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Development of Guardrail Runout Length Calculation Procedures

Dean L. Sicking and Daniel F. Wolford

Department of Civil Engineering
College of Engineering and Technology

W348 Nebraska Hall
Lincoln, Nebraska 68588-0531
Telephone (402) 472-2371
FAX (402) 472-8934

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16. Abstract

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ABSTRACT

Guardrail runout length recommendations contained in AASHTO's Roadside Design Guide were re-evaluated in order to determine optimal guardrail length of need. Encroachment data collected along Canadian highways (4) was evaluated in a manner similar to that used with Hutchinson and Kennedy encroachment data (3) to reproduce procedures contained in AASHTO's Roadside Design Guide. A benefit cost analysis of extending guardrails was also conducted as an alternative procedure for defining appropriate guardrail runout lengths. Both analysis procedures indicate that guardrail runout lengths recommended by the Roadside Design Guide are excessive. Highway agencies are recommended to select one of the two new procedures for identifying appropriate guardrail runout lengths.

CHAPTER 1. INTRODUCTION

Strong-post W-beam guardrails are often installed along highways to protect traffic from serious roadside hazards such as bridge piers, bridge abutments, and other rigid hazards. In these situations, guardrail must be placed both in front of and upstream from the roadside hazard in order to provide a reasonable level of protection for motorists. The theoretical distance required for a vehicle that has left the roadway to come to a stop is called the guardrail runout length, and the guardrail length-of-need is the distance that the guardrail is extended upstream from the hazard. As runout lengths are increased, the likelihood of a vehicle running behind the barrier and impacting the shielded hazard is reduced. However, the number of vehicles impacting the guardrail grow as guardrail runout lengths are increased. Unfortunately, guardrail itself is a moderately severe hazard and causes approximately 1300 fatalities along our nation's highways every year. In fact, accident data analysis indicates that approximately 13 percent of reported guardrail accidents involve vehicle rollover and almost 2 percent of all reported guardrail accidents produce a fatality (1). As guardrail runout lengths become large, the reductions in serious impacts on the shielded hazard will become less than the increase in serious guardrail accidents. Thus, there is an optimum guardrail runout length that will produce a minimum number of injury and fatal accidents.

Runout length recommendations contained in the 1989 AASHTO Roadside Design Guide (RDG) (2) are based on findings from Hutchinson and Kennedy's (3) study of encroachments into snow covered medians on rural interstate highways.

Researchers analyzed distances that errant vehicles traveled along the snow covered medians in order to select the appropriate design values for guardrail length-of-need calculations. Thus, the current procedures for determining guardrail runout lengths do not consider the severity of guardrail impacts. Instead they are designed to reduce, to a very low level, the number of vehicles impacting the shielded hazard.

However, inherent flaws in the Hutchinson and Kennedy research may increase guardrail runout lengths to excessive levels, even if guardrails did not produce any injuries. Snow covered medians included in Hutchinson and Kennedy's study produced much longer stopping distances for these vehicles than would normally occur on roadsides not covered with ice. Although Nebraska's roadsides are sometimes covered with ice and snow, it is unreasonable to design guardrail installations based solely on ice covered conditions due to the relatively limited portion of each year that this condition exists. The 70-mph speed limit associated with rural Illinois interstates during the late 1960's and early 1970's further increased distances that vehicle traveled along the medians during Hutchinson and Kennedy's study. Encroachment data collected along Canadian highways (4) provides the potential for revising the Roadside Design Guide procedures to eliminate some of these flaws.

Recently developed benefit/cost analysis routines are capable of assessing the safety benefits of increased runout lengths. These programs estimate the number of serious injury and fatal accidents that will occur as a result of vehicles impacting a guardrail and traveling behind the barrier to impact the roadside hazard. Thus, benefit/cost analysis procedures offer another method to determine appropriate guardrail runout lengths.

The research described in this report was undertaken to re-evaluate the appropriateness of guardrail runout length calculations contained in AASHTO's Roadside Design Guide. The objectives of the research included: (1) revise the RDG runout length calculations in consideration of the more recent encroachment data collected in Canada; (2) utilize a state-of-the-art benefit cost analysis program to determine optimum guardrail runout lengths; and (3) develop simplified guidelines for determining the most appropriate guardrail runout lengths based on the previous findings.

CHAPTER 2. RESEARCH APPROACH

The research described in this report involved a two-phase approach to re-examining procedures for determining guardrail length-of-need. The first method involved revising runout length tables included in AASHTO's Roadside Design Guide (2) to reflect findings from a major encroachment study conducted in Canada (4). The second approach involved conducting a benefit/cost analysis of guardrail installations to determine the optimum runout lengths. Details of each of these procedures are described in the following two sections.

Revised Runout Length Tables

The RDG guardrail length-of-need calculation procedures are based on the philosophy that barriers should be designed to give errant vehicles sufficient opportunity to come to a controlled stop prior to impacting a roadside hazard. Thus, guardrails are designed to successfully redirect all vehicles leaving the roadway within a given distance of the hazard. This is called the runout length. Figure 1, excerpted

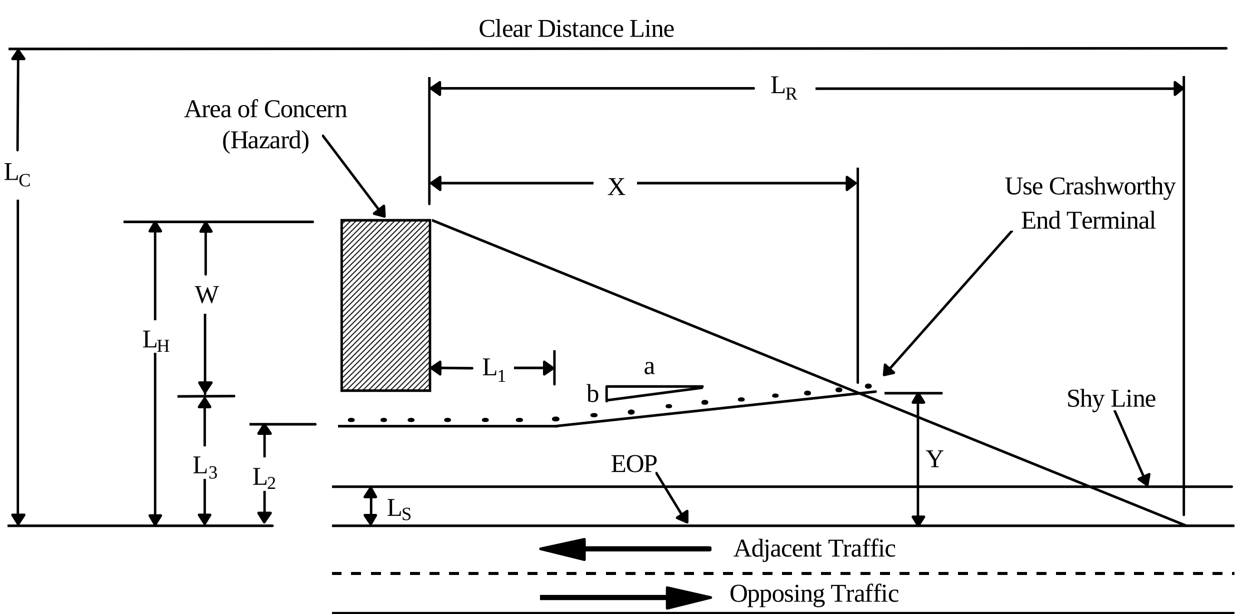

FIGURE 1. APPROACH BARRIER DIAGRAM.

from AASHTO's Roadside Design Guide, illustrates this basic philosophy for guardrail length-of-need calculation. As shown in this figure, the required guardrail length-of-need, X, is selected based on the runout length, L_R, the lateral offset to the back of the hazard, L_H, and the barrier flare configuration.

This procedure is based on the assumption that runout lengths are selected to reduce, to an acceptable level, the number of vehicles running behind the barrier and impacting the hazard. As shown in Table 1, runout lengths recommended in the Roadside Design Guide are based on highway design speed and traffic volume. The relationship between runout length and design speed is associated with the correlation between vehicle operating speeds and stopping distances. The adjustment in L_R and traffic volume is based on a recognition of the reduced level of service provided by low volume roadways.

TABLE 1. RECOMMENDED RUNOUT LENGTHS.

	Traffic Volume (ADT)			
	>6000	2000-6000	800-2000	<800
Design Speed (mph)	Runout Length Lr(ft)	Runout Length Lr(ft)	Runout Length Lr(ft)	Runout Length Lr(ft)
70	480	440	400	360
60	400	360	330	300
50	320	290	260	240
40	240	220	200	180
30	170	160	140	130

An investigation of the source of the Roadside Design Guide procedures revealed that this technique originated during the development of the 1977 AASHTO Guide for Selecting, Locating, and Designing Traffic Barriers. The procedure used to develop the distances in Table 1 is contained in an unpublished report by James Hatton(16). The runout lengths in this table have their basis in findings from Hutchinson and Kennedy's study of median encroachments on interstate highways in Illinois (3). Runout lengths, L_R, for 70 mph design speeds were obtained from a distribution of distances traveled by encroaching vehicles along the highways included in this study. These distances were then reduced proportionately for lower design speeds based on stopping distance reductions associated with lower operating speeds.

Cooper (4) collected encroachment data along Canadian highways during the summer of 1979. Vehicle tracks in the grass along rural highways were studied in order to determine the frequency and nature of roadside encroachments. Cooper's research project ignored winter driving conditions while Hutchinson and Kennedy's study included only winter driving conditions. Cooper's data is believed to be more appropriate for use in the selection of guardrail length-of-need, since friction on grassy roadsides during summer months should be more representative of average accident conditions than snow covered medians.

As described in Chapter 3, Cooper's data was used to recreate Table 1 in a manner similar to the way Hutchinson and Kennedy's data was used to develop the original table.

Benefit/Cost Analysis Procedure

A Benefit/Cost analysis is often utilized to evaluate the relative merits of two safety treatment options. These techniques attempt to estimate the number and severity of roadside accidents associated with each safety treatment option. The benefits of a safety improvement, measured in terms of reductions in accident costs, are then compared to the direct highway agency costs associated with the improvement. A safety improvement may be installed if the estimated benefits of a specific design exceed the cost of constructing and maintaining that design over a period of time. The research approach incorporated for the runout length study involved evaluating increasing lengths of guardrail until the benefits of extending the barrier become less than the associated cost.

The most important component of any benefit/cost analysis is the severity of accidents predicted to occur. For evaluation of lengths-of-need, the severity of a guardrail accident is of primary importance. Guardrail severities were estimated by a combination of computer simulation of guardrail impacts and examination of guardrail accident data collected in five states. The severity of the roadside hazard behind the barrier was selected to be representative of a rigid obstacle. Note that this hazard severity would cause the procedure to select longer barrier lengths-of-need than if less severe obstacles were analyzed.

CHAPTER 3. UPDATE OF ROADSIDE DESIGN GUIDE PROCEDURE

As described above, runout lengths recommended in the Roadside Design Guide are based primarily on findings from an encroachment study conducted along medians on Interstates 74 and 57 in Illinois. Encroachments were detected by regular visual inspections of the medians. At the time of the data collection, Interstate 74 had 80 ft depressed grass medians. Data was collected on this highway continuously for over three years from the fall of 1960 to the spring of 1964. This period included four winter travel periods during which snow covered medians were not uncommon. Interstate 57 incorporated a narrow median averaging 24 ft wide, much of which was paved. In order to detect encroachments on the paved portion of the median, data was collected from Interstate 57 during three winter driving seasons between 1957 and 1960.

