design standards employed by many associations such as AASHTO, NACE and others have been developed based on economy, safety, environmental conditions and experience. Traffic accidents data is analyzed on study by study basis, however no comprehensive data collection and management system for traffic accident data presently exists. These data would yield the relative frequency on which accident costs for highway design features may be based. The AASHTO Policy on Geometric Design for Rural Highway, known as the Blue Book, is a primary source for standards of highway design, based on safety criteria. The following are the safety and economic criteria explained by AASHTO.

1. **Surface Width.** The values in Table 6 are minimum widths to provide safe and economic design suggested in accordance with volume, design speed and traffic composition. The AASHTO Blue Book suggests that where economically feasible, as in flat terrain or where large truck combinations are prevalent, the use of a wider surface up to 24 feet should be encouraged.

2. **Shoulder Width.** AASHTO explains that shoulder width is not as directly related to speed and traffic volume as is surface width, and the economic feasibility exerts a greater influence on the width of shoulder than on the width of surfacing. Table 7 shows suggested minimum widths of shoulder for 2-lane rural highways, summarized broadly in terms of ranges for volume classifications. AASHTO recommends usable shoulders of 10 ft width as desirable on all highways.

3. **Sight Distance.** Due to the limited conflicts on passing
Table 6  Minimum Widths of Surfacing for 2-Lane Highways

Source: Reference 2  p. 261

<table>
<thead>
<tr>
<th>Design Speed MPH</th>
<th>Current ADT 50-250</th>
<th>Current ADT 250-400</th>
<th>Current ADT 400-750</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>20</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>40</td>
<td>20</td>
<td>20</td>
<td>22</td>
</tr>
<tr>
<td>50</td>
<td>20</td>
<td>20</td>
<td>22</td>
</tr>
<tr>
<td>60</td>
<td>20</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>70</td>
<td>20</td>
<td>22</td>
<td>24</td>
</tr>
</tbody>
</table>

Table 7  Widths of Shoulder for 2-Lane Rural Highways

Source: Reference 2  p. 261

<table>
<thead>
<tr>
<th>Design Volume (Current ADT)</th>
<th>Usable shoulder width, in feet.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
</tr>
<tr>
<td>50-250</td>
<td>4</td>
</tr>
<tr>
<td>250-400</td>
<td>4</td>
</tr>
<tr>
<td>400-750</td>
<td>6</td>
</tr>
</tbody>
</table>

Table 8  Suggested Earth Slopes for Design

Source: Reference 2  p. 248

<table>
<thead>
<tr>
<th>Height of cut or fill, in feet</th>
<th>Earth slope, horizontal to vertical, for type of terrain:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flat or Rolling</td>
</tr>
<tr>
<td>0-4</td>
<td>6:1</td>
</tr>
<tr>
<td>4-10</td>
<td>4:1</td>
</tr>
<tr>
<td>10-15</td>
<td>3:1</td>
</tr>
<tr>
<td>15-20</td>
<td>2:1</td>
</tr>
<tr>
<td>over 20</td>
<td>2:1</td>
</tr>
</tbody>
</table>

* In clay or silty soils subject to erosion, slopes steeper than 2:1 should be avoided.
of vehicles on low volume roads, passing sight distance may be neglected. Stopping sight distance is an important factor related to safety. Stopping sight distance requirements with variation of speed are shown in Table 12.

4. Right-of-Way. AASHTO requires 18 feet to 25 feet or more from the edge of shoulder to the right-of-way limits to maintain good balance for cross-section elements on low volume roads.

5. Slope. Table 8 shows the slopes used as a general basis for design of earth side slopes. These values are suggested for use where the topography limits the use of flatter slopes. Even in steep terrain there are places where flatter slopes are desirable to increase safety or improve appearance, than those shown. Typically this is justified where only small additional cost is incurred.

It is generally accepted that side slopes of 4:1 are reasonably safe and very often can be provided at less cost than the cost of a guardrail. General Motors Proving Grounds tests indicate that side slope of 6:1 provide insurance against overturning, and often recovery of control, even under adverse conditions.

4.4 Criteria for Evaluating Trade-offs

1. Economic Criteria

Oglesby and Altenhofen (51) studied the economics of design standards for low volume roads, and they recommended:

(a) For straight roads free of sight distance impairments and with a current ADT of 250-400 vehicles per day, the maximum recom-
mended width of roadbed, that is surfacing and shoulders combined might be 20 or 22 feet. Provisions should be made in the standards to permit narrower roadbeds where economic or other advantages of narrower roadbeds can be demonstrated.

(b) For similar roads with a current ADT lower than 250 vehicles per day, the maximum recommended width might be set at 20 feet, but agencies would be urged to consider narrower designs, particularly at volumes less than 150 vehicles per day.

(c) They emphasized that highway engineers and administrators should recognize that low volume roads and major highways serve different functions. Often the demands on them in serving these functions are not the same. To require that low volume roads fit the needs associated with major highways would mean overdesigning them and making them far more costly than they need be. They added that accidents on low volume roads are rare so that even if all accidents could be eliminated, the economic gain would be extremely small. Furthermore, since accident records show that higher standards do not seem to reduce non-intersection accidents, it is pointless to adopt high standards with this aim in mind (51). Also, where roads are to be reconstructed, the flexibility would cause engineers and administrators to think in terms of improvements, rather than complete rebuilding.

2. Safety Criteria

The previous findings and accident records related to either individual or combined roadway elements are adopted to explain the trade-offs between design standards and their associated safety only. Many relationships between roadway elements and accident rates are
shown in both the literature review and the body sections.

Shannon and Stanley (56) have analyzed the accident data of States of Idaho and Washington by using both statistical and economic methods. They recommended that for the current ADT under 400 vehicles per day, minimum pavement width should be 20 feet. Their recommendation is given in Table 9.

Table 9 Recommended Minimum Paved Width for Safety.
Source: Reference 56

<table>
<thead>
<tr>
<th>Current ADT</th>
<th>Minimum Paved Width, feet.</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-299</td>
<td>20</td>
</tr>
<tr>
<td>250-399</td>
<td>20</td>
</tr>
<tr>
<td>400-749</td>
<td>24</td>
</tr>
<tr>
<td>750-999</td>
<td>28</td>
</tr>
<tr>
<td>1000-1999</td>
<td>34</td>
</tr>
<tr>
<td>2000-2999</td>
<td>40</td>
</tr>
</tbody>
</table>
3. **Summary of Safety and Economic Trade-offs for Existing Design Standards**

Lane width and shoulder width standards can be evaluated in general to determine the economic and safety trade-offs.

(a) **Lane width.** It has been found that a 10-foot lane width is employed extensively on low volume roads. The average lane width for ADT 100-400 for Federal and State level, as shown in Table 2, is 9.9 feet. This value meets the requirements of width safety and economy. County roads are designed with lane widths of 10 to 12 feet. Oglesby and Altenhofen indicate that 11 and 12 feet lane widths are not economically justified because they do not provide a sufficient increase in safety for low volume roads since accidents are infrequent.

(b) **Shoulder width.** Almost all the data shows that shoulder width varies from 2 to 4 feet while AASHTO Blue Book has a minimum requirement of 4 feet. With a 10-foot lane width on a 24-foot roadbed as recommended by Oglesby and Altenhofen (51), only a 2-foot shoulder remains. A study by Belmont (10), for an ADT of 500, indicates that a 2 ft. shoulder has an average accident rate of 0.25 accidents per mile per year and a 4 ft. shoulder has 0.2 accidents per mile per year. With the difference of 0.05 accidents per mile per year, very little safety or economic gain, in terms of accident cost is found. Therefore, a 2 foot shoulder width is a more practical design.

(c) **Horizontal and vertical alignment.** The design of horizontal and vertical alignment is based on terrain and road class. It has been found that the combination of horizontal and vertical elements influences the accidents on the roadway.
5.1 Probability of Accidents in Passing Maneuvers for Low Volume Roads

Accidents on low volume roads are rare events. The probability of conflicts in passing maneuvers is now analyzed to determine this effect. A number of assumptions are made for the development of probability of conflicts:

1. Sight distances effects are not considered.
2. Average speed of 40 MPH or approximately 60 feet per second is used.
3. Headways of approximately 2 seconds are assumed.
4. All vehicles arrive at the given section during a 12-hour period from 7 a.m. to 7 p.m.
5. All arrivals follow a Poisson distribution.

The basic situation is described as follows. A driver in vehicle A, traveling 50 MPH, overtakes vehicle B, traveling 40 MPH. Without regard for safe passing sight distance, the driver in vehicle A pulls into the opposing traffic lane to pass vehicle B. Before vehicle A can return to the right lane, vehicle C, traveling in the opposite direction, approaches generating a conflict with vehicle A. The necessary determination in this evaluation is the probability of the above situation occurring.

The probability of simultaneous arrivals of two (A and B) or more vehicles is then given by,

\[ P(X) = 1 - [P(0) + P(1)] \]
For a 200 ADT facility with a 50/50 directional distribution, the ADT in either direction is 100. The arrival rate \( q \) equals 0.0023 vehicles per second. Then the arrivals in a 2-second interval equals \( 2 \times 0.0023 = 0.0046 = qt \).

The probability of having \( X \) vehicles is given by Poisson distribution,

\[
P(X) = \frac{e^{-qt}qt^X}{X!}
\]

\[
P(0) = \frac{e^{-0.0046} (0.0046)^0}{0!} = 0.99541
\]

\[
P(1) = \frac{e^{-0.0046} (0.0046)^1}{1!} = 0.00458
\]

where, \( q = \text{arrival rate} \quad t = \text{time interval} \)

Therefore, \( P(X \geq 2) = 1 - [P(0) + P(1)] = 0.0000105 \)

i.e., Probability of passing maneuver in a 15-minute period equals

\[
= 0.0000105 \times 15 \times 60
\]

\[
= 0.004725
\]

In other words, the probability of two vehicles close enough for the rear vehicle to pass in 15-minute period is 0.004725.