Thus, Hutchinson and Kennedy data incorporated an unreasonably large proportion of winter driving conditions when encroachment distances could be expected to be unreasonably large. Further, encroachment lengths appear to be measured not parallel to the roadway, but along the path of the encroaching vehicle's left front tire. As defined in the Hutchinson and Kennedy study, the length of travel would be significantly larger than the actual distance the vehicle travels parallel to the roadway. Although Hutchinson and Kennedy's study does not appear to be well suited for determining appropriate guardrail lengths-of-need, it was the only source of encroachment research available at the time the Roadside Design Guide procedures were first developed. Figure 2 presents length of travel distances observed by Hutchinson and Kennedy. As shown in this Figure, the 80^{th} percentile length of travel

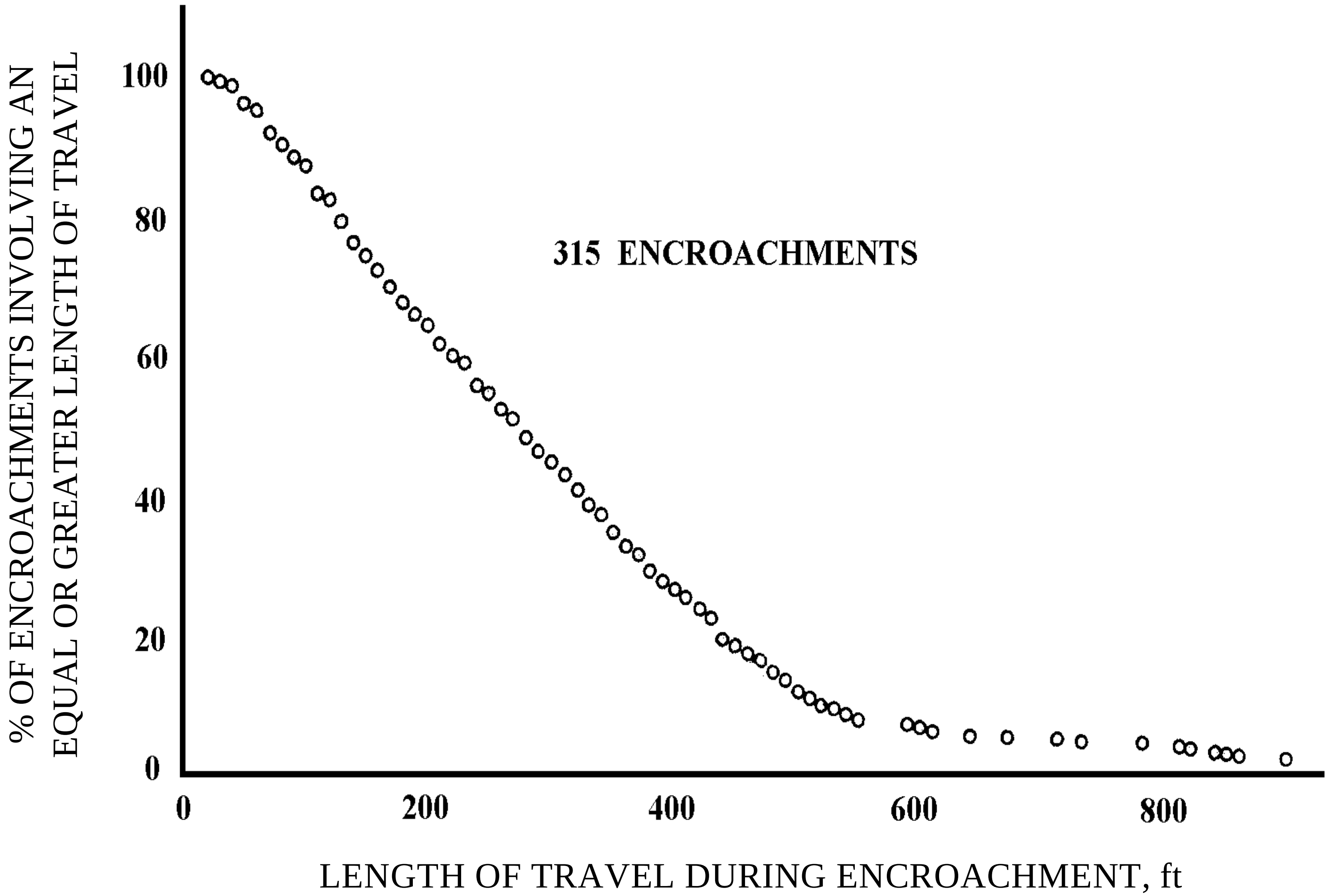

FIGURE 2. HUTCHINSON AND KENNEDY'S TRAVEL DISTANCE CURVE.

observed during this study was approximately 440 ft. The design speed for the two interstate highways included in Hutchinson and Kennedy's study was 70 mph. The majority of data was collected on roadways with traffic volumes ranging from 1700 to just under 6000 vehicles per day. In fact, only 25 encroachments out of 315 originated on highways with a traffic volume greater than 6000 ADT. The runout length, L_R , recommended by the Roadside Design Guide for design speeds of 70 mph with traffic volumes ranging between 2000 and 6000 ADT was set at 440 ft. Thus, it has been assumed that this entry in the runout length selection table appears to be set at the 80th percentile length of travel distance from the Hutchinson and Kennedy's study. As shown in Table 2, runout lengths for lower volume roadways appear to have been selected based upon higher allowable levels of risk from this same study. This method does not imply that actual vehicle runout lengths are expected to be directly correlated with traffic volumes for a given design speed. Higher allowable levels of risk were incorporated in recognition of the generally lower levels of service provided on low volume roadways.

TABLE 2. RUNOUT LENGTH TABLE BASED ON ALLOWABLE RISK.

	Traffic Volume (ADT)			
	>6000	2000-6000	800-2000	<800
Design Speed (mph)	Runout Length (ft)	Runout Length (ft)	Runout Length (ft)	Runout Length (ft)
Percentile of Runout	85%	80%	75%	70%
70	480	440	400	360

The procedure for reducing runout lengths for lower design speeds was carefully investigated. The first attempts were based on the assumption that lower design speeds should be directly correlated to reduced operating speeds and that runout length recommendations should be directly related to estimated stopping distances. Vehicle stopping distances can be calculated as shown in Equation 1.

$$X = 1.47\, V_d\, t + \frac{(1.47\, V_d)^2}{2\, \mu\, g} \qquad \textbf{(1)}$$

where:

X	= vehicle stopping distance, ft.
V_d	= design speed, mph.
t	= driver perception/reaction time, seconds.
μ	= coefficient of friction along roadside.
g	= acceleration due to gravity, ft/sec^2.

This analysis indicated that runout distances shown in Table 1 could be replicated by setting the perception reaction time to 3 seconds and the coefficient of friction to 0.95. Both of these values were believed to be unreasonably high and further efforts were made to determine how the Roadside Design Guide runout length table was generated.

The actual equation used to extrapolate the runout lengths to lower design speeds was obtained from James Hatton's unpublished report "A Roadside Design Procedure" (16) and is shown as Equation 2. This equation was derived mainly from engineering judgement. The author reasoned that the equation could have some relationship to perception-reaction time and breaking distance with an assumed coefficient of friction of .5. The runout length equation was then calibrated to match Hutchinson and Kennedy's data by multiplying coefficient of 0.3 and 0.7 to the perception and braking distances respectively.

$$L_{R_d} = L_{R_{70}} \left[.3 \left(\frac{V_d}{V_{70}} \right)^2 + .7 \left(\frac{V_d}{V_{70}} \right) \right] \qquad \textbf{(2)}$$

where:

LR_d = runout length associated with the design speed of interest, Vd.
LR_{70} = runout length associated with the 70 mph design speed.
V_d = design speed of interest.
V_{70} = 70 mph design speed.

During the search for the procedure described above, an equivalent procedure that is move intuitive was developed using a proportionate reduction technique based on highway design speeds, as shown in Equation 3. Table 3 presents runout lengths that would be generated by implementing the reduction procedure shown in Equation 3.

$$L_{R_{(i)}} = \frac{L_{R_{(i-1)}} [\, 2V_{(i)} - V_{(i-1)} \,]}{V_{(i)}} \qquad \textbf{(3)}$$

where:

i = index ranging from 2 to 5 with increments of 1.
$LR_{(i)}$ = runout length associated with design speed i.
$LR_{(i-1)}$ = runout length associated with design speed (i-1).
V_i = design speed i.
$V_{(i-1)}$ = design speed (i-1).

Only minor differences are apparent when comparing runout lengths shown in Tables 1 and 3. These relatively minor differences were attributed to adjustments in the procedure based on engineering judgement.

TABLE 3. RUNOUT LENGTH REDUCTION PROCEDURE.

		Traffic Volume (ADT)			
		>6000	2000-6000	800-2000	<800
Design Speed (mph)	i	Runout Length (ft)	Runout Length (ft)	Runout Length (ft)	Runout Length (ft)
70	1	480	440	400	360
60	2	400	367	333	300
50	3	320	293	267	240
40	4	240	220	200	180
30	5	160	147	133	120

Revised Runout Lengths Based on Canadian Data

The philosophy of selecting appropriate runout lengths based on the nature of observed encroachments was implemented using findings from a larger encroachment study conducted along Canadian highways (4). Cooper collected encroachment data along rural highways by observing tire tracks along grassy roadside during five summer months. Data was collected along approximately 6300 miles of rural highway, including 48 two-way, undivided roadway sections and 12 sections along four-lane, divided highways scattered across Canada. The speed limits along the study sections ranged from 75-100 km/h or 47-62 mph. In total, data was collected on approximately 1950 roadside encroachments.

Although the Canadian data suffers from some problems similar to those found in the Hutchinson and Kennedy data, the magnitude of these problems should be significantly reduced. Of primary concern to the guardrail runout length problem is the effects of these study errors on the distances that vehicles travel parallel to the highway. As discussed previously, over representation of snowy and icy conditions, measuring travel distances along the vehicle path instead of parallel to roadway, and excessive speed limits are all believed to increase the distribution of longitudinal travel distances obtained from Hutchinson and Kennedy. Alternatively, Cooper's data was collected only during summer months; and therefore, under represents snowy and icy roadside conditions. However, Cooper correctly measured longitudinal runout distances and collected data on highways with a variety of speed limits in the general range of those found on most modern U. S. highways. Further, even though snow and ice generally covers Nebraska roadsides during some portion of the winter, summer weather patterns in Canada are probably more representative of average roadside conditions in Nebraska than are snow covered medians. Therefore, Cooper's longitudinal travel distances are believed to be more appropriate for use in developing guardrail runout length selection tables than Hutchinson and Kennedy's findings.

Evaluation of Cooper's Data - Encroachment data collected by Cooper was evaluated to determine appropriate methods for refining and segregating the data. A preliminary analysis of the distributions of longitudinal and lateral distances collected during this study revealed that a number of encroachments were recorded as having zero lateral or longitudinal distance. Further, some of the encroachment records were missing important data elements or had serious inconsistencies. After all of these records were eliminated from the study 1620 encroachments remained for use in establishing longitudinal runout length distributions.

The remaining encroachment records were evaluated to determine the traffic volume, speed limit, and highway class combinations for which reasonable numbers of encroachments had been collected. Unfortunately, this effort revealed a strong correlation between all of the variables. For example, nearly all of the divided highways had traffic volumes greater than 10,000 ADT and speed limits greater than 90 kph. Similarly, undivided highways were found generally to have traffic volumes less than 10,000 ADT and speed limits less than or equal to 90 kph. A correlation between posted speed limit and longitudinal travel distance was also observed as illustrated in Figure 3. However, differences in observed longitudinal runout distances between the two low volume classes were much less than expected. Further, only minor correlations between traffic volume and longitudinal runout length distributions were observed as shown in Figure 4.

Based on these findings, it was determined that the 1620 encroachments could be reliably used to develop longitudinal encroachment distance distributions for two different roadway and traffic volume situations. Table 4 shows the two longitudinal encroachment distance categories selected for this study.

TABLE 4. LONGITUDINAL DISTANCE CATEGORIES.