Assuming that the following vehicle passes at a constant speed of 50 MPH, the length of time that vehicle A is encroaching on the opposing lane is determined as follows:

\[
d = \frac{1.47 V t}{2}
\]

where, \( d = \text{distance traveled in left lane, in feet} \)

\( V = \text{average speed, in MPH} \)

\( t = \text{time left lane occupied, in second} \)

Then,

\[
t = \frac{d}{1.47 V}
\]
Assuming \( d \) is approximately 700 feet at the speed of 50 MPH based on AASHTO Blue Book (2), Figure III-2, p. 143.

\[
t = \frac{800}{1.47(50)} = 10.9 \text{ or } 11 \text{ seconds}
\]

If an opposing vehicle arrives in that 11-second interval, there would be a conflict. The probability of an arrival in the opposing lane is given,

\[
P(X > 1) = 1 - P(0) = 1 - e^{-qt} \frac{(qt)^x}{x!}
\]

\[
qt = (0.0023)(11) = 0.0253
\]

\[
P(X > 1) = 1 - e^{-0.0253} \frac{(0.0253)^0}{0!} = 1 - 0.97502 = 0.02498
\]

The probability of the passing maneuver occurring during the 11-second critical interval is

\[
P(P) = 0.0000105 \times \frac{11}{2} = 0.000055
\]

Thus, the probability of conflict, \( P(C) \) is

\[
P(C) = P(P) \times P(X > 1) = (0.000055)(0.02498) = 1.3739 \times 10^{-6}
\]

The probability of such an occurrence within a 12-hour period equals \( P(C) \) times the number of 11-second intervals in that period:

\[
P(C_D) = 1.3739 \times 10^{-6} \times \frac{12 \times 3600}{11} = 0.005396
\]
Also, it is assumed that this figure still represents 100 percent non-compliance and 100 percent of limited sight distance.

Over a 365-day period, the expected number of conflicts, \( E(C_y) \) becomes:

\[
E(C) = 0.005396(365)
\]
\[
= 1.970
\]

In determining the probability of an accident, given a conflict, a study (72) showed 33 accidents occurred in 100,000 conflicts, or probability of an accident, given a conflict, \( P(A,C) = 0.00033 \). Therefore, the probability of an accident \( P(A) \) is given by:

\[
P(A) = P(A,C)P(C_D)
\]
\[
= (0.00033)(0.005396)
\]
\[
= 1.7807 \times 10^{-6}
\]

For ADT of 200 vehicles per day, the expected number of accidents in one year is, \( E(A) \),

\[
E(A) = (1.7807 \times 10^{-6})(365)
\]
\[
= 0.00065
\]

Table 10 shows the expected number of accidents resulting from passing maneuvers with variation of the traffic carried by a low volume roads. An analysis made of data collected over a three-year period for roads in San Joaquin, California carrying less than 400 vehicles per day showed a frequency of reported non-intersection accidents of 179 per 100 million vehicle miles. The fatality rate, based on only three fatal accidents in three years, is 1.9. These rates, applied to traffic volumes of 100 and 400 vehicles per day, give the results on Table 11.

In the previous computations, accidents on low volume roads are analyzed as rare events. This results in low accident costs per year(51).
Table 10  Expected Number of Accidents Related to ADT
for Low Volume Roads

<table>
<thead>
<tr>
<th>ADT</th>
<th>Expected Number of Accidents per Year in Passing Maneuvers</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>$2.7056 \times 10^{-8}$</td>
</tr>
<tr>
<td>100</td>
<td>$2.435 \times 10^{-4}$</td>
</tr>
<tr>
<td>200</td>
<td>$6.500 \times 10^{-4}$</td>
</tr>
<tr>
<td>300</td>
<td>$2.325 \times 10^{-3}$</td>
</tr>
<tr>
<td>400</td>
<td>$5.578 \times 10^{-3}$</td>
</tr>
<tr>
<td>500</td>
<td>$1.773 \times 10^{-2}$</td>
</tr>
</tbody>
</table>

Table 11  Non-intersection Accidents for Low volume Roads

Source: Reference 51

<table>
<thead>
<tr>
<th>Accident Class</th>
<th>Probability of at least one non-intersection accident in one year on a given mile of road</th>
<th>100 ADT</th>
<th>400 ADT</th>
</tr>
</thead>
<tbody>
<tr>
<td>All reported accidents</td>
<td></td>
<td>0.065</td>
<td>0.26</td>
</tr>
<tr>
<td>Accidents involving fatalities</td>
<td></td>
<td>0.008</td>
<td>0.003</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Accident Class</th>
<th>Length of road required to account for one accident in one year (in mile)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>All reported accidents</td>
<td></td>
<td>15</td>
<td>4</td>
</tr>
<tr>
<td>Accidents involving fatalities</td>
<td></td>
<td>1200</td>
<td>300</td>
</tr>
</tbody>
</table>
5.2 Effect of Sight Distance on Horizontal Curves on Safety

Horizontal curvature is varied with terrain and environment, and to a lesser degree with the engineering judgment of design engineers. Design speed and stopping sight distance also limit the curvature of a particular location. Horizontal sight distance is often restricted in terrain conditions where a cut slope is on the inside of the curve. This often occurs on low volume, low cost roads where the topography is too rough to build roads to high geometric design standards economically. For design of horizontal curve the sight line is a chord of the curve, and the minimum stopping sight distance is measured along the centerline of the inside lane around the curve. For stopping sight distance, a height criteria of 3.75 ft height of eye and 6-inch height of object is used. A height of 2.0 feet can be used to approximate the midpoint of the sight line where the cut slope usually obstructs sight.

The stopping sight distance employed is based on the AASHTO Blue Book standards. The World Bank (16) has surveyed the design standards used in 34 developing countries and found that the average stopping sight distance standards are almost the same as those of AASHTO. The AASHTO standards are shown in Table 12.

The horizontal sight distance is limited by the middle ordinate distances, that is, centerline of the inside lane to sight obstruction. These middle ordinate distances are computed from the following equations and plotted on Figure 22.

\[ m = \frac{5730}{D} \text{ vers } \frac{SD}{200} \quad (4) \]
Table 12  Minimum Stopping Sight Distance

Source: Reference 2  p. 136

<table>
<thead>
<tr>
<th>Design Speed</th>
<th>Assumed Speed for Condition</th>
<th>Perception &amp; Brake Reaction</th>
<th>Coefficient of Friction</th>
<th>Braking Distance on Level</th>
<th>Stopping Sight Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>MPH</td>
<td>MPH</td>
<td>sec.</td>
<td>feet</td>
<td>f</td>
<td>feet</td>
</tr>
<tr>
<td>20</td>
<td>18</td>
<td>2.5</td>
<td>67</td>
<td>0.40</td>
<td>27</td>
</tr>
<tr>
<td>30</td>
<td>28</td>
<td>2.5</td>
<td>103</td>
<td>0.36</td>
<td>73</td>
</tr>
<tr>
<td>40</td>
<td>36</td>
<td>2.5</td>
<td>132</td>
<td>0.33</td>
<td>131</td>
</tr>
<tr>
<td>50</td>
<td>44</td>
<td>2.5</td>
<td>161</td>
<td>0.31</td>
<td>208</td>
</tr>
<tr>
<td>60</td>
<td>52</td>
<td>2.5</td>
<td>191</td>
<td>0.30</td>
<td>300</td>
</tr>
</tbody>
</table>
Figure 22  Stopping Sight Distance on Horizontal Curves

Source: Reference 2, p. 188
or \[ m = R \left( \text{vers} \frac{28.65 S}{R} \right) \]  \hspace{1cm} (5)  

\[ S = \frac{R}{28.65} \cos^{-1} \frac{R-m}{R} \]  \hspace{1cm} (6) 

where,  
- \( m \) = middle ordinate, in feet  
- \( D \) = degree of curvature  
- \( R \) = radius of curve, in feet  
- \( S \) = stopping sight distance, in feet  
- \( \text{vers } \theta = 1 - \cos \theta \)

The cross sections shown in Figure 23 are assumed for various lane and shoulder conditions to calculate the lateral distances, in feet, from the centerline of the inside lane to a point 2 ft. up on the backslope. The cross-section dimensions assumed for this analysis for single and two-lane roads, with and without shoulders on low volume roads, are:

1. 10-foot lane width,  
2. 4-foot shoulder,  
3. 1-foot side ditch.

With the given lateral distances, the maximum degree of curvatures for each design speed can be either calculated from the equations (4), or interpolated from Figure 22. Table 13 shows the values of maximum curvature obtained with variations in assumptions.

The maximum degree of curvature and design speed has been plotted on Figure 24 (a) through 24 (d). These relationships indicate the critical values of degree of curvature limited by horizontal sight distance with the variations of design speed, backslopes and types of roadway. The required increase in middle ordinate to provide horizontal sight distance for safe operation must be considered. If the approach
Figure 23  Assumed Cross-Section Elements

(a) Single-lane with 4 ft shoulder

(b) Single-lane without shoulder

(c) Two-lane with 4 ft shoulder

(d) Two-lane without shoulder
### Table 13 (a) Maximal Degree of Curvature versus Lateral Distance for Single Lane Road with 4 ft. Shoulder

<table>
<thead>
<tr>
<th>Backslope</th>
<th>Lateral Distance (feet)</th>
<th>Maximum Degree of Curvature when ( e = 0.10 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>V=20 MPH</td>
<td>V=30 MPH</td>
</tr>
<tr>
<td>Vertical</td>
<td>10</td>
<td>12</td>
</tr>
<tr>
<td>1:1</td>
<td>11</td>
<td>13</td>
</tr>
<tr>
<td>2:1</td>
<td>12</td>
<td>14</td>
</tr>
<tr>
<td>3:1</td>
<td>14</td>
<td>16.5</td>
</tr>
<tr>
<td>4:1</td>
<td>16</td>
<td>18</td>
</tr>
<tr>
<td>5:1</td>
<td>18</td>
<td>-</td>
</tr>
<tr>
<td>6:1</td>
<td>20</td>
<td>23.5</td>
</tr>
<tr>
<td></td>
<td>22</td>
<td>-</td>
</tr>
</tbody>
</table>

* Lateral distance is the distance from the centerline of inside lane to a point 2 ft. up on the backslopes.

### Table 13 (b) Maximal Degree of Curvature versus Lateral Distance for Single Lane Road without Shoulder

<table>
<thead>
<tr>
<th>Backslope</th>
<th>Lateral Distance (feet)</th>
<th>Maximum Degree of Curvature when ( e = 0.10 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>V=20 MPH</td>
<td>V=30 MPH</td>
</tr>
<tr>
<td>Vertical</td>
<td>6</td>
<td>19</td>
</tr>
<tr>
<td>1:1</td>
<td>7</td>
<td>22</td>
</tr>
<tr>
<td>2:1</td>
<td>8</td>
<td>26</td>
</tr>
<tr>
<td>3:1</td>
<td>10</td>
<td>32</td>
</tr>
<tr>
<td>4:1</td>
<td>12</td>
<td>38</td>
</tr>
<tr>
<td>5:1</td>
<td>14</td>
<td>45</td>
</tr>
<tr>
<td>6:1</td>
<td>16</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>-</td>
</tr>
</tbody>
</table>

* Lateral distance is the distance from the centerline of inside lane to a point 2 ft. up on the backslopes.
Table 13 (c) Maximum Degree of Curvature versus Lateral Distance for Two-Lane Road with 4 ft. Shoulder

<table>
<thead>
<tr>
<th>Backslope</th>
<th>Lateral Distance* (feet)</th>
<th>Maximum Degree of Curvature when $e = 0.10$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$V=20$ MPH</td>
</tr>
<tr>
<td>Vertical</td>
<td>15</td>
<td>48</td>
</tr>
<tr>
<td>1:2:1</td>
<td>16</td>
<td>51</td>
</tr>
<tr>
<td>1:1</td>
<td>17</td>
<td>53.5</td>
</tr>
<tr>
<td>2:1</td>
<td>19</td>
<td>-</td>
</tr>
<tr>
<td>3:1</td>
<td>21</td>
<td>-</td>
</tr>
<tr>
<td>4:1</td>
<td>23</td>
<td>-</td>
</tr>
<tr>
<td>5:1</td>
<td>25</td>
<td>-</td>
</tr>
<tr>
<td>6:1</td>
<td>27</td>
<td>-</td>
</tr>
</tbody>
</table>

* Lateral distance is the distance from the centerline of inside lane to a point 2 ft. up on the backslopes.

Table 13 (d) Maximum Degree of Curvature versus Lateral Distance for Two-Lane Road without Shoulder

<table>
<thead>
<tr>
<th>Backslope</th>
<th>Lateral Distance* (feet)</th>
<th>Maximum Degree of Curvature when $e = 0.10$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$V=20$ MPH</td>
</tr>
<tr>
<td>Vertical</td>
<td>11</td>
<td>34.5</td>
</tr>
<tr>
<td>1:2:1</td>
<td>12</td>
<td>38</td>
</tr>
<tr>
<td>1:1</td>
<td>13</td>
<td>42</td>
</tr>
<tr>
<td>2:1</td>
<td>15</td>
<td>48</td>
</tr>
<tr>
<td>3:1</td>
<td>17</td>
<td>53.5</td>
</tr>
<tr>
<td>4:1</td>
<td>19</td>
<td>-</td>
</tr>
<tr>
<td>5:1</td>
<td>21</td>
<td>-</td>
</tr>
<tr>
<td>6:1</td>
<td>23</td>
<td>-</td>
</tr>
</tbody>
</table>

* Lateral distance is the distance from the centerline of inside lane to a point 2 ft. up on the backslopes.
Figure 24 (a) Critical Degree Of Curvature versus Design Speed with Variation of Sideslopes for Single-Lane Road with 4-foot shoulder
Figure 24 (b) Critical Degree of Curvature versus Design Speed with Variation of Sideslopes for Single-Lane Road without Shoulder
Figure 24 (c) Critical Degree of Curvature versus Design Speed with Variation of Sideslopes for Two-Lane Road with 4-foot shoulder
Figure 24 (d) Critical Degree of Curvature versus Design Speed with Variation of Sideslopes for Two-Lane Road without shoulder.
* The numbers in ( ) indicate involvement rates per 100 MVM conforming to speed increment

Figure 25  Speed Increment versus Middle Ordinate Increment with Variation of Curvature
speed is higher than the safe speed permitted by horizontal sight distance, an increase in middle ordinate or lateral distance is required to provide sufficient sight distance. For a given approach speed, the middle ordinate increment is increased more for curves of high degree of curvature than for curves of low degree of curvature. This is illustrated in the Figure 25. Solomon (58) has found the relationship between accident involvements and speed change from average speed as shown on Figure 5. Accident rates were found to be highest at very low speeds, lowest at about the average speed, and increase again at the speeds higher than average speed designed, forming a V-shaped distribution. The accident involvement rates from the Solomon's finding have been superimposed on Figure 25 showing the increase in accident involvement rates associated with speed increment from design speed for the curve and middle ordinate increment needed. This demonstrates the trade-off between safety and the required sight distance for horizontal curves with maximum degree of curvature corresponding the design speed for a low volume road.

The middle ordinate increment to provide safe horizontal sight distance can be used to estimate the construction or improvement costs that would be incurred in providing safe sight distance for a given curvature. That cost of improvement can be compared to the expected accident costs based on the accident involvement expected.
5.3 Effect of Horizontal and Vertical Alignments on Vehicle Running Cost and Travel Time

The primary objective is to provide measures to evaluate the cost trade-offs between design requirements and the costs and other consequences of accidents.

The vehicle running costs and time consumed on curves of varying degrees must be evaluated compared to the accident costs expected with that degree of curvature. Running costs, time consumed and accident costs typically increase as a curve becomes sharper. On the other hand, in the situation discussed, the longer radius or lower degree of curvature of the flatter curves can substantially increase construction costs. Thus, the designer must consider the trade-offs between vehicle running costs, accident costs and time consumed on the one hand and construction costs on the other.

The relationships between vehicle running cost and horizontal alignment conditions have been determined for various of vehicles. The horizontal curve selection must take account of these relationships:

1. Running costs increase proportionally with distance, they also vary with speed.

2. It is somewhat more costly to operate vehicles at the same speed on curves than on tangents, the sharper the curve the greater the cost per unit distance.

3. The cost of slowing vehicles and accelerating them again is substantial, this cost is a function of both higher and lower travel speeds.
It is assumed that driver behavior conforms to the assumptions underlying the AASHTO standards. According to these standards, speeds on curves never exceed those at which the stopping sight distance makes the driver uncomfortable due to sight restriction. Provision is made to recognize the influence of sight distance for these speed controls on running costs and times. It is also assumed that low volume roads have provided good side friction. Side friction of the road surface would govern the travel speed if the surface is loosened and the coefficient of side friction reaches the level of discomfort. Many returned questionnaires indicate that paved surfaces are generally employed in Oregon.

5.3.1 Variables to be evaluated

As indicated above, construction costs are unique to each site and cannot be treated in a general way. On the other hand, vehicle running costs and time consumed can be generalized, and that has been done here.

Vehicle running cost on curves has been divided into three parts:

(a) Cost of traveling on tangents before entering and after leaving the particular curve,

(b) Cost of slowdown from travel speed to the limited curve speed and speed up from curve speed to travel speed on tangent, and

(c) Cost of traveling on curve.

The procedures for determining added running cost due to the effects of horizontal curve and vertical grade are as follows:
Figure 26  Flow Chart for Finding Added Running Cost
Source: Reference 50.
DETERMINE BEFOREHAND

\[ \Delta = 60^\circ \]

CENTRAL ANGLE

\[ m = 1:1 \]

SIDE SLOPE

80K

VEHICLE TYPE

50

APPROACH SPEED

100

TRAFFIC VOLUME

VEHICLES

PER DAY

DETERMINE CURVE IN

QUESTION

Fig. 24 FIND CRITICAL SPEED

THROUGH CHOSEN CURVE

DETERMINE DISTANCE AND UNIT

COSTS

LENGTH OF CURVE (FROM FIG. A-1)

110

LENGTH OF TANGENT (FROM FIG. A-2)

65

SLOWING SPEED (FROM FIG. A-3)

18

COST ON TANGENT (FROM FIG. A-4)

25

EXCESS COST ON CURVE (FROM FIG. A-4)

2.00

TOTAL COST OF TRAVELING ON CURVE ($/100 VEH.)

2

COST OF TRAVELING ON TANGENTS ($/100 VEH.)

65 x 0.28 = 18

COST OF TRAVELING ON CHOSEN CURVE ($/100 VEH.)

+ 40 = 60

COST OF TRAVELING ON 1° CURVE ($/100 VEH.)

60 x 3.65 = 219

COST OF TRAVELING ON 1° CURVE ($/100 VEH.)

19

41

$15,000

Figure 26 (a) Flow Chart for Finding Added Running Cost (Revised)

Source: Reference 50.
Step 1. Critical velocities on curves are determined by using Figures 24(a) to 24(d). These are valid for cases fitting the assumptions stated earlier.

Step 2. Calculate the costs as mentioned above: cost of traveling on tangents, cost of slowdown-speedup and cost of traveling on a curve from the charts available in Appendix A. These charts have been developed by Oglesby, Arias and Clark (50) using the available data of Winfrey's textbook (66). Figure 26 illustrates the approach to find added running cost in a flowchart. The results are the difference between the annual operating costs of a given vehicle running on a specific curve and that of a vehicle on 1-degree curve.

These costs have been analyzed during the years 1969-1970. An update to incremental running costs can be made using a multiplying factor of 1.2 for the year 1978 with the average compound inflation rate of approximately 2 percent. This factor is derived from:

$$\text{Multiplication Factor} = (1 + i)^n$$

$$= (1 + 0.02)^9$$

$$= 1.20$$

where $i = 2\%$

$n = 9\text{ years} \ (1969-1978)$. 