Highway Class	Posted Speed Limit (kph)	Posted Speed Limit (mph)	Mean Posted Speed (mph)	Average Daily Traffic (veh/d)	Number of Observations
Divided	95-100	59-62	60.5	>10,000	435
Undivided	75-90	46.6-56	50.3	1,000 to 10,000	1185

The next step required the selection of appropriate design runout lengths from Cooper’s longitudinal extent distributions. A suitable confidence interval must be selected in order to provide an acceptable level of risk to traveling motorists. The runout lengths were then selected using the appropriate confidence interval. Recall that the Roadside Design Guide procedure incorporated an 80th percentile confidence

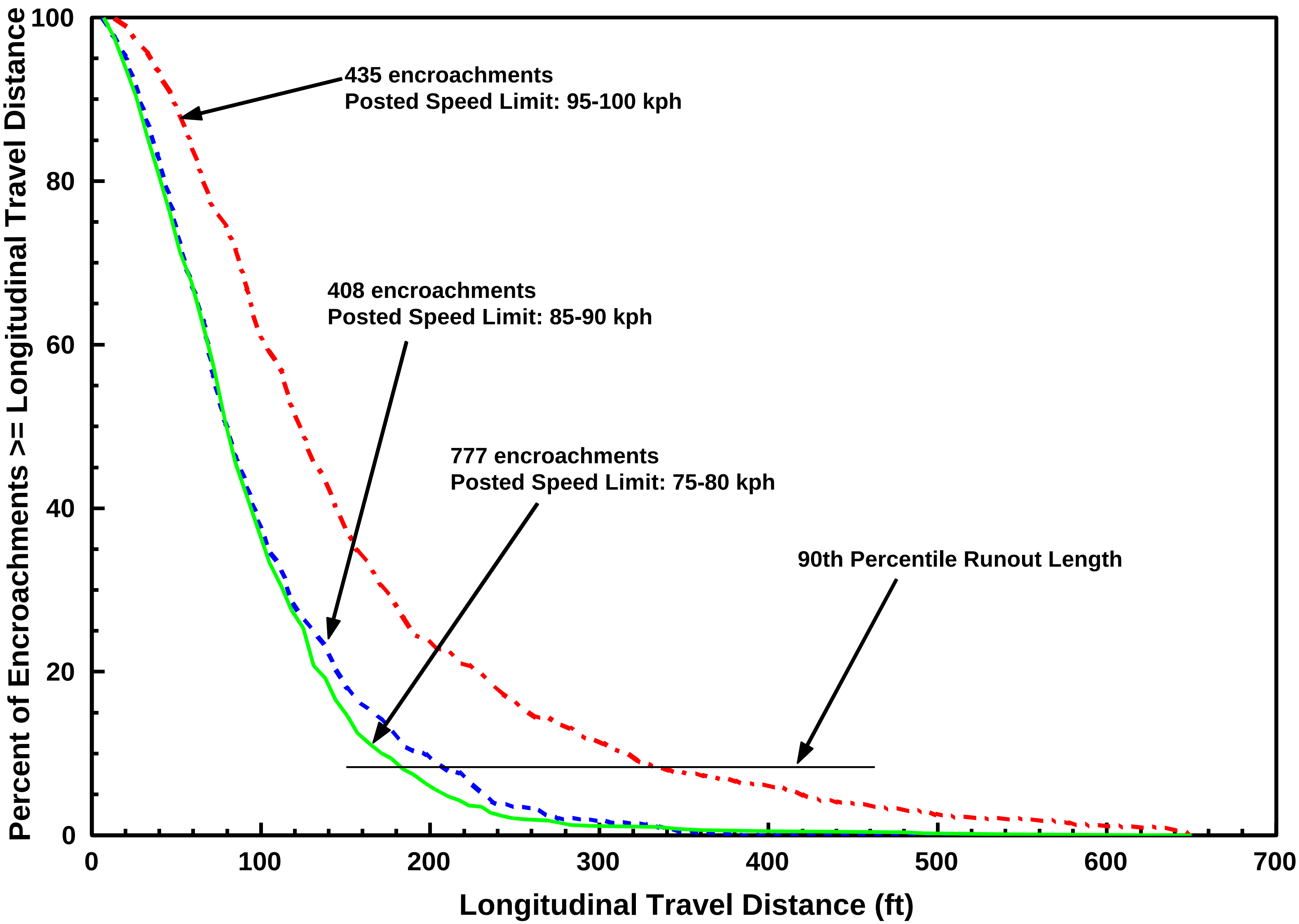

FIGURE 3. LONGITUDINAL PROBABILITY DISTRIBUTION BY SPEED.

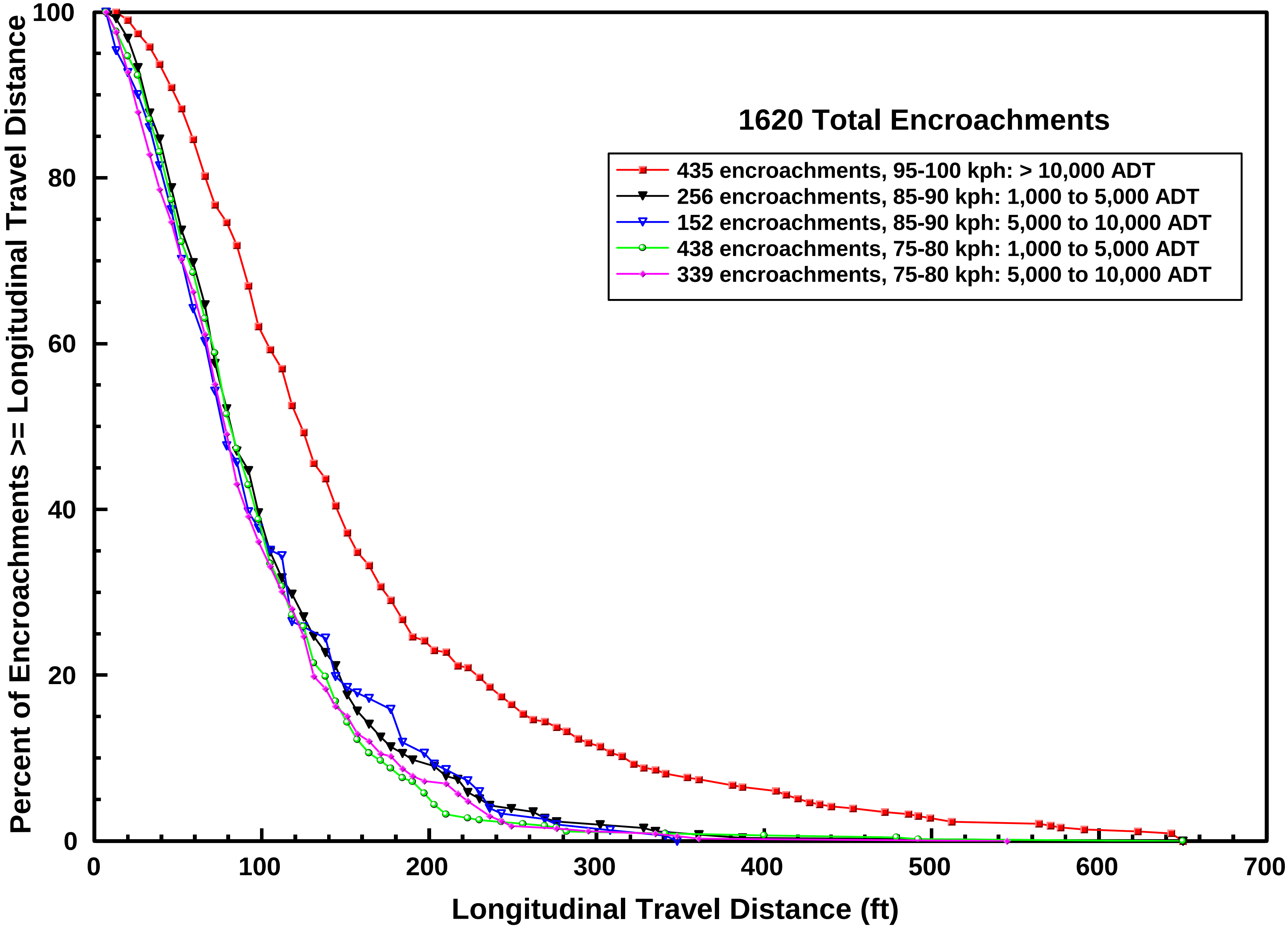

FIGURE 4. LONGITUDINAL PROBABILITY DISTRIBUTION BY TRAFFIC VOLUME.

interval for high volume, high speed roadways. On the surface, designing traffic barriers to prevent 80 percent of vehicles running off of the road from impacting a hazard may appear to be low. However, there are many mitigating factors that must be considered. The most important of these mitigating factors is the fact that errant vehicles traveling behind the barrier to impact a roadside obstacle will undoubtedly slow down significantly before reaching the hazard. Thus, even though the barrier theoretically allows 20 percent of vehicles running off of the road to impact the hazard, the severity of these accidents could be expected to be greatly reduced as the speed decreases. Further, vehicles that run behind the barrier to impact the hazard would have to leave the road before the upstream end of the barrier in order to strike the hazard.

Nevertheless, selection of an 80^{th} percentile design condition was probably based partially in recognition of the flaws in the Hutchinson and Kennedy study that tended to increase observed longitudinal runout lengths. Therefore, it is appropriate to reconsider the confidence interval to be used when employing Cooper's data to develop revised runout length recommendations. Similar approaches have been used in selecting design parameters for other areas of roadside safety design. Consider for example the impact angle and speed combinations selected for barrier design. Until recently longitudinal traffic barriers have been designed to meet all occupant risk criteria for impacts up to 60 mph and 15 degree impact angles. This condition has been identified as an 85^{th} percentile impact condition for accidents occurring along high speed rural highways (8). Recently, this impact condition has been revised to increase the impact angle to 20 degrees, which represents a 90^{th} percentile impact condition.

Further, barriers have also been designed to contain and redirect large automobiles impacting at 60 mph and 25 degrees, corresponding to a 95th percentile impact condition. As illustrated by this example, the appropriate design condition or tolerable level of risk for roadside safety applications is not rigidly defined. However, levels of risk in the 80th to 95th percentile range are appropriate. Table 5 shows the two longitudinal runout length categories developed from Cooper's data with Hutchinson and Kennedy's findings in a tabular format. This table clearly shows that runout lengths observed by Cooper are much shorter than those found in the earlier study. Since Cooper's findings are believed to err toward lower runout lengths while the earlier study erred toward the high side, it is reasonable to reduce the level of risk from that associated with the Roadside Design Guide. Therefore, a 90th percentile design condition was chosen for incorporation into the revised guardrail runout length calculation procedures. Table 5 can then be used to identify appropriate runout lengths for two combinations of speed limit and traffic volume.

TABLE 5. PERCENTILE RUNOUT LENGTH.

Researcher	Mean Speed Limit (mph)	Percentile Runout Length (ft)					
		95%	90%	85%	80%	75%	70%
Hutchinson and Kennedy	70	740	520	480	440	400	360
Cooper	60.5	420	316	258	227	188	172
	50.3	220	180	154	139	126	114

Unfortunately, the Roadside Design Guide is based on design speed rather than speed limit. Therefore it was necessary to convert speed limits from Cooper's study to corresponding design speeds. Cooper reported some comparisons between speed limits and operating speeds observed on highway sections included in his study. Table 6 shows comparisons between speed limit, median speed, and 85th percentile operating speed for several classes of highway included in the study. Note that there is a strong correlation between all three parameters. The median operating speed was found to be approximately the same as the speed limit; while the 85th percentile speed was found to be between 7 and 15 percent higher than the speed limit.

Although, design speeds for highways included in Cooper's study were clearly higher than the posted speed limits, as demonstrated by operating speed observations shown in Table 6, the extent of the increase is unclear. This problem is further

TABLE 6. RESULTS FROM SPOT SPEED STUDIES.