The consumer price compound rate was 4.0 percent in 1940-1957 and 1.9 percent during 1957-1963. The inflation rate of approximately 6 percent is presently employed to determine the accident costs (29). Lee and Grant (73) suggest the use of 2 percent for long term inflation rate.
Figure 27 (a) Annual Vehicle Running Costs for Traversing Circular Curves compared with Costs for a one-degree curve
Figure 27 (b) Annual Vehicle Running Costs for Traversing Circular Curves compared with Costs for One-Degree Curve
Figure 27 (c) Annual Vehicle Running Costs for Traversing Circular Curves compared with Costs for One-Degree Curve.
Figure 27 (d) Annual Vehicle Running Costs for Traversing Circular Curves compared with Costs for One-Degree Curve
Following these procedures, Figures 27(a) to 27(d) indicate the annual vehicle running costs in dollars (during the years 1969-1970) for 100 ADT with variations of degree of curvature for a 4-kip passenger car and a 12-kip single unit truck running on 2-lane roads. The costs are also stratified to give conditions with and without 4-foot shoulders. These figures show that the running costs increase rapidly for the curves with a degree of curvature higher than 5-8 degrees, depending on type of vehicles.

The costs are also affected by sideslopes, as shown on Figures 27(a) to 27(d) comparing sideslopes of 4:1 to 1:1. At sideslope 4:1, the critical speed is higher so the vehicles can travel faster than on 1:1 sideslope on the same degree of curvature. Therefore, the running cost is less than that of sideslope 1:1 due a lesser effect of sight restriction.

In addition, these costs have been developed for high type pavement such as asphaltic concrete or portland cement concrete. For low type pavement, such as gravel or bituminous surfaces, factors are used to convert vehicle running costs from the basis of high type pavements. These factors are included in Table 14. This table shows the variations of factors on various types of vehicles and speed.
Table 14  Factor to Convert Motor Vehicle Running Cost on High Type Pavements to Cost on Gravel and Stone Roadway Surfaces

Source: Reference 66, p. 727.

<table>
<thead>
<tr>
<th>Speed, mph</th>
<th>4-kip Passenger Car</th>
<th>5-kip Commercial Delivery</th>
<th>12-kip Single-Unit Truck</th>
<th>40-kip 2-S2 Gasoline</th>
<th>50-kip 3-S2 Diesel</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1.079</td>
<td>1.074</td>
<td>1.090</td>
<td>1.114</td>
<td>1.129</td>
</tr>
<tr>
<td>71/2</td>
<td>1.106</td>
<td>1.109</td>
<td>1.122</td>
<td>1.148</td>
<td>1.172</td>
</tr>
<tr>
<td>10</td>
<td>1.132</td>
<td>1.125</td>
<td>1.152</td>
<td>1.181</td>
<td>1.210</td>
</tr>
<tr>
<td>121/2</td>
<td>1.157</td>
<td>1.149</td>
<td>1.180</td>
<td>1.212</td>
<td>1.245</td>
</tr>
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<td>15</td>
<td>1.181</td>
<td>1.172</td>
<td>1.207</td>
<td>1.241</td>
<td>1.278</td>
</tr>
<tr>
<td>171/2</td>
<td>1.205</td>
<td>1.193</td>
<td>1.232</td>
<td>1.267</td>
<td>1.307</td>
</tr>
<tr>
<td>20</td>
<td>1.228</td>
<td>1.215</td>
<td>1.256</td>
<td>1.291</td>
<td>1.334</td>
</tr>
<tr>
<td>221/2</td>
<td>1.250</td>
<td>1.237</td>
<td>1.279</td>
<td>1.314</td>
<td>1.359</td>
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<td>25</td>
<td>1.272</td>
<td>1.259</td>
<td>1.300</td>
<td>1.336</td>
<td>1.382</td>
</tr>
<tr>
<td>271/2</td>
<td>1.294</td>
<td>1.281</td>
<td>1.321</td>
<td>1.357</td>
<td>1.404</td>
</tr>
<tr>
<td>30</td>
<td>1.315</td>
<td>1.302</td>
<td>1.341</td>
<td>1.377</td>
<td>1.424</td>
</tr>
<tr>
<td>321/2</td>
<td>1.337</td>
<td>1.323</td>
<td>1.361</td>
<td>1.397</td>
<td>1.444</td>
</tr>
<tr>
<td>35</td>
<td>1.358</td>
<td>1.344</td>
<td>1.381</td>
<td>1.417</td>
<td>1.464</td>
</tr>
<tr>
<td>371/2</td>
<td>1.380</td>
<td>1.365</td>
<td>1.401</td>
<td>1.437</td>
<td>1.483</td>
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<td>40</td>
<td>1.402</td>
<td>1.387</td>
<td>1.421</td>
<td>1.457</td>
<td>1.518</td>
</tr>
<tr>
<td>421/2</td>
<td>1.424</td>
<td>1.409</td>
<td>1.441</td>
<td>1.476</td>
<td>1.540</td>
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<td>45</td>
<td>1.447</td>
<td>1.431</td>
<td>1.462</td>
<td>1.495</td>
<td>1.553</td>
</tr>
<tr>
<td>471/2</td>
<td>1.471</td>
<td>1.455</td>
<td>1.484</td>
<td>1.515</td>
<td>1.553</td>
</tr>
<tr>
<td>50</td>
<td>1.498</td>
<td>1.479</td>
<td>1.507</td>
<td>1.536</td>
<td>1.572</td>
</tr>
<tr>
<td>521/2</td>
<td>1.526</td>
<td>1.504</td>
<td>1.531</td>
<td>1.557</td>
<td>1.592</td>
</tr>
<tr>
<td>55</td>
<td>1.553</td>
<td>1.531</td>
<td>1.557</td>
<td>1.579</td>
<td>1.614</td>
</tr>
<tr>
<td>571/2</td>
<td>1.582</td>
<td>1.561</td>
<td>1.587</td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>1.631</td>
<td>1.592</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.3.2 Effect of Grades on Vehicle Operating Cost

Both upgrades and downgrades influence vehicle running costs. Oglesby, Arias and Clark (50) demonstrate that the relationships between running costs and grades are not linear. Furthermore, these cost functions show that operating costs increase rather than decrease when downgrades become steeper than 3 or 4 percent. Figure 26 (a) is a revised flowchart for finding added running costs with the effects of curvatures, sideslopes and grades. The figures indicated in the flowchart are included in Appendix A.

5.3.3 Effect of Horizontal Alignment on Travel Times

Curvatures influence vehicle travel times. Travel time consumed when vehicles travel on a sharp curve is higher than traveling on a flatter curve. Figure 28 presents the approach for finding added time consumed in a flowchart. Figures 29 (a) and 29 (b) illustrate the relations of annual time for traversing circular curves compared with time for traversing a one-degree curve for both a 4-kip passenger car and a 12-kip single unit truck. The sight distance restrictions on degree of curvature due to various sideslopes for the assumed roadway sections of this study are also shown in these figures. These figures indicate the rapid increase in total time consumed if the degree of curvature is greater than 6.5 degrees for a sideslope 1:1 and greater than 8 degrees for a sideslope 4:1. For other types of vehicles and sideslopes, or different central angles and approach speeds, figures can be developed to determine travel time around curves by utilizing
DETERMINE DEFOREHAND $\Delta = 60^\circ$

CENTRAL ANGLE
20° to 160°

DETERMINE CURVE IN QUESTION $D = 58^\circ$ (100' R)

FROM Fig. 24 FIND CRITICAL SPEED THRU CHOSEN CURVE (mph)

FIND CRITICAL I = 18

DETERMINE DISTANCES AND UNIT TIMES

TOTAL TIME TRAVELING ON CURVE
TOTAL TIME TRAVELING ON TANGENT
TOTAL TIME TRAVELING ON CHOSEN CURVE

TIME OF TRAVELING A 1° CURVE

ANNUAL TIME DIFFERENCE

Figure 28 Flow Chart for Finding Added Time Consumed
Source: Reference 50.
Figure 29 (a) Annual Time for Traversing Circular Curves compared with Time for Traversing a One-Degree Curve
Figure 29 (b) Annual Time for Traversing Circular Curves compared with Time for Traversing a One-Degree Curve
5.4 Guardrail Installation for Low Volume Roads

The primary reason for the installation of guardrails is to reduce the severity of collisions involving run-off-the-road vehicles. A guardrail is warranted only where the expected severity of striking the guardrail is less than the severity of the accident that would have occurred if the guardrail had not been present. There is widespread agreement that no guardrail is needed if the free slope is flatter than 4:1 and no other hazards are present. The primary factors associated with evaluation of guardrail use are embankment height, fill slope, shoulder width, ditch design, horizontal curvature and other roadside obstacles.

The information and analysis on guardrails and median barriers has been developed for high traffic volume roads (43). The criteria for use of guardrails on low volume roads has not been developed.

Economic analysis comparing the use of guardrail to a flattened slope of a given embankment must analyzed to understand the economic trade-offs involved. Since guardrails are recognized as hazards in themselves, emphasis is placed on reducing the number of such installations to those that can be firmly justified.

Figures 30 and 31 depict the increase in severity index with increasing embankment height and embankment slope, based on California's 1963 single vehicle embankment accidents. The research was done by Glennon and Tamburi (24). The accident costs quoted in their study, describing the severity of accidents, were for the State of Illinois in
### Table 15  Accident Severity for Embankment Slopes

**Source:** Reference 24.

<table>
<thead>
<tr>
<th>Slope Ratio</th>
<th>Fatal &amp; Nonfatal Injury Accidents</th>
<th>Total Accidents</th>
<th>Severity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>5:1</td>
<td>4</td>
<td>15</td>
<td>0.27</td>
</tr>
<tr>
<td>4:1</td>
<td>10</td>
<td>24</td>
<td>0.42</td>
</tr>
<tr>
<td>3:1</td>
<td>15</td>
<td>31</td>
<td>0.48</td>
</tr>
<tr>
<td>2:1</td>
<td>215</td>
<td>348</td>
<td>0.62</td>
</tr>
<tr>
<td>1 1/2:1</td>
<td>305</td>
<td>439</td>
<td>0.70</td>
</tr>
<tr>
<td>1:1</td>
<td>115</td>
<td>147</td>
<td>0.78</td>
</tr>
</tbody>
</table>

### Table 16  Accident Severity for Embankment Heights

**Source:** Reference 24.

<table>
<thead>
<tr>
<th>Embankment Height (feet)</th>
<th>Fatal &amp; Nonfatal Injury Accidents</th>
<th>Total Accidents</th>
<th>Severity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>41</td>
<td>93</td>
<td>0.44</td>
</tr>
<tr>
<td>8</td>
<td>111</td>
<td>184</td>
<td>0.60</td>
</tr>
<tr>
<td>15</td>
<td>179</td>
<td>279</td>
<td>0.64</td>
</tr>
<tr>
<td>25</td>
<td>107</td>
<td>165</td>
<td>0.65</td>
</tr>
<tr>
<td>35</td>
<td>50</td>
<td>67</td>
<td>0.75</td>
</tr>
<tr>
<td>45</td>
<td>33</td>
<td>44</td>
<td>0.75</td>
</tr>
<tr>
<td>60</td>
<td>41</td>
<td>53</td>
<td>0.77</td>
</tr>
<tr>
<td>85</td>
<td>32</td>
<td>40</td>
<td>0.80</td>
</tr>
<tr>
<td>125</td>
<td>25</td>
<td>30</td>
<td>0.83</td>
</tr>
<tr>
<td>175</td>
<td>21</td>
<td>24</td>
<td>0.88</td>
</tr>
<tr>
<td>350</td>
<td>19</td>
<td>20</td>
<td>0.95</td>
</tr>
</tbody>
</table>
Figure 30  Accident Severity Related to Embankment Heights

Source: Reference 24.
Figure 31  Accident Severity Related to Embankment Slopes

Source: Reference 24.
1958 and accident data for California in 1963. The severity index is defined in the following form:

The severity index, $SI = \frac{25F + 6I + P}{N}$

where $F =$ number of fatal accidents, $I =$ number of injury accidents, $P =$ number of property-damage-only accidents, and $N =$ total number of accidents.

Numerous multiple regression equations were developed, and the best relationship found is;

$$\log (SI) = 0.566 + 0.160 \log (H) + 0.324 \log (S)$$

where $SI =$ severity index, $H =$ embankment height, and $S =$ embankment slope.

This relationship has an $R^2$ of 0.804, reflecting that 80 percent of the variation in the data were explained by this relation. In addition to the above equation, Glennon and Tamburi (24) reviewed the severity of accidents where guardrails could be used to develop warrants for guardrail usage. The accident reports were used to verify that embankment guardrail was involved. Table 17 gives the severity breakdown of these accidents.

Table 17 1963-1964 Single Vehicle Struck Embankment Guardrail Accidents.

Source: Reference 24.

<table>
<thead>
<tr>
<th>Fatal</th>
<th>Injury</th>
<th>PDO*</th>
<th>Total</th>
<th>SI</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>147</td>
<td>170</td>
<td>331</td>
<td>4.24</td>
</tr>
</tbody>
</table>

* Property-Damage-Only accidents
This relationship from Table 17 has been plotted and used as the warrant for guardrails. Figure 32 can be used to determine the need for a guardrail along an embankment. The curve with an index 50, represents approximately the line of equal accident severity for different combinations of embankment heights and slopes. To each combination of embankment height and steepness of the sideslope, a value of basic "need index" is assigned. The procedure developed in Highway Research Board Special Report number 81 (33) assigned a basic value of 50 for the minimum height of fill on a primary highway requiring guardrails under the approval of a subcommittee. To each combination of slope and height of fill indicated in Table 18, a basic value of 50 was assigned. Then, larger values were assigned to higher fills and smaller values were assigned to lower fills for each slope to produce a full range of values, as given in Table 19.

The warranting values assigned to two types of highways are:

(1) Primary highways (high traffic volume) ...... 50

(2) Secondary highways (carrying generally low volumes) ...... 70

Glennon and Tamburi's finding has been suggested as a reasonable warrant in other recent Highway Research Board reports (43, 44). The procedure is applicable primarily to new systems with high traffic volume. Michie and Calcote (43) developed three basic need index curves having values of 50, 60 and 70 from Glennon and Tamburi's findings. The "50" curve is identical to a curve with a severity index of 4.24 from Glennon research, as indicated in Table 17. The "60" and "70" curves have an approximate severity index of 4.6 and 5.0, respect-
Table 18 Minimum Height of Fill on Primary Highways Requiring Guardrail

Source: Reference 33.

<table>
<thead>
<tr>
<th>Fill Slope</th>
<th>Height of Fill (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 1/2:1</td>
<td>8</td>
</tr>
<tr>
<td>2:1</td>
<td>10</td>
</tr>
<tr>
<td>2 2/3:1</td>
<td>12</td>
</tr>
<tr>
<td>3:1</td>
<td>15</td>
</tr>
</tbody>
</table>

Table 19 Basic Guardrail Need Index for Embankment Conditions.

Source: Reference 33.

<table>
<thead>
<tr>
<th>Height of Fill (feet)</th>
<th>Need Index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 1/2:1</td>
</tr>
<tr>
<td>4</td>
<td>40</td>
</tr>
<tr>
<td>6</td>
<td>45</td>
</tr>
<tr>
<td>8</td>
<td>50</td>
</tr>
<tr>
<td>10</td>
<td>55</td>
</tr>
<tr>
<td>12</td>
<td>60</td>
</tr>
<tr>
<td>15</td>
<td>65</td>
</tr>
<tr>
<td>20</td>
<td>70</td>
</tr>
<tr>
<td>30</td>
<td>75</td>
</tr>
<tr>
<td>40+</td>
<td>80</td>
</tr>
</tbody>
</table>
Guards are warranted for embankment geometry in low volume roads. The graph shows the relationship between embankment height and slope, indicating when guardrails are necessary for safety. Source: Reference 33, 43.

Figure 32
Guardrail Requirement for Embankment Geometry, A Development of Basic Need for Guardrail Installation on Low Volume Roads.
ively. From HRB Special Report number 81 (33), Glennon and Tamburi (24) and Michie and Calcote (43), the warranting values of need index have been superimposed in this study on Figure 32. The curve for a need index of "80" also has a severity index of about 5.4. It is obvious from Figure 32 that the "50" and "70" curves developed by both procedures compare closely. Therefore the need index "70" is accepted as the warranting relationship for guardrail installation on low volume roads. Embankment with a combination of embankment height and slope lower than this line do not warrant the use of guardrails.

5.5 Trade-Off Between Guardrail Installation and Accident Cost

The guardrail installation must be compared to the accident costs for embankments of various heights and slopes. The effectiveness of guardrail installation depends on the embankment features. Glennon and Tamburi (24) summarized all reports of 1963 single vehicle down-the-embankment accidents in California. The data are grouped and categorized in each category of embankment height and slope. The form of the accident data used in the analysis is indicated in Table 15 and Table 16. An index of the severity of accidents is expressed as follow:

\[
\text{A.S.I.} = \frac{\text{Number of Fatal and Nonfatal Injury Accidents}}{\text{Total Number of Accidents}}
\]

This severity index is different than the SI severity index used before. Figure 30, 31 represent the relationship between embankment slopes, heights and accident severity index. Increases in embankment slope and height result in increases in accident severity.

Using the composite accident cost (CAC) of 3,139 dollars per accident as computed in Section 5.6 and the number of encroachments per mile
### Table 20  Accident Costs Related to Embankment Heights

<table>
<thead>
<tr>
<th>Embankment Height (feet)</th>
<th>Accident Severity Index A.S.I.</th>
<th>Number of Encroachment / mile/year</th>
<th>CAC ** (dollars) / Accident</th>
<th>Accident Costs (dollars) / mile / year</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>w/o shoulder</td>
</tr>
<tr>
<td>3</td>
<td>0.44</td>
<td>1</td>
<td>3,139</td>
<td>1</td>
</tr>
<tr>
<td>8</td>
<td>0.60</td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>15</td>
<td>0.64</td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>25</td>
<td>0.65</td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>35</td>
<td>0.75</td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>45</td>
<td>0.75</td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>60</td>
<td>0.77</td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>85</td>
<td>0.80</td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>125</td>
<td>0.83</td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>175</td>
<td>0.88</td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>350</td>
<td>0.95</td>
<td></td>
<td></td>
<td>1</td>
</tr>
</tbody>
</table>

* Probability of obstacle based on G.M. Proving Ground curve, see Figure 33.

** CAC is the composite accident cost.
Table 21  Accident Costs Related to Embankment Slopes

<table>
<thead>
<tr>
<th>Embankment Slopes</th>
<th>Accident Severity Index A.S.I.</th>
<th>Number of Encroachment/mile/year</th>
<th>CAC ** (dollars)/Accident</th>
<th>Accident Costs (dollars)/mile/year</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>w/o shoulder</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Prob.* Acc. Cost</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2-foot shoulder</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Prob.* Acc. Cost</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4-foot shoulder</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Prob.* Acc. Cost</td>
</tr>
<tr>
<td>5:1</td>
<td>0.27</td>
<td>1</td>
<td>3,139</td>
<td>1</td>
</tr>
<tr>
<td>4:1</td>
<td>0.42</td>
<td>1</td>
<td>3,139</td>
<td>1</td>
</tr>
<tr>
<td>3:1</td>
<td>0.48</td>
<td>1</td>
<td>3,139</td>
<td>1</td>
</tr>
<tr>
<td>2:1</td>
<td>0.62</td>
<td>1</td>
<td>3,139</td>
<td>1</td>
</tr>
<tr>
<td>1 1/2:1</td>
<td>0.70</td>
<td>1</td>
<td>3,139</td>
<td>1</td>
</tr>
<tr>
<td>1:1</td>
<td>0.78</td>
<td>1</td>
<td>3,139</td>
<td>1</td>
</tr>
</tbody>
</table>

* Probability of obstacle based on G.M. Proving Ground curve, see Figure 33.