Speed Limit	Mean Speed	85th Percentile
100 kph	1.01SL	1.08SL
90 kph	1.04SL	1.15SL
80 kph	1.04SL	1.15SL
All	1.02SL	1.11SL

Note: SL denotes the speed limit.

aggravated by the fact that Cooper's data does not clearly identify the exact speed limit associated with each encroachment. As shown in Table 4, speed limits are lumped together into two groups with two speed limits each. Thus, it is not clear which speed limit predominates within each of these categories. In the absence of more definitive data, the mean of the speed limit range was selected as the best indicator of the average speed limit associated with each category. Further, since design speeds are generally set 5 to 10 mph above the speed limit, the average design speed assigned to each speed limit category was set at 5 mph above the average speed limit. Table 7 shows the assigned design speeds and associated runout lengths based on these assumptions.

Several attempts were made to correlate the runout lengths and design speeds shown in Table 7 in an effort to develop a more appropriate method for extrapolating this information to other design speed conditions. Unfortunately, all methods based on stopping distance formulas proved to be unsatisfactory due to the large difference in runout length between the mean posted speed limit of 60.5 and 50.3 mph. Therefore a simple proportionate reduction technique similar to that incorporated in the Roadside Design Guide procedures was used to estimate runout lengths at other speeds. The runout lengths shown in Table 7 are the runout lengths from Table 5 adjusted using Equation 3. This simplified procedure will tend to yield larger than expected runout lengths for lower design speeds and higher than expected runout lengths when design speed is increased. Fortunately, the proportionate reduction procedure was used primarily to estimate runout lengths for lower service level roadways and only involved minor upward design speed adjustments.

TABLE 7. RUNOUT LENGTHS BASED ON ASSIGNED DESIGN SPEEDS.

Design Speed (mph)	Percent of Longitudinal Runout Length Accounted For (ft)				
	95%	90%	85%	80%	75%
70	482	363	296	260	216
60	261	213	182	165	150

Runout lengths estimated from Cooper's data were extrapolated for lower volume highways in a manner similar to that used with the Roadside Design Guide procedure. Table 8 presents adjusted runout lengths for traffic volume and design speed categories corresponding with those found in the RDG. Note that the traffic volume categories shown in Table 1 were originally selected based on highways included in the original Hutchinson and Kennedy study (3). Thus, rearrangement of these traffic volumes to correspond with Cooper's study or design classifications incorporated by the Nebraska Department of Roads is justifiable. Table 9 presents recommended runout lengths for highway classifications currently employed by NDOR.

TABLE 8. ADJUSTED RUNOUT LENGTHS.

Design Speed (mph)	Runout Length (Lr) given Traffic Volume (ADT)			
	> 10,000 Lr (ft)	5,000 to 10,000 Lr (ft)	1,000 to 5,000 Lr (ft)	< 1,000 Lr (ft)
70	360	300	260	220
60	260	210	180	170
50	210	170	150	130
40	160	130	110	100
30	110	90	80	70

TABLE 9. RECOMMENDED NDOR RUNOUT LENGTH TABLE.

Design Speed (mph)	Runout Length (Lr) given Traffic Volume (ADT)			
	> 3,000 Lr (ft)	1,700 to 3,000 Lr (ft)	850 to 1,700 Lr (ft)	< 850 Lr (ft)
70	360	300	260	220
60	260	210	180	170
50	210	170	150	130
40	160	130	110	100
30	110	90	80	70

Revised guardrail runout lengths shown in Tables 8 and 9 are believed to be appropriate for use with the length-of-need procedures presented in the Roadside Design Guide. Although the new runout length recommendations are much lower than contained in the RDG, this technique should provide reasonable levels of protection for all classes of highways. Comparisons between the current techniques and the revised procedures are included in Chapter 5.

CHAPTER 4. BENEFIT-COST ANALYSIS

The primary objective of a benefit/cost analysis procedure is normally to provide a tool for prioritizing funding choices. The length of guardrail to be installed at a particular site is an essential funding choice. Highway agencies should utilize sufficient guardrail to provide a reasonable level of protection for motorists running off the road but not so much that funds are expended unnecessarily and the number of injuries and fatalities associated with roadside accidents actually begin to increase. For the guardrail length-of-need analysis, benefits are measured in terms of a reduction in accident costs associated with extending the barrier. The costs include the additional installation, maintenance, and repair costs associated with extending the barrier. Extending the guardrail is not considered to be a good investment unless the benefits out weigh the additional costs.

Only encroachment probability based, benefit/cost analyses can be used to study basic design questions such as the appropriate length of guardrail to be used at a particular site. These procedures attempt to relate the rate that vehicles run off the road to roadside accident rates through a probabilistic model (5,6,7). These techniques generally utilize relationships between traffic volume and encroachment rates developed from studies by Hutchinson and Kennedy (3) or Cooper (4) to predict encroachment rates. Accident rates are then estimated based on the assumption that errant vehicles generally follow a straight path until the vehicle is stopped or brought under control. This assumption leads to a hazard envelope, shown in Figure 5, within

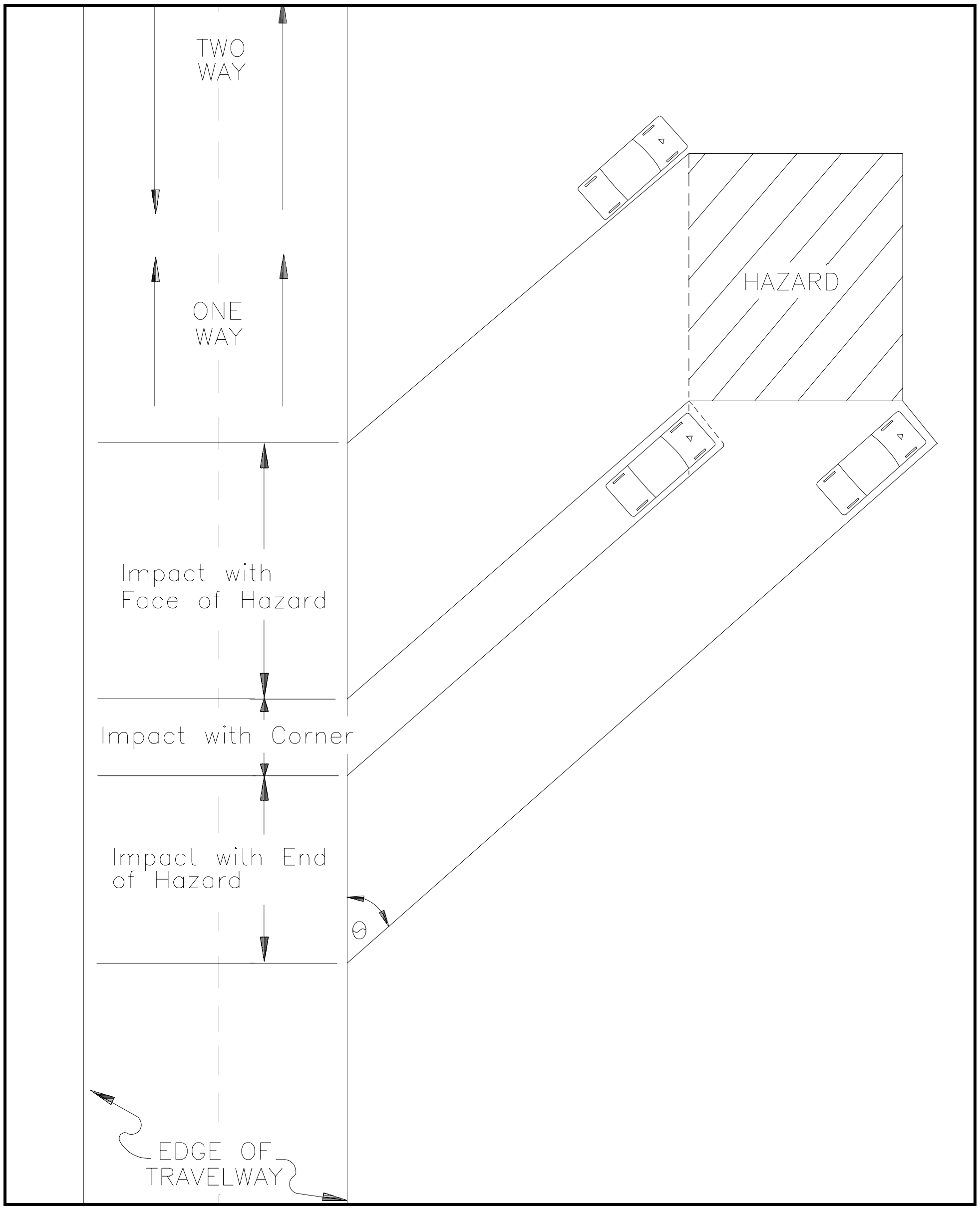

FIGURE 5. HAZARD ENVELOPE.

which vehicles encroaching at a given angle will impact a roadside hazard unless stopped or brought under control. Distributions of encroachment speeds, angles, distances, and vehicle types are incorporated into the analysis to estimate the frequency and nature of each type of roadside accident.

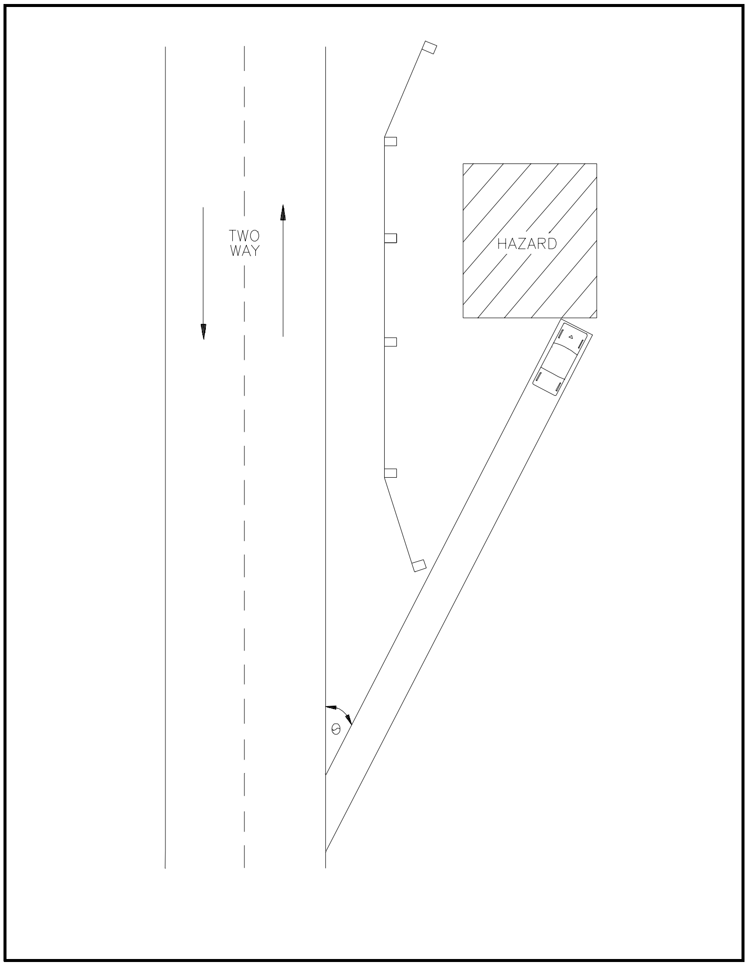

FIGURE 6. HAZARD IMAGING TECHNIQUE

Benefit/cost analyses can be used to study the guardrail length-of-need problem only if it can predict the number of accidents prevented as guardrail is extended upstream from a roadside hazard. As shown in Figure 6, a hazard imaging technique can be employed to estimate the risk associated with vehicles running behind the barrier to impact a roadside hazard. Only two procedures have been fully developed to date that incorporate such hazard imaging techniques, the Benefit to Cost Analysis Program, BCAP (9) and ABC (5). These two benefit-cost models are actually two different computer codes that evolved from the same original model; therefore, the programs are very similar. ABC was selected for use in the current study because the researchers are more familiar with this program and the input routines are generally better suited to studying the current problem. The following section presents a brief discussion of the benefit-cost analysis model, much of which is excerpted from a paper by Sicking and Ross (5). This is followed by a presentation of a new set of guardrail length-of-need criteria which are based on results developed with ABC.

Benefit-Cost Methodology

ABC is a computerized approach which compares the benefits derived from a safety improvement to the direct highway agency costs incurred as a result of the improvement. Benefits are measured in terms of reductions in societal costs due to decreases in the number and/or severity of accidents. Direct highway agency costs are comprised of initial, maintenance, and accident repair costs associated with a proposed improvement. The ratio between the benefits and costs of an improvement, called the B/C ratio, is used to determine if a safety improvement is cost beneficial:

$$BC_{2-1} = \frac{SC_1 - SC_2}{DC_2 - DC_1} \qquad \textbf{(4)}$$

where:

BC_{2-1} = Benefit/Cost ratio of alternative 2 compared to alternative 1.
SC_i = Societal accident costs associated with alternative I.
DC_i = Direct costs associated with alternative I.

In this approach, alternative 2 is initially assumed to be an improvement relative to alternative 1. If the benefit-cost ratio is less than 1.0, the predicted benefits are less than the predicted costs. Hence, the improvement is not justifiable and it should not normally be implemented. If the benefit-cost ratio for a safety improvement is greater than 1.0, the expected benefits are believed to be equal to or greater than the expected costs. Hence, the safety improvement is justifiable. Although budgetary limitations generally preclude funding of all projects that have a benefit-cost ratio of 1.0 or more, the benefit-cost ratio can still be used as a guide to prioritize safety improvements.

Factors which must be taken into account in the formulation of the benefit-cost analysis include: encroachment characteristics, accident costs, hardware installation costs, and repair costs. Details of the assumptions inherent in the general formulation of the benefit-cost analysis are presented elsewhere (5) and are not fully restated in this report. Details of the assumptions which are specific to the guardrail length-of-need study and which are needed for a proper interpretation of the results are discussed below.

Uncontrolled encroachment characteristics that are required for use in the benefit-cost methodology include frequency, speed, angle, and lateral movement. There are relatively few sources of such data available. The largest database available which contains pure encroachment information was collected on Canadian highways by Cooper (4). The Cooper study involved highways with operating speeds in the same range as those on most U.S. highways today. Therefore, the Cooper data were used to develop the necessary encroachment model. These data are available elsewhere and are not reproduced in this report (4,5).

As implemented in the ABC benefit/cost methodology, development of a relationship between encroachment characteristics (both angle and speed) and societal costs is a two step process. First a relationship between the impact speed, the impact angle and severity index must be established. This process involves estimating the likelihood of vehicle occupants being killed or injured during an impact at a given speed and angle. A variety of techniques, including full-scale crash testing, computer simulation, and accident data analysis, have been used to develop these relationships.

Full-scale crash testing and computer simulations of vehicular impacts generate surrogate measures of occupant risk, such as, maximum accelerations and estimated speeds at which occupants strike the vehicle interior. Unfortunately, very few studies have attempted to link these measures of occupant risk to probability of injury. The most successful of these efforts involved comparing vehicle damage during crash testing to vehicle damage arising from bridge rail accidents (11). Correlations between the Traffic Accident Damage (TAD) scales for these vehicles were then used to develop a relationship between maximum 50 millisecond, ms, average accelerations and the probability of injury as shown in Figure 7. Probabilities of injury can then be correlated with severity index by combining distributions of all injury and fatal accident probabilities for the severity index scale as shown in Table 10 (2). In this manner, severity of impact with guardrails was estimated for full-size and small automobiles using computer simulations and full-scale crash test results. As shown in Figures 8 and 9, the resulting severities for large automobiles appeared to be reasonable, while the severity associated with small cars seemed excessive. This finding should not be surprising because of the type of vehicles associated with the development of the relationship between vehicle accelerations and probability of injury. During the 1960's and early 1970's, the vast majority of vehicles sold in the U.S. were in the full-size category.

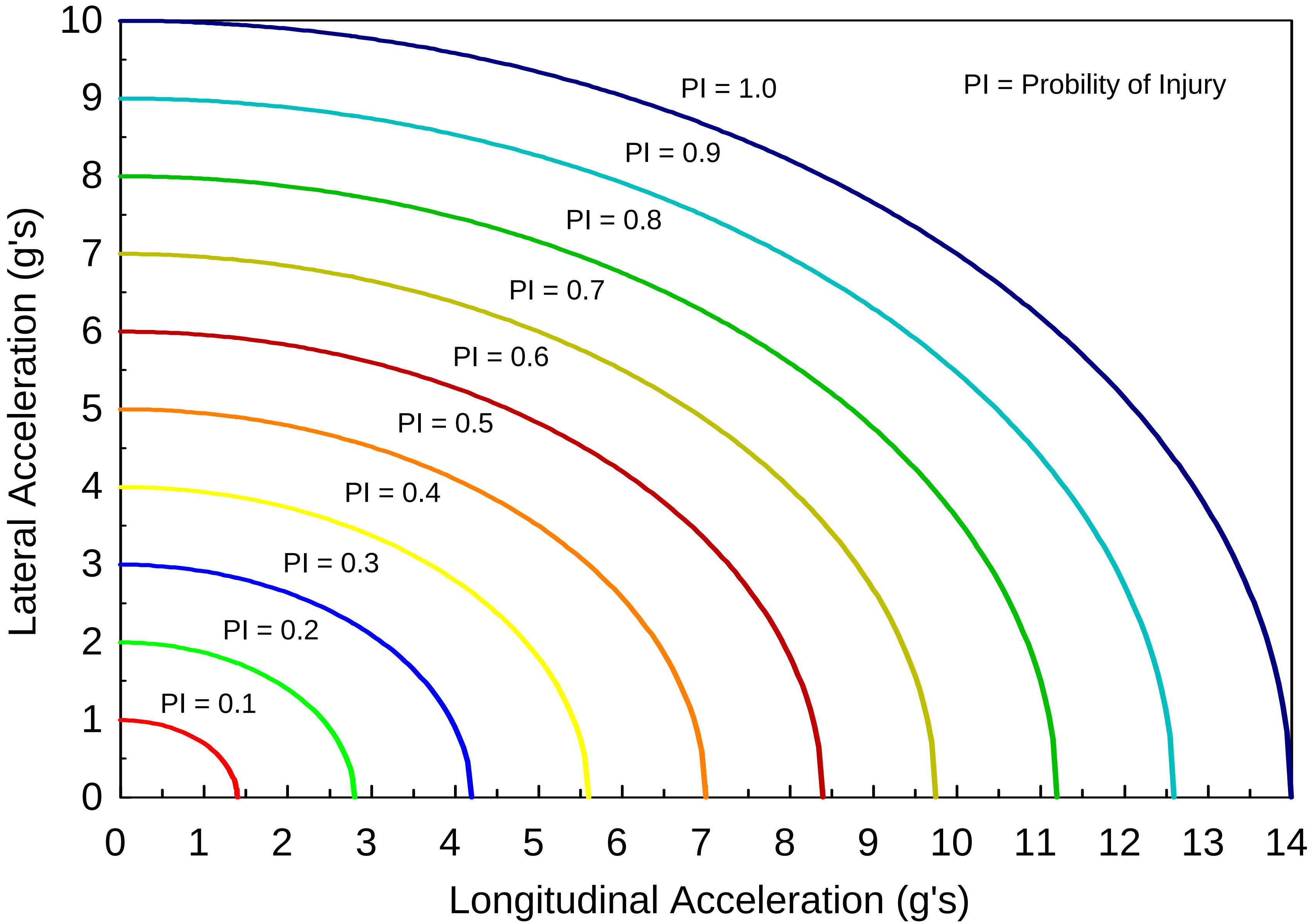

FIGURE 7. PROBABILITY OF INJURY ELLIPSE.

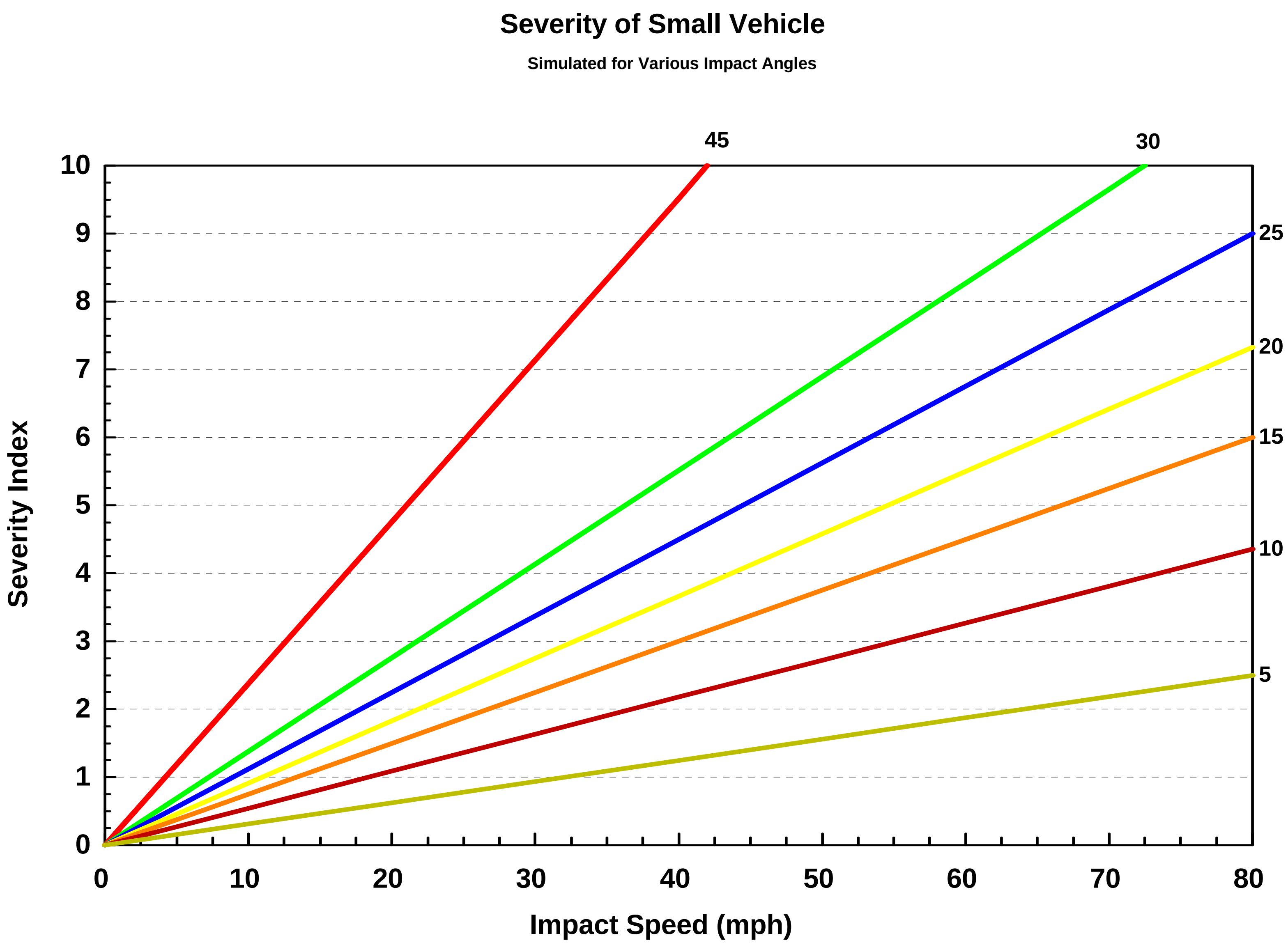

FIGURE 8. SEVERITY OF LARGE VEHICLE

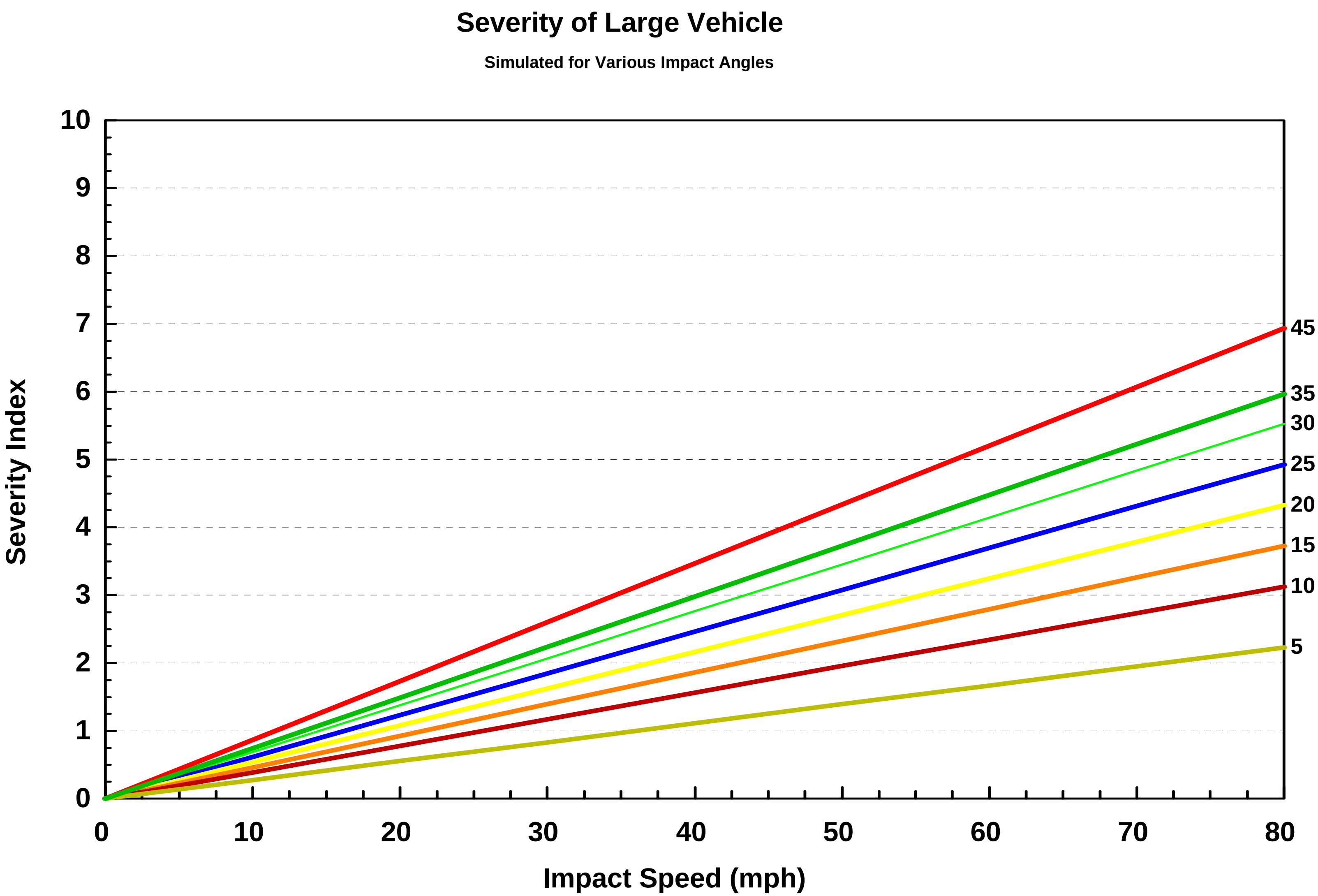

FIGURE 9. SEVERITY OF LARGE VEHICLE.

Therefore, most of the vehicles involved in the development of Figure 7 were in this category. Since this procedure may not be valid for use with small automobiles, all automobile guardrail impact severities were developed from Figure 9.

After the relationships between encroachment characteristics and the severity index are established, a relationship between the severity index and societal costs is needed to evaluate societal costs. A relationship presented in the 1989 AASHTO Roadside Design Guide (2) was initially used in the benefit-cost analysis described in this report. After discussions with engineers at the Nebraska Department of Roads and more careful consideration of the appropriate societal costs for use with roadside safety analysis, relationships found in the 1995 update to the AASHTO Roadside Design Guide (12) were incorporated into the study. This relationship, between the severity index and societal costs, is presented in Table 10. As shown in this table, the cost of a fatal accident, an accident with severity index of 10, is set at $1,000,000.

The severity of guardrail impact shown in Figure 9 was then compared with accident data as another check on its validity. First, accident data from the Highway Safety Information System (HSIS) was obtained to determine accident severities associated with guardrail impacts. Table 11 shows the gross guardrail accident severities generated from four different HSIS states and a detailed study of guardrail accidents in Texas (1). Note that the average severities shown in this table are somewhat higher than would be expected for guardrail impacts. However, these data do not include unreported accidents. Therefore, a direct comparison with the accident prediction model is inappropriate. In order to make such a comparison, the effects of unreported accidents on gross accident severities must be estimated.

TABLE 10. SEVERITY INDEX AND COST BY ACCIDENT TYPE DISTRIBUTION.

Severity Index	Property Damage (1)	Property Damage (2)	Slight Injury	Moderate Injury	Severe Injury	Fatal Injury	Total	Probability of Injury	Accident Cost ($)
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0
0.5	100.0	0.0	0.0	0.0	0.0	0.0	100.0	0	625
1.0	66.7	23.7	7.3	2.3	0.0	0.0	100.0	9.6	1,719
2.0	0.0	71.0	22.0	7.0	0.0	0.0	100.0	29	3,919
3.0	0.0	43.0	34.0	21.0	1.0	1.0	100.0	57	17,244
4.0	0.0	30.0	30.0	32.0	5.0	3.0	100.0	70	46,063
5.0	0.0	15.0	22.0	45.0	10.0	8.0	100.0	85	106,919
6.0	0.0	7.0	16.0	39.0	20.0	18.0	100.0	93	225,694
7.0	0.0	2.0	10.0	28.0	30.0	30.0	100.0	98	363,938
8.0	0.0	0.0	4.0	19.0	27.0	50.0	100.0	100	556,525
9.0	0.0	0.0	0.0	7.0	18.0	75.0	100.0	100	786,875
10.0	0.0	0.0	0.0	0.0	0.0	100.0	100.0	100	1,000,000

TABLE 11. GROSS GUARDRAIL ACCIDENT SEVERITIES.

Injury Level	Texas Guardrail Accidents	North Carolina Towaways	Michigan Towaways	Utah Towaways	Illinois Towaways
PDO	51.2%	49.9%	55.8%	68.6%	63.7%
C-Injury	19.5%	18.3%	17.9%	8.6%	9.8%
B-Injury	18.7%	19.1%	16.9%	13.1%	13.7%
A-Injury	8.8%	11.9%	9.0%	8.9%	12.1%
Fatal	1.8%	0.8%	0.4%	0.8%	0.7%
%(A+K)	10.6%	12.7%	9.4%	9.7%	12.8%

Several researchers have attempted to estimate the magnitude of the unreported accident problem by comparing reported accident frequency with the rate that marks appear on longitudinal barriers (13) or barrier repair frequencies (14). Studies of marks on longitudinal barriers such as W-beam guardrail indicate an 8:1 ratio between unreported and reported accidents (13). This ratio is believed to be somewhat high due to the fact that marks on roadside barriers can be caused by something other than traffic accidents. For example, items that become dislodged from vehicles and fall into the roadway are often knocked off the travelway and impact a roadside barrier with sufficient force to cause detectable damage or marks. Further, crash testing and accident investigations indicate that vehicles impacting roadside barriers are often redirected away from the barrier only to return a short time later as a result of damage to vehicle suspensions. Thus, two or more distinct and separate areas of damage often result from a single impact.

Efforts to compare barrier repair frequencies to reported accident rates tend to indicate a ratio between unreported and reported accidents in the range of 1:1.6. Although these studies involved cable barriers that should require repairs even for relatively minor impacts, some portion of the low speed, low angle accidents would be expected to require no repair. Thus, this procedure probably underestimates the magnitude of the unreported accident problem.

In light of the above discussion, it can be concluded that between 38 and 89 percent of longitudinal barrier accidents go unreported. For purposes of comparing reported accident severities with encroachment probability model predictions, it was assumed that approximately 75% of longitudinal barrier impacts go unreported. Data shown in Table 11 was then adjusted for unreported accidents based on the assumption that no severe injury or fatal accidents would go unreported and that moderate and minor injuries would be reported at rates of 67% and 33% respectively. Table 12 shows the adjusted accident severities and estimated average accident costs for the four HSIS states and the Texas guardrail accident study (1). Average accident

TABLE 12. ADJUSTED GUARDRAIL ACCIDENT SEVERITIES.

Injury Level	Texas G.R. Accidents	North Carolina Towaways	Michigan Towaways	Utah Towaways	Illinois Towaways	Average Distribution
PDO	67.6%	68.0%	70.5%	81.6%	79.0%	73.22%
C-Injury	19.5%	18.2%	17.9%	8.6%	9.8%	14.94%
B-Injury	9.4%	9.6%	8.5%	6.6%	6.9%	8.08%
A-Injury	2.9%	3.9%	3.0%	2.9%	4.1%	3.46%
Fatal	0.6%	0.3%	0.1%	0.3%	0.2%	0.30%
Total %	100%	100%	100%	100%	100%	100%
RSDG Costs	$14,720	$13,703	$9,791	$11,172	$12,615	$12,589

costs were estimated based on accident costs for PDO, injury, and fatal accidents published in the 1995 Roadside Design Guide (12) and shown in Table 13.

TABLE 13. ESTIMATED COSTS BY ACCIDENT SEVERITY LEVELS.

Injury Description	Police Injury Code (PIC)	Accident Cost
Fatal Accident	K	$ 1,000,000
Severe Injury Accident	A	$ 200,000
Moderate Injury Accident	B	$ 12,500
Slight Injury Accident	C	$ 3,750
Property Damage Only Accident	PDO	$ 1500

The ABC model was then run, using severity index and impact angle relationships shown in Figure 9, to determine predicted severity levels and average accident costs. As shown in Table 14, the predicted severity distributions are not too different from the adjusted accident data findings, and average accident costs are comparable. Although the accident severities used in the analysis cannot be completely validated due to problems associated with unreported accidents, guardrail impact severities used in Figure 9 appear to correlate reasonably well with available accident data.

TABLE 14. PREDICTED SEVERITY DISTRIBUTIONS.

Injury Level		PREDICTED INJURY LEVEL	
		Benefit/Cost Analysis	Accident Data
Property Damage Only	PDO	68.6%	73.2%
Slight Injury	C	18.2%	14.9%
Moderate Injury	B	10.8%	8.1%
Severe Injury	A	1.25%	3.5%
Fatal Injury	K	1.0%	.3%
Average Accident Cost		$16,093	$12,589

Direct costs associated with W-beam guardrail use include installation, repair, and maintenance costs of the barrier. For the analysis presented in this report, the initial costs of W-beam guardrails were obtained from bid summaries obtained from NDOR engineers. The average installation cost for strong-post W-beam guardrail was approximately $11.00 per linear foot. The ABC benefit/cost analysis program requires that the repair cost be entered as a slope representing the repair cost per ft-lb of energy due to an impact with a vehicle. This relationship between impact severity, IS, and the repair costs is shown in Figure 10. In this relationship the impact severity is given as a function of vehicle speed and angle of impact as follows:

$$IS = \frac{1}{2} m \left(V \sin\theta \right)^2 \qquad \textbf{(5)}$$

where:

IS	= impact severity,
m	= mass of the vehicle,
V	= speed of the vehicle, and
θ	= impact angle (11).

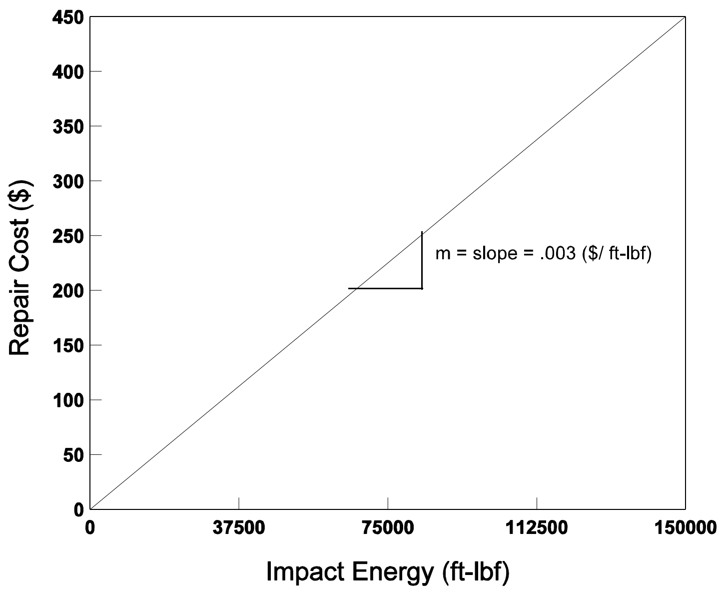

FIGURE 10. REPAIR COST RELATIONSHIP.

For example, an impact with a guardrail at 60 mph and 25 degrees for a 4500 lb vehicle would produce an impact severity of 96,700 ft-lb. The associated repair cost for such an event would be approximately $290. However, an actual impact of this type would most likely be more expensive. In fact, the mobilization cost to repair the guardrail would probably at least equal this value. The repair cost is therefore a conservative estimate, erring on the side of longer guardrail installations.

It must be noted that not all vehicles impacting W-beam guardrails are successfully redirected. In some cases the errant vehicle goes through or over the barrier. In order to accurately evaluate accident costs associated with such barrier impacts, a benefit-cost analysis must incorporate a provision for guardrail penetration. Impact severity, as calculated in Equation 5, has been shown to be a reasonably good predictor of the propensity for a vehicle to penetrate through or over a longitudinal barrier (15). For purposes of this benefit-cost analysis, the capacity of W-beam guardrail was estimated to be 90,000 ft-lb for small automobiles and 150,000 ft-lb for full-size automobiles and trucks. However, these W-beam guardrail penetration thresholds are believed to be somewhat high. The effect of using high penetration thresholds is to introduce conservatism into the process. If fewer vehicles are predicted to penetrate the barrier, the accident costs associated with the barrier are reduced and the benefit-cost ratio associated with barrier installation will improve.

The severity of accidents that involve vehicles penetrating W-beam guardrails has never been well established. However, crash test data and computer simulation results indicate that most guardrail penetrations result in vehicle rollover. Accident data on TxDOT standard W-beam guardrail indicates a fatality rate of 27 percent for impacts involving automobile rollover (1). Although similar data for trucks are not available, accident data collected on rural highways in the State of Washington indicate that only 50 percent of truck rollover accidents involve an injury or fatality (15). These fatality and injury rates were used to assign a severity index of 6.5 for automobile penetration accidents and 3.0 for truck penetration accidents.

Finally, it is necessary to estimate the severity of impact with a roadside hazard that would normally be shielded with W-beam guardrail. Although guardrail is routinely placed in front of a variety of hazards, rigid obstacles, such as bridge piers, are probably among the most hazardous. Guardrail length-of-need would be expected to be higher for more hazardous objects. Therefore, in order to develop length-of-need procedures that are conservative, a rigid obstacle was chosen as the hazard to be used with the benefit/cost analysis. The severity of impact with a rigid obstacle has been estimated by modeling a vehicle as a spring-mass, single degree of freedom, dynamics model (9). This analysis yielded the following equation:

$$SI = 0.183\,V \qquad \textbf{(6)}$$

where:

SI = severity index, and
V = impact velocity (mph).

Complete details of the formulation of the ABC benefit-cost analysis are available elsewhere (5).

Length-of-Need Selection Charts

The primary goal of the benefit cost analysis effort was to develop simplified charts for determining appropriate guardrail lengths-of-need. The first step in developing the charts involved examining the sensitivity of guardrail length-of-need to various roadway and roadside variables. The variables found to have a significant effect on guardrail length-of-need are listed in Table 15, with a list of all variables investigated, classified by significance. However, some of these variables are strongly correlated, such as the offset to the face of the hazard, L_3, the offset to back of hazard,

L_H, and the width of the hazard, W. Thus, the importance of some of these variables may be eliminated by controlling other parameters.

TABLE 15. VARIABLES INVESTIGATED.

Description	Variable	Significant
Functional Classification	FUNC	yes
Highway Type	HWYTYP	yes
Average Daily Traffic	ADT	yes
Lateral Offset Difference	$L_{OD} = L_H - Y$	yes
Offset to Back of Hazard	L_H	yes
Offset to Front of Guardrail	L2	yes
Offset to Front of Hazard	L3	yes
Width of Hazard	W	yes
	(LH-L2)/L2	no
	(LH-L2)/LH	no
	LH+L2	no
	LH*L2	no
	LH*L3	no
	LH/(LH-L2)	no
	LH+L3	no
	LH/(L3-L2)	no
	LH/L2	no
	L2/(L3-L2)	no

The variables found to have a significant effect on length-of-need were then systematically evaluated to determine the variables or combination of variables that had the most effect on guardrail length-of-need. This process involved holding a combination of variables constant and evaluating the sensitivity of the calculated length-of-need to all of the other variables found to have a significant influence on recommended runout lengths. This process ultimately lead to the conclusion that guardrail run-out-lengths are relatively insensitive to changes in other variables when traffic volume and the difference between the lateral offsets of the barrier and the back of the hazard are held constant. This latter term has been dubbed the "offset difference" and was then incorporated into the process of developing guardrail length-of-need selection charts.

The ABC model was then used to develop length-of-need selection charts as a function of these two variables. This process involved using the ABC model to determine optimal lengths-of-need for a wide variety of roadside situations, then using linear regressions to develop relationships between runout length and lateral offset difference for several different traffic volumes. Figures 11 and 12 show resulting runout length selection charts for upstream and downstream guardrail applications, respectively.

The application of charts shown in Figures 11 and 12 should be much less complicated than procedures now contained in the Roadside Design Guide. This process first involves identifying areas where guardrail is needed. As discussed in the Roadside Design Guide, guardrail should only be considered if a roadside hazard cannot be eliminated, redesigned to reduce the hazard, or moved out of the clear zone. In these cases, if the severity of the hazard is sufficient to warrant guardrail placement, a guardrail layout such as that shown in Figure 13 must be selected.

UPSTREAM GUARDRAIL RUNOUT LENGTH
FOR RURAL ROADWAYS

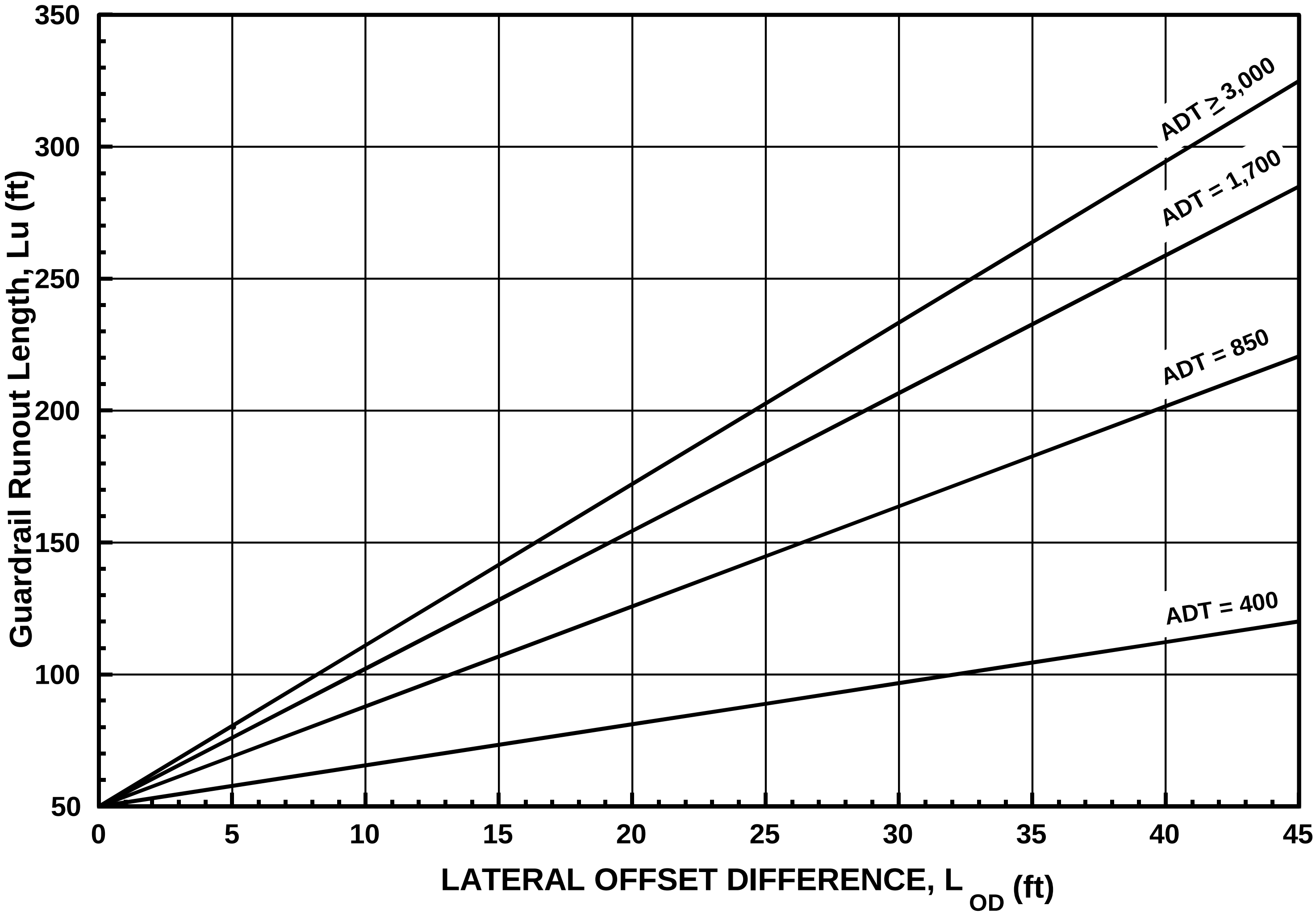

FIGURE 11. UPSTREAM LENGTH-OF-NEED DESIGN CHART.

DOWNSTREAM GUARDRAIL RUNOUT LENGTH
FOR RURAL ROADWAYS

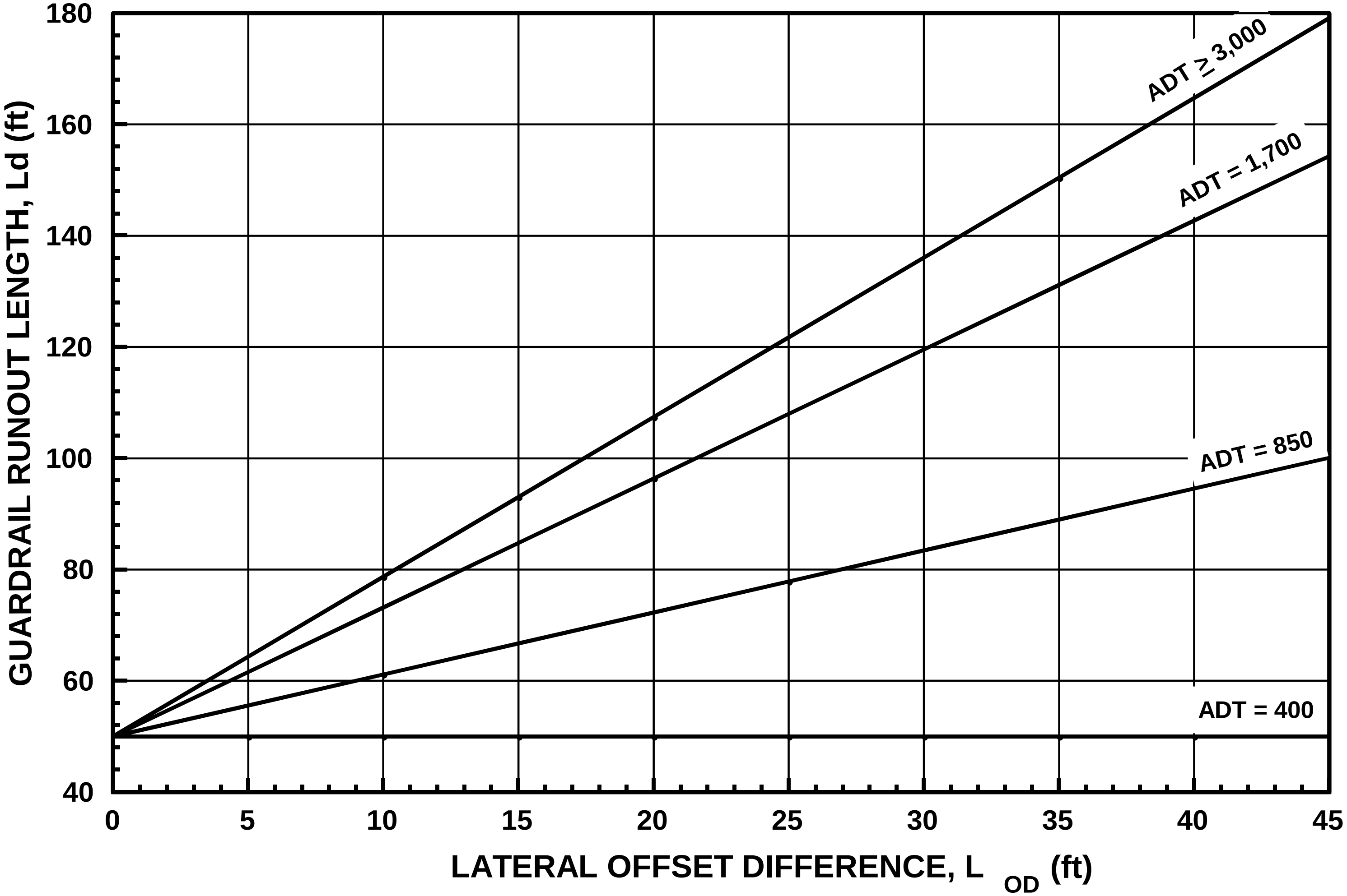

FIGURE 12. DOWNSTREAM LENGTH-OF-NEED DESIGN CHART.

When selecting a guardrail layout, the designer must determine if a flare is to be used and how far from the travelway it can be placed. In most situations, roadside slopes and ditches preclude the use of long flared sections that extend far from the travelway. In these cases, the lateral offset of the beginning of the length-of-need, Y, is easily identified. The design must then determine the lateral offset difference, L_{OD}. As described above and shown in Figure 13, L_{OD} is the difference between the maximum

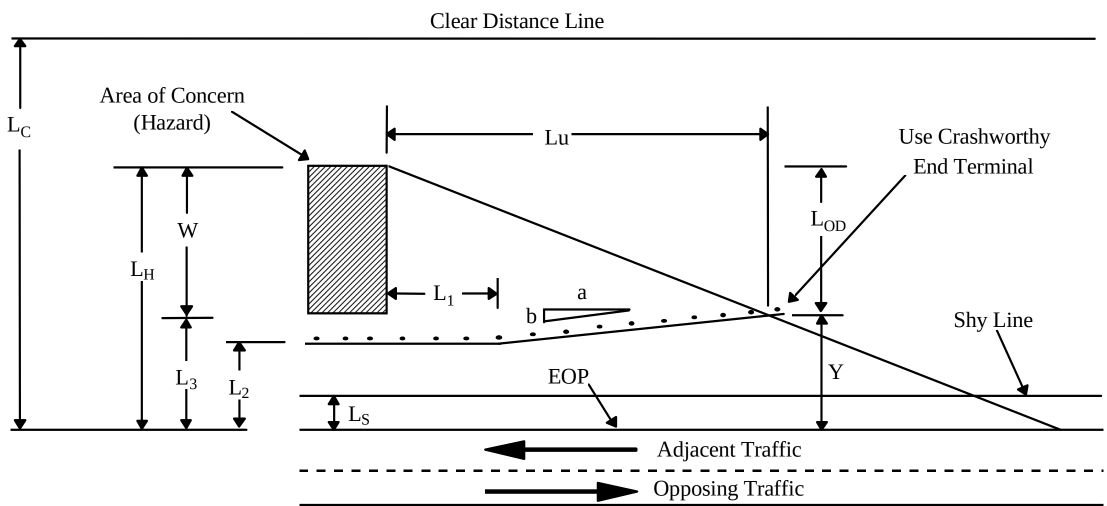

FIGURE 13. APPROACH BARRIER DIAGRAM.

lateral extent of the area of concern and the lateral offset of the beginning of the length-of-need. Note that the lateral offset of the area of concern is the lesser of the lateral offset to the back of the hazard and the clear zone distance. The design year traffic volume and the lateral offset difference can then be used to directly determine appropriate upstream, L_U, and down stream, L_D, runout lengths from Figures 11 and 12. The selected runout lengths and lateral offset distances must be checked against recommended flare rates presented in the Roadside Design Guide. If flare rate and runout lengths are compatible, the guardrail layout is completed.

In the rare circumstance that a roadside is sufficiently flat to allow the barrier to be flared as far from the roadway as possible, this process becomes a little more complicated. The designer must first estimate an appropriate offset to the beginning of length-of-need for the upstream and downstream guardrail ends. Note that these two numbers would seldom be the same since upstream runout lengths are always longer than downstream lengths. The two lateral offsets can then be used to determine estimated upstream and downstream runout lengths. These estimated runout lengths must then be compared with recommended flare rates from the Roadside Design Guide. If the lateral offsets of the beginning of the length-of-need are significantly different than the estimated values, the runout lengths should be revised using the new lateral offset values.

CHAPTER 5. COMPARISON OF EXISTING AND NEW PROCEDURES

Published examples of guardrail length-of-need calculations from the Roadside Design Guide were repeated using the new procedures calculated in this report. Summary calculation tables for both flared and parallel guardrail installations are shown in tables 16 and 17, respectively. The variables Lu and Ld represent the upstream and the downstream barrier runout lengths for both flared and parallel configurations.

As shown in Table 16, the revised Level of Risk procedures reduced the recommended lengths of guardrail by an average of 27 percent for the flared installation. Similarly, a 40 percent reduction in the recommended lengths of guardrail was obtained for parallel installation of guardrail. These reductions were found to be fairly consistent for all types of highways for both parallel and flared guardrail configurations.

The benefit/cost analysis based procedures produced recommended reductions of only 20 percent for flared configurations. This method slightly over estimates the downstream length-of-need for very short lateral offset differences. However, for larger lateral offset differences this method predicts shorter lengths-of-need than the other two methods. Further, length-of-need reductions for highways with both low traffic volumes and low speeds were found to be relatively small, while reductions for higher speed highways with higher traffic volumes were found to approach that associated with the revised Level of Risk analysis.

TABLE 16. SUMMARY TABLE FOR FLARED END BARRIER.

Description of Example in Roadside Design Guide	Variables	Bridge Approach 4 Lane divided right side	Percent Reduction from RDG	Bridge Approach 4 Lane divided median	Percent Reduction from RDG	Bridge Pier 2-lane 2-way	Percent Reduction from RDG	Nontraversable Embankment 2-lane 2-way	Percent Reduction from RDG	Average
ADT	(veh/day)	6200		6200		850		3000		
Design Speed	(mph)	60		60		50		70		
LH	(ft)	36		30		12		28		
L2	(ft)	12		8		8		6		
Flare Rate	a	15		15		11		30		
Roadside	LR (ft)	400		400		260		440		
Design	Ld (ft)	N/A*		N/A*		34		184		
Guide	Lu (ft)	164		167		46		235		
Canadian	LR (ft)	260	35%	260	35%	130	50%	300	32%	38%
Encroachment	Ld (ft)	N/A*		N/A*		23	34%	137	25%	29%
Data	Lu (ft)	125	24%	130	22%	34	25%	180	23%	24%
Benefit/Cost										
Analysis										
Design	Ld (ft)	N/A*		N/A*		52	-53%	106	43%	-5%
Chart	Lu (ft)	148	10%	142	15%	56	-22%	158	33%	9%

TABLE 17. SUMMARY TABLE FOR PARALLEL BARRIER.

Description of Example In Roadside Design Guide	Variables	Bridge Approach 4 Lane divided right side	Percent Reduction from RDG	Bridge Approach 4 Lane divided median	Percent Reduction from RDG	Bridge Pier 2-lane 2-way	Percent Reduction from RDG	Nontraversable Embankment 2-lane 2-way	Percent Reduction from RDG	Average
ADT	(veh/day)	6200		6200		850		3000		
Design Speed	(mph)	60		60		50		70		
LH	(ft)	36		30		12		28		
L2	(ft)	12		8		8		6		
Flare Rate	a	15		15		11		30		
Roadside	LR (ft)	400		400		260		440		
Design	Ld (ft)	N/A*		N/A*		43		242		
Guide	Lu (ft)	267		293		87		346		
Canadian	LR (ft)	260	35%	260	35%	130	50%	300	32%	38%
Encroachment	Ld (ft)	N/A*		N/A*		22	50%	165	32%	41%
Data	Lu (ft)	173	35%	191	35%	43	50%	236	32%	38%
Benefit/Cost	LP dn(ft)									
Analysis	LP up(ft)									
Design	Ld (ft)	N/A*		N/A*		54	-26%	114	53%	14%
Chart	Lu (ft)	194	27%	182	38%	65	24%	182	47%	34%

CHAPTER 6. SUMMARY AND CONCLUSIONS

As discussed in Chapters 2 and 3 above, the Roadside Design Guide (2) length-of-need calculation procedures are based on the assumption that roadside barriers should be designed to reduce, to a specified level, the level of risk associated with running behind the barrier. Further, the procedures adjust the allowable level of risk according to the roadway's level of service, measured by the traffic volume. The revised "Level of Risk" procedures incorporating Cooper's encroachment data provide an improved method of estimating actual levels of risk associated with guardrail length-of-need calculations. The revised procedure will provide reasonably safe roadside barrier designs and will likely reduce the overall level of injuries and fatalities associated with sites where guardrail is installed.

Guardrail length-of-need calculations based on benefit/cost analysis presented in Chapter 3 use an entirely different approach to selecting guardrail length-of-need. This procedure is based on the assumption that guardrail should be designed to provide the most cost-effective guardrail installation possible. The general approach chosen to develop the revised length-of-need selection procedures is believed to be relatively conservative due to relatively low installation, maintenance, accident, and repair costs incorporated into the study. In this instance, conservative means that the procedure should tend to recommend longer guardrail lengths than are optimum.

Both of the new length-of-need calculation procedures are believed to provide better estimates of optimum guardrail length-of-need calculations than the Roadside Design Guide techniques. Therefore, highway agencies are recommended to select one of the two revised length-of-need calculation procedures. Agencies familiar and comfortable with the RDG procedures should probably implement the revised Level of Risk approach. Agencies that are interested in simplifying the length-of-need calculations may wish to use the technique developed through benefit/cost analysis.

Ultimately, the benefit/cost analysis techniques should be improved to the point that this technique should provide the most appropriate analysis of this problem. Unfortunately, even the most sophisticated benefit/cost analysis techniques do not yet consider relatively important encroachment characteristics such as longitudinal extent of encroachment distribution or curved encroachments. This problem should be revisited when these considerations are incorporated into new benefit/cost analysis procedures during the next century.

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