** CAC is the composite accident cost.
per year, accident cost per mile per year can be calculated as follow:

Accident Cost/mile/year = (Number of encroachments per mile per year) x (Accident Severity Index) x (Probability of Obstacle) x (Composite Accident Cost per Accident)

Encroachment frequency is obtained from Figure 34. It is 1 encroachment per mile per year for an ADT of 400. Figure 33 indicates the probability of obstacle. The calculations of accident cost per mile per year related to the category of embankment heights and slopes are shown on Table 20 and Table 21, respectively. Figures 35 and 36 illustrate the accident costs per mile per year related to embankment heights and slopes for the roadway with 2-foot, 4-foot shoulders and without shoulders. If the cost of guardrail installation is known, the cost and accident cost of not having a guardrail can be expressed. The following is an example to define the trade-off between guardrail cost and accident cost.

Example: The following assumptions have been made:

1. Guardrail cost is 5 dollars per linear foot, thus guardrail cost per mile = $5 \times 5280 = $26,400
2. Service life (n) of guardrail is 20 years.
3. Salvage value (L) is 10 percent of the present cost.
4. Interest rate (i) is 6 percent.

Annual Capital Recovery Cost of guardrail installation per mile

\[
= (P-L) \left( \frac{A}{P}, i, n \right) + Li
= (26400-2640) \left( \frac{A}{P}, 6\%, 20 \right) + 2640(0.06)
\]
Figure 33  Distribution of Impacted Roadside Obstacles versus Distance from Edge of Pavement

Source: Reference 29.

Figure 34  Roadside Encroachment Frequency

Source: Reference 29.
Figure 35  Accident Costs versus Embankment Heights
Figure 36  Accident Costs versus Embankment Slopes
= 23760(.087^{1/3}) + 158.40

= $2,229.80

Based on Figures 35 and 36, for the roadway with 2-foot shoulder, an annual guardrail cost per mile ($2,230) can be plotted. From these analyses, an embankment height of 35 feet and an embankment slope of 1.5:1 should be used to obtain the balance between guardrail cost and accident cost of not having a guardrail. With an embankment height less than 35 feet and slopes flatter than 1.5:1, guardrail installation cost would exceed the accident cost.

When a new low volume road is being designed and a wider shoulder is required, this procedure can be used to determine economic locations for guardrail. Any increase in embankment earthwork cost to provide wider shoulders must be taken into account in addition to guardrail installation cost.

5.6 Trade-Off Between Clear Zone Area and Safety.

The accident potential and severity of lateral obstructions, or highway furniture, are perhaps the least understood contributors to safety on highways. Nearly one-third of all highway facilities occur when vehicles inadvertently leave the roadway and strike fixed objects. According to the 1976 Traffic Accident Report (68), approximately 50 percent of total rural accidents were run-off-the roadway accidents for county and local roads in the State of Oregon. By removing these obstacles and thus providing traversible roadsides, drivers of the errant vehicles would
have the opportunity to regain control of their cars. Figure 33 shows a plot of cases in which cars left the pavement based on studies by General Motors, Cornell Laboratory, Hutchinson and Huelke (29). These curves show close correlation except for the Cornell data. A 30 foot clear zone may be warranted on a high volume road. This zone is based on the fact that approximately 80 percent of the drivers who leave the road travel 30 feet from the edge of pavement (43). If the 30 feet can not be cleared of roadside obstacles due to practical or economic reasons, a guardrail may be warranted for the roadside areas. However, this concept cannot be adopted for low traffic volume roads because the lower frequency of off-roadway accidents make it impossible to justify it based on safety or economy. Figure 34 indicates the expected roadside encroachment per mile per year corresponding to average daily traffic, ADT. Roadside encroachment for low volume roads is quite low, 1 encroachment per mile per year at an ADT of 400, compared to 6 encroachments per mile per year for a high traffic volume of 10,000 ADT. The trade-off between clear zone and safety is evaluated by computing the balance between accident costs and clearing and grubbing costs.

Calculation of Accident Costs

The cost of accidents cannot be exactly determined because of various uncertainties. Based on The U.S. National Safety Council (34), the calculable costs of motor-vehicle accidents are wage losses, medical expense, insurance administration costs, and property damage. The costs of all these items per case in 1972 were:
Fatality $ 82,000
Non-fatal Disabling Injury $ 3,400
Property Damage Accident (include minor injuries) $ 480

Based on 1976 Traffic Accidents Report of Oregon for Rural, County and Local Road (68), the off-roadway accidents in Oregon were:

<table>
<thead>
<tr>
<th>Off-roadway accidents</th>
<th>Fatality</th>
<th>Injury</th>
<th>Property Damage Only</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>75</td>
<td>1,398</td>
<td>945</td>
</tr>
<tr>
<td>Percent of total accidents</td>
<td>3.1%</td>
<td>57.8%</td>
<td>39.1%</td>
</tr>
</tbody>
</table>

The total accident cost for run-off-the-road accidents can be calculated by:

\[
\text{Composite Accident Cost, } CAC = (FR \times CF) + (IR \times CI) + (PR \times CP)
\]

in which

\[
\begin{align*}
CAC &= \text{Composite accident cost,} \\
FR &= \text{Fatality rate per accident,} \\
IR &= \text{Injury rate per accident,} \\
PR &= \text{Property damage rate per accident,} \\
CF &= \text{Cost of a fatality,} \\
CI &= \text{Cost of a injury, and} \\
CP &= \text{Cost of a property damage.}
\end{align*}
\]

thus,

\[
CAC = (0.031 \times 82,000) + (0.578 \times 3,400) + (0.391 \times 480) = $ 4,696
\]

Experience has shown that normally only from 20 to 40 percent of property-damage-only accidents are reported to public authorities (74). To compensate, property-damage-only accident rates should be adjusted in the absence of more precise information, based on the degree of underreporting. The reported rates are multiplied by 2.5 to correct for
underreporting, which assumes a 20 percent reporting level. The adjusted composite accident cost would be:

<table>
<thead>
<tr>
<th></th>
<th>Fatality</th>
<th>Injury</th>
<th>Property-Damage-Only</th>
</tr>
</thead>
<tbody>
<tr>
<td>1976 Off-roadway accidents</td>
<td>75</td>
<td>1,398</td>
<td>945 \times 2.5 = 2,362</td>
</tr>
</tbody>
</table>

Percent of total accidents

\[
\text{Composite Accident Cost, CAC} = (0.02 \times 82,000) + (0.365 \times 3,400) + (0.616 \times 480) \\
= \$ 3,139
\]

Thus, off-roadway accident costs per mile per year can be computed from the following equation:

\[
\text{Accident Cost per mile per year} = \left( \text{Probability of a Roadside Obstacle} \right) \times \left( \text{Number of Encroachments per mile per year} \right) \times \left( \text{Composite Accident Cost} \right)
\]

**Example:** The following example demonstrates the use of this relationship. The calculation is made for a low volume road with an ADT of 400. The composite accident cost is $ 4,696 (with the absence of unreported PDO) per accident. The steps in calculation of accident cost per mile per year at distance 10 feet from the edge of pavement are as follows:

(a) At distance 10 feet from the edge of pavement, from Figure 33 percent of obstacle is 58% based on G.M. Proving Ground curve,

(b) For low volume road, ADT of 400, see Figure 34, number of encroachment per mile per year is 1,

(c) CAC with the absence of unreported PDO is $ 4,696 per accident.

\[
\text{Accident Cost / mile / year} = 0.58 \times 1 \times $ 4,696 = $ 2,724
\]
At other widths, such as 20 and 30 feet from the edge of pavement, the calculation can be made using the same procedure as above.

Calculation of Clearing and Grubbing Cost

Construction bidding prices for the item of clearing and grubbing cost has been surveyed from Federal Highway Projects in National Forests to obtain an estimate of these costs. Clearing and grubbing costs are found to vary from site to site and from project to project, depending on the density of trees in the area to be cleared. Table 22 summarizes the 1977 bidding cost of clearing and grubbing for both low and high bid costs in the Pacific Northwest.

The clearing and grubbing costs assumed here for calculation of trade-offs are:

<table>
<thead>
<tr>
<th></th>
<th>Low Estimate</th>
<th>High Estimate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Assumed clearing and grubbing cost</td>
<td>$ 500 / acre</td>
<td>$ 2,000 / acre</td>
</tr>
<tr>
<td></td>
<td>$60/mile/foot</td>
<td>$242/mile/foot</td>
</tr>
</tbody>
</table>

Using the accident costs and assumed clearing and grubbing costs, a family of curves has been plotted to determine the trade-off or the balance between accident cost and clearing and grubbing cost with varying distance from edge of pavement. The accident costs are both composite accident costs with and without unreported PDO. The accident costs per mile per year are also computed and plotted with varying distances from edge of pavement. The accident costs decrease with increasing distance from edge of pavement. Clearing and grubbing costs per mile required to eliminate the obstacles at varying distances are
Table 22  1977 Clearing and Grubbing Bidding Cost of Pacific Northwest

<table>
<thead>
<tr>
<th>Project Site</th>
<th>Low Bidding Cost</th>
<th>High Bidding Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wayan Freedom Highway, Caribou National Forest, Caribou County, Idaho</td>
<td>$ 120 / acre or</td>
<td>$ 700 / acre or</td>
</tr>
<tr>
<td></td>
<td>$ 14.78/mile/ft</td>
<td>$ 84.75/mile/ft</td>
</tr>
<tr>
<td>Yakima Indian Reservation, National Forest, Yakima, Washington</td>
<td>$ 2,000 / acre or</td>
<td>$ 4,000 / acre or</td>
</tr>
<tr>
<td></td>
<td>$ 202.62/mile/ft</td>
<td>$ 484.82/mile/ft</td>
</tr>
<tr>
<td>Mt. Hood National Forest, Clackamas County, Oregon</td>
<td>$ 2,700 / acre or</td>
<td>$ 10,000 / acre or</td>
</tr>
<tr>
<td></td>
<td>$ 327.27/mile/ft</td>
<td>$ 1,212.12 /mile/ft</td>
</tr>
<tr>
<td>St. Joe National Forest, Shoshone County, Idaho</td>
<td>$ 2,000 / acre or</td>
<td>$ 3,500 / acre or</td>
</tr>
<tr>
<td></td>
<td>$ 242.42/mile/ft</td>
<td>$ 424.26 /mile/ft</td>
</tr>
<tr>
<td>Defence Access Road, National Forest Kitsap County, Washington</td>
<td>$ 1,500 / acre or</td>
<td>$ 3,000 / acre or</td>
</tr>
<tr>
<td></td>
<td>$ 181.81/mile/ft</td>
<td>$ 363.63 /mile/ft</td>
</tr>
</tbody>
</table>
Figure 37  Accident Cost and Clearing&Grubbing Cost Related to Distance from Edge of Pavement
also calculated. Both high and \( \frac{1}{3} \) estimates of assumed costs are used for calculation. The costs of clearing and grubbing increase proportional to the increase of distance from edge of pavement. These relationships have been superimposed on Figure 37. From this analysis the required clear zone from the intersection points of accident cost and clearing and grubbing cost can be determined. These points represent the balance between accident cost and clearing cost. Thus, the right-of-way zone can be determined directly from these relationships. In addition, the lines are plotted from the points of 10 and 20 feet from edge of pavement, parallel to the line passing through the origin to represent the conditions resulting from construction or other prior clearing. If the area is already cleared during construction to 10 feet, the clear distance needed is the intersection point of the accident curve and clearing and grubbing line drawn from a 10 ft. point.

The use of this graph is as follows: if clearing and grubbing cost of a project is known, the relationship between the cost and distance must be drawn. The intersection of that line and accident curve is the requirement for clear zone and right-of-way. For example, if a project has high assumed clearing and grubbing cost and accident cost with absence of unreported PDO, the required clear zone is 10.5 feet from edge of pavement. If the area already cleared during construction is 10 feet, the clear zone of 16.5 feet is required or only 6.5 feet to be cleared.
6.1 A Criteria for Optimizing Level of Safety

The trade-off analysis developed here is based on safety and economic criteria. The trade-offs represent the relationships between design standards and accident costs. Based on these findings and relationships, the accident cost savings to improve a roadway to an acceptable standard can be computed. The accident cost savings or the expected total loss in safety and operation, can be used to optimize the design standards and safety measures. A higher level of safety can be achieved if the designers or decision makers attempt to minimize the expected total loss or maximize the accident cost saving. Equation (8) is used to calculate the expected loss in safety and vehicle operation in terms of accident cost, vehicle running cost and travel time cost.

Dart, Mann (18) and Sparks (59) have found that the probability of accidents explained by roadway characteristics is 0.45. The remaining 55 percent is accountable to other factors, the driver and vehicle errors.

The expected loss due to safety and vehicle operation can be calculated from the following equation.

\[
TLS = p_a \ AC + RC + TC
\]

where

\( TLS \) = the expected total loss in safety and operation,
\( AC \) = the accident cost changes associated with variations in roadway elements, such as curvature, downgrade, lane width (pavement width) and embankment conditions,
RC = the incremental vehicle running costs due to variations of degree of curvature and grades,

TC = the added travel time costs due to the effects of horizontal and vertical alignments, and

\( p_a = 0.45 \), the probability of accidents explained by roadway characteristics.

If a roadway has been constructed with low standards, the accident costs and vehicle operation costs are typically high. A higher level of safety and reduced accident costs can be achieved with improved roadway standards. For existing low volume roads, spot improvements are preferred rather than the improvement of extended lengths of highway (51). To select the best solution, comparison must be made between benefit of reduced accident costs and vehicle operating costs and increased costs in construction and maintenance. An accepted interest rate level should be used to consider the time value of benefits and costs. The benefit-cost ratio method is an accepted approach for economic analysis. The following example calculates the reduction in accident costs, vehicle running cost and time cost.

6.2 Example of Calculation

Existing and improved geometric features are assumed. The related figures and charts are available in Chapter II through Chapter V.

<table>
<thead>
<tr>
<th>Existing Geometric Features</th>
<th>Improved Geometric Features</th>
</tr>
</thead>
<tbody>
<tr>
<td>9-foot lane width</td>
<td>10-foot lane width</td>
</tr>
<tr>
<td>2-foot shoulder width</td>
<td>4-foot shoulder width</td>
</tr>
<tr>
<td>30 degree of curvature</td>
<td>20 degree of curvature</td>
</tr>
</tbody>
</table>
Sideslope 1:1
Side ditch 1 foot
Central Angle = 60°
Approach Speed 50 mph
ADT = 100
Traffic Stream: (Assumed)
90% of 4-kip passenger car
10% of 12-kip single unit truck

Accident Rates:
9' Lane Width; 3 accidents/MVM

2' shoulder; .25 accidents/mile/year

30° curvature; 6 accidents/MVM

Vehicle Running Cost:
4-kip Passenger Car;
30° curvature; $763/year

12-kip Single Unit Truck;
30° curvature; $1,838/year

10' Lane Width; 2.2 accidents/MVM (After Raff, Figure 8)
4' Shoulder; .20 accidents/mile/year (After Belmont, Figure 11)
20° curvature; 4 accidents/MVM (After Raff, Figure 16)

20° curvature; $664/year (After Figure 27a)

20° curvature; $1,493/year (After Figure 27b)
Time Consumed:

4-kip Passenger Car;
30° curvature; 160 hrs/year 20° curvature; 136 hrs/year
(After Figure 29a)

12-kip Single Unit Truck;
30° curvature; 222 hrs/year 20° curvature; 180 hrs/year
(After Figure 29b)

Calculation of Accident Cost per Million Vehicle Mile (MVM)

According to 1975 Accident Rates on Federal-Aid and Other Highway Systems issued by U.S. Department of Transportation (69), the accident rates in Oregon are as follows:

Rural Non-State System: (Local Roads);

<table>
<thead>
<tr>
<th></th>
<th>Fatality</th>
<th>Injury</th>
<th>Property Damage Only</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accident rates per MVM</td>
<td>3.56</td>
<td>197.96</td>
<td>183.72</td>
</tr>
<tr>
<td>Percent of total accidents</td>
<td>0.92%</td>
<td>51.35%</td>
<td>47.72%</td>
</tr>
</tbody>
</table>

Thus, the composite accident cost per MVM = 82,000 x 0.0092 + 3,400 x 0.5135 + 480 x 0.4772

= $ 1,975 per MVM

Reduction in Accident Costs of Improved Geometric Features over the Existing Features.

Lane Width; Accident Reduction = 3-2.2 = 0.8 accidents/MVM
Shoulder; Accident Reduction = 0.25-0.20 = 0.05 accidents/MVM
Degree of Curvature; Accident Reduction

= 6-4 = 2 accidents/MVM

Total Accident Reduction = 2.85 accidents/MVM

Reduction in Accident Cost = 2.85 x $ 1,975 = $ 5,629/year
Reduction in Vehicle Running Cost:

4-kip Passenger Car;

Vehicle Running Cost Reduction

= $ 763 - $ 664
= $ 99 / year

12-Single Unit Truck Reduction

= $ 1,838 - $ 1,493
= $ 345 / year

Total Vehicle Running Cost Reduction

= 0.9 x $99 + 0.1 x $345
= $ 124 / year

Reduction in Time Costs

4-kip Passenger Car;

Time Reduction

= 160 - 136 = 24 hrs/year

12-Single Unit Truck;

Time Reduction

= 222 - 180 = 42 hrs/year

Total Time Reduction

= 0.9 x 24 + 0.1 x 42
= 26 hrs/year

The time cost per average vehicle hour as determined by Oglesby and Altenhofen (51) is assumed. Trips for market and extra-market activities have been arbitrarily assumed for calculating the travel time cost. Time cost is $ 1.84 per average vehicle hour.

Total Reduction in Time Cost

= 26 x $ 1.84
= $ 48 / year

substituting the costs into equation [8],

The Expected Total Loss, T\(T\)L\(S\) = 0.45 AC + RC + TC

= 0.45($5,629) + $124 + $48
= $ 2,705 per year
Decision Making. Based on analysis and evaluation using these costs, an informed decision on the appropriate standards can be made. By using the procedures as shown, it is possible to determine geometric standards that are economic, safe and provide for a high level of service. These costs must be evaluated against incremental construction costs. The decision maker must determine if the improved safety and increased level of service are worth the added road costs. An accepted economic analysis technique, such as benefit-cost ratio or rate of return, must be used.
7.1 Conclusions

Many procedures are developed in this study to determine the trade-offs between geometric design standards and safety for low volume roads. Existing design standards for low volume roads are evaluated relative to safety. Data and previous findings are used to analyze the trade-offs. The results have been concluded as follows:

(1) Based on safety and economic criteria, 10-foot lane width is suggested for low volume roads. From the surveyed data of Oregon, 10-foot lane width is generally employed on much of the county road system.

(2) Two-foot shoulder should be used rather than using a wider or no shoulder. This is to provide safety and emergency uses for vehicles.

(3) The required sight distance corresponding to speed effects on safety for driver on various curves with the restriction of backslopes is analyzed. The critical speeds that vehicles can travel on various curvatures for each backslope conditions, can be determined. Assumptions must be made to re-evaluate for a particular roadway if it differs from the cross-section elements assumed for this study.

(4) Horizontal and vertical alignment conditions influence vehicle running costs and travel times. Both added vehicle running costs and travel time consumed are computed. They increase with increasing curvature and grades. Downgrades greater than 3 percent
show increases in running costs.

(5) Trade-off between guardrail installation and accident cost have been developed. The annual cost of guardrail installation on a particular location is compared to the costs for expected accidents. The balance between potential accident cost and guardrail installation cost can be used to obtain critical embankment heights and slopes.

(6) The balance between clear zone from the edge of pavement (for consideration of right-of-way requirement) and accident cost can be used to determine the area of clear zone that must be provided. Due to the variations of clearing and grubbing cost from project to project, the relationship between clear zone and accident cost should be performed for a specific project using the procedure described.

7.2 **Recommendations**

A number of recommendations are made:

(1) To obtain a precise result for decision making, accident data used for trade-off analysis on a particular location should be obtained and categorized relating to geometric features of a roadway. Statistical analysis may be employed to predict the accident rates.

(2) The procedures to determine the balance between guardrail installation cost, clearing and grubbing cost and accident costs are recommended for low volume roads. Different assumptions may be made to fit a particular project.

(3) For decision making, the comparison between accident savings
due to improvements and the incremental construction cost must be made. Not only safety criteria but also economic, financial, social and environmental criteria must be considered for selection of design standards for low volume roads.

(4) Additional research is recommended to relate accident rates to shoulder width for low volume roads since little research has been done in this area.
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APPENDIX A

CHARTS AND FIGURES FOR COMPUTING VEHICLE RUNNING COST AND TRAVEL TIME
Appendix A

Charts in Appendix A are used for computing vehicle running costs influencing by horizontal and vertical alignments, and travel times consumed. Figure 26, 26 (a) and 28 are the flow charts for finding added running cost and travel time, respectively. Computations can be made by utilizing charts and figures in Appendix A as described in each flow chart. These charts have been developed by Oglesby, et al (50) of Stanford University. Figure A-1 to A-13 are utilized to calculate vehicle running cost and Figure A-14 to A-18 are of travel time calculations.

Figure A-3 represents the running cost for vehicle operating at uniform speed on level tangents. On the contrary, Figure A-3a to A-3e show the running costs for vehicle operating at uniform speeds on grades. As shown in Figure 26 (a), the different is just the use of Figure A-3a to A-3e in lieu of Figure A-3 which is used for the procedures in Figure 26.
Fig A-2: Total distances on two tangents for various central angles and sharpnesses of curves.

Note: Loch gronhed Ilne represents curves of equal central angle from 20 degrees to 160 degrees.
Fig. A-3  RUNNING COSTS FOR 100 VEHICLES OPERATING AT UNIFORM SPEED ON LEVEL TANGENTS.
Fig A-3a  RUNNING COST PER 100 STATIONS - 4 KIP PASSENGER CARS OPERATING AT UNIFORM SPEED.

Fig A-3b  RUNNING COST PER 100 STATIONS - 5 KIP PICKUPS OPERATING AT UNIFORM SPEEDS.
Fig. A-3a  RUNNING COST PER 100 STATIONS-12 KIP SINGLE UNIT TRUCK OPERATING AT UNIFORM SPEED.

Fig. A-3b  RUNNING COST PER 100 STATIONS-23 KIP UNLOADED LOGGING TRUCKS OPERATING AT UNIFORM SPEED.
Fig A-3a RUNNING COST PER 100 STATIONS - 80 KIP LOADED LOGGING TRUCKS OPERATING AT UNIFORM SPEED.
Fig. A-4  EXCESS RUNNING COST FOR 100 VEHICLES OPERATING AT UNIFORM SPEED THROUGH HORIZONTAL CURVES

Fig. A-5  EXCESS RUNNING COST FOR 100 VEHICLES OPERATING AT UNIFORM SPEED THROUGH HORIZONTAL CURVES
Fig. A-6 EXCESS RUNNING COST FOR 100 VEHICLES OPERATING AT UNIFORM
SPEED THROUGH HORIZONTAL CURVES

Fig. A-7 EXCESS RUNNING COST FOR 100 VEHICLES OPERATING AT UNIFORM
SPEED THROUGH HORIZONTAL CURVES
Fig. A-8 EXCESS RUNNING COST FOR 100 VEHICLES OPERATING AT UNIFORM SPEED THROUGH HORIZONTAL CURVES

Note: These lines represent equal radii of curve (from 175' to 25').

Vehicle type: 80 kip loaded logging truck.

Fig. A-9 INTERIM SPEED IN MILES PER HOUR FOR 100 VEHICLES SPEEDING AND RETURNING BACK TO INITIAL SPEED.

Vehicle type: 4 kip automobile.
Fig. A-17  EXCESS RUNNING COST FOR 100 VEHICLES SLOWING FROM INITIAL SPEED TO INTERIM SPEED AND RETURNING BACK TO INITIAL SPEED
Vehicle type: 12 kip single unit truck

Fig. A-18  EXCESS RUNNING COST FOR 100 VEHICLES SLOWING FROM INITIAL SPEED TO INTERIM SPEED AND RETURNING BACK TO INITIAL SPEED
Vehicle type: 5 kip commercial delivery
Note: All curves represent lines of equal initial speed (in mph).

Fig. A-13 EXCESS RUNNING COST FOR 100 VEHICLES SLOWING FROM INITIAL SPEED TO INTERIM SPEED AND RETURNING BACK TO INITIAL SPEED

Vehicle type: 80 kip loaded logging truck

Fig. A-12 EXCESS RUNNING COST FOR 100 VEHICLES SLOWING FROM INITIAL SPEED TO INTERIM SPEED AND RETURNING BACK TO INITIAL SPEED

Vehicle type: 23 kip unloaded logging truck
Fig. A-14  TIME CONSUMED BY 100 VEHICLES TRAVELING AT UNIFORM SPEED
Note: Graphed curves represent lines of equal initial speed (in miles per hour).

Fig. A-18
Excess time consumed for 100 speed change cycles slowing from initial speed to interim speed and returning back to initial speed.

Vehicle type: 5 kip commercial delivery.

Fig. A-15
Excess time consumed for 100 speed change cycles slowing from initial speed to interim speed and returning back to initial speed.

Vehicle type: 4 kip automobile.
Fig A-18 EXCESS TIME CONSUMED FOR 100 SPEED CHANGE CYCLES SLOWING FROM INITIAL SPEED TO INTERIM SPEED AND RETURNING TO INITIAL SPEED

Vehicle type: 80 kip loaded logging truck

Note: Graphed curves represent lines of equal initial speed (in miles per hour).
APPENDIX B

QUESTIONNAIRE FORMAT
QUESTIONNAIRE

NOTE: If you do not have sufficient room to write your standards or data in the available space, please write on the back of the sheets. If you have formal (published) standards that are unique for your jurisdiction, please enclose a copy.

GEOMETRIC DESIGN STANDARDS FOR LOW VOLUME ROADS:

The following items deal with existing design standards being used for roads in the county network carrying volumes of 500 vehicles per day.

1) Design Speed:

Variations with traffic volume?

Variations with road class?

Variations with terrain?

Other?

2) Design Vehicle:

Vehicle type

Gross vehicle weight (max.) No. of axles

Single axle load (max.) Double axle load (max.)

Vehicle length Wheel base

3) Cross-section Elements:

Shoulder width:

Variations with road class?
Variations with traffic volume?

Variations with terrain?

Shoulder type:
Variations with road class?

Variations with traffic volume?

Variations with terrain?

Lane Width:
Variations with road class?

Variations with traffic volume?

Variations with terrain?

Sideslope (typical in natural soil):
Cut section
Variations with depth of cut?

Fill section
Variations with height of fill?

Crown slope (for drainage):
Surface crown slope

Shoulder slope
4) **Right-of-way (Distance from edge of shoulder):**

   Minimum _______________ feet
   Desirable _______________ feet

5) **Pavement type:**

   Variations with traffic volume?
   ____________________________
   ____________________________
   ____________________________
   ____________________________

   Variations with gross vehicle weight?
   ____________________________
   ____________________________
   ____________________________
   ____________________________

6) **Horizontal alignment:**

   Maximum degree of curvature _______________
   ____________________________
   ____________________________
   ____________________________

   Variations with design speed?
   ____________________________
   ____________________________
   ____________________________

   Variations with annual average daily traffic volume?
   ____________________________
   ____________________________
   ____________________________
   ____________________________

   Variations with terrain:
   Flat topography _______________
   ____________________________
   ____________________________

   Rolling topography _______________
   ____________________________
   ____________________________

   Mountainous topography _______________
   ____________________________
   ____________________________

   Maximum superelevation _______________
   ____________________________
   ____________________________

   Criteria to select less than maximum superelevation?
   ____________________________
   ____________________________
   ____________________________
   ____________________________

   Conditions to modify maximum superelevation?
   ____________________________
   ____________________________
   ____________________________
   ____________________________

   Safe side friction factor _______________
   ____________________________
   ____________________________

   Variations with speed?
   ____________________________
   ____________________________
   ____________________________
   ____________________________
7) Vertical alignment:

   Maximum grade

   Variations with terrain?

   Variations with traffic volume?

   Others?

Sight distance:

   Stopping sight distance

   Passing sight distance

   Variations with design speed?

8) Roadsides:

   Side ditch (for cut section), Depth____ ft, Width____ ft

   Sideslopes

   Culvert headwalls:

   Have they been used for secondary roads, local roads, and other low volume roads?

   Distance from edge of shoulder

   Guardrail:

   Type of guardrail

   Distance from edge of shoulder?

   Maximum sideslope before guardrails must be used?

   Height of fill
<table>
<thead>
<tr>
<th>Traffic signs and markings:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance from edge of shoulder to signposts?</td>
</tr>
<tr>
<td>Delineators used?</td>
</tr>
<tr>
<td>Fog stripe used?</td>
</tr>
<tr>
<td>Variations with road class (secondary, local roads)?</td>
</tr>
<tr>
<td>Other</td>
</tr>
</tbody>
</table>

9) General comments of relation of design geometrics to safety: