Roadside Design Guide

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and Transportation Officials

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PREFACE

The *Roadside Design Guide* is developed and maintained by the AASHTO Subcommittee on Design, Technical Committee for Roadside Safety. The guide presents a synthesis of current information and operating practices related to roadside safety and is written in dual units—metric and U.S. Customary units. The 2006 edition of the guide supersedes the 1996 AASHTO publication of the same name and includes an update to Chapter 6, "Median Barriers," which replaces Chapter 6 of the 2002 edition.

In this guide, the roadside is defined as that area beyond the traveled way (driving lanes) and the shoulder (if any) of the roadway itself. Consequently, roadside delineation, shoulder surface treatments, and similar on-roadway safety features are not extensively discussed. While it is a readily accepted fact that safety can best be served by keeping motorists on the road, the focus of the guide is on safety treatments that minimize the likelihood of serious injuries when a driver runs off the road.

A second noteworthy point is that this document is a guide. It is not a standard, nor is it a design policy. It is intended for use as a resource document from which individual highway agencies can develop standards and policies. While much of the material in the guide can be considered universal in its application, there are several recommendations that are subjective in nature and may need modification to fit local conditions. However, it is important that significant deviations from the guide be based on operational experience and objective analysis.

To be consistent with AASHTO's *A Policy on Geometric Design of Highways and Streets,* design speed is as the basic speed parameter to be used in this guide. However, since the design speed is often selected based on the most restrictive physical features found on a specific project, there may be a significant percentage of a project length where that speed will be exceeded by a reasonable and prudent driver. Conversely, there will be other instances where roadway conditions will prevent most motorists from driving as fast as the design speed. Because roadside safety design is intended to minimize the consequences of a motorist leaving the roadway inadvertently, the designer should consider the speed at which encroachments are most likely to occur when selecting an appropriate roadside design standard or feature.

Design values are presented in this document in both metric and U.S. Customary units. The relationship between the metric and U.S. Customary values is neither an exact (soft) conversion nor a completely rationalized (hard) conversion. The metric values are those that would have been used had the guide been presented exclusively in metric units; the U.S. Customary values are those that would have been used if the guide had been presented exclusively in U.S. Customary units. Therefore, the user is advised to work entirely in one system and not to attempt to convert directly between the two.

The reader is cautioned that roadside safety is a rapidly changing field of study, and changes in policy, criteria, and technology are certain to occur after this document is published. Efforts should be made to incorporate the appropriate current design elements into the project development. Comments from users of this guide on suggested changes or modifications resulting from further developmental work or hands-on experience are appreciated. All such comments should be addressed to the American Association of State Highway and Transportation Officials, Engineering Program, 444 North Capitol Street NW, Suite 249, Washington, DC 20001.

GLOSSARY

Area of Concern—An object or roadside condition that may warrant safety treatment.

Barricade—A device which provides a visual indicator of a hazardous location or the desired path a motorist should take. It is not intended to contain or redirect an errant vehicle.

Barrier—A device which provides a physical limitation through which a vehicle would not normally pass. It is intended to contain or redirect an errant vehicle.

Breakaway—A design feature which allows a device such as a sign, luminaire, or traffic signal support to yield or separate upon impact. The release mechanism may be a slip plane, plastic hinges, fracture elements, or a combination of these.

Bridge Railing—A longitudinal barrier whose primary function is to prevent an errant vehicle from going over the side of the bridge structure.

Clearance—Lateral distance from edge of traveled way to a roadside object or feature.

Clear Runout Area—The area at the toe of a non-recoverable slope available for safe use by an errant vehicle.

Clear Zone—The total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry.

Cost-effective—An item or action taken that is economical in terms of tangible benefits produced for the money spent.

Crash Cushion—Device that prevents an errant vehicle from impacting fixed objects by gradually decelerating the vehicle to a safe stop or by redirecting the vehicle away from the obstacle.

Crash Tests—Vehicular impact tests by which the structural and safety performance of roadside barriers and other highway appurtenances may be determined. Three evaluation criteria are considered, namely (1) structural adequacy, (2) impact severity, and (3) vehicular post-impact trajectory.

Crashworthy—A feature that has been proven acceptable for use under specified conditions either through crash testing or in-service performance.

Design Speed—A selected speed used to determine the various geometric design features of the roadway. The assumed design speed should be a logical one with respect to the topography, anticipated operating speed, the adjacent land use, and the functional classification of highway.

Drainage Feature—Roadside items whose primary purpose is to provide adequate roadway drainage such as curbs, culverts, ditches, and drop inlets.

End Treatment—The designed modification of the end of a roadside or median barrier.

Experimental Barrier—A barrier that has performed satisfactorily in full-scale crash tests and promises, but has not yet demonstrated satisfactory in-service performance.

Flare—The variable offset distance of a barrier to move it farther from the traveled way; generally in reference to the upstream end of the barrier.

Frangible—A structure quality or feature that makes the structure readily or easily broken upon impact.

Fuse Plate—The plate which provides structural reinforcement to the sign post hinge to resist wind loads but which will release or fracture upon impact of a vehicle with the post.

Glare Screen—A device used to shield a driver's eye from the headlights of an oncoming vehicle.

Hinge—The weakened section of a sign post designed to allow the post to rotate upward when impacted by a vehicle.

Impact Angle—For a longitudinal barrier, it is the angle between a tangent to the face of the barrier and a tangent to the vehicle's path at impact. For a crash cushion, it is the angle between the axis of symmetry of the crash cushion and a tangent to the vehicle's path at impact.

Impact Attenuator—See Crash Cushion.

Length of Need—Total length of a longitudinal barrier needed to shield an area of concern.

Level of Performance—The degree to which a longitudinal barrier, including bridge railing, is designed for containment and redirection of different types of vehicles.

Longitudinal Barrier—A barrier whose primary function is to prevent penetration and to safely redirect an errant vehicle away from a roadside or median obstacle.

Median—The portion of a divided highway separating the traveled ways for traffic in opposite directions.

Median Barrier—A longitudinal barrier used to prevent an errant vehicle from crossing the highway median.

Non-Recoverable Slope—A slope which is considered traversable but on which the errant vehicle will continue on to the bottom. Embankment slopes between 1V:3H and 1V:4H may be considered traversable but non-recoverable if they are smooth and free of fixed objects.

Offset—Lateral distance from edge of traveled way to a roadside object or feature.

Operating Speed—The highest speed at which reasonably prudent drivers can be expected to operate vehicles on a section of highway under low traffic densities and good weather. This speed may be higher or lower than posted or legislated speed limits or nominal design speeds where alignment, surface, roadside development, or other features affect vehicle operations.

Operational Barrier—One that has performed satisfactorily in full-scale crash tests and has demonstrated satisfactory in-service performance.

Performance Level—See Level of Performance.

Recoverable Slope—A slope on which a motorist may, to a greater or lesser extent, retain or regain control of a vehicle. Slopes flatter than 1V:4H are generally considered recoverable.

Recovery Area—Generally synonymous with clear zone.

Roadside—That area between the outside shoulder edge and the right-of-way limits. The area between roadways of a divided highway may also be considered roadside.

Roadside Barrier—A longitudinal barrier used to shield roadside obstacles or non-traversable terrain features. It may occasionally be used to protect pedestrians or "bystanders" from vehicle traffic.

Roadside Signs—Roadside signs can be divided into three main categories: overhead signs, large roadside signs, and small roadside signs. Large roadside signs may be defined as those greater than or equal to 5 m^2 [50 ft^2] in area. Small roadside signs may be defined as those less than 5 m^2 [50 ft^2] in area.

Roadway—The portion of a highway, including shoulders, for vehicular use.

Rounding—The introduction of a vertical curve between two transverse slopes to minimize the abrupt slope change and to maximize vehicle stability and maneuverability.

Severity Index—A severity index (SI) is a number from zero to ten used to categorize accidents by the probability of their resulting in property damage, personal injury, or a fatality, or any combination of these possible outcomes. The resultant number can then be translated into an accident cost and the relative effectiveness of alternate safety treatments can be estimated.

Shielding—The introduction of a barrier or crash cushion between the vehicle and an obstacle or area of concern to reduce the severity of impacts of errant vehicles.

Shy Distance—The distance from the edge of the traveled way beyond which a roadside object will not be perceived as an obstacle by the typical driver to the extent that the driver will change the vehicle's placement or speed.

Slip Base—A structural element at or near the bottom of a post or pole which will allow release of the post from its base upon impact while resisting wind loads.

Slope—The relative steepness of the terrain expressed as a ratio or percentage. Slopes may be categorized as positive (backslopes) or negative (foreslopes) and as parallel or cross slopes in relation to the direction of traffic.

Temporary Barrier—Temporary barriers are used to prevent vehicular access into construction or maintenance work zones and to redirect an impacting vehicle so as to minimize damage to the vehicle and injury to the occupants while providing worker protection.

Traffic Barrier—A device used to prevent a vehicle from striking a more severe obstacle or feature located on the roadside or in the median or to prevent crossover median accidents. As defined herein, there are four classes of traffic barriers, namely, roadside barriers, median barriers, bridge railings, and crash cushions.

Transition—A section of barrier between two different barriers or, more commonly, where a roadside barrier is connected to a bridge railing or to a rigid object such as a bridge pier. The transition should produce a gradual stiffening of the approach rail so vehicular pocketing, snagging, or penetration at the connection can be avoided.

Traveled Way—The portion of the roadway for the movement of vehicles, exclusive of shoulders.

Through Traveled Way—The portion of roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.

Traversable Slope—A slope from which a motorist will be unlikely to steer back to the roadway but may be able to slow and stop safely. Slopes between 1V:3H and 1V:4H generally fall into this category.

Vehicle—A motorized unit for use in transporting passengers or freight, ranging from an 820-kg [1,800-lb] automobile to a 36000-kg [80,000-lb] van-type tractor-trailer.

Warrants—The criteria by which the need for a safety treatment or improvement can be determined.

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Chapter 1
An Introduction to Roadside Safety

1.0 HISTORY OF ROADSIDE SAFETY

Roadside safety design, as one component of total highway design, is a relatively recent concept. Most of the highway design components were established in the late 1940s and the 1950s. These components included horizontal alignment, vertical alignment, hydraulic design, and sight distance to name some of the most common highway design elements. These elements have been revised and refined over the years through experience and research. However, the highway design components themselves have remained about the same for several decades.

Roadside safety design did not become a much discussed aspect of highway design until the late 1960s, and it was the decade of the 1970s before this type of design was regularly incorporated into highway projects. Because most highways are designed for twenty- to thirty-year projected traffic volumes, many roadway projects placed in service before the 1970s are only now becoming candidates for major reconstruction. This reconstruction offers an opportunity to incorporate cost-effective roadside safety concepts and design features. The purpose of this Guide is to present the concepts of roadside safety to the designer in such a way that the most practical, appropriate, and beneficial roadside design can be accomplished for each project.

1.1 THE BENEFITS OF ROADSIDE SAFETY

Roadside design might be defined as the design of the area between the outside shoulder edge and the right-of-way limits. Some have referred to this aspect of highway design as off-pavement design. A question commonly asked revolves around whether spending resources off the pavement is really beneficial given the limited nature of infrastructure funds. Perhaps, some statistics bring the potential of crash reduction and roadside safety into focus.

The United States suffers approximately 40,000 traffic fatalities each year. The actual number has fluctuated around this level since the mid-1960s. At the same time, the number of vehicle kilometers [miles] traveled each year has increased approximately two and one-half times since the mid-1960s. Therefore, the traffic fatality rate per one billion vehicle kilometers [miles] given in Figure 1.1 has fallen by more than half since the mid-1960s.

This significant reduction is due to several factors. Motor vehicles are much safer than they were in the past. Protected passenger compartments, padded interiors, and occupant restraints are some features that have added to passenger safety during impact situations. Roadways have been made safer through design improvements such as increased superelevation, intersection geometry, and the addition of grade separations. Drivers are more educated about safe vehicle operation as evidenced by the increased use of occupant restraints and a decrease in driving under the influence of alcohol or drugs. All these contributing factors have reduced the motor vehicle fatality rate.

How significant is the involvement of the roadside environment in highway crashes? Unfortunately, roadside crashes account for far too great a portion of the total fatal highway crashes. About thirty percent, or almost one in every three fatalities, are the result of a single vehicle run-off-the-road crash. These figures mean that the roadside environment comes into play in a very significant percentage of fatal and serious-injury crashes.

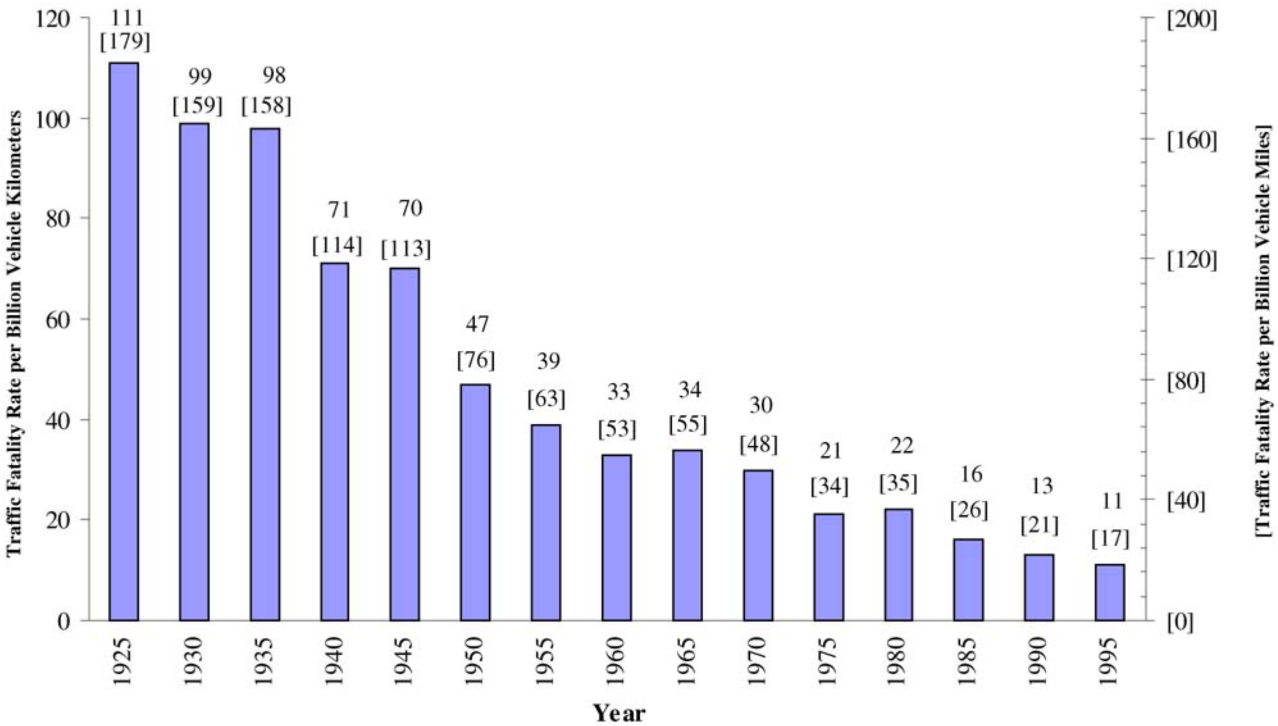


FIGURE 1.1 Traffic fatality rate per billion vehicle kilometers [miles] by year

1.2 THE FORGIVING ROADSIDE CONCEPT

There are many reasons why a vehicle will leave the pavement and encroach on the roadside, including:

- driver fatigue or inattention
- excessive speed
- driving under the influence of drugs or alcohol
- crash avoidance
- roadway conditions such as ice, snow, or rain
- vehicle component failure
- poor visibility

Regardless of the reason for a vehicle leaving the roadway, a roadside environment free of fixed objects with stable, flattened slopes enhances the opportunity for reducing crash severity. The forgiving roadside concept allows for errant vehicles leaving the roadway and supports a roadside design where the serious consequences of such an incident are reduced.

Through decades of experience and research, the application of the forgiving roadside concept has been refined to the point where roadside design is an integral part of transportation design criteria. Design options for reducing roadside obstacles, in order of preference, are as follows:

1. Remove the obstacle.
2. Redesign the obstacle so it can be safely traversed.
3. Relocate the obstacle to a point where it is less likely to be struck.
4. Reduce impact severity by using an appropriate breakaway device.
5. Shield the obstacle with a longitudinal traffic barrier designed for redirection or use a crash cushion.
6. Delineate the obstacle if the above alternatives are not appropriate.

One on-roadway safety feature that is becoming more prevalent nationwide on facilities experiencing a significant number of run-off-the-road crashes is the use of transverse milled shoulder rumble strips to supplement pavement edge lines. These indentations in the roadway shoulders alert motorists through noise and vibration that their vehicles have crossed the edge line and afford many motorists an opportunity to remain on or return to the traveled way safely. Several transportation agencies have reported significant reductions in single-vehicle crashes after installing shoulder rumble strips.

1.3 THE CONTENT AND FORMAT

This Guide replaces the 2nd Edition of the AASHTO *Roadside Design Guide* (1996). This publication can be considered a companion document for such current publications as *A Policy on Geometric Design of Highways and Streets* and *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*. There are also several research publications and additional reference literature given at the end of each chapter.

Chapter 2 discusses methods for selecting appropriate alternative roadside safety enhancements. The discussion involves benefit/cost analysis to determine a ranking of alternatives in the absence of better local information. Appendix A offers an example of one methodology for accomplishing a benefit/cost analysis of various alternatives.

Chapter 3 contains a discussion of the clear roadside concept. It gives some relative clear zone values from which design guidance may be derived. Examples of the application of the clear-zone values are also given. The chapter also includes a discussion of the treatment of drainage features.

Chapter 4 provides information on the use of sign and luminaire supports within the roadside environment. Both small and large signs are included and criteria for breakaway and non-breakaway supports are presented. The chapter concludes with discussions of miscellaneous roadside features such as mailbox supports, utility poles, and trees.

Chapters 5, 6, 7, and 8 provide information concerning roadside barriers and crash cushions. Chapter 5 discusses roadside barriers. Appendix B gives selected details for these roadside barriers. Chapter 6 provides equivalent information for median barriers and Appendix C gives selected median barrier details. Chapter 7 includes information on appropriate bridge railings. Chapter 8 offers the latest state-of-the-practice information on barrier end treatments and crash cushions.

Chapter 9 discusses the application of the roadside safety concept for the temporary conditions found in construction or maintenance work zones. For example, the chapter contains information on clear zones in a work zone, temporary barriers, truck-mounted attenuators, and temporary traffic control devices.

Chapter 10 discusses the application of roadside safety in the urban environment. While much of the information presented in this publication applies to rural high-speed conditions, this chapter offers information on urban roadside practices.

Chapter 11 with Appendices D and E provides information on mailboxes and mailbox turnout design.

1.4 CRASH TESTING ROADSIDE SAFETY FEATURES AND APPURTENANCES

This publication has several references to crash-testing criteria and crash-tested hardware. The intended implication of referring to a device as crash tested is that the roadside hardware was tested to the applicable criteria in existence at the time when full-scale crash testing was done. While full-scale crash-testing criteria subjects roadside devices to severe vehicle impact conditions, the testing can not duplicate every roadside condition or vehicle impact situation. The testing provides for an acceptable level of performance under normalized conditions. However, every roadside device or installation has limitations dictated by physical laws, the crashworthiness of vehicles, and the limitation of resources. Some in-service impact situations may have more severe consequences if they occur beyond the design limits, which the testing was intended to replicate.

National Cooperative Highway Research Program (NCHRP) Report 350, *Recommended Procedures for the Safety Performance Evaluation of Highway Features* (1), contains the current recommendations for testing and evaluating the performance of longitudinal barriers, terminals, crash cushions, breakaway or yielding supports for signs and luminaries, breakaway utility poles, truck-mounted attenuators, and work zone traffic control devices. NCHRP Report 350 establishes uniform procedures for the testing and in-service evaluation of permanent and temporary safety features and supersedes previous recommendations provided in NCHRP Report 230 (2). Major revisions from NCHRP Report 230 included test vehicle changes, number of impact conditions, adoption of the "Test Level" concept, widened ranges of devices, and metrication. The uniform testing and evaluation procedures set forth in NCHRP Report 350 provide the following benefits:

- a basis for comparison of impact performance merits of candidate safety features;
- guidance for safety feature manufacturers; and
- a basis for the formulation of safety feature performance specifications.

NCHRP Report 350 presents specific impact conditions for conducting vehicle crash tests. The conditions include vehicle mass [weight], speed, approach angle, and point on the safety feature to be hit. Standard test vehicle types are defined for small passenger cars, standard ¾-ton pickup trucks, single-unit van trucks, tractor/van-type trailer units, and tractor/tanker trailer units. The impact speeds range from 35 to 100 km/h [approximately 20 to 60 mph] and approach angles vary from 0 to 25 degrees. The specific NCHRP Report 350 test conditions and evaluation criteria for each type of roadside device are summarized in the chapters that address that type of device. The report itself is out-of-print but can be viewed and downloaded from the following web site: http://www4.nationalacademies.org/trb/crp.nsf/NCHRP+projects. From this site, NCHRP Report 350 can be found by clicking on Area 22, then on Project 22-7. The file is very large and is primarily intended for research personnel who conduct the actual crash testing.

1.5 THE APPLICATION OF THIS GUIDE

This publication is intended to present information on the latest state-of-the-practice in roadside safety. The concepts, designs, and philosophies presented in the following chapters can not, and should not, be included in their totality on every single project. Each project is unique and offers an individual opportunity to enhance that particular roadside environment from a safety perspective.

The guidelines presented in this publication are most applicable to new construction or major reconstruction projects. These projects, which often include significant changes in horizontal or vertical alignment, offer the greatest opportunity for implementing many of the roadside safety enhancements presented in this document. For resurfacing, rehabilitation, or restoration (3R) projects, the primary emphasis is generally placed on the roadway itself to maintain the structural integrity of the pavement. It will generally be necessary to selectively incorporate roadside safety guidelines on 3R projects only at locations where the greatest safety benefit can be realized. Because of the scope of 3R projects and the limited nature of most rehabilitation programs, the identification of areas that offer the greatest safety enhancement potential is critical. Accident reports, site investigations, and maintenance records offer starting points for identifying these locations.

The importance of designing the roadside to be as clear as practical can be seen by noting which objects and slope conditions are most frequently associated with fatal run-off-the-road crashes. Table 1.1 shows the numbers of fatalities in the United States from 1993 to 1999 resulting from collisions with specific roadside objects or slope conditions. This information was obtained through the National Highway Traffic Safety Administration's (NHTSA) Fatality Analysis Reporting System (FARS) and identifies the first harmful event in a series of events resulting in a fatal crash. In some cases, the first harmful event is also identified as the most harmful event. For example, if a motorist strikes a tree, the impact with the tree is likely to be classified as both the first and most harmful event. On the other hand, if the first harmful event is striking an embankment, the most harmful event is often a rollover. FARS data for each State can be accessed directly at http://www-fars.nhtsa.dot.gov.

The amount of monetary resources available for all roadside safety enhancements is limited. The objective of designers has to be to maximize roadside safety on a system-wide basis with the given funds. Accomplishing this objective means addressing those specific roadside features that can contribute the most to the safety enhancement of that individual highway project. If the inclusion of the highest level of roadside design criteria is routinely required in each highway design project, regardless of cost or safety effectiveness, it is likely that system-wide safety may stay static or may be degraded. This potential will certainly exist if other roadside needs are not improved because funds were not judiciously applied to the most viable safety enhancement need.

Given the fact that objects and slope changes must be introduced at varying points off the pavement edge, the enhancement of roadside safety involves selecting the "best" choice among several acceptable design alternatives. The experience gained from decades of selecting design alternatives, the research done on vehicle dynamics, and the technological advances in materials offers the potential for maintaining and enhancing one of the safest national transportation systems in existence.

This document is intended to represent the spectrum of commonly available roadside design alternatives. In most cases, these alternatives have shown significant benefits under appropriately selected field conditions. Many of these roadside enhancements have, over time, demonstrated their ability in the field to improve roadside safety conditions. In many areas, this publication strives to give the advantages and disadvantages of roadside technology. With this information, designers can make more knowledgeable decisions about the best applications for individual projects. It should be noted that no attempt is made, or implied, to offer every single roadside enhancement design technique or technology.

Finally, this publication is not intended to be used as a standard or a policy statement. This document is made available to be a resource for current information in the area of roadside design. Agencies may choose to use this information as one reference upon which to build the roadside design criteria best suited to their particular location

TABLE 1.1 First harmful event fixed-object fatalities by object type

FIXED OBJECT	YEAR						
	1993	1994	1995	1996	1997	1998	1999
Boulder	82	96	90	93	87	90	91
Bridge/ Overpass	448	434	459	435	431	402	409
Building	100	77	77	62	96	78	81
Concrete Barrier	229	183	229	221	239	259	280
Culvert/Ditch	1,359	1,380	1,476	1,437	1,396	1,491	1,481
Curb/Wall	810	830	921	947	915	823	753
Embankment	1,060	1,143	1,269	1,239	1,186	1,206	1,268
Fence	397	441	432	478	429	473	512
Guardrail	1,128	1,125	1,191	1,137	1,159	1,248	1,185
Impact Attenuator	23	28	35	26	19	19	24
Sign or Light Support	471	453	580	634	514	504	546
Tree/Shrub	3,035	3,014	3,198	3,128	3,220	3,226	3,348
Utility Pole	1,274	1,096	1,135	1,096	1,111	1,092	1,070
Other Fixed Objects	575	587	564	569	534	508	508
Other Pole/ Support	301	350	359	404	359	312	352
Total Fatalities	11,292	11,237	12,015	11,906	11,695	11,731	11,908

and projects. Knowledgeable design, practically applied at the local level, offers the greatest potential for a continually improved transportation system.

REFERENCES

1. Ross, H. E., Jr., D. L. Sicking, and R. A. Zimmer. *National Cooperative Highway Research Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features*. Transportation Research Board, Washington, DC, 1993.
2. Michie, J. D. *National Cooperative Highway Research Program Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenance*. Transportation Research Board. Washington, DC, 1981.

Chapter 2
Roadside Safety and Economics

2.0 OVERVIEW

The consistent application of geometric design standards for roads and streets provides motorists with a high degree of safety. Design features, such as horizontal and vertical curvature, lane and shoulder width, signing, shoulder rumble strips, and pavement markings, play an important role in keeping motorists on the traveled way. Roadside safety features, such as breakaway supports, barriers, and crash cushions provide an extra margin of safety to motorists who inadvertently leave the roadway. Most of these appurtenances are routinely installed based on a subjective analysis of their benefits to the motorist; however, in some instances it may not be immediately obvious that the benefits to be gained from a specific safety design or treatment equal or exceed the additional costs. Thus, a designer must decide how and where limited funds should be spent to achieve the greatest overall benefit. One method that can be used to make this determination is a benefit/cost analysis.

2.1 BENEFIT/COST ANALYSIS

A benefit/cost analysis is a method by which the estimated benefits to be derived from a specific course of action are compared to the costs of implementing that action. If the estimated benefits of a specific design exceed the cost of constructing and maintaining that design over a period of time, the safer design may be implemented; however, simply having a benefit/cost ratio greater than one is not ample justification for the construction of a roadside safety treatment. Each project must compete with others for limited safety funds. The designer must attempt to build those projects that best meet the public's need for safety and mobility.

The primary benefit obtained from selecting one design over another, relative to safety, is the expected reduction in the future costs of crashes. These costs typically include property damage costs and personal injury costs. To estimate these costs, the expected number and severity of crashes that may occur for each roadside treatment must be estimated. In some cases, the total number of crashes may be reduced by a given treatment, such as providing a significantly wider roadside recovery area than previously existed. In other instances, the safety treatment may not reduce the total number of crashes but may reduce their severity. The installation of a median or roadside traffic barrier may have this effect.

The costs used in a benefit/cost analysis are generally the direct construction and maintenance costs incurred by the highway agency. They can usually be estimated with a high degree of accuracy.

A benefit/cost analysis must also consider the period of time (project life) over which each alternative treatment provides a benefit. Because different treatments can have different project lives, both benefits and costs must be annualized so direct comparisons between alternative design treatments can be made. To reduce total (life-cycle) costs to annualized costs, discount rates must be considered. An annualized benefit/cost ratio thus compares the expected savings (benefits) to society, through reduced costs from crashes, to the costs (construction and maintenance) incurred by the highway agency to provide a specific treatment.

The following subsections identify the type of data that are needed to conduct a benefit/cost analysis and the general availability of this information. The major factors include:

- encroachments,
- roadside geometry, and
- crash costs.

2.1.1 Encroachments

The benefits derived from a roadside safety treatment can be calculated by first estimating the number of vehicles that are likely to run off the road at a particular location. By definition, an encroachment occurs when a motorist strays from the traveled way. The primary factors that affect the number of encroachments are traffic volume, roadway alignment, and lane widths. The number of estimated encroachments is determined by multiplying an encroachment rate by the number of vehicles using the facility, resulting in a figure representing the number of encroachments per kilometer [mile] per year. Current encroachment rates are derived from a limited number of studies conducted over the past 30 years (1, 2). These rates should be adjusted when actual data at a specific location are available. They may also be modified based on engineering judgment for non-typical conditions.

It should be further noted that not all encroachments result in crashes. For example, for small-angle encroachments, even a narrow recovery area may provide enough space for a driver to regain control and return safely to the roadway. To estimate the number of crashes that may result from encroachments, the angles of departure from the roadway and the speeds and types of vehicles involved must be considered.

2.1.2 Roadside Geometry

Once a vehicle has left the roadway, a crash may or may not occur. The end result of an encroachment depends upon the physical characteristics of the roadside environment. As noted earlier, the highway designer has a significant degree of control over roadside geometry and appurtenances. Flat, traversable, stable slopes will minimize overturning crashes, which are usually severe. Elimination of roadside hardware, its relocation to less vulnerable areas, or the use of breakaway-type devices remain the options of choice in the development of safer roadsides. Obstructions that cannot otherwise be treated should be shielded by properly designed and installed traffic barriers or crash cushions if it is cost-effective to do so. Finally, if a fixed object or other roadside obstacle cannot be eliminated, relocated, modified, or shielded for whatever reason, consideration should be given to delineating the feature so it is readily visible to a motorist.

2.1.3 Crash Costs

Once an estimate has been made of the number of crashes that can be expected at a given location, this information must be translated into a cost that is directly related to crash severity. One method of accomplishing this is by assigning a Severity Index (SI) to individual crashes. This SI will vary with the type of vehicle involved, its speed and impact angle, and the type of obstacle struck. A crash may range in severity from minor to fatal. If an SI system is used, a crash involving no personal injuries and negligible property damage might be assigned an SI of zero, while a crash with a 100 percent chance of a fatality might be assigned an SI of 10. Between these extremes, crashes typically involve varying degrees of property damage coupled with slight, moderate, or severe personal injuries.

Converting severity indices to crash cost is a relatively easy process, but it does require that a dollar cost be assigned to each type of crash. This step involves considerable judgment because it requires that a value be assigned to each crash classification, including fatal crashes. Primary sources of crash cost data include the National Safety Council, the National Highway Traffic Safety Administration, and the Federal Highway Administration.

2.2 BENEFIT/COST ANALYSIS PROGRAMS

Several highway agencies have used the ROADSIDE analysis program presented in the two earlier editions of the Roadside Design Guide to both analyze site-specific alternative safety treatments and to develop design charts and tables using local data. Information on an updated and significantly revised version of ROADSIDE, called the Roadside Safety Analysis Program (RSAP), is included in Appendix A.

REFERENCES

1. Hutchinson, J. W., and T. W. Kennedy. *Medians of Divided Highways—Frequency and Nature of Vehicle Encroachments*. Bulletin 487. University of Illinois Engineering Experiment Station, 1966.

2. Cooper, P. "Analysis of Roadside Encroachments—Single Vehicle Run-off-Road Accident Data Analysis for Five Provinces," *B.C. Research*. Vancouver, British Columbia, Canada, March 1980.

Chapter 3
Roadside Topography and Drainage Features

3.0 OVERVIEW

This chapter includes a discussion on the development and evaluation of the clear roadside concept and its application to roadside design. It also discusses embankment slopes and ditches and how these features influence roadside features such as curbs, culverts, and drop inlets, whose purpose is to provide adequate roadway drainage. The designer is presented with several options that enhance safety without affecting the capabilities of these elements to drain the highway.

Most of the clear roadside design guidelines discussed in this chapter have been practiced to varying degrees for several years. This chapter attempts to reemphasize and collect the currently accepted design principles to provide guidance in the area of roadside clearances. However, to include every recommendation or design value in this chapter on every future highway project is neither feasible nor possible. Engineering judgment will have to play a part in determining the extent to which improvements can reasonably be made with the limited resources available.

As the designer studies the options available, some consideration should be given to the future maintenance of drainage facilities and roadside topography. Ongoing repair and upkeep will be necessary to ensure the continued function and safety of various roadside drainage features. Personnel, materials, equipment, and cost are some of the considerations in every maintenance program. The designer should take into account the exposure of crews to traffic conditions while completing repairs. Also, maintenance activities can cause various levels of disruption in the traffic flow, which may increase the potential for crashes.

3.1 THE CLEAR ROADSIDE CONCEPT

Beginning in the early 1960s, as more Interstate highways and other freeways were opened to traffic, the nature and characteristics of the typical rural highway crash began to change. Instead of head-on crashes with other vehicles or crashes involving trees immediately adjacent to the roadway, many drivers were running off the new freeways and colliding with man-made objects such as bridge piers, sign supports, culverts, ditches, and other design features of the roadway. In 1967, the AASHO Traffic Safety Committee (currently the AASHTO Standing Committee on Highway Traffic Safety) issued a report entitled *Highway Design and Operational Practices Related to Highway Safety* (1). This document became known as the "Yellow Book" and its principles were widely applied to highway construction projects, particularly high-speed controlled access facilities. A second edition of the Yellow Book, published by AASHTO in 1974, stated that "for adequate safety, it is desirable to provide an unencumbered roadside recovery area that is as wide as practical on a specific highway section. Studies have indicated that on high-speed highways, a width of 9 meters [30 feet] or more from the edge of the through traveled way permits about 80 percent of the vehicles leaving a roadway out of control to recover."

Subsequently, most highway agencies began to try to provide a traversable and unobstructed roadside area (clear zone) extending approximately 9 m [30 ft] beyond the edge of the through traveled way, particularly on high-volume, high-speed roadways. Many obstacles located within this clear-zone distance were removed, relocated, redesigned, or shielded by traffic barriers or crash cushions. It soon became apparent, however, that in some limited situations where the embankment sloped significantly downward, a vehicle could encroach farther from the through traveled way; thus, a 9 m [30 ft] recovery area might not be adequate. Conversely, on most low-volume or low-speed facilities, a 9 m [30 ft] clear-zone distance was considered excessive and could seldom be justified for engineering, environmental, or economic reasons.

The 1977 AASHTO *Guide for Selecting, Locating and Designing Traffic Barriers* (2) modified the earlier clear-zone concept by introducing variable clear-zone distances based on traffic volumes, speeds and roadside geometry. Figure 3.1 or Table 3.1 can be used to determine the suggested clear-zone distance for selected traffic volumes and speeds. However, Figure 3.1 and Table 3.1 only provide a general approximation of the needed clear-zone distance. The curves are based on limited empirical data that was extrapolated to provide information for a wide range of conditions. The designer must keep in mind site-specific conditions, design speeds, rural versus urban locations, and practicality. The distances obtained from Figure 3.1 and Table 3.1 should suggest only the approximate center of a range to be considered and not a precise distance to be held as absolute.

The designer may choose to modify the clear-zone distance for horizontal curvature obtained from either Figure 3.1 or Table 3.1 by using Table 3.2. These modifications are normally considered only when crash histories indicate a need, or a specific site investigation shows a definitive crash potential that could be significantly lessened by increasing the clear-zone width, and when such increases are cost effective. Horizontal curves, particularly for high-speed facilities, are usually superelevated to increase safety and provide a more comfortable ride. Increased banking on curves where the superelevation is inadequate is an alternate method of increasing roadway safety within a horizontal curve, except where snow and ice conditions limit the use of increased superelevation.

For relatively flat and level roadsides, the clear-zone concept is simple to apply. However, it is less clear when the roadway is in a fill or cut section where roadside slopes may be either positive, negative, or variable, or where a drainage channel exists near the through traveled way. Consequently, these features must be discussed before a full understanding of the clear-zone concept is possible. The AASHTO publication *A Policy on Geometric Design of Highways and Streets* (3) may be referenced for additional clear-zone discussion.

3.2 ROADSIDE GEOMETRY

If a roadside is not flat, a motorist leaving the roadway will encounter a foreslope, a backslope, a transverse slope, or a drainage channel (change in sideslope from a foreslope to a backslope). Each of these features has an effect on a vehicle's lateral encroachment and trajectory as discussed in the following sections.

3.2.1 Foreslopes

Foreslopes parallel to the flow of traffic may be identified as recoverable, non-recoverable, or critical. Recoverable foreslopes are 1V:4H or flatter. If such slopes are relatively smooth and traversable, the suggested clear-zone distance may be taken directly from Figure 3.1 or Table 3.1. Motorists who encroach on recoverable foreslopes can generally stop their vehicles or slow them enough to return to the roadway safely. Fixed obstacles such as culvert headwalls will normally not extend above the foreslope within the clear-zone distance. Examples of suggested roadside design practices for recoverable foreslopes and the application of the clear-zone concept are in Section 3.3.1.

A non-recoverable foreslope is defined as one that is traversable, but from which most vehicles will be unable to stop or to return to the roadway easily. Vehicles on such slopes typically can be expected to reach the bottom. Foreslopes between 1V:3H and 1V:4H generally fall into this category. Since a high percentage of encroaching vehicles will reach the toe of these slopes, the clear-zone distance cannot logically end on the slope. Fixed obstacles will normally not be constructed along such slopes and a clear runout area at the base is desirable. Section 3.3.2 discusses non-recoverable foreslopes. Example C provides an example for a clear-zone computation.

A critical foreslope is one on which a vehicle is likely to overturn. Foreslopes steeper than 1V:3H generally fall into this category. If a foreslope steeper than 1V:3H begins closer to the through traveled way than the suggested clear-zone distance for that specific roadway, a barrier might be warranted if the slope cannot readily be flattened. Barrier warrants for critical foreslopes are discussed in Chapter 5.

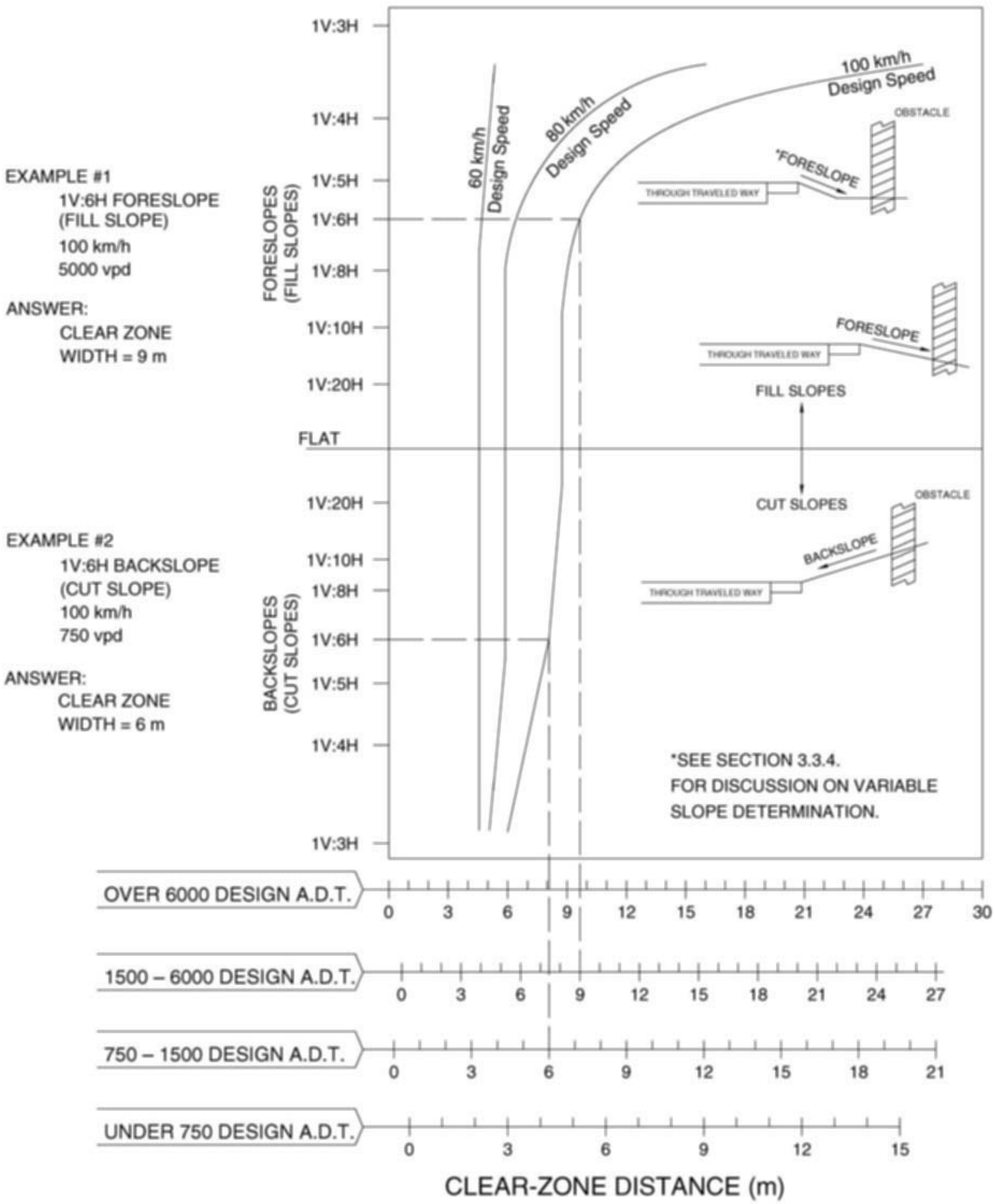


FIGURE 3.1a Clear-zone distance curves [metric units]

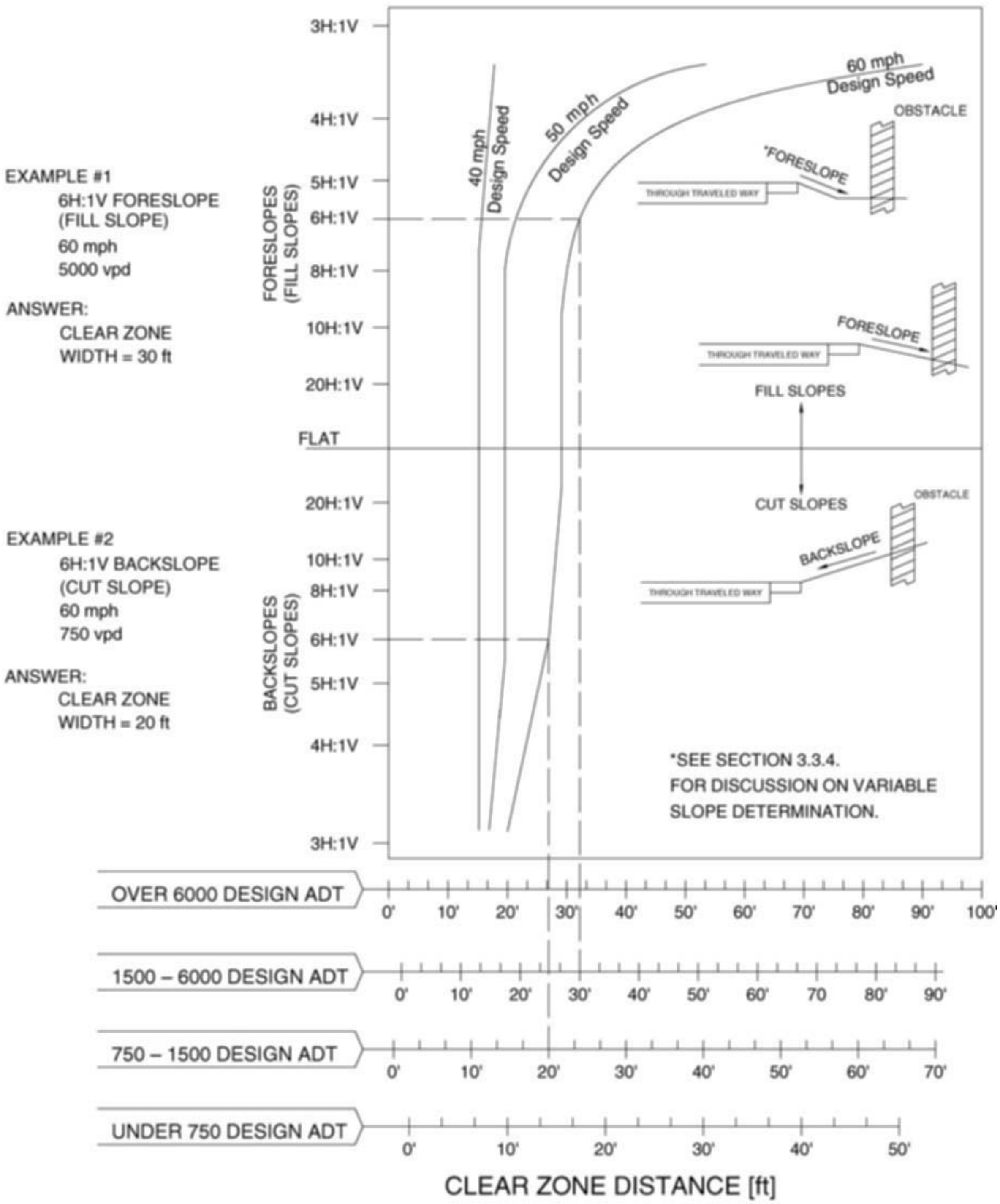


FIGURE 3.1b Clear-zone distance curves [U.S. customary units]

TABLE 3.1 Clear-zone distances in meters [feet] from edge of through traveled way

Metric Units

DESIGN SPEED	DESIGN ADT	FORESLOPES			BACKSLOPES		
		1V:6H or flatter	1V:5H TO 1V:4H	1V:3H	1V:3H	1V:5H TO 1V:4H	1V:6H or flatter
60 km/h or less	UNDER 750	2.0 – 3.0	2.0 – 3.0	**	2.0 – 3.0	2.0 – 3.0	2.0 – 3.0
	750 – 1500	3.0 – 3.5	3.5 – 4.5	**	3.0 – 3.5	3.0 – 3.5	3.0 – 3.5
	1500 – 6000	3.5 – 4.5	4.5 – 5.0	**	3.5 – 4.5	3.5 – 4.5	3.5 – 4.5
	OVER 6000	4.5 – 5.0	5.0 – 5.5	**	4.5 – 5.0	4.5– 5.0	4.5 – 5.0
70–80 km/h	UNDER 750	3.0 – 3.5	3.5 – 4.5	**	2.5 – 3.0	2.5 – 3.0	3.0 – 3.5
	750 – 1500	4.5 – 5.0	5.0 – 6.0	**	3.0 – 3.5	3.5 – 4.5	4.5 – 5.0
	1500 – 6000	5.0 – 5.5	6.0 – 8.0	**	3.5 – 4.5	4.5 – 5.0	5.0 – 5.5
	OVER 6000	6.0 – 6.5	7.5 – 8.5	**	4.5 – 5.0	5.5 – 6.0	6.0 – 6.5
90 km/h	UNDER 750	3.5 – 4.5	4.5 – 5.5	**	2.5 – 3.0	3.0 – 3.5	3.0 – 3.5
	750 – 1500	5.0 – 5.5	6.0 – 7.5	**	3.0 – 3.5	4.5 – 5.0	5.0 – 5.5
	1500 – 6000	6.0 – 6.5	7.5 – 9.0	**	4.5 – 5.0	5.0 – 5.5	6.0 – 6.5
	OVER 6000	6.5 – 7.5	8.0 – 10.0*	**	5.0 – 5.5	6.0 – 6.5	6.5 – 7.5
100 km/h	UNDER 750	5.0 – 5.5	6.0 – 7.5	**	3.0 – 3.5	3.5 – 4.5	4.5 – 5.0
	750 – 1500	6.0 – 7.5	8.0 – 10.0*	**	3.5 – 4.5	5.0 – 5.5	6.0 – 6.5
	1500 – 6000	8.0 – 9.0	10.0 – 12.0*	**	4.5 – 5.5	5.5 – 6.5	7.5 – 8.0
	OVER 6000	9.0 – 10.0*	11.0 – 13.5*	**	6.0 – 6.5	7.5 – 8.0	8.0 – 8.5
110 km/h	UNDER 750	5.5 – 6.0	6.0 – 8.0	**	3.0 – 3.5	4.5 – 5.0	4.5 – 5.0
	750 – 1500	7.5 – 8.0	8.5 – 11.0*	**	3.5 – 5.0	5.5 – 6.0	6.0 – 6.5
	1500 – 6000	8.5 – 10.0*	10.5 – 13.0*	**	5.0 – 6.0	6.5 – 7.5	8.0 – 8.5
	OVER 6000	9.0 – 10.5*	11.5 – 14.0*	**	6.5 – 7.5	8.0 – 9.0	8.5 – 9.0

* Where a site specific investigation indicates a high probability of continuing crashes, or such occurrences are indicated by crash history, the designer may provide clear-zone distances greater than the clear-zone shown in Table 3.1. Clear zones may be limited to 9 m for practicality and to provide a consistent roadway template if previous experience with similar projects or designs indicates satisfactory performance.

** Since recovery is less likely on the unshielded, traversable 1V:3H slopes, fixed objects should not be present in the vicinity of the toe of these slopes. Recovery of high-speed vehicles that encroach beyond the edge of the shoulder may be expected to occur beyond the toe of slope. Determination of the width of the recovery area at the toe of slope should take into consideration right-of-way availability, environmental concerns, economic factors, safety needs, and crash histories. Also, the distance between the edge of the through traveled lane and the beginning of the 1V:3H slope should influence the recovery area provided at the toe of slope. While the application may be limited by several factors, the foreslope parameters which may enter into determining a maximum desirable recovery area are illustrated in Figure 3.2.

TABLE 3.1 (Cont'd)

[U.S. Customary Units]

DESIGN SPEED	DESIGN ADT	FORESLOPES			BACKSLOPES		
		1V:6H or flatter	1V:5H TO 1V:4H	1V:3H	1V:3H	1V:5H TO 1V:4H	1V:6H or flatter
40 mph or less	UNDER 750	7 – 10	7 – 10	**	7 – 10	7 – 10	7 – 10
	750 – 1500	10 – 12	12 – 14	**	10 – 12	10 – 12	10 – 12
	1500 – 6000	12 – 14	14 – 16	**	12 – 14	12 – 14	12 – 14
	OVER 6000	14 – 16	16 – 18	**	14 – 16	14 – 16	14 – 16
45–50 mph	UNDER 750	10 – 12	12 – 14	**	8 – 10	8 – 10	10 – 12
	750 – 1500	14 – 16	16 – 20	**	10 – 12	12 – 14	14 – 16
	1500 – 6000	16 – 18	20 – 26	**	12 – 14	14 – 16	16 – 18
	OVER 6000	20 – 22	24 – 28	**	14 – 16	18 – 20	20 – 22
55 mph	UNDER 750	12 – 14	14 – 18	**	8 – 10	10 – 12	10 – 12
	750 – 1500	16 – 18	20 – 24	**	10 – 12	14 – 16	16 – 18
	1500 – 6000	20 – 22	24 – 30	**	14 – 16	16 – 18	20 – 22
	OVER 6000	22 – 24	26 – 32 *	**	16 – 18	20 – 22	22 – 24
60 mph	UNDER 750	16 – 18	20 – 24	**	10 – 12	12 – 14	14 – 16
	750 – 1500	20 – 24	26 – 32 *	**	12 – 14	16 – 18	20 – 22
	1500 – 6000	26 – 30	32 – 40 *	**	14 – 18	18 – 22	24 – 26
	OVER 6000	30 – 32 *	36 – 44 *	**	20 – 22	24 – 26	26 – 28
65–70 mph	UNDER 750	18 – 20	20 – 26	**	10 – 12	14 – 16	14 – 16
	750 – 1500	24 – 26	28 – 36 *	**	12 – 16	18 – 20	20 – 22
	1500 – 6000	28 – 32 *	34 – 42 *	**	16 – 20	22 – 24	26 – 28
	OVER 6000	30 – 34 *	38 – 46 *	**	22 – 24	26 – 30	28 – 30

* Where a site specific investigation indicates a high probability of continuing crashes, or such occurrences are indicated by crash history, the designer may provide clear-zone distances greater than the clear-zone shown in Table 3.1. Clear zones may be limited to 30 ft for practicality and to provide a consistent roadway template if previous experience with similar projects or designs indicates satisfactory performance.

** Since recovery is less likely on the unshielded, traversable 1V:3H slopes, fixed objects should not be present in the vicinity of the toe of these slopes. Recovery of high-speed vehicles that encroach beyond the edge of the shoulder may be expected to occur beyond the toe of slope. Determination of the width of the recovery area at the toe of slope should take into consideration right-of-way availability, environmental concerns, economic factors, safety needs, and crash histories. Also, the distance between the edge of the through traveled lane and the beginning of the 1V:3H slope should influence the recovery area provided at the toe of slope. While the application may be limited by several factors, the foreslope parameters which may enter into determining a maximum desirable recovery area are illustrated in Figure 3.2.

TABLE 3.2 Horizontal Curve Adjustments

K_{cz} (Curve Correction Factor) (Metric Units)

RADIUS (m)	DESIGN SPEED (km/h)					
	60	70	80	90	100	110
900	1.1	1.1	1.1	1.2	1.2	1.2
700	1.1	1.1	1.2	1.2	1.2	1.3
600	1.1	1.2	1.2	1.2	1.3	1.4
500	1.1	1.2	1.2	1.3	1.3	1.4
450	1.2	1.2	1.3	1.3	1.4	1.5
400	1.2	1.2	1.3	1.3	1.4	—
350	1.2	1.2	1.3	1.4	1.5	—
300	1.2	1.3	1.4	1.5	1.5	—
250	1.3	1.3	1.4	1.5	—	—
200	1.3	1.4	1.5	—	—	—
150	1.4	1.5	—	—	—	—
100	1.5	—	—	—	—	—

K_{cz} (Curve Correction Factor) [U.S. Customary Units]

RADIUS [ft]	DESIGN SPEED [mph]						
	40	45	50	55	60	65	70
2860	1.1	1.1	1.1	1.2	1.2	1.2	1.3
2290	1.1	1.1	1.2	1.2	1.2	1.3	1.3
1910	1.1	1.2	1.2	1.2	1.3	1.3	1.4
1640	1.1	1.2	1.2	1.3	1.3	1.4	1.5
1430	1.2	1.2	1.3	1.3	1.4	1.4	—
1270	1.2	1.2	1.3	1.3	1.4	1.5	—
1150	1.2	1.2	1.3	1.4	1.5	—	—
950	1.2	1.3	1.4	1.5	1.5	—	—
820	1.3	1.3	1.4	1.5	—	—	—
720	1.3	1.4	1.5	—	—	—	—
640	1.3	1.4	1.5	—	—	—	—
570	1.4	1.5	—	—	—	—	—
380	1.5	—	—	—	—	—	—

$CZ_C = (L_C)(K_{CZ})$

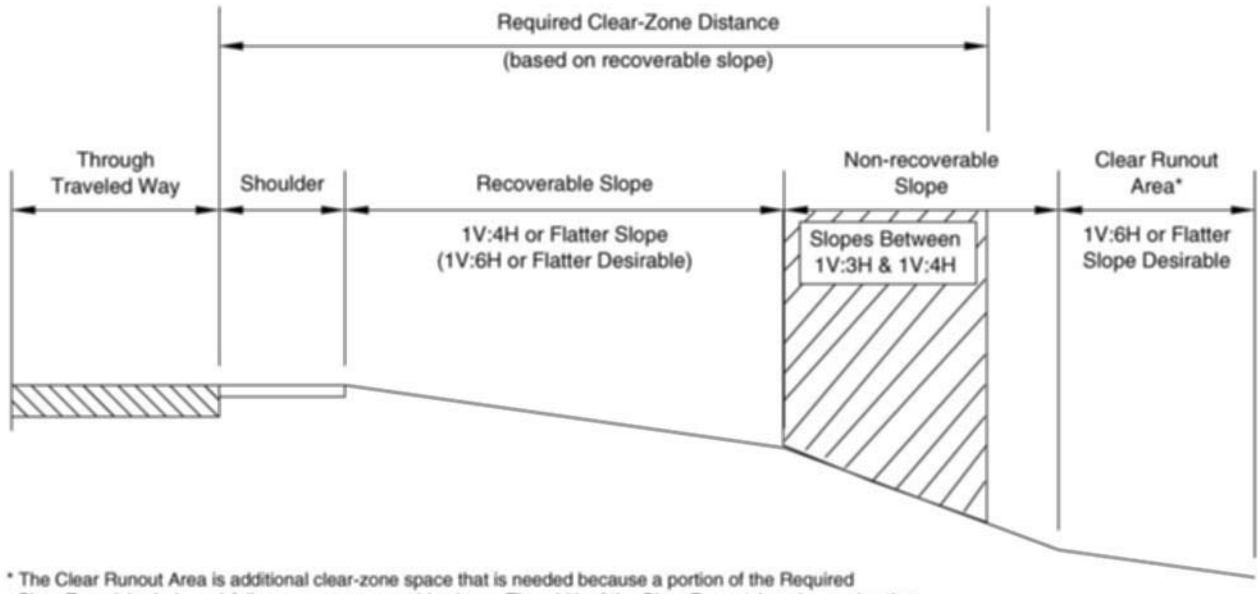
Where:

CZ_C = clear zone on outside of curvature, meters [feet]

L_C = clear-zone distance, meters [feet] (Figure 3.1 or Table 3.1)

K_{CZ} = curve correction factor

Note: The clear-zone correction factor is applied to the outside of curves only. Curves flatter than 900 m [2860 ft] do not require an adjusted clear zone.



* The Clear Runout Area is additional clear-zone space that is needed because a portion of the Required Clear Zone (shaded area) falls on a non-recoverable slope. The width of the Clear Runout Area is equal to that portion of the Clear Zone Distance that is located on the non-recoverable slope.

FIGURE 3.2 Example of a parallel foreslope design

Many State highway agencies typically construct "barn roof" sections, providing a relatively flat recovery area adjacent to the roadway for some distance, followed by a steeper foreslope. Such a cross section is more economical than providing a continuous flat foreslope from the edge of the through traveled way to the original ground line and is generally perceived as safer than constructing a continuous steeper foreslope from the edge of the shoulder. Figure 3.2 depicts the clear-zone distance reaching a non-recoverable parallel foreslope and the subsequent clear runout area that may be provided at the toe of the non-recoverable slope to provide a maximum desirable adjusted clear-zone distance. Example clear-zone calculations for this type of cross section are also included in Section 3.3.4.

3.2.2 Backslopes

When a highway is located in a cut section, the backslope may be traversable depending upon its relative smoothness and the presence of fixed obstacles. If the foreslope between the roadway and the base of the backslope is traversable (1V:3H or flatter) and the backslope is obstacle-free, it may not be a significant obstacle, regardless of its distance from the roadway. On the other hand, a steep, rough-sided rock cut should normally begin outside the clear zone or be shielded. A rock cut is normally considered to be rough-sided when the face will cause excessive vehicle snagging rather than provide relatively smooth redirection. Warrants for the use of a roadside barrier in conjunction with backslopes are included in Chapter 5.

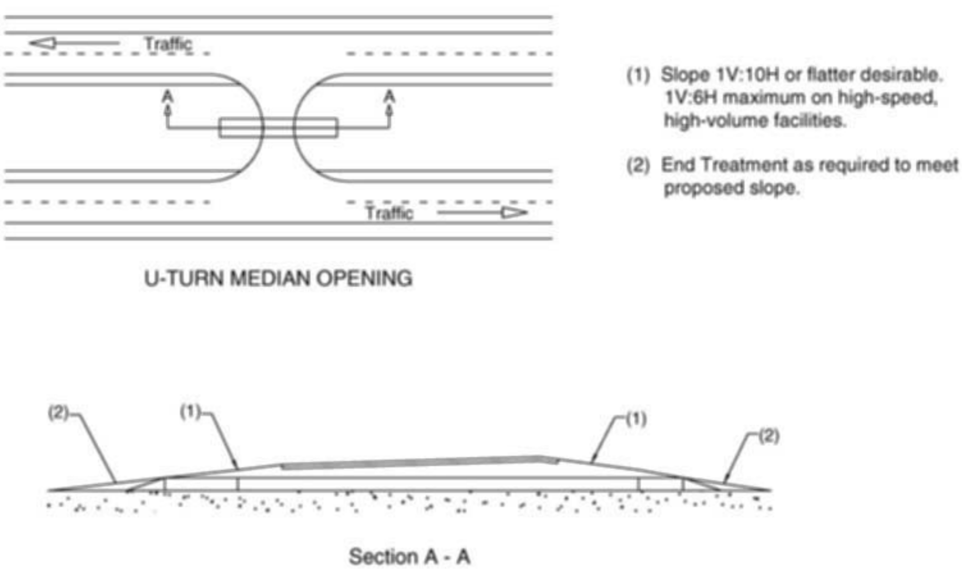
3.2.3 Transverse Slopes

Common obstacles on roadsides are transverse slopes created by median crossovers, berms, driveways, or intersecting side roads. These are generally more critical to errant motorists than foreslopes or backslopes because they are typically struck head on by run-off-the-road vehicles. Transverse slopes of 1V:6H or flatter are suggested for high-speed roadways, particularly for that section of the transverse slope that is located immediately adjacent to traffic. This slope can then be transitioned to a steeper slope as the distance from the through traveled way increases.

Transverse slopes of 1V:10H are desirable; however, their practicality may be limited by width restrictions and the maintenance problems associated with the long tapered ends of pipes or culverts. Transverse slopes steeper than 1V:6H may be considered for urban areas or for low-speed facilities. Figures 3.3 and 3.4 show suggested designs for these slopes. Safety treatments for parallel drainage structures are discussed in Section 3.4.3.

Some alternative designs for drains at median openings are shown in Figure 3.5. The water flows into a grated drop inlet in the median to a cross-drainage structure or

FIGURE 3.3 Preferred cross slope design



*Use of the flattest possible median cross slopes on high-speed highways, particularly within the appropriate clear-zone area, can provide an improved roadside. Safety treatment of culverts as discussed in Section 3.4.3 may further enhance the improvement.

FIGURE 3.4 Median transverse slope design

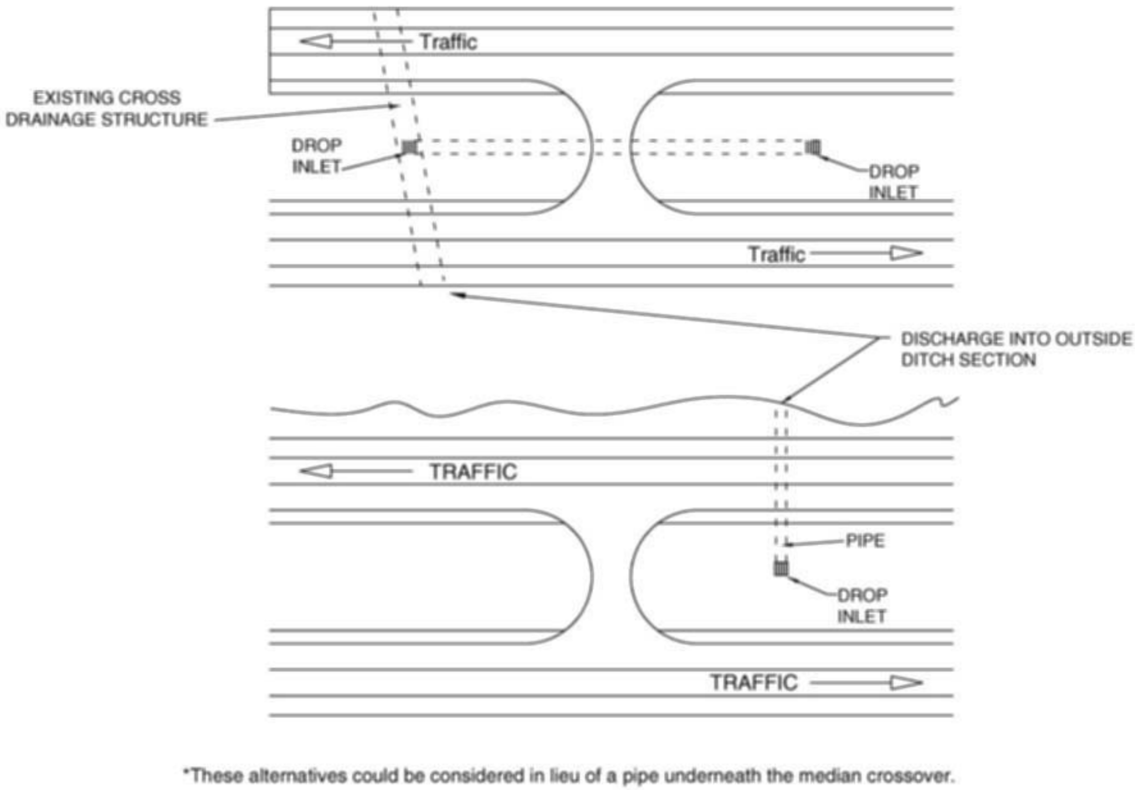


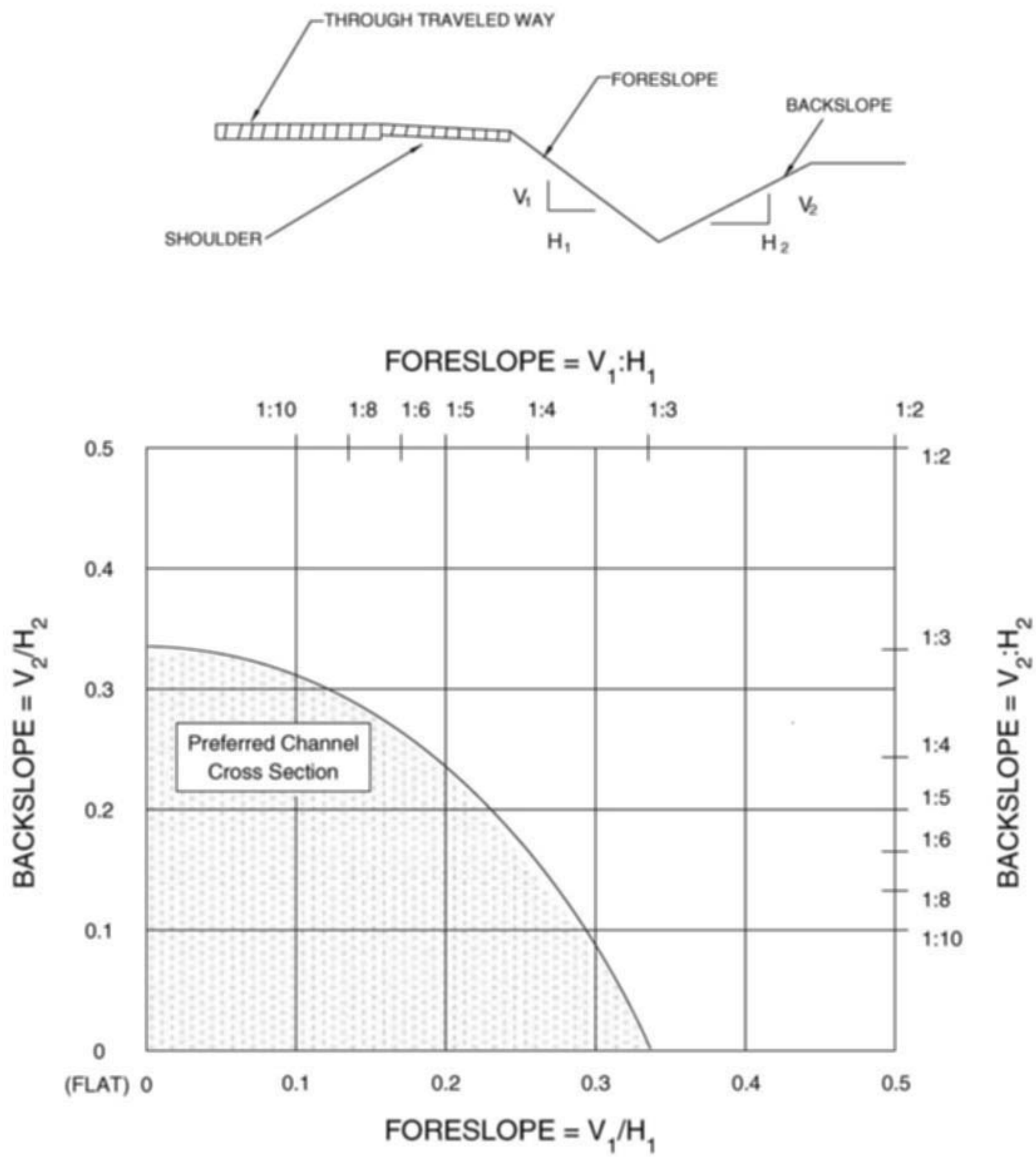
FIGURE 3.5 Examples of alternate median drainage

directly underneath the travel lanes to an outside channel. This eliminates the two pipe ends that would be exposed to traffic in the median. The transverse slopes of the median opening would then be desirably sloped at 1V:10H or flatter.

3.2.4 Drainage Channels

A drainage channel is defined as an open channel usually paralleling the highway embankment and within the limits of the highway right-of-way. The primary function of drainage channels is to collect surface runoff from the highway and areas that drain to the right-of-way and convey the accumulated runoff to acceptable outlet points. Channels must be designed to carry the design runoff and to accommodate excessive storm water with minimal highway flooding or damage. However, channels should also be designed, built, and maintained with consideration given to their effect on the roadside environment. Figures 3.6 and 3.7 present preferred foreslopes and backslopes for basic ditch configurations. Cross sections shown in the shaded region of each of the figures are considered to have traversable cross sections. Channel sections that fall outside the shaded region are considered less desirable and their use should be limited where high-angle encroachments can be expected, such as the outside of relatively sharp curves. Channel sections outside the shaded region may be acceptable for projects having one or more of the following characteristics: restrictive right-of-way; rugged terrain; resurfacing, restoration, or rehabilitation (3R) construction projects; or on low-volume or low-speed roads and streets, particularly if the channel bottom and backslopes are free of any fixed objects.

If practical, drainage channels with cross sections outside the shaded regions and located in vulnerable areas may be reshaped and converted to a closed system (culvert or pipe) or, in some cases, shielded by a traffic barrier. Warrants for the use of roadside barrier to shield nontraversable channels within the clear zone are included in Chapter 5.



*This chart is applicable to all Vee ditches, rounded channels with a bottom width less than 2.4 m [8 ft] and trapezoidal channels with bottom widths less than 1.2 m [4 ft].

FIGURE 3.6 Preferred cross sections for channels with abrupt slope changes

3.3 APPLICATION OF THE CLEAR-ZONE CONCEPT

A basic understanding of the clear-zone concept is critical to its proper application. The numbers obtained from Figure 3.1 or Table 3.1 imply a degree of accuracy that does not exist. Again, the curves are based on limited empirical data that was then extrapolated to provide data for a wide range of conditions. Thus, the numbers obtained from these curves represent a reasonable measure of the degree of safety suggested for a particular roadside, but they are neither absolute nor precise. In some cases, it is reasonable to leave a fixed object within the clear zone; in other instances, an object beyond the clear-zone distance may require removal or shielding. Use of an appropriate clear-zone distance amounts to a compromise between maximizing safety and minimizing construction costs. Appropriate application of the clear-zone concept

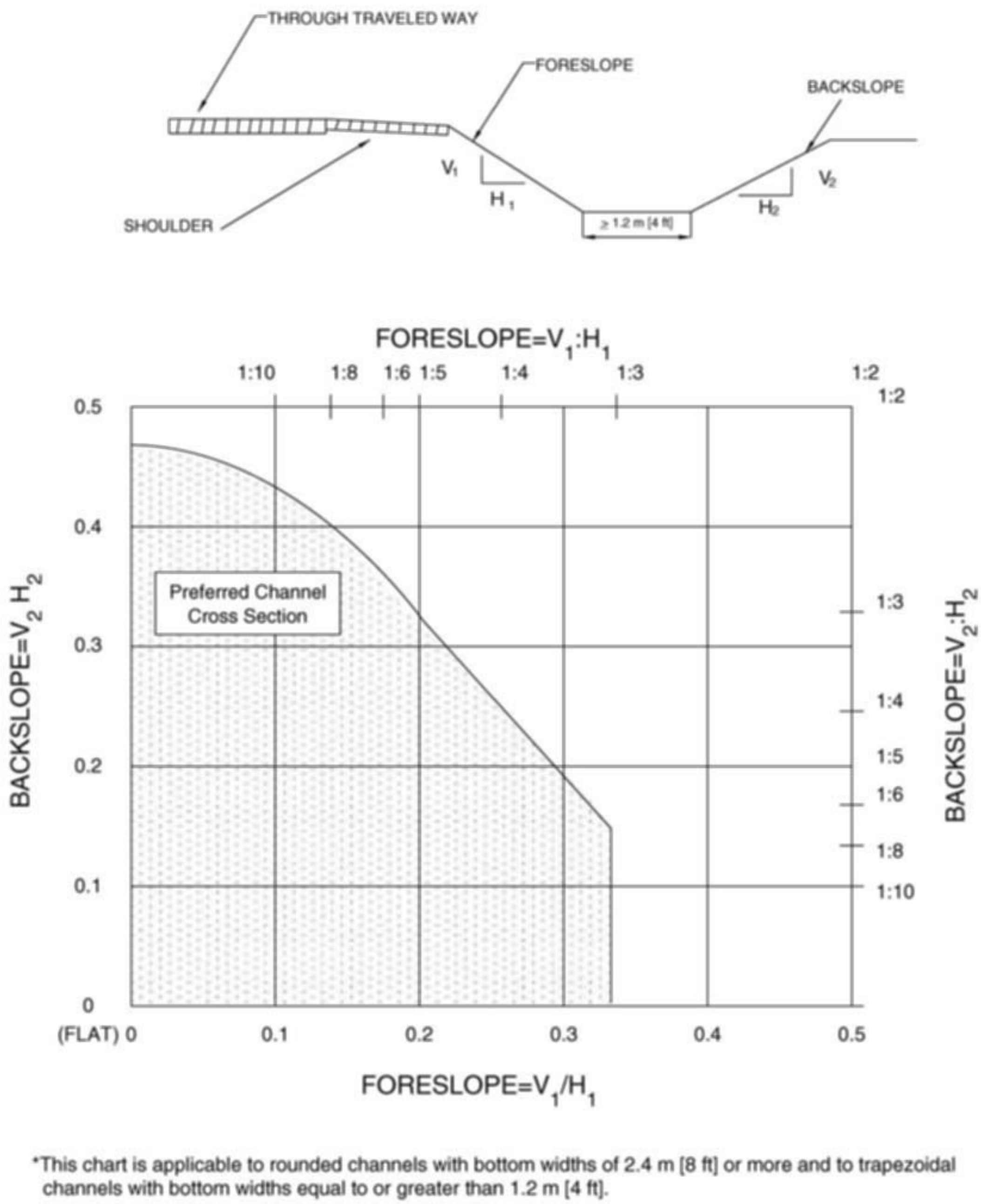


FIGURE 3.7 Preferred cross sections for channels with gradual slope changes

will often result in more than one possible solution. The following sections are intended to illustrate a process that may be used to determine if a fixed object or non-traversable terrain feature warrants relocation, modification, removal, shielding, or no treatment.

The guidelines in this chapter may be most applicable to new construction or major reconstruction. On resurfacing, rehabilitation, or restoration (3R) projects, the primary emphasis is placed on the roadway itself. The actual performance of an existing facility may be measurable through an evaluation of crash records and on-site inspections as part of the design effort or in response to complaints by citizens or officials. Consequently, it may not be cost-effective or practical because of environmental impacts or limited right-of-way to bring a 3R project into full compliance with all of the clear-zone recommendations provided in this guide. Because of the scope of such projects and the limited funding available, emphasis should be placed on correcting or protecting areas within the project that have identifiable safety problems related to clear-zone widths. Bodies of water and escarpments are the types of areas that may be considered for special emphasis.

3.3.1 Recoverable Foreslopes

The clear-zone distance for recoverable foreslopes of 1V:4H or flatter may be obtained directly from Figure 3.1 or Table 3.1. On new construction or major reconstruction, smooth slopes with no significant discontinuities and with no protruding fixed objects are desirable from a safety standpoint. It is desirable to have the top of the slope rounded so an encroaching vehicle remains in contact with the ground. It is also desirable for the toe of the slope to be rounded to make it essentially traversable by an errant vehicle. Designing smooth cross slopes is normally accomplished by using standard or typical cross sections. The flatter the selected slope, the easier it is to mow or otherwise maintain and the safer it becomes to negotiate. The examples at the end of this chapter illustrate the application of the clear-zone concept to recoverable foreslopes.

3.3.2 Non-Recoverable Foreslopes

Foreslopes from 1V:3H up to 1V:4H are considered traversable if they are smooth and free of fixed objects. However, because many vehicles on slopes this steep will continue on to the bottom, a clear runout area beyond the toe of the non-recoverable foreslope is desirable. The extent of this clear runout area could be determined by first finding the available distance between the edge of the through traveled way and the breakpoint of the recoverable foreslope to the non-recoverable foreslope. (See Figure 3.2.) This distance is then subtracted from the recommended clear-zone distance based on the steepest recoverable foreslope before or after the non-recoverable foreslope. The result is the desirable clear runout area that should be provided beyond the non-recoverable foreslope if practical. The clear runout area may be reduced in width based on existing conditions or site investigation. Such a variable sloped typical section is often used as a compromise between roadside safety and economics. By providing a relatively flat recovery area immediately adjacent to the roadway, most errant motorists can recover before reaching the steeper foreslope beyond. The foreslope break may be liberally rounded so that an encroaching vehicle does not become airborne. It is suggested that the steeper slope be made as smooth as practical and rounded at the bottom. Figure 3.2 illustrates a recoverable foreslope followed by a non-recoverable foreslope. Example C demonstrates the method for calculating the desirable runout area.

3.3.3 Critical Foreslopes

Critical foreslopes are those steeper than 1V:3H. They will cause most vehicles to overturn and should be treated if they begin within the clear-zone distance of a particular highway and meet the warrants for shielding contained in Chapter 5. Examples C, D, and E illustrate the application of the clear-zone concept to critical foreslopes.

3.3.4 Examples of Clear-Zone Application on Variable Slopes

A variable foreslope is often specified on new construction to provide a relatively flat recovery area immediately adjacent to the roadway followed by a steeper foreslope. This design requires less right-of-way and embankment material than a continuous, relatively flat foreslope and is commonly called a "barn-roof" section. If an adequate recovery area (as determined from Figure 3.1 or Table 3.1) exists on the flatter foreslope, the steeper slope may be critical or non-traversable. Clear-zone distances for embankments with variable foreslopes ranging from essentially flat to 1V:4H may be averaged to produce a composite clear-zone distance. Slopes that change from a foreslope to a backslope cannot be averaged and should be treated as drainage channel sections and analyzed for traversability using Figure 3.6 or 3.7.

Although a "weighted" average of the foreslopes may be used, a simple average of the clear-zone distances for each foreslope is accurate enough if the variable slopes are approximately the same width. If one foreslope is significantly wider, the clear-zone computation based on that slope alone may be used.

3.3.5 Clear-Zone Applications for Drainage Channels and Backslopes

Drainage channel cross sections that are considered preferable in Figures 3.6 or 3.7 are not obstacles and need not be constructed at or beyond the clear-zone distance for a specific roadway. It is important that roadside hardware not be located in or near ditch bottoms or on the backslope near the drainage channel. Any vehicle leaving the roadway may be funneled along the drainage channel bottom or encroach to some extent on the backslope, thus making an impact more likely. Breakaway hardware may not function as designed if the vehicle is airborne or sliding sideways when contact is made. Non-yielding fixed objects should be located beyond the clear-zone distance for these cross sections as determined from Figure 3.1 or Table 3.1.

3.4 DRAINAGE FEATURES

Effective drainage is one of the most critical elements in the design of a highway or street. However, drainage features should be designed and built with consideration given to their consequences on the roadside environment. In addition to drainage channels, which were addressed in Section 3.2.4, curbs, parallel and transverse pipes and culverts, and drop inlets are common drainage system elements that should be designed, constructed, and maintained with both hydraulic efficiency and roadside safety in mind.

In general, the following options, listed in order of preference, are applicable to all drainage features:

- eliminate non-essential drainage structures;
- design or modify drainage structures so they are traversable or present a minimal obstruction to an errant vehicle;
- if a major drainage feature cannot effectively be redesigned or relocated, shield it using a suitable traffic barrier if it is in a vulnerable location.

The remaining sections of this chapter identify the safety problems associated with curbs, pipes and culverts, and drop inlets, and offer recommendations concerning the location and design of these features to improve their safety characteristics without adversely affecting their hydraulic capabilities. The information presented applies to all roadway types and projects. However, as with many engineering applications, the specific actions taken at a given location often rely heavily on the exercise of good engineering judgment and on a case-by-case assessment of the costs and benefits associated with alternative designs.

3.4.1 Curbs

Curbs are commonly used for drainage control, pavement edge support and delineation, right-of-way reduction, aesthetics, sidewalk separation, and reduction of maintenance operations. Curb designs are classified as vertical or sloping. Vertical curbs are defined as those having a vertical or nearly vertical traffic face 150 mm [6 in.] or higher. These are intended to discourage motorists from deliberately leaving the roadway. Sloping curbs are defined as those having a sloping traffic face 150 mm [6 in.] or less in height. These can be readily traversed by a motorist when necessary, but a designer may prefer a height for sloping curbs of no greater than 100 mm [4 in.] because higher curbs may drag the underside of some vehicles.

In general, curbs are not desirable along high-speed roadways. If a vehicle is spinning or slipping sideways as it leaves the roadway, wheel contact with a curb could cause it to trip and overturn. Under other impact conditions, a vehicle may become airborne, which may result in loss of control by the motorist. The distance over which a vehicle may be airborne and the height above (or below) normal bumper height attained after striking a curb may become critical if secondary crashes occur with traffic barriers or other roadside appurtenances.

If a curb is used in conjunction with a metal beam traffic barrier, it should ideally be located flush with the face of the railing or behind it. Where the curb height is 150 mm [6 in.] or higher, the barrier should be stiffened to reduce its deflection to avoid the potential of a vehicle vaulting the rail. Curbs should not be used in front of sloping faced concrete barriers because such placement may result in unsatisfactory barrier performance. Curb/barrier combinations, particularly for bridge railings, should be crash tested if extensive use of the combination exists or is planned and a similar combination has not been previously tested. Refer to Chapter 5, Section 5.6.2, "Terrain Effects" for additional guidelines about curb usage with traffic barriers. Also see Chapter 4 of *A Policy on Geometric Design of Highways and Streets* (2) for more information on curb configuration and placement. A National Cooperative Highway Research Program project, scheduled for completion in March 2003, is under way to develop design guidelines for the use of curbs and curb-barrier combinations.

When obstructions exist behind curbs, a minimum horizontal clearance of 0.5 m [1.5 ft] should be provided beyond the face of curbs to the obstructions. This offset may be considered the minimum allowable horizontal clearance (or operational offset), but it should not be construed as an acceptable clear zone distance. Since curbs do not have a significant redirectional capability, obstructions behind a curb should be located at or beyond the minimum clear-zone distances shown in Table 3.1. In many instances, it will not be feasible to obtain the recommended clear zone distances on existing facilities. On new construction where minimum recommended clear zones cannot be provided, fixed objects should be located as far from traffic as practical on a project-by-project basis, but in no case closer than 0.5 m [1.5 ft] from the face of the curb.

3.4.2 Cross-Drainage Structures

Cross-drainage structures are designed to carry water underneath the roadway embankment and vary in size from 460 mm [18 in.] for concrete or corrugated metal pipes to 3 m [10 ft] or more for multibarreled concrete box culverts or structural plate pipes. Typically, their inlets and outlets consist of concrete headwalls and wingwalls for the larger structures and beveled-end sections for the smaller pipes. While these types of designs are hydraulically efficient and minimize erosion problems, they may represent an obstacle to motorists who run off the road. This type of design may result in either a fixed object protruding above an otherwise traversable embankment or an opening into which a vehicle can drop, causing an abrupt stop. The options available to a designer to minimize these obstacles are:

- use a traversable design;
- extend the structure so that it is less likely to be hit;
- shield the structure;
- delineate the structure if the above alternatives are not appropriate.

Each of these options is discussed in the following subsections.

3.4.2.1 Traversable Designs

A roadside designed with optimal safety features could be defined as one that is almost flat, is completely traversable from the edge of the through traveled way to the right-of-way line, and would include sufficient area for all desirable clear-zone distance requirements. Such a facility would resemble a landing strip or runway at an airport. Thus, it is readily apparent from the start that roadside design must be a series of compromises between "absolute" safety and engineering, environmental, and economic constraints. The designer should strive for embankments as smooth or traversable as practical for a given facility. As indicated in Sections 3.1 and 3.2, traversable, nonrecoverable foreslopes may be rounded at top and bottom and may provide a relatively flat runout area at the bottom.

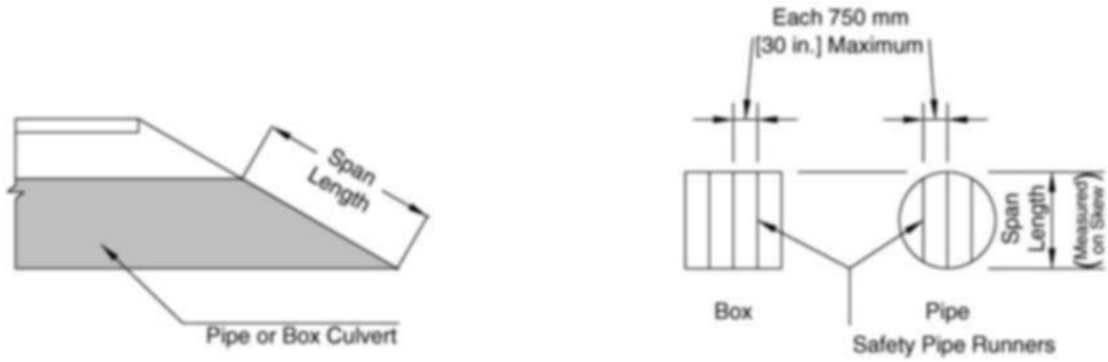
If a foreslope is traversable, the preferred treatment for any cross-drainage structure is to extend (or shorten) it to intercept the roadway embankment and to match the inlet or outlet slope to the foreslope. For small culverts, no other treatment is required. For cross-drainage structures, a small pipe culvert is defined as a single round pipe with a 900 mm [36 in.] or less diameter or multiple round pipes each with a 750 mm [30 in.] or less diameter. Extending culverts to locate the inlets/outlets a fixed distance from the through traveled way is not recommended if such treatment introduces discontinuities in an otherwise traversable slope. Extending the pipe results in warping the foreslopes in or out to match the opening, which produces a significantly longer area that affects the driver who has run off the road. Matching the inlet to the foreslope is desirable because it results in an extremely small "target" to hit, reduces erosion problems, and simplifies mowing operations.

Single structures and end treatments wider than 1 m [3 ft] can be made traversable for passenger size vehicles by using bar grates or pipes to reduce the clear opening width. Modifications to the culvert ends to make them traversable should not significantly decrease the hydraulic capacity of the culvert. Safety treatments must be hydraulically efficient. In order to maintain hydraulic efficiency, it may be necessary to apply bar grates to flared wingwalls, flared end sections, or to culvert extensions that are larger in size than the main barrel. The designer should consider shielding the structure if significant hydraulic capacity or clogging problems could result.

Full-scale crash tests have shown that automobiles can cross grated-culvert end sections on slopes as steep as 1V:3H at speeds as low as 30 km/h [20 mph] and as high as 100 km/h [60 mph], when steel pipes spaced on 750 mm [30 in.] centers are used for these cross-drainage structures. This spacing does not significantly change the flow capacity of a pipe unless debris accumulates and causes partial clogging of the inlet. This underscores the importance of accurately assessing the clogging potential of a structure during design and the importance of keeping the inlets free of debris. Figure 3.8 shows recommended sizes to support a full-sized automobile, and is based on a 750 mm [30 in.] bar spacing. It is important to note that the toe of the foreslope and the ditch or stream bed area immediately adjacent to the culvert must be more or less traversable if the use of a grate is to have any significant safety benefit. Normally, grading within the right-of-way limits can produce a satisfactory runout path.

For median drainage where flood debris is not a concern and where mowing operations are frequently required, much smaller openings between bars may be tolerated and grates similar to those commonly used for drop inlets may be appropriate. It should also be noted that both the hydraulic efficiency and the roadside environment may be improved by making the culverts continuous and adding a median drainage inlet. This alternative eliminates two end treatments and is usually a practical design when neither median width nor height of fill are excessive. Figure 3.9 shows a traversable pipe grate on a concrete box culvert constructed to match the 1V:6H side slope.

SPAN LENGTH		INSIDE DIAMETER	
up to 3.65 m	[12 ft]	75 mm	[3 in.]
3.65–4.90 m	[12–16 ft]	87 mm	[3.5 in.]
4.90–6.10 m	[16–20 ft]	100 mm	[4 in.]
6.10 m [20 ft] or less with center support		75 mm	[3 in.]



*The chart above shows recommended safety pipe runner sizes for various span lengths for cross-drainage structures. The safety pipe runners are Schedule 40 pipes spaced on centers of 750 mm [30 in.] or less.

FIGURE 3.8 Design criteria for safety treatment of pipes and culverts

FIGURE 3.9 Safety treatment for cross-drainage culvert

3.4.2.2 Extension of Structure

For intermediate sized pipes and culverts whose inlets and outlets cannot readily be made traversable, an option often exercised by the designer is to extend the structure so the obstacle is located at or just beyond the appropriate clear zone. While this practice reduces the likelihood of the pipe end being hit, it does not completely eliminate the possibility. As noted in Section 3.1, the clear-zone distance should not be viewed as a discrete, exact distance but as the center of a zone, which should then be analyzed on a site-specific basis.

If the extended culvert headwall remains the only significant man-made fixed object immediately at the edge of the clear zone along the section of roadway under design, and the roadside is generally traversable to the right-of-way line elsewhere, simply extending the culvert to the edge of the clear zone may not be the best alternative, particularly on freeways and other high-speed, access-controlled facilities. On the other hand, if the roadway has numerous fixed objects, both natural and man-made, at the edge of the clear zone, extending individual structures to the same minimum distance from traffic may be appropriate. However, redesigning the inlet/outlet so it is no longer an obstacle is usually the preferred safety treatment.

3.4.2.3 Shielding

For major drainage structures that are costly to extend and whose end sections cannot be made traversable, shielding with an appropriate traffic barrier is often the most effective safety treatment. Although the traffic barrier is longer and closer to the roadway than the structure opening and is likely to be hit more often than an unshielded culvert located farther from the through traveled way, a properly designed, installed, and maintained barrier system may provide an increased level of safety for the errant motorist.

3.4.3 Parallel Drainage Features

Parallel drainage culverts are those that are oriented parallel to the main flow of traffic. They are typically used at transverse slopes under driveways, field entrances, access ramps, intersecting side roads, and median crossovers. Most such culverts are designed to carry relatively small flows until the water can be discharged into outfall channels or other drainage facilities and carried away from the roadbed. However, these drainage features can present a significant roadside obstacle because they can be struck head-on by impacting vehicles. As with cross-drainage structures, the designer's primary concern should be to design generally traversable slopes and to match the culvert openings with adjacent slopes. Section 3.2.3 recommends that transverse slopes that can be struck at 90 degrees by run-off-the-road vehicles be constructed as flat as practical, with 1V:6H or flatter suggested for locations susceptible to high-speed impacts. On low-volume or low-speed roads, where crash history does not indicate a high number of run-off-the-road occurrences, steeper transverse slopes may be considered as a cost-effective approach. Using these guidelines, safety treatment options are similar to those for cross-drainage structures, in order of preference:

- eliminate the structure
- use a traversable design
- move the structure laterally to a less vulnerable location
- shield the structure
- delineate the structure if the above alternatives are not appropriate

3.4.3.1 Eliminate the Structure

Unlike cross-drainage pipes and culverts which are essential for proper drainage and operation of a road or street, parallel pipes can sometimes be eliminated by constructing an overflow section on the field entrance, driveway, or intersecting side road. To ensure proper performance, care should be taken when allowing drainage to flow over highway access points, particularly if several access points are closely spaced or the water is subject to freezing. This treatment will usually be appropriate only at low-volume locations where this design does not decrease the sight distance available to drivers entering the main road. Care must also be exercised to avoid erosion of the entrance and the area downstream of the crossing. This can usually be accomplished by paving the overflow section (assuming the rest of the facility is not paved) and by adding an upstream and downstream apron at locations where water velocities and soil conditions make erosion likely.

Closely spaced driveways with culverts in drainage channels are relatively common as development occurs along highways approaching urban areas. Since traffic speeds and roadway design elements are usually characteristic of rural highways, these culverts may constitute a significant roadside obstacle. In some locations, such as along the outside of curves or where records indicate concentrations of run-off-the-road crashes, it may be desirable to convert the open channel into a storm drain and

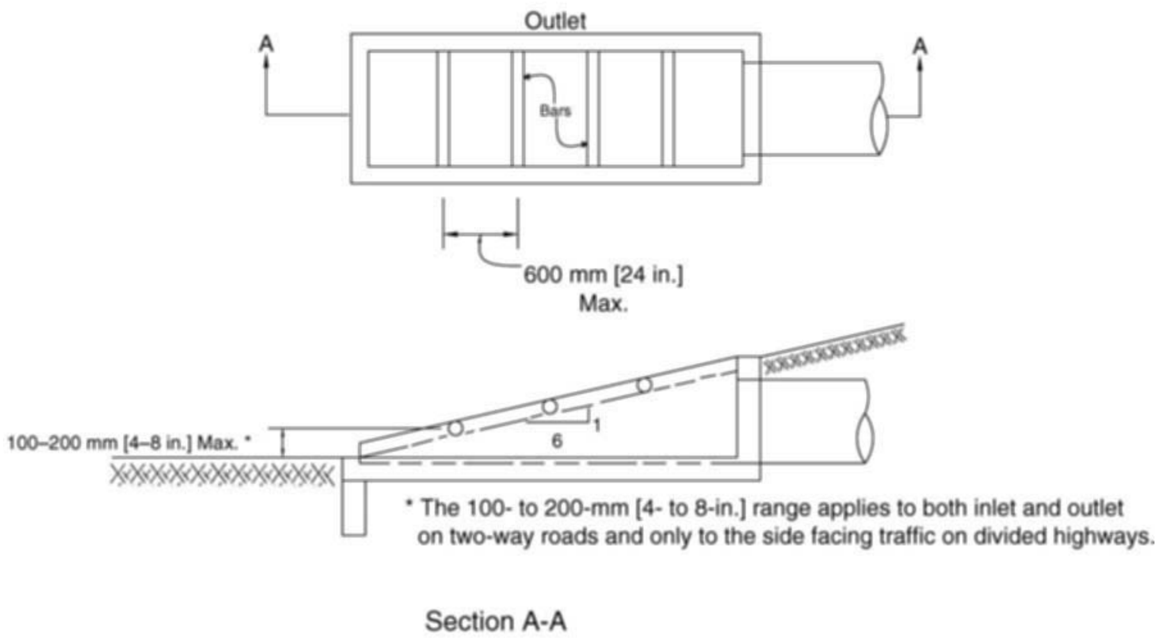


FIGURE 3.10 Inlet/outlet design example for parallel drainage

backfill the areas between adjacent driveways. This treatment will eliminate the ditch section as well as the transverse slopes with pipe inlets and outlets.

3.4.3.2 Traversable Designs

As emphasized earlier in this chapter, transverse slopes should be designed with consideration given to their effect on the roadside environment. The designer should try to provide the flattest transverse slopes practical in each situation, particularly in areas where the slope has shown a high probability of being struck head-on by a vehicle. Once this has been done, parallel drainage structures should match the selected transverse slopes and should be safety treated if possible when they are located in a vulnerable position relative to main road traffic. While many of these structures are small and present a minimal target, the addition of pipes and bars perpendicular to traffic can reduce wheel snagging in the culvert opening. Research has shown that, for parallel drainage structures, a grate consisting of pipes set on 600 mm [24 in.] centers will significantly reduce wheel snagging. It is recommended that the center of the bottom bar or pipe be set at 100 to 200 mm [4 to 8 in.] above the culvert invert.

Generally, single pipes with diameters of 600 mm [24 in.] or less will not require a grate. However, when a multiple pipe installation is involved, consideration of a grate for smaller pipes may be appropriate. Reference may be made to the Texas Transportation Institute Research Study 2-8-79-280, *Safe End Treatment for Roadside Culverts* (4), in which researchers concluded that a passenger vehicle should be able to traverse a pipe/slope combination at speeds up to 80 km/h [50 mph] without rollover. To achieve this result, both the roadway (or ditch) foreslope and the driveway foreslope should be 1V:6H or flatter and have a smooth transition between them. Ideally, the culvert should be cut to match the driveway slope and fitted with cross members perpendicular to the direction of traffic flow as described above. This study suggests that it could be cost-effective to flatten the approach slopes to 1V:6H and match the pipe openings to these slopes for all sizes of pipes up to 900 mm [36 in.] in diameter for traffic volumes above 100 vehicles per day. The addition of grated inlets to these pipes was considered cost-effective for pipes 900 mm [36 in.] or greater for ADTs over 500, and for pipes over 600 mm [24 in.] in diameter for traffic volumes over 13,000 vehicles per day. Because these numbers were based in part on assumptions by the researchers, they should be interpreted as approximations and not as absolute numbers. Figure 3.10 illustrates a possible design for the inlet and outlet end of a parallel culvert. When channel grades permit, the inlet end may use a drop-inlet type design to reduce the length of grate required.

The recommended grate design may affect culvert capacity if significant blockage by debris is likely; however, because capacity is not normally the governing design criteria for parallel structures, hydraulic efficiency may

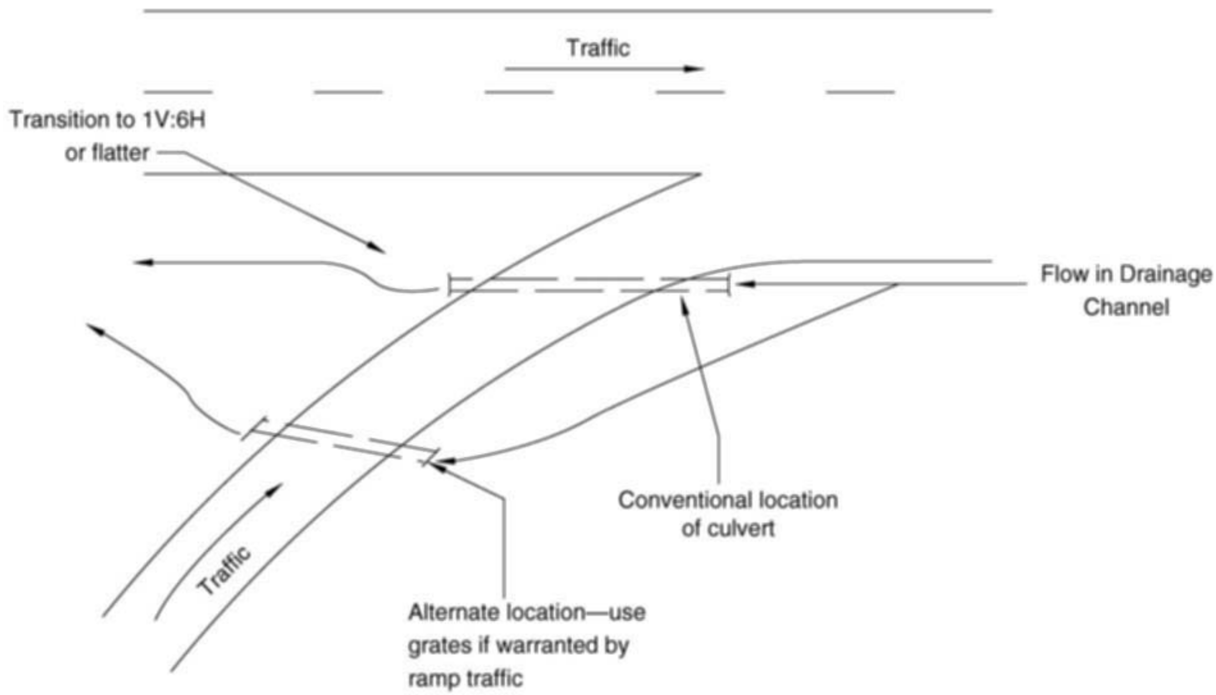


FIGURE 3.11 Alternate location for a parallel drainage culvert

not be an overriding concern. A report issued by the University of Kansas suggested that a 25 percent debris blockage factor should be sufficiently conservative for use as a basis for culvert design in these cases (5). This report also suggests that under some flow conditions, the capacity of a grated culvert may be equal to that of a standard headwall design as a result of decreased entrance turbulence. In those locations where headwater depth is critical, a larger pipe should be used or the parallel drainage structure may be positioned outside the clear zone as discussed in the following section.

3.4.3.3 Relocate the Structure

Some parallel drainage structures can be moved laterally farther from the through traveled way. This treatment often affords the designer the opportunity to flatten the transverse slope within the selected clear-zone distance of the roadway under design. If the embankment at the new culvert locations is traversable and likely to be encroached upon by either main road or side road traffic, safety treatment should be considered. It is suggested that the inlet or outlet match the transverse slope regardless of whether or not additional safety treatment is deemed necessary. A suggested design treatment is shown in Figure 3.11. Figure 3.12 shows a recommended safety treatment for parallel drainage pipes.

3.4.3.4 Shielding

In cases where the transverse slope cannot be made traversable, the structure is too large to be safety treated effectively, and relocation is not feasible, it may be necessary to shield the obstacle with a traffic barrier. Specific information on the selection, location, and design of an appropriate barrier system is contained in Chapter 5.

3.4.4 Drop Inlets

Drop inlets can be classified as on-roadway or off-roadway structures. On-roadway inlets are usually located on or alongside the shoulder of a street or highway and are designed to intercept runoff from the road surface. These include curb opening inlets, grated inlets, slotted drain inlets, or combinations of these three basic designs. Since they are installed flush with the pavement surface, they do not constitute a significant safety problem to errant motorists. However, they must be selected and sized to accommodate design water runoff. In addition, they must be capable of supporting vehicle wheel loads and present no obstacle to pedestrians or bicyclists.

Off-roadway drop inlets are used in medians of divided roadways and sometimes in roadside ditches. While their purpose is to collect runoff, they should be designed and located to present a minimal obstacle to errant motorists.

FIGURE 3.12 Safety treatment for parallel drainage pipe

This can be accomplished by building these features flush with the channel bottom or slope on which they are located. No portion of the drop inlet should project more than 100 mm [4 in.] above the ground line. The opening should be treated to prevent a vehicle wheel from dropping into it; but unless pedestrians are a consideration, grates with openings as small as those used for pavement drainage are not necessary. Neither is it necessary to design for a smooth ride over the inlet. It is sufficient to prevent wheel snagging and the resultant sudden deceleration or loss of control associated with it.

REFERENCES

1. AASHO. *Highway Design and Operational Practices Related to Highway Design.* American Association of State Highway Officials, Washington, DC, 1967.

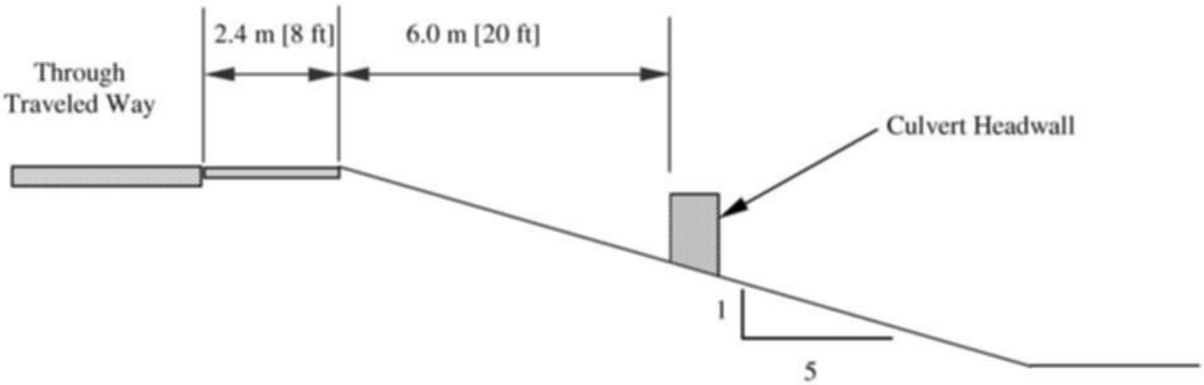
2. AASHTO. *Guide for Selecting, Locating and Designing Traffic Barriers.* American Association of State Highway and Transportation Officials, Washington, DC, 1977.

3. AASHTO. *A Policy on Geometric Design of Highways and Streets.* American Association of State Highway and Transportation Officials, Washington, DC, 2001.

4. Texas Transportation Institute. *Research Study 2-8-79-280: Safe End Treatments for Roadside Culverts.*

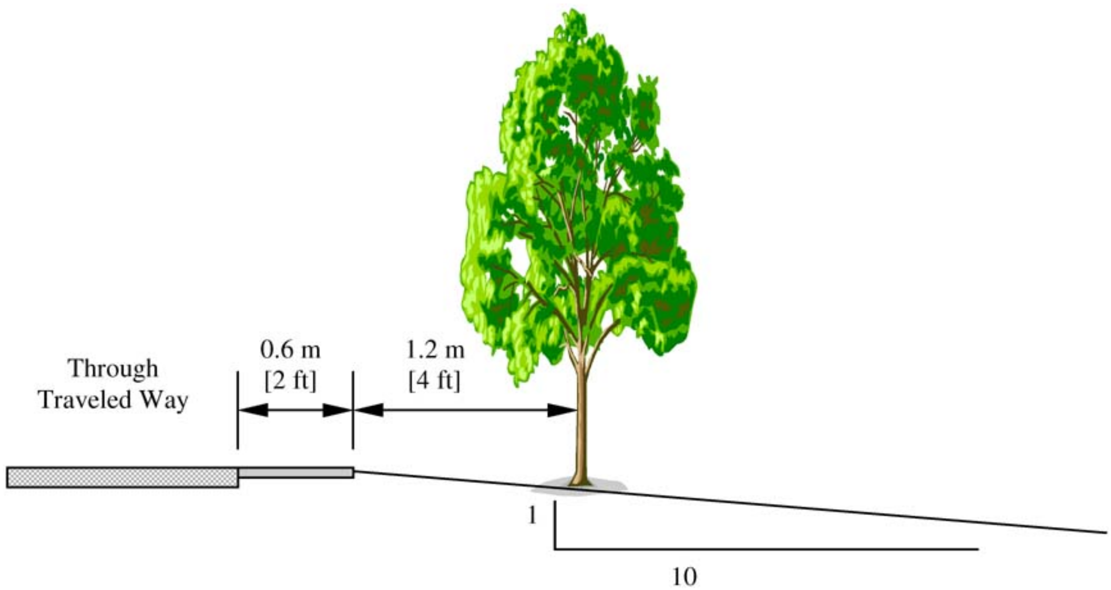
5. University of Kansas. *Development of Hydraulic Design Charts for Type IV End Sections for Pipe Culverts.* KU-93-5. 1993.

EXAMPLE A
Design ADT: 4000
Design Speed: 100 km/h [60 mph]
Recommended clear-zone distance for 1V:5H foreslope: 10 m to 12 m [32 ft to 40 ft] (from Table 3.1)



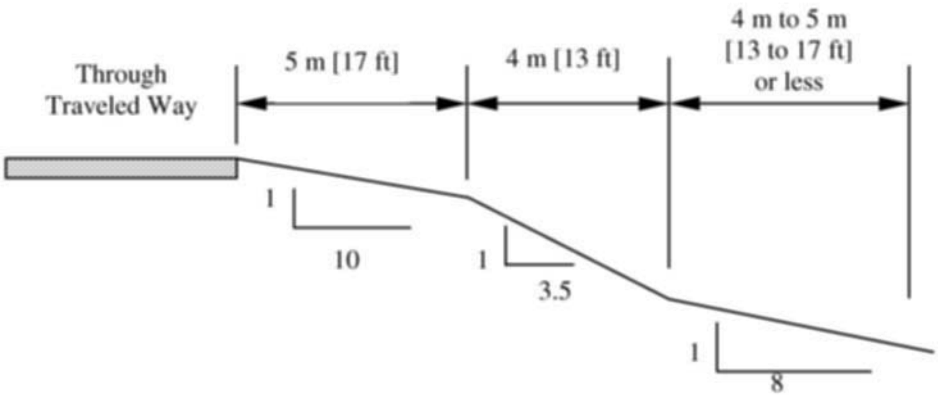
Discussion: The available recovery area of 8.4 m [28 ft] is less than the recommended 10 m to 12 m [32 ft to 40 ft]. If the culvert headwall is greater than 100 mm [4 in.] in height and is the only obstruction on an otherwise traversable foreslope, it should be removed and the inlet modified to match the 1V:5H foreslope. If the foreslope contains rough outcroppings or boulders and the headwall does not significantly increase the obstruction to a motorist, the decision to do nothing may be appropriate. A review of the highway's crash history, if available, may be made to determine the nature and extent of vehicle encroachments and to identify any specific locations that may require special treatment.

EXAMPLE B
Design ADT: 300
Design Speed: 60 km/h [40 mph]
Recommended clear-zone distance for 1V:10H slope: 2 m to 3 m [7 ft to 10 ft] (from Table 3.1)



Discussion: The available clear-zone distance is 1.8 m [6 ft], 0.2 m to 1.2 m [1 ft to 4 ft] less than the recommended recovery area. When an area has a significant number of run-off-the-road crashes, it may be appropriate to consider shielding or removing the entire row of trees within the crash area. If this section of road has no significant history of crashes and is heavily forested with most of the other trees only slightly farther from the road, this tree would probably not require treatment. If, however, none of the other trees are closer to the roadway than, for example, 4.5 m [15 ft], this individual tree represents a more significant obstruction and should be considered for removal. If a tree were 4.5 m [15 ft] from the edge of through traveled way, and all or most of the other trees were 7.5 m [25 ft] or more, its removal may still be appropriate. This example emphasizes that the clear-zone distance is an approximate number at best and that individual objects should be analyzed in relation to other nearby obstacles.

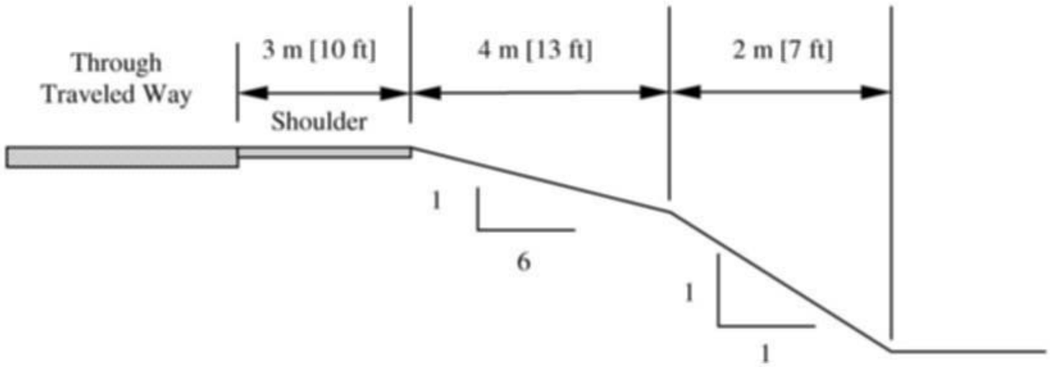
EXAMPLE C
Design ADT: 7000
Design Speed: 100 km/h [60 mph]
Recommended clear-zone distance for 1V:10H foreslope: 9 to 10 m [30 to 32 ft] (from Table 3.1)
Recommended clear-zone distance for 1V:8H foreslope: 9 to 10 m [30 to 32 ft] (from Table 3.1)
Available recovery distance before breakpoint of non-recoverable foreslope: 5 m [17 ft]
Clear runout area at toe of foreslope: 9 to 10 m [30 to 32 ft] minus 5 m [17 ft] or 4 to 5 m [13 to 17 ft]



Discussion: Since the non-recoverable foreslope is within the required clear-zone distance of the 1V:10H foreslope, a runout area beyond the toe of the non-recoverable foreslope is desirable. Using the steepest recoverable foreslope before or after the non-recoverable foreslope, a clear-zone distance is selected from Figure 3.1 or Table 3.1. In this example, the 1V:8H foreslope beyond the base of the fill dictates a 9 to 10 m [30 to 32 ft] clear-zone distance. Since 5 m [17 ft] are available at the top, an additional 4 to 5 m [13 to 17 ft] could be provided at the bottom. All foreslope breaks may be rounded and no fixed objects would normally be built within the upper or lower portions of the clear-zone or on the intervening foreslope.

It may be practical to provide less than the entire 4 to 5 m [13 to 17 ft] at the toe of the non-recoverable foreslope. A smaller recovery area could be applicable based on the rounded foreslope breaks, the flatter 1V:10H foreslope at the top, or past crash histories. A specific site investigation may be appropriate in determining an approximate runout area beyond the toe of the non-recoverable foreslope.

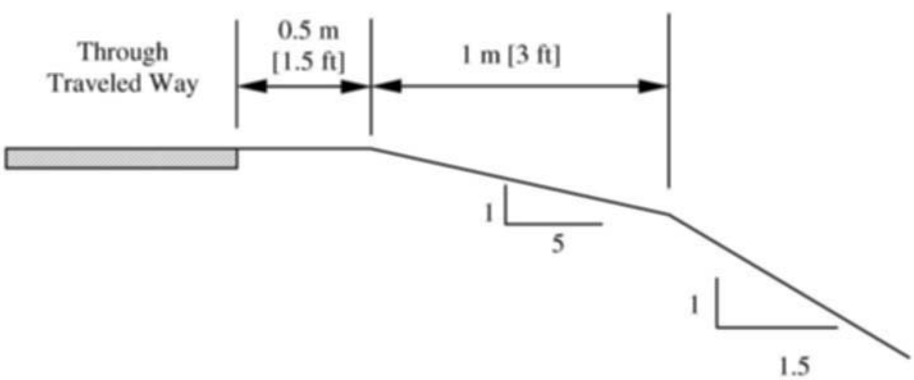
EXAMPLE D
Design ADT: 12,000
Design Speed: 110 km/h [70 mph]
Recommended clear-zone distance for 1V:6H foreslope: 9 to 10.5 m [30 to 34 ft] (from Table 3.1)



Discussion: Since the critical foreslope is only 7 m [23 ft] from the through traveled way, instead of the suggested 9 to 10.5 m [30 to 34 ft], it should be flattened if practical or considered for shielding. However, if this is an isolated obstacle and the roadway has no significant crash history, it may be appropriate to do little more than delineate the drop off in lieu of foreslope flattening or shielding.

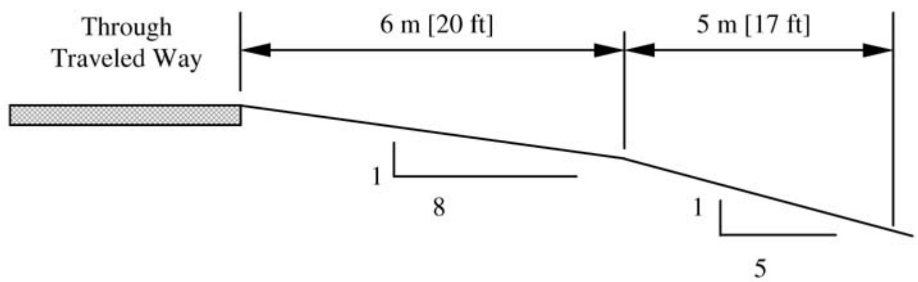
Although a "weighted" average of the foreslopes may be used, a simple average of the clear-zone distances for each foreslope is accurate enough if the variable foreslopes are approximately the same width. If one foreslope is significantly wider, the clear-zone computation based on that foreslope alone may be used.

EXAMPLE E
Design ADT: 350
Design Speed: 60 km/h [40 mph]
Recommended clear-zone distance for 1V:5H foreslope: 2 m to 3 m [7 ft to 10 ft] (from Table 3.1)



Discussion: The available 1.5 m [4.5 ft] is 0.5 m to 1.5 m [2.5 ft to 5.5 ft] less than the recommended recovery area. If much of this roadway has a similar cross section and no significant run-off-the-road crash history, neither foreslope flattening nor a traffic barrier would be recommended. On the other hand, even if the 1V:5H foreslope were 3 m [10 ft] wide and the clear-zone requirement were met, a traffic barrier might be appropriate if this location has noticeably less recovery area than the rest of the roadway and the embankment was unusually high.

EXAMPLE F
Design ADT: 5000
Design Speed: 100 km/h [60 mph]
Recommended clear-zone distance for 1V:8H foreslope: 8 m to 9 m [26 ft to 30 ft] (from Table 3.1)
Recommended clear-zone distance for 1V:5H foreslope: 10 m to 12 m [32 ft to 40 ft] (from Table 3.1)

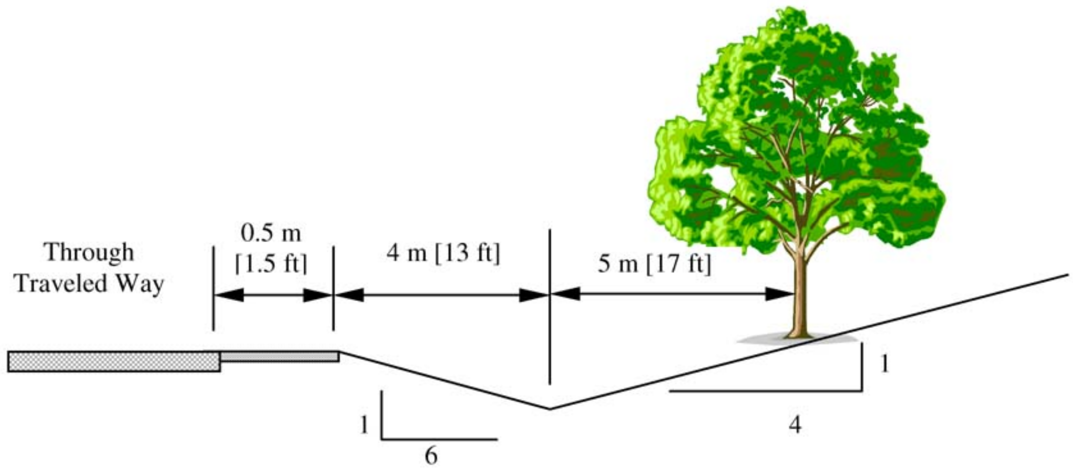


Discussion: The 1V:8H foreslope and the 1V:5H foreslope may be averaged taking into account the distance available on each foreslope. The distance (6 m [20 ft]) along the 1V:8H slope is multiplied by the slope of 1/8 (V/H). The distance (5 m [17 ft]) along the 1V:5H foreslope is multiplied by the slope of 1/5. The resulting distances are added together and divided into the sum of the two distances (6 m [20 ft] plus 5 m [17 ft]) available. The result is an "average" foreslope which may be used in Figure 3.1 or Table 3.1. For sections flatter than or equal to 1V:10H, a slope of 1V:10H is used. Decimal results of 0.5 or greater may be rounded up to the next even numbered slope while decimal results less than 0.5 may be rounded down to the next even numbered slope. The calculations are given below:

1. 6 m [20 ft] x (1/8) + 5 m [17 ft] x (1/5) = 1.75 m [5.9 ft]
2. 6 m [20 ft] + 5 m [17 ft] = 11 m [37 ft]
3. 11 m [37 ft]/1.75 m [5.9 ft] = 6.3 rounded to 6
4. Enter Table 3.1 for the 1V:6H or flatter slopes
5. The clear-zone distance from Table 3.1 is 8 m to 9 m [26 ft to 30 ft] for the given speed and traffic volume. Since the example has 11 m [37 ft] available on the two foreslopes, it is acceptable without further treatment.

In this example, it would be desirable to have no fixed objects constructed on any part of the 1V:5H foreslope. Natural obstacles such as trees or boulders at the toe of the slope would not be shielded or removed. However, if the final foreslope were steeper than 1V:4H, a clear runout area should be considered at the toe of the foreslope. The designer may choose to limit the clear-zone distance to 9 m [30 ft] if that distance is consistent with the rest of the roadway template, a crash analysis or site investigation does not indicate a potential run-off-the-road problem in this area, and the distance selected does not end at the toe of the non-recoverable foreslope.

EXAMPLE G
Design ADT: 1400
Design Speed: 100 km/h [60 mph]
Recommended clear-zone distance for 1V:6H foreslope (fill): 6 to 7.5 m [20 to 24 ft] (from Table 3.1)
Recommended clear-zone distance for 1V:4H backslope (cut): 5 to 5.5 m [16 to 18 ft] (from Table 3.1)



Discussion: For channels within the preferred cross-section area of Figures 3.6 or 3.7, the clear zone may be determined from Figure 3.1 or Table 3.1. However, when the recommended clear zone exceeds the available clear zone for the foreslope, an adjusted clear zone may be determined as follows:

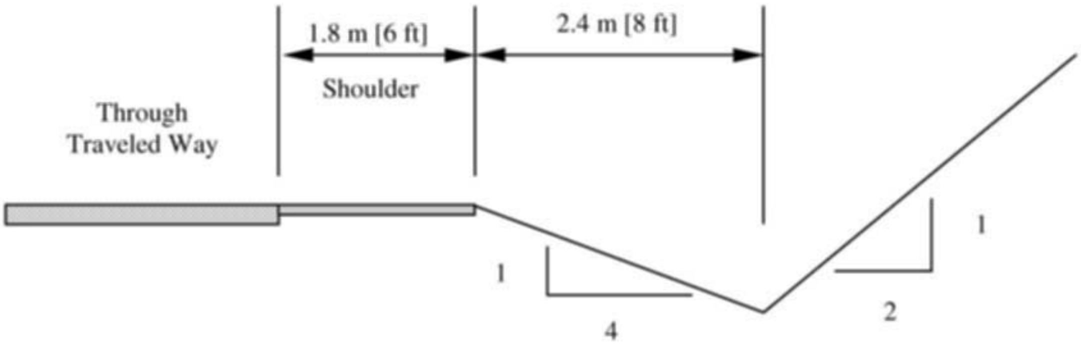
1. Calculate the percentage of the recommended clear-zone range available from the edge of through traveled way to the PVI of the foreslope (4.5 m /6 x 100 = 75% and 4.5 m / 7.5 x 100 = 60% [14.5 ft / 20 ft x 100 = 73% and 14.5 ft / 24 ft x 100 = 60 %]).
2. Subtract these percentages from 100% and multiply results by the recommended range of clear zones for the backslope (100% – 75%) x 5 m = 1.25 m , and (100% – 60%) x 5.5 m = 2.2 m [(100% – 73%) x 16 ft = 4.3 ft and (100% – 60%) x 18 ft = 7.2 ft]. The range of required clear zone on the backslope is 1.25 m to 2.2 m [4.3 to 7.2 ft].
3. Add the available clear zone on the foreslope to the range of values determined in Step 2 (4.5 m + 1.25 m = 5.75 m and 4.5 m + 2.2 m = 6.7 m [14.5 ft + 4.3 ft = 18.8 ft and 14.5 ft + 7.2 ft = 21.7 ft]). The adjusted clear-zone range is 5.75 to 6.7 meters [18.8 to 21.7 ft].

Because the tree is located beyond the adjusted clear zone, removal is not required. Removal should be considered if this one obstacle is the only fixed object this close to the through traveled way along a significant length.

To determine the recommended clear zone for the foreslope in the trapezoidal channel, an average foreslope must be calculated. See Example F for the method of foreslope averaging.

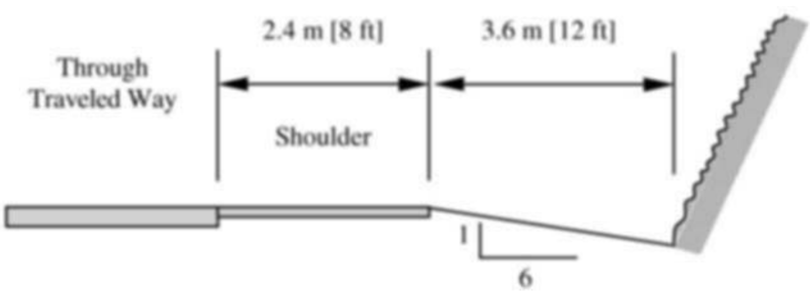
Drainage channels not having the preferred cross section (see Figure 3.6 or 3.7) should be located at or beyond the clear zone. However, backslopes steeper than 1V:3H are typically located closer to the roadway. If these slopes are relatively smooth and unobstructed, they present little safety problem to an errant motorist. If the backslope consists of a rough rock cut or outcropping, shielding may be warranted as discussed in Chapter 5.

EXAMPLE H
Design ADT: 800
Design Speed: 80 km/h [50 mph]
Recommended clear-zone distance for 1V:4H foreslope: 5 m to 6 m [16 ft to 20 ft] (from Table 3.1)



Discussion: The ditch is not within the preferred cross section area of Figure 3.6 and is 0.6 m to 1.8 m [2 ft to 6 ft] less than the recommended clear-zone distance. However, if the ditch bottom and backslope are free of obstacles, no additional improvement is suggested. A similar cross section on the outside of a curve where encroachments are more likely and the angle of impact is sharper would probably be flattened if practical.

EXAMPLE I
Design ADT: 3000
Design Speed: 100 km/h [60 mph]
Recommended clear-zone distance for 1V:6H foreslope: 8.0 to 9.0 m [26 to 30 ft] (from Table 3.1)



Discussion: The rock cut is within the given clear-zone distance but would probably not warrant removal or shielding unless the potential for snagging, pocketing, or overturning a vehicle is high. Steep backslopes are clearly visible to motorists during the day, thus lessening the risk of encroachments. Roadside delineation of sharper than average curves through cut sections can be an effective countermeasure at locations having a significant crash history or potential.

Chapter 4
Sign, Signal, and Luminaire Supports, Utility Poles, Trees, and Similar Roadside Features

4.0 OVERVIEW

Although a traversable and unobstructed roadside is highly desirable from a safety standpoint, some appurtenances simply must be placed near the traveled way. Man-made fixed objects that frequently occupy highway rights-of-way include highway signs, roadway lighting, traffic signals, railroad warning devices, motorist-aid callboxes, mailboxes, and utility poles. Approximately 15 percent of all fixed-object fatalities each year involve crashes with sign and lighting supports and utility poles. Although of a lesser order of magnitude, collisions with other roadside hardware are frequently severe as well. Finally, it must be recognized that approximately 3,000 motorists a year are killed as a result of crashes with trees and other vegetation.

This chapter is not intended to provide technical design details. Virtually all highway agencies use standard drawings for their roadside device installations and it is assumed that these drawings will comply with the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* (1). Similarly, information on existing operational hardware is included only to the extent necessary to familiarize the designer with the types of breakaway devices available and how each is intended to function.

The highway designer is charged with providing the safest facility practicable within given constraints. As noted in Chapter 1, there are 6 options from which to choose a safe design. In order of preference, these are:

1. Remove the obstacle.
2. Redesign the obstacle so it can be traversed safely.
3. Relocate the obstacle to a point where it is less likely to be struck.
4. Reduce impact severity by using an appropriate breakaway device.
5. Shield the obstacle with a longitudinal traffic barrier or crash cushion or both if it cannot be eliminated, relocated, or redesigned.
6. Delineate the obstacle if the above alternatives are not appropriate.

While options 1 and 2 above are the preferred choices, these solutions are not always practical, especially for highway signing and lighting which must remain near the roadway to serve their intended functions. This chapter deals primarily with option 4, the use of breakaway hardware, which has become a cornerstone of the forgiving roadside concept since its inception in the mid-1960s. Emphasis is placed on the selection of the most appropriate device to use in a given location and on installing the support to ensure acceptable performance when it is hit. The final section of this chapter addresses the problems associated with trees and shrubs and provides the designer with some guidelines to follow on this frequently sensitive topic.

4.1 ACCEPTANCE CRITERIA FOR BREAKAWAY SUPPORTS

The term "breakaway support" refers to all types of sign, luminaire, and traffic signal supports that are designed to yield when impacted by a vehicle. The release mechanism may be a slip plane, plastic hinge, fracture element, or a combination of these. The criteria used to determine if a support is considered breakaway are found in Reference 1 and NCHRP Report 350, *Recommended Procedures for the Safety Performance Evaluation of Highway Features* (2). Breakaway support hardware previously found acceptable under the requirements of either the 1985 or 1994 editions of the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* (1) are acceptable under NCHRP Report 350 guidelines. The Federal Highway Administration maintains lists of acceptable, crashworthy supports.

These criteria require that a breakaway support fail in a predictable manner when struck head-on by an 820 kg [1,800 lb] vehicle, or its equivalent, at speeds of 35 km/h [20 mph] and 100 km/h [60 mph]. It is desirable to limit the longitudinal component of the occupant impact velocity to 3.0 m/s [10 ft/s]; but values as high as 5.0 m/s [16 ft/s] are considered acceptable. These specifications also establish a maximum stub height of 100 mm [4 in.] to lessen the possibility of snagging the undercarriage of a vehicle after a support has broken away from its base.

In addition to the change in velocity criterion, satisfactory breakaway support performance depends on the crash vehicle remaining upright during and after the impact with no significant deformation or intrusion of the passenger compartment. The appropriate procedures for acceptance testing of breakaway supports are described in NCHRP Report 350.

Full-scale crash tests, bogie tests, and pendulum tests are used in the acceptance testing of breakaway devices. In full-scale testing, an actual vehicle is accelerated to the test speed and impacted into the device being tested. The point of initial impact is the front of the vehicle, either centered or at the quarter point of the bumper. Full-scale tests produce the most accurate results; however, their main disadvantage is cost. Bogie vehicles are also used to test breakaway hardware. A bogie is a reusable, adjustable surrogate vehicle used to model actual vehicles. A nose, similar to a pendulum nose, is used to duplicate the crush characteristics of the vehicle being modeled. Bogie vehicles are designed to be used in the speed range of 35 to 100 km/h [20 to 60 mph].

To reduce testing costs, pendulum tests are also used to evaluate breakaway hardware. Pendulum nose sections have been developed that model the fronts of vehicles. Pendulum tests have typically been used to test luminaire support hardware. However, due to the physical limitations of pendulums, pendulum testing is limited to 35 km/h [20 mph] impacts. Pendulum test results for impacts at 35 km/h [20 mph] may be extrapolated to predict 100 km/h [60 mph] impact behavior providing the support breaks free with little or no bending in the support. This extrapolation method should not be used with base-bending or yielding supports.

4.2 DESIGN AND LOCATION CRITERIA FOR BREAKAWAY AND NON-BREAKAWAY SUPPORTS

Sign, luminaire, and similar supports must first be structurally adequate to support the device mounted on them and to resist ice and wind loads as specified in the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* (1). Other concerns are that they be properly designed and carefully located to ensure that the breakaway devices perform properly and to minimize the likelihood of impacts by errant vehicles. For example, supports should not be placed in drainage ditches where erosion and freezing might affect the proper operation of the breakaway mechanism. It is also possible that a vehicle entering the ditch might be inadvertently guided into the support. Signs and supports that are not needed should be removed. If a sign is needed, then it should be located where it is least likely to be hit. Whenever possible, signs should be placed behind existing roadside barriers (beyond the design deflection distance), on existing structures, or in similar non-accessible areas. If this cannot be achieved, then breakaway supports should be used. Only when the use of breakaway supports is not practicable should a traffic barrier or crash cushion be used exclusively to shield sign supports.

As a general rule, breakaway supports should be used unless an engineering study indicates otherwise. However, concern for pedestrian involvement has led to the use of fixed supports in some urban areas. Examples of sites where breakaway supports may be imprudent are adjacent to bus shelters or in areas of extensive pedestrian concentrations.

Supports placed on roadside slopes must not allow impacting vehicles to snag on either the foundation or any substantial remains of the support. Surrounding terrain must be graded to permit vehicles to pass over any non-breakaway portion of the installation that remains in the ground or rigidly attached to the foundation. Figure 4.1, adopted from the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* (1), illustrates the method used to measure the required 100 mm [4 in.] maximum stub height.

Breakaway support mechanisms are designed to function properly when loaded primarily in shear. Most mechanisms are designed to be impacted at bumper height, typically about 500 mm [20 in.] above the ground. If impacted

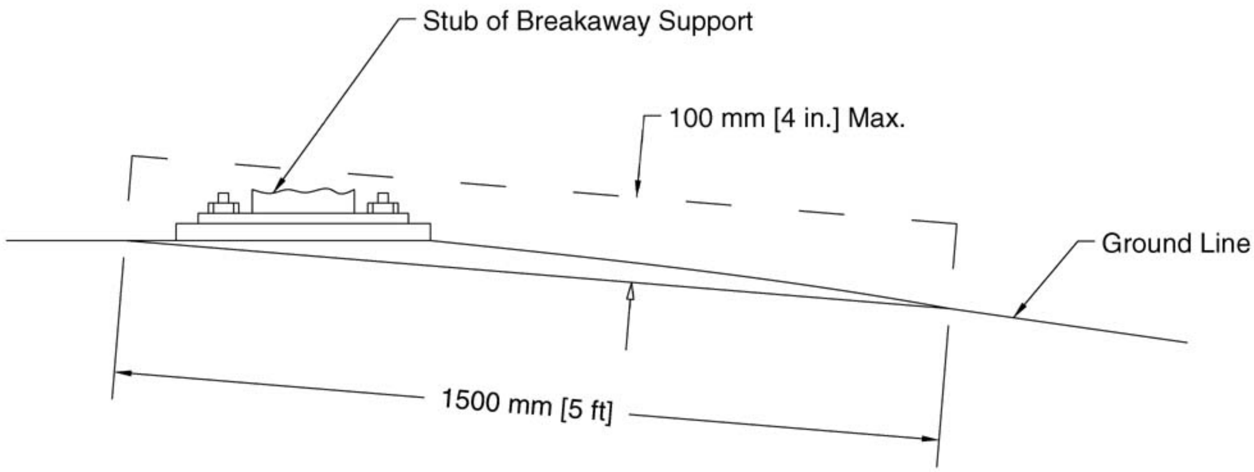


FIGURE 4.1 Breakaway support stub height measurements

at a significantly higher point, the bending moment in the breakaway base may be sufficient to bind the mechanism, resulting in non-activation of the breakaway device. For this reason, it is critical that breakaway supports not be located near ditches, on steep slopes, or at similar locations where a vehicle is likely to be partially airborne at the time of impact.

The type of soil may also affect the activation mechanisms of some breakaway supports. Fracture-type supports (i.e., high-carbon, U-channel posts, telescoping tubes, wood supports, etc.) could push through loose or saturated soils, absorbing energy and possibly adversely affecting a support's fracture mechanism. Usually this is not a problem for a fracture-type support embedded less than 1 m [3 ft] because the support will likely pull out of the soil, unless it has a special anchor plate designed to ensure that it does not. However, for fracture-type supports with pull-out-resisting anchors, supports embedded more than 1 m [3 ft], or any other support that might be sensitive to foundation movement, consideration should be given to qualifying them through crash testing in the "weak soil" described in the NCHRP Report 350 testing guidelines in addition to qualifying them through the "standard soil" crash tests called for in NCHRP Report 350. As explained in the Commentary on Chapter 2 in the NCHRP Report 350 (2):

> The weak soil should be used, in addition to the standard soil, for any feature whose impact performance is sensitive to soil-foundation or soil-structure interaction if identifiable areas of the state or local jurisdiction in which the feature will be installed contain soil with similar properties, and if there is a reasonable uncertainty regarding the performance of the feature in weak soil. Tests have shown that some base-bending or yielding small sign supports readily pull out of the weak soil upon impact. For features of this type, the strong soil is generally more critical and tests in the weak soil may not be necessary.

Special anchor plates or design details may also be used to accommodate expected wind loads. Since these design details could affect proper performance, it is recommended that these designs also be tested in both soils. To affect a truly cost-effective program of breakaway supports, there are other items that need to be considered. Availability of a particular support will affect installation costs and replacement costs. Durability of the support will affect the expected life of a non-struck support. Also, there may be some supports that can be reused after being impacted by a vehicle, which may be more cost-effective even though the initial costs are high. Thus, the expected impact frequency and simplicity of maintenance may influence an agency's selection.

4.3 SIGN SUPPORTS

Roadway signs can be divided into three main categories: overhead signs, large roadside signs, and small roadside signs. The hardware and corresponding safety treatment of sign supports varies with the sign category.

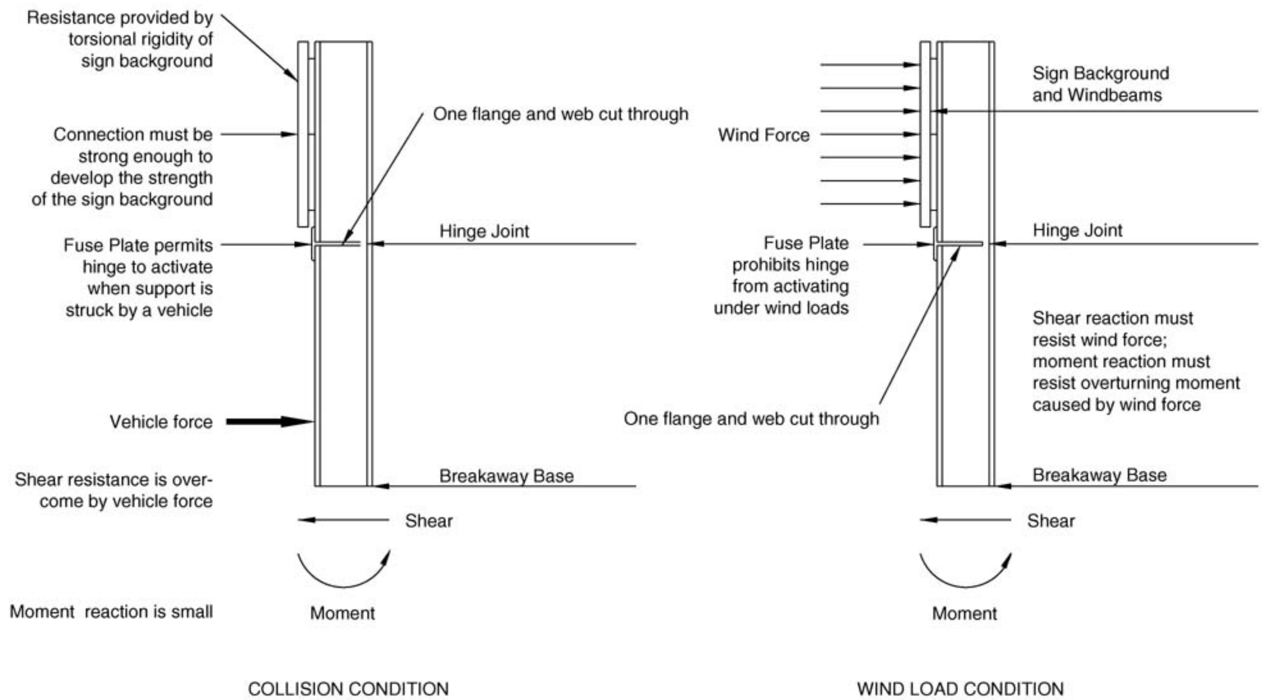


FIGURE 4.2 Wind and impact loads on roadside signs

4.3.1 Overhead Signs

Where possible, overhead signs should be installed on or relocated to nearby overpasses or other existing structures. Overhead signs, including cantilevered signs, generally require massive support systems that cannot be made breakaway. All overhead sign supports located within the clear zone should be shielded with a crashworthy barrier.

4.3.2 Large Roadside Signs

Large roadside signs may be defined as those greater than 5 m^2 [50 ft^2] in area. They typically have two or more breakaway support posts. The basic concept of the breakaway sign support is to provide a structure that will resist wind and ice loads, yet fail in a safe and predictable manner when struck by a vehicle. The loading conditions for which the support must be designed are shown in Figure 4.2. The desired impact performance is depicted in Figure 4.3. To achieve satisfactory breakaway performance, the following criteria should be met:

- The hinge should be at least 2.1 m [7 ft] above the ground so that no portion of the sign or upper section of the support is likely to penetrate the windshield of an impacting vehicle.

- A single post, if 2.1 m [7 ft] or more from another post, should have a mass [weight] less than 67 kg/m [45 lb/ft]. The total mass [weight] below the hinge, but above the shear plate of the breakaway base, should not exceed 270 kg [600 lb]. For two posts spaced less than 2.1 m [7 ft] apart, each post should have a mass [weight] less than 27 kg/m [18 lb/ft].

- No supplementary signs should be attached below the hinges if such placement is likely to interfere with the breakaway action of the support post or if the supplemental sign is likely to strike the windshield of an impacting vehicle.

The breakaway mechanisms of large roadside sign supports are either a fracture or a slip-base type. Fracture mechanisms consist of either couplers or wood posts with reduced cross sections. Most couplers are considered to be multidirectional, i.e., they are expected to work satisfactorily when struck from any direction. Figure 4.4 shows one type of multidirectional coupler in common use.

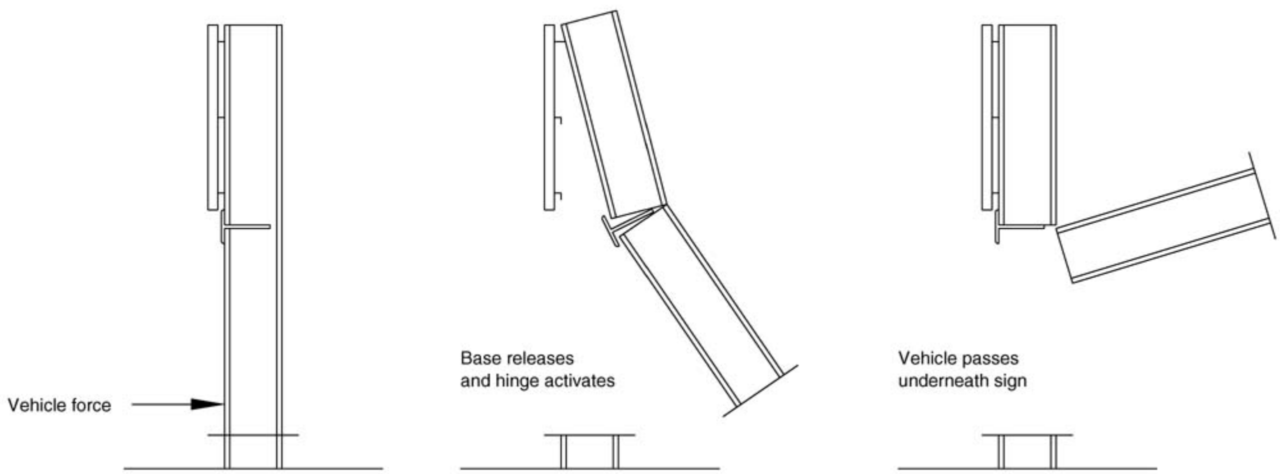


FIGURE 4.3 Impact performance of a multiple-post sign support

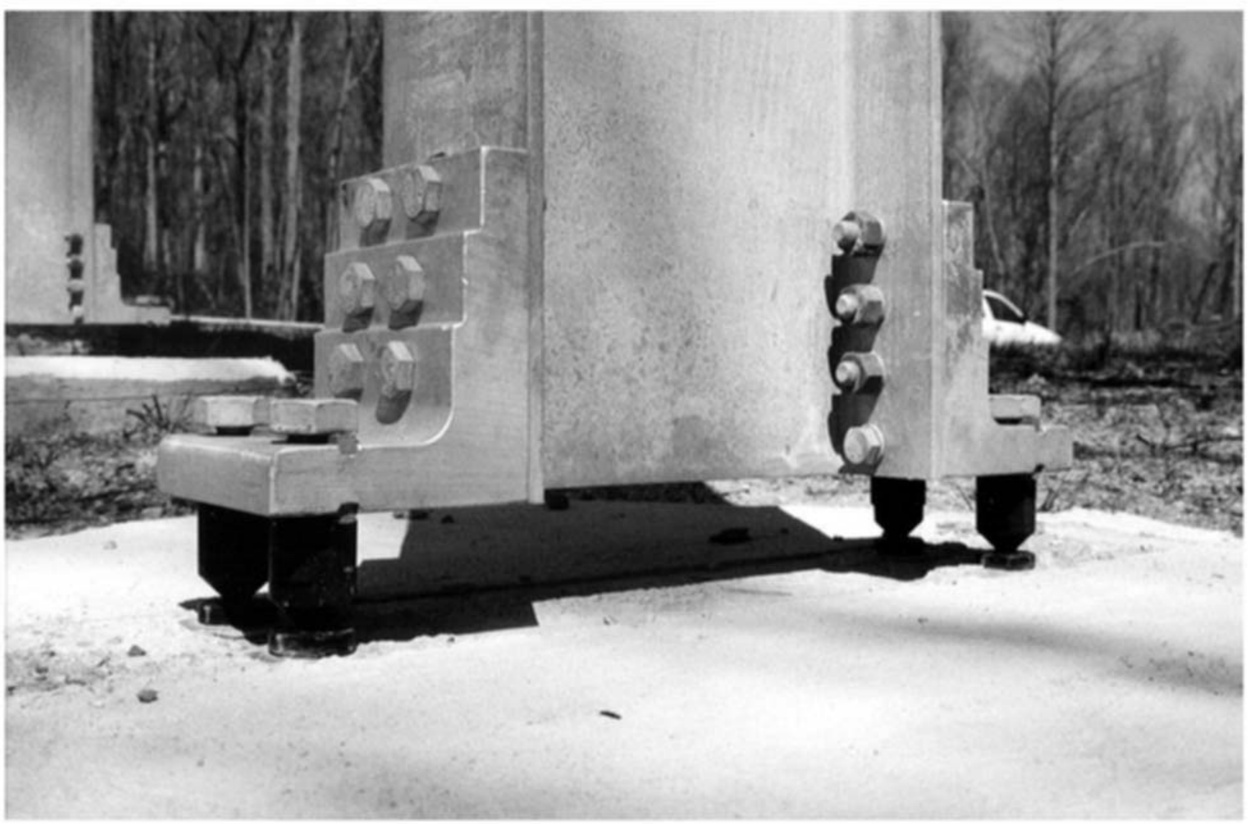


FIGURE 4.4 Multidirectional coupler

FIGURE 4.5 Typical uni-directional slip base

Slip-base type mechanisms activate when two parallel plates slide apart as the bolts are pushed out under impact. The designs may be either uni-directional or multidirectional. Horizontal slip bases using the four-bolt pattern shown in Figure 4.5 are uni-directional.

The upper hinge design for uni-directional impacts consists of a slotted fuse plate on the expected impact side and a saw cut through the web of the post to the rear flange. The rear flange then acts as a hinge when the post rotates upward. This commonly used design is shown in Figure 4.6. Slotted plates may be used on both sides of the post if impacts are expected from either direction.

Proper functioning of the slip base and fuse plate designs requires proper torque of the bolts. If the bolted connection is too tight, friction forces between the plates may prevent activation of the breakaway base under intended loading conditions. If the bolts are under-torqued, the posts may "walk" off the base under wind and other vibration loads. The use of keeper plates is recommended to retain the clamping bolts in place even if the bolted connection relaxes over time.

Designing for wind load is necessary for large signs. A check of wind load designs on the fuse plates should also be made. A perforated steel fuse plate meeting the requirements of ASTM A 36/A 36M has been shown to perform satisfactorily when used as the fuse plate on a steel post. Since this design does not require its connections to be torqued to a specific value, it is relatively fail-safe and recommended for use in lieu of slotted fuse plates. The perforated design is shown in Figure 4.7.

In some low-speed tests, the fuse plates on large roadside sign supports have failed to activate and the support has pulled away from the sign panel. The change in vehicle speed has still been acceptable. However, fuse and hinge plates should not be eliminated based on these low-speed tests. While they are more likely to activate in a high-speed impact, they act as a back-up safety feature in low-speed impacts.

Although the *Manual on Uniform Traffic Control Devices* (MUTCD) (3) specifies the general location of large roadside signs, the highway designer has a significant degree of latitude in the exact placement of any given sign. Crash test results show that breakaway supports installed on level terrain will perform as intended when struck head-on by a vehicle. However, if these supports are installed on a slope or the possibility exists that a vehicle may be spinning or sliding on impact, the breakaway feature may not function as well as when it is installed on level terrain. Even if a sign is erected on breakaway supports, it can cause significant damage to an impacting vehicle and injuries to the vehicle occupants. Once hit, the sign becomes a maintenance problem. These

FIGURE 4.6 Slotted fuse plate design

FIGURE 4.7 Perforated fuse plate design

are obvious reasons for locating all signs where they are least likely to be hit and, when feasible, outside the clear zone, even if they are breakaway.

4.3.3 Small Roadside Signs

Small roadside signs may be defined as those supported on one or more posts and having a sign panel area not greater than 5 m^2 [50 ft^2]. Although not usually perceived as an obstruction, small signs can cause substantial damage to impacting automobiles. Small sign supports are typically either driven directly into the soil, set in drilled holes, or mounted on a separately installed base. The breakaway mechanisms for small sign supports consist of either a base-bending, a fracture, or a slip base design. The most commonly used small sign support hardware and the characteristics of each are described below.

Base-bending or yielding sign supports typically consist of U-channel steel posts, perforated square steel tubes, thin-walled aluminum tubes, or thin-walled fiberglass tubes. A steel plate measuring approximately 100 mm x 300 mm x 6 mm [4 in. x 12 in. x $^1/_4$ in.] may be welded or bolted to the pipe support to prevent the sign from twisting from wind loads. Performance of these base-bending supports is much more difficult to predict than other support types. Variations in the depth of embedment, the soil resistance, stiffness of the sign support, mounting height of the sign, and many other factors influence their dynamic behavior.

Splicing of steel U-channel posts is not recommended unless tested because the impact performance of a spliced post cannot be accurately predicted. Unless crash tested with bracing in place, diagonal bracing of a sign support should be avoided because such bracing could significantly affect the crash performance of an otherwise acceptable design. This is particularly true for base-bending or yielding supports. When it is absolutely necessary to increase the strength of a post support system, larger breakaway or multiple breakaway posts should be considered.

For single sign posts with bending or yielding characteristics, the sign panels should be adequately bolted to the post with oversized washers to prevent the panel from separating on impact and penetrating a windshield. At higher speeds, base-bending or yielding sign supports bend around the bumper causing the top of the support to impact the windshield or roof. Early research indicated that the shorter mounted signs would destroy the windshield and penetrate the occupant compartment. A minimum height of 2.7 m [9 ft] to the top of the sign panel was recommended to alleviate the situation because it was expected that the top of the sign would then impact the roof rather than the windshield. This suggestion was valid at the time the research was conducted because of the prevalence of small cars. Recent computer simulation of 100 km/h [60 mph] sign impacts with mid-size automobiles and light trucks show that the 2.7 m [9 ft] height results in the same undesirable location of impact that the lower signs had on the small cars. Therefore, there is no net crashworthiness benefit to requiring signs to be mounted higher. Agencies concerned with this undesirable behavior may wish to consider a small sign support system that incorporates a slip base or breakaway coupling mechanism for use on high-speed streets and highways.

Fracturing sign supports are either wood posts, steel posts/pipes, or aluminum supports connected at ground level to a separate anchor. Wood posts are typically set in drilled holes and backfilled, while anchors for steel pipe and steel post systems are normally driven into the ground.

Slip base designs for small sign supports may be broadly classified as uni-directional or multidirectional. The most basic types of uni-directional breakaway sign supports are the horizontal and inclined slip bases. The design shown in Figure 4.8 is typical. The inclined design shown uses a 4-bolt slip base inclined in the direction of traffic at 10 to 20 degrees from horizontal. This angle ensures that the sign will move upward to allow the impacting vehicle to pass under the sign without its hitting the windshield or top of the car. When this type of slip base is used for small signs, hinges in the posts are not needed. The major limitation of this slip base design is its directional property. The inclined slip base can only be struck from one direction to yield satisfactorily. Neither the horizontal nor the inclined slip-base designs should be used in medians, traffic islands, or other locations where impacts from more than one direction are possible.

Multidirectional slip bases are typically triangular and are designed to release when struck from any direction. A typical design is shown in Figure 4.9. These types of breakaway supports are ideally suited for use on medians, channelizing islands, T-intersections, ramp terminals, and other locations where a sign may be impacted from several directions.

Slip base breakaway sign supports are subject to installation and maintenance problems that do not exist for rigid supports. Wind and other vibration loads may cause the bolts in the slip base to loosen. A keeper plate is recommended to prevent the clamping bolts, which have low torque requirements, from "walking" or migrating from the slots under wind loads.

A more common problem is the failure of a slip base to release properly due to over-torquing of the clamping bolts in the slip base and in the hinge of small sign supports. Because the slip base operates on the weakened shear plane concept, over-torquing creates high friction between the slip base elements and may prevent the post from releasing properly when hit. For this reason, breakaway designs not dependent upon specific torque requirements

FIGURE 4.8 Uni-directional slip base for small signs

FIGURE 4.9 Multidirectional slip base for small signs

FIGURE 4.10 Oregon 3-bolt slip base

are highly desirable. The Oregon 3-bolt slip base, shown in Figure 4.10, has 90-degree notch openings and thick washers. This design has very good breakaway performance and is much less sensitive to over-torquing. Problems with thread fabrication on clamping bolt nuts, improper assembly of slip base parts, and anchor bolts projecting into the slip base are other common deficiencies that should be avoided.

In areas where critical wind velocities are prevalent, sign flutter can be a problem that should be considered. This phenomenon, where rapid rotation and twisting of the posts occur, can cause failure of the posts by fatigue.

4.4 MULTIPLE POST SUPPORTS FOR SIGNS

All breakaway supports within a 2.1 m [7 ft] spacing are considered to act together. This criterion is based on a need to minimize the potential for unacceptable performance of breakaway hardware. In some cases, a vehicle could leave the roadway at a sufficiently high angle such that two posts within a 2.1 m [7 ft] spacing would be struck. In other cases, a vehicle could be yawing in the roadside to such an extent that two posts within a 2.1 m [7 ft] spacing would be struck. In many instances, the greatest change in vehicle velocity occurs when impacting breakaway hardware at slower speeds because less energy is available to activate the breakaway mechanism. Since vehicles leaving the roadway at very high angles or in a yawing mode would likely be traveling at slower speeds, the 2.1 m [7 ft] criterion is a reasonable safety factor that should be used in roadside design of breakaway hardware.

4.5 BREAKAWAY LUMINAIRE SUPPORTS

Breakaway luminaire supports are typically classified as frangible bases (cast aluminum transformer bases), slip bases, or frangible couplings (couplers). Examples of each type in common use are shown in Figures 4.11 to 4.13. Breakaway luminaire supports can be similar to breakaway sign hardware. The breakaway mechanism properly activates if loaded in shear rather than bending and is designed to release when impacted at typical bumper height

FIGURE 4.11 Example of a cast aluminum transformer base

FIGURE 4.12 Example of a luminaire slip base design

FIGURE 4.13 Example of a frangible coupling design

of about 500 mm [20 in.]. The devices may not perform properly when the supports are located along the roadside where impacts would result in bending rather than shear. Superelevation, slope rounding and offset, and vehicle departure angle and speed will influence the striking height of a typical bumper. If the foreslopes are limited to 1V:6H or flatter between the roadway and the luminaire support, vehicles should strike the support at an acceptable height.

As a general rule, a luminaire support will fall near the line of the path of an impacting vehicle. The mast arm usually rotates so it is pointing away from the roadway when resting on the ground. This action generally prevents the pole from going into other traffic lanes. However, the designer must remain aware that these falling poles may endanger bystanders such as pedestrians, bicyclists, and motorists.

At the present time, the height of poles with breakaway features should not exceed 18.5 m [60 ft]. This maximum height is recommended because it is the approximate maximum height of currently accepted hardware and is also the height that can accommodate modern lighting design practices when foundations are set at about roadway grade. To prevent a situation with potentially serious consequences should a pole fall on a vehicle, the mass [weight] of a breakaway luminaire support should not exceed 450 kg [1000 lb].

The type of soil surrounding a luminaire foundation may affect the performance of the breakaway mechanism. Experience shows that if foundations are allowed to push through the soil, the luminaire support will be placed in bending rather than shear, resulting in non-activation of the breakaway mechanism. Foundations should be properly designed to prevent their movement or rotation or both in surrounding soils.

Non-direct-burial luminaire supports generally require a substantial foundation. It is important that any such foundation is essentially flush with the ground because the 100 mm [4 in.] stub height criterion in the AASHTO breakaway specifications includes all non-breakaway elements above the ground line.

In all breakaway supports housing electrical components, efforts should be made to effectively reduce fire and electrical hazards should an errant vehicle impact a structure. Upon knockdown, the electricity in the support/structure should disconnect as close to the foundation as possible.

When luminaire supports are located near a traffic barrier, breakaway bases may or may not be applicable, depending upon the type and characteristics of the barrier. Luminaire supports should not be placed within the deflection distance of a barrier. For the most part, the impact performance of barriers interacting with a luminaire support breakaway device during a crash has not been determined. This situation should be avoided unless crash testing of a particular combination of devices indicates that the performance is acceptable. If the support must be within the design deflection distance of the barrier, it should be a breakaway design or the railing should be stiffened locally to minimize the resultant deflection. Details on traffic barrier types and characteristics can be found in Chapters 5 and 6.

Several state agencies mount luminaires on top of concrete median barriers, a practice that often requires modification to the luminaire support or median barrier or both. This type of installation generally does not use breakaway supports because of the risk a downed pole might present to opposing traffic. A consideration in this design is the likelihood of truck impacts with the barrier, since a truck bed will typically overhang short barriers during an impact and could snag on the support that is located there. The resultant vehicle deceleration may be unacceptable.

A final consideration on roadway lighting is a reduction in the total number of luminaires used along a section of highway. Higher mounting heights may significantly reduce the total number of supports needed. The ultimate design in this respect is the use of tower or high-mast lighting that requires far fewer supports located much farther from the roadway. From a roadside safety perspective, this is a preferred method for lighting major interchanges.

4.6 SUPPORTS FOR TRAFFIC SIGNALS AND MISCELLANEOUS TRAFFIC SERVICE DEVICES

Other relatively narrow objects that are usually located adjacent to the roadway include traffic signals, motorist-aid callboxes, railroad warning devices, fire hydrants, and mailboxes. These are discussed in the following sections.

4.6.1 Traffic Signals

Traffic signal posts present a special situation where a breakaway support may not be practical or desirable. As with luminaire supports, a fallen signal post may become an obstruction. However, the potential risks associated with the temporary loss of full signalization at the intersection should be considered.

When traffic signals are installed on high-speed facilities (generally defined as those having speed limits of 80 km/h [50 mph] or greater), the signal supports, and the signal support box if not mounted on one of the signal support poles, should be placed as far away from the roadway as practicable. Shielding these supports can be considered if they are within the clear zone for that particular roadway. Consideration should be given to using breakaway supports for post-mounted signals installed in wide medians.

4.6.2 Motorist-Aid Callboxes

Motorist-aid callboxes should be treated as roadside obstacles. Their proximity to the traveled way warrants the use of crashworthy breakaway supports. Because of their size and weight, they can usually be designed to meet vehicle change-in-velocity requirements. A callbox must be securely attached to its support to prevent its separating and penetrating the windshield.

To the extent possible, callboxes should be located behind traffic barriers warranted for other reasons. Not only does this make them less likely to be hit, but it also reduces the risk of a motorist using a callbox being struck by a vehicle. Callboxes must be accessible to wheelchair users.

4.6.3 Railroad Crossing Warning Devices

Highway and railroad officials must cooperatively decide on the type of warning device needed at a particular crossing, e.g., crossbucks, flashing light signals, or gates. As a minimum, crossbucks are required and should be installed on an acceptable sign support. Other warning device supports, such as signals or gates, can cause an increase in the severity of injuries to vehicle occupants if struck at high speeds. In these cases, if the support is located in the clear zone, consideration should be given to shielding the support with a crash cushion. Longitudinal barrier is not often used because there is seldom sufficient space for a proper downstream end treatment, a longer obstacle is created by installing a guardrail, and a vehicle striking a longitudinal barrier when a train is occupying the crossing may be redirected into the train. The designer must also be aware of the immediate risk to other motorists just after the devices are knocked down by impacting vehicles.

4.6.4 Fire Hydrants

Fire hydrants are another type of roadside feature that may be an obstacle. While most fire hydrants are made of cast iron and could be expected to fracture upon impact, crash testing meeting current testing procedures has not been done to verify that designs meet breakaway criteria. However, at least one fire hydrant stem and coupling design is available which provides for immediate water shutoff if struck by a vehicle.

Whenever possible, fire hydrants should be located sufficiently far away from the roadway so that they do not become obstructions for the motorist, yet are still readily accessible to and usable by emergency personnel. Any portion of the hydrant not designed to break away should be within 100 mm [4 in.] of the ground.

4.6.5 Mailbox Supports

Mailbox supports are addressed in Chapter 11.

4.7 UTILITY POLES

Motor vehicle crashes with utility poles account for approximately 10 percent of all fixed-object fatal crashes annually. This degree of involvement is related to the number of poles in use, their proximity to the traveled way, and their unyielding nature.

As with sign and luminaire supports, the most desirable solution is to locate utility poles where they are least likely to be struck. One alternative unique to power and telephone lines is to bury them, thereby eliminating the obstacles. For poles that cannot be eliminated or relocated, breakaway designs have been developed and successfully crash tested. This alternative is briefly discussed below. Since utility poles are generally privately owned and installed devices permitted on publicly owned rights-of-way, they are not under the direct control of a highway agency. This dual responsibility sometimes complicates the implementation of effective countermeasures.

For new construction or major reconstruction, every effort should be made to install or relocate utility poles as far from the traveled way as practical. Two AASHTO publications provide more detailed information on locating utility facilities within highway rights-of-way (4, 5).

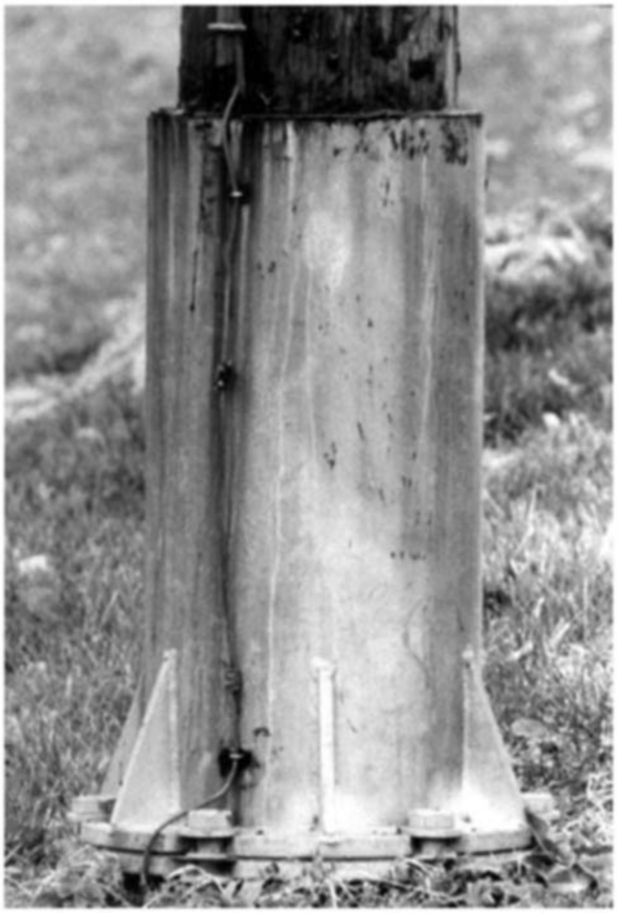
For existing utility pole installations, a concentration of crashes at a site or a certain type of crash that seems to occur frequently in a given jurisdiction may indicate that the highway/utility system is contributing to the crash potential. Utility pole crashes are subject to the same patterns as other types of roadway crashes; thus, they are subject to traditional highway crash study procedures. A detailed study of crash records may identify high-frequency crash locations and point out improvements that will reduce the number and severity of future crashes. Road users (the public and utility firms) can also provide input into the nature and causes of highway/utility crashes. The steps that are normally included in a comprehensive crash-reduction program are:

- setting up a traffic records system,
- identifying high-frequency crash locations,
- analyzing high-frequency crash locations,
- correcting the high-frequency crash locations, and
- reviewing the results of the program.

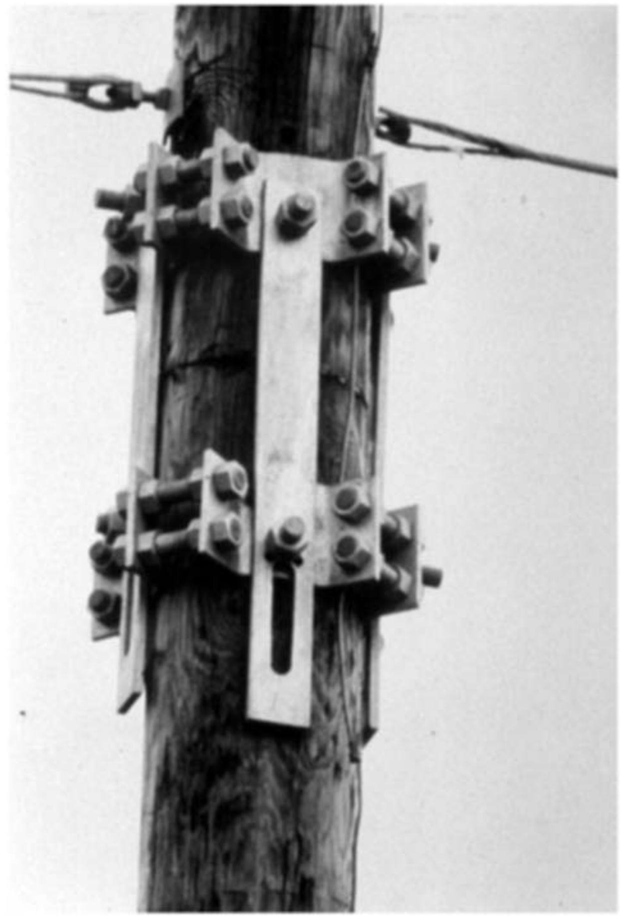
The size of the agency conducting the program may affect the degree of sophistication and complexity needed. Small highway agencies or utility firms may find it sufficient to place pins on a city map to identify high-crash locations, and then to review copies of police crash reports in order to select the best safety treatment. Large utility firms, units of local government, and state highway agencies may resort to computers to handle enormous volumes of data. Safety programs of the latter type frequently use sophisticated statistical software to select the best sites for treatment and to identify the most appropriate countermeasures. A manual procedure and a microcomputer program have been developed that enable the designer to determine which countermeasures could effectively reduce the frequency or severity of accidents at a given site. Details of this model are contained in *Transportation Research Record (TRR) 970* (6). The following specific countermeasures were included in the analysis:

- placing utility lines underground,
- increasing lateral pole offset,
- increasing pole spacing,
- multiple pole use (joint usage), and
- breakaway design.

Unlike the first three countermeasures, the use of a breakaway design is intended to reduce the severity of an accident rather than its frequency. The design shown in Figure 4.14, consisting of ground level slip base and upper hinge assembly, has been successfully crash tested. This design may be considered for poles in vulnerable locations that cannot economically be removed or relocated, such as gore areas, the outside of sharp curves, and opposite the intersecting roadway at T-intersections. Several variations of the breakaway utility pole are available and have demonstrated satisfactory in-service performance in the limited field trials to date.



(a)



(b)

FIGURE 4.14 Prototype breakaway design for utility poles

Another countermeasure that can be considered is adequate shielding of selected poles, particularly the massive supports used for major electrical transmission lines within the clear zone or in other vulnerable locations. An increasingly common practice is the delineation of poles that are not otherwise treated, particularly along streets and highways where nighttime run-off-the-road crashes are prevalent.

4.8 TREES

Single vehicle crashes with trees account for nearly 25 percent of all fixed-object fatal crashes annually and result in the deaths of approximately 3,000 persons each year. Unlike the roadside hardware previously addressed in this chapter, trees are not generally a design element over which highway designers have direct control. With the exception of landscaping projects where the types and locations of trees and other vegetation can be carefully chosen, the problem most often faced by designers is the treatment of existing trees that are likely to be impacted by an errant vehicle. To promote consistency within a State, each highway agency should develop a formal policy to provide guidance to design, landscape, construction, and maintenance personnel for this situation. This section is intended to provide general guidelines from which a specific policy on trees may be developed.

Trees are potential obstructions by virtue of their size and their location in relation to vehicular traffic. Generally, an existing tree with an expected mature size greater than 100 mm [4 in.] is considered a fixed object. When trees or shrubs with multiple trunks or groups of small trees are close together, they may be considered as having the effect of a single tree with their combined cross-sectional area. Maintenance forces can minimize future problems by mowing clear zones to prevent seedlings from becoming established. The location factor is more difficult to address than tree size. Typically, large trees should be removed from within the selected clear zone for new construction and for reconstruction. As noted in Chapter 3, the extent of the clear zone is dependent upon several variables, including highway speeds, traffic volumes, and roadside slopes. Segments of a highway can be analyzed to identify individual trees or groups of trees that are candidates for corrective measures. County and township roads, which generally have restrictive geometric designs and narrow, off-road recovery areas, account for a large percentage of the annual tree-related fatal crashes, followed by State and U.S. numbered highways on curved alignment. Fatal crashes involving trees along Interstate highways are relatively rare in most states.

Following several years of research by the Michigan Department of Transportation, a *Guide to Management of Roadside Trees* (7) was distributed nationally by the Federal Highway Administration as Report No. FHWA-IP-86-17. This document contains detailed information on identifying and evaluating higher risk roadside environments and provides guidance for implementing roadside tree removal. It also addresses environmental issues, alternative treatments, mitigation efforts, and maintenance practices. The remainder of this section is basically a summary of the information and recommendations included in that report.

Essentially, there are two methods for addressing the issue of roadside trees. The first is to keep the motorist on the road whenever possible, and the second is to mitigate the danger inherent in leaving a roadway with trees along it.

On-roadway treatments include

- pavement marking,
- rumble strips,
- signs,
- delineators, and
- roadways improvements.

Pavement markings are one of the most effective and least costly improvements that can be made to a roadway. Centerline and edge line markings are particularly effective for roads with heavy nighttime traffic, frequent fog, and narrow lanes. Shoulder rumble strips can also be used to warn motorists that their vehicles have crossed the edgeline and may run off the road.

The installation of advance warning signs and roadway delineators can also be used to notify motorists of sections of roadway where extra caution is advised. Typically, these will be used in advance of curves that are noticeably sharper than those immediately preceding it.

Roadway improvements such as curve reconstruction to provide increased superelevation, shoulder widening, and paving are relatively expensive countermeasures that may not be cost-effective in all cases.

Off-roadway treatments consist primarily of two options

- tree removal, and
- shielding.

The removal of individual trees should be considered when those trees are determined both to be obstructions and to be in a location where they are likely to be hit. Such trees can often be identified by past crash histories at similar sites, by scars indicating previous crashes, or by field reviews. Removal of individual trees will not reduce the probability that a vehicle will leave the roadway at that point, but should reduce the severity of any resulting crash. Because tree removal can be expensive and often has adverse environmental impacts, it is important that this countermeasure be used only when it is an effective solution. For example, 1V:3H and flatter slopes may be traversable, but a vehicle on a 1V:3H slope will usually reach the bottom. If there are numerous trees at the toe of the slope, removal of isolated trees on the slope will not significantly reduce the risk of a crash. Similarly, if the recommended clear zone for a particular roadway is 7 m [23 ft], including the shoulder, removal of trees 6 m to 7 m [20 ft to 23 ft] from the road will not materially change the risk to motorists if an unbroken tree line remains at 8 m [26 ft] and beyond. However, isolated trees noticeably closer to the roadway may be candidates for removal. If a tree or group of trees is in a vulnerable location but cannot be removed, a properly designed and installed traffic barrier can be used to shield them. Roadside barriers should only be used when the severity of striking the tree is greater than striking the barrier. Specific information on the selection, location, and design of roadside barriers is contained in Chapter 5.

REFERENCES

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4. AASHTO. *A Policy on the Accommodation of Utilities within Freeway Right-of-Way*. American Association of State Highway and Transportation Officials, Washington, DC, 1989.

5. AASHTO. *A Guide for Accommodating Utilities within Highway Right-of-Way*. American Association of State Highway and Transportation Officials, Washington, DC, 1994.

6. Transportation Research Board. *Safety Appurtenances and Utility Accomodation. Transportation Research Record 970*. Transportation Research Board, Washington, DC, 1984.

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Chapter 5
Roadside Barriers

5.0 OVERVIEW

A roadside barrier is a longitudinal barrier used to shield motorists from natural or man-made obstacles located along either side of a traveled way. It also may be used to protect bystanders, pedestrians, and cyclists from vehicular traffic under special conditions.

This chapter summarizes performance requirements and warrants for roadside barriers and contains guidelines for selecting and designing an appropriate barrier system. The structural and safety characteristics of selected roadside barriers and transition sections are presented here. For similar information on end treatments, see Chapter 8. Finally, placement guidelines are included, and a methodology is presented for identifying and upgrading existing substandard installations.

5.1 PERFORMANCE REQUIREMENTS

The primary purpose of all roadside barriers is to prevent a vehicle from leaving the traveled way and striking a fixed object or terrain feature that is less forgiving than striking the barrier itself. Containing and redirecting the impacting vehicle using a barrier system accomplishes this. Because the dynamics of a crash are complex, the most effective means of assessing barrier performance is through full-scale crash tests. By standardizing such tests, designers can compare the safety performance of alternative designs.

5.1.1 Current Crash Test Criteria

A series of standard crash tests are presented in National Cooperative Highway Research Program Report No. 350, *Recommended Procedures for the Safety Performance Evaluation of Highway Features* (NCHRP Report 350) (1). This report establishes six test levels (TLs) for longitudinal barriers to evaluate occupant risk, structural integrity of the barrier, and post-impact behavior of the vehicle for a variety of vehicle masses at varying speeds and angles of impact.

TL-1, TL-2, and TL-3 require successful tests of an 820 kg [1,800 lb] car impacting a barrier at an angle of 20 degrees and a 2000 kg [4,400 lb] pickup truck impacting a barrier at an angle of 25 degrees, at speeds of 50 km/h, 70 km/h, and 100 km/h [30 mph, 45 mph, and 60 mph], respectively. TL-4 adds an 8000 kg [17,600 lb] single-unit truck at an impact angle of 15 degrees and 80 km/h [50 mph] to the TL-3 matrix. TL-5 substitutes a 36000 kg [80,000 lb] tractor-trailer (van) for the single-unit truck and TL-6 substitutes a 36000 kg [80,000 lb] tractor-trailer (tanker).

For barrier approvals and performance acceptance, the designer is encouraged to contact the Federal Highway Administration's (FHWA) Office of Highway Safety and to access FHWA's web site at http://safety.fhwa.dot.gov/fourthlevel/hardware/longbarriers.htm to view the acceptance letters for longitudinal barriers under NCHRP Report 350.

5.1.2 Barrier Classifications

NCHRP Report 350 describes both experimental and operational acceptance phases for roadside barriers. In the experimental phase, a barrier that has acceptably passed crash testing is subjected to in-service evaluation. In the operational phase, a barrier that has been found acceptable through an in-service evaluation is classified as operational and it is recommended that its performance continues to be monitored. In practice, the determination of whether a barrier must undergo an experimental in-service evaluation is at the discretion of the user agency. Additionally, a barrier may be considered operational if it has been used for an extended period and has demonstrated satisfactory field performance in terms of construction, maintenance, and crash experience. All the barriers cited in this chapter have been found acceptable through crash testing and may be considered operational, although this would not preclude a user agency from treating any of the barriers as experimental to determine if they meet its needs.

No matter what status, experimental or operational, that an agency ascribes to a barrier, it is strongly recommended that the barrier's performance be monitored for any problems in construction, maintenance, or crashworthiness. See NCHRP Report 350 for guidance on conducting in-service evaluations.

Omission of a barrier system does not necessarily imply that it is not acceptable for use. There are barriers in use today that have not been subjected to full-scale crash tests but have performed satisfactorily over time.

5.2 WARRANTS

Barrier warrants are based on the premise that a traffic barrier should be installed only if it reduces the severity of potential crashes. It is important to note that the probability or frequency of run-off-the-road crashes is not directly related to the severity of potential crashes. The mere installation of barriers could lead to higher incident rates due to the proximity of the barriers to the traveled way.

Typically, guardrail warrants have been based on a subjective analysis of certain roadside elements or conditions. If the consequences of a vehicle striking a fixed object or running off the road are believed to be more serious than hitting a traffic barrier, then the barrier is considered warranted. While this approach can be used, often there are instances where it is not immediately obvious whether the barrier or the unshielded condition presents the greater risk. Furthermore, the subjective method does not directly consider either the probability of a crash occurring or the costs associated with shielded and unshielded conditions.

Warrants may also be established by using a benefit-to-cost analysis whereby factors such as design speed and traffic volume can be evaluated in relation to barrier need. Costs associated with the barrier (installation costs, maintenance costs, and crash costs) are compared to similar costs associated without barriers. This procedure is typically used to evaluate three options: (1) remove or reduce the area of concern so that it no longer requires shielding, (2) install an appropriate barrier, or (3) leave the area of concern unshielded. The third option would normally be cost-effective only on facilities with low volume, low speed, or both, or where engineering studies show the probability of crashes is low. Appendix A presents an analysis procedure that can be used to compare several alternative safety treatments and provide guidance to the designer in selecting an appropriate design.

Highway conditions that warrant shielding by a roadside barrier can be placed into one of two basic categories: embankments or roadside obstacles. Pedestrians or other "bystanders" may also warrant protection from vehicular traffic. Specific highway features contained in each of these categories are discussed in the following sections.

5.2.1 Embankments

Embankment height and side slope are the basic factors considered in determining barrier need as shown in Figure 5.1. These criteria are based on studies of the relative severity of encroachments on embankments versus impacts with roadside barriers. Embankments with slope and height combinations on or below the curve do not warrant shielding unless they contain obstacles within the clear zone. Figure 5.1, however, does not take into account either the probability of an encroachment occurring or the relative cost of installing a traffic barrier versus leaving the slope unshielded. Figure 5.2 is a modified warrant chart developed by a state that addresses the decreased probability of encroachments on lower volume roads. Figure 5.3 is another example of a modified warrant chart, one which considers the cost-effectiveness of barrier installation for the site-specific conditions noted on the chart. Figures 5.2 and 5.3 are presented as examples only and are not intended for direct application. Highway agencies are encouraged to develop similar warranting criteria based upon their own cost-effectiveness evaluations.

A rounded slope reduces the chances of an errant vehicle becoming airborne and affords the driver more control over the vehicle. Optimum rounding is arbitrarily defined as the minimum radius a standard-sized automobile can negotiate without losing tire contact. It is dependent on the encroachment angle and speed as well as the characteristics of individual vehicles.

5.2.2 Roadside Obstacles

Roadside obstacles include both non-traversable terrain and fixed objects, and may be either man-made (such as culvert inlets) or natural (such as trees). Together, these highway conditions account for over thirty percent of all highway fatalities each year. Barrier warrants for roadside obstacles are a function of the obstacle itself and the likelihood that it will be hit. However, a barrier should be

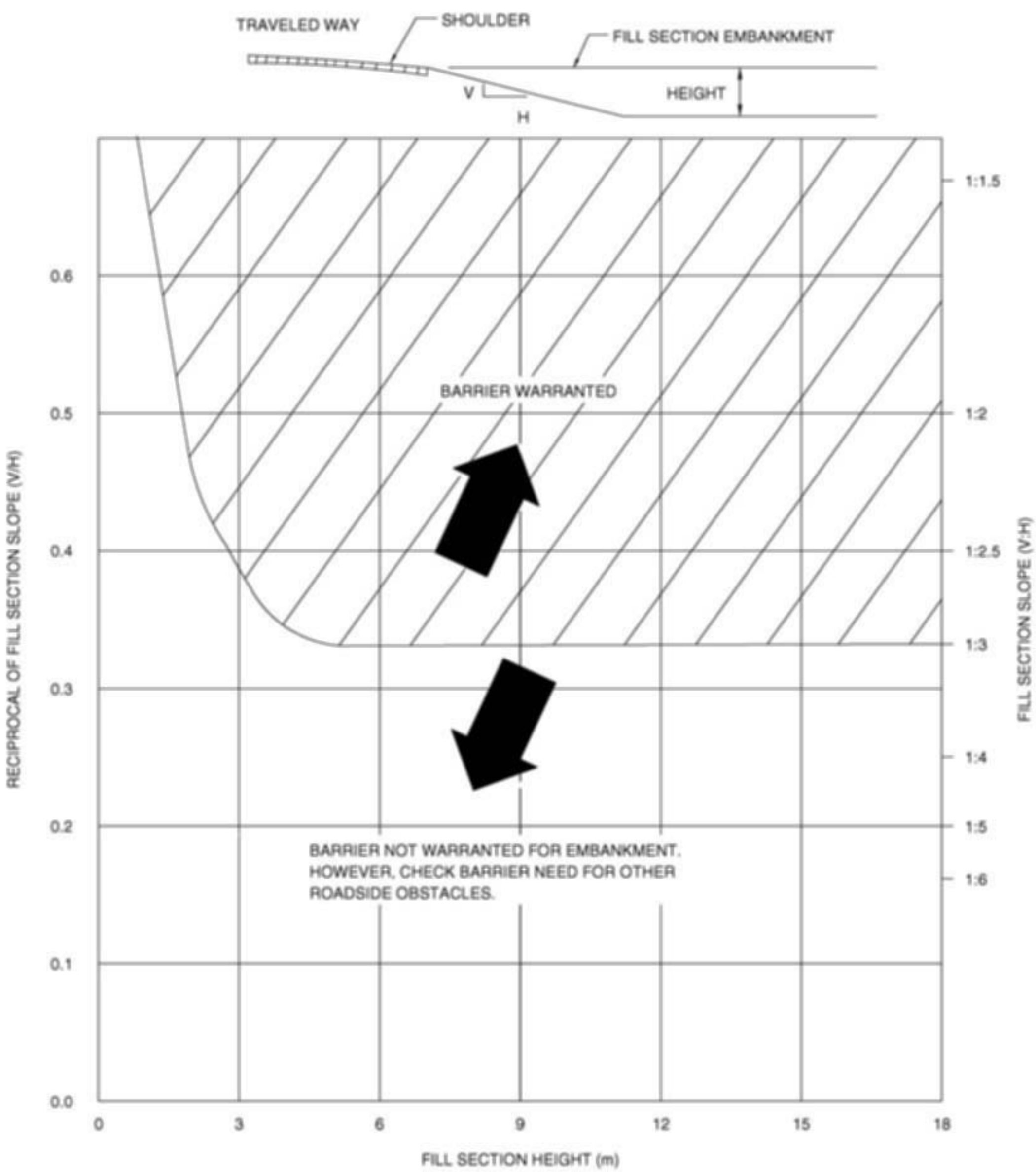


FIGURE 5.1a Comparative risk warrants for embankments (metric units)

installed only if it is clear that the result of a vehicle striking the barrier will be less severe than the crash resulting from hitting the unshielded object.

Non-traversable terrain and roadside obstacles that normally warrant shielding are listed in Table 5.1. While roadside obstacles immediately adjacent to the traveled way are usually removed, relocated, modified, or shielded, the optimal solution becomes less evident as the distance between the obstruction and the traveled way increases. Table 3.1, Clear-Zone Distances, is intended as a guide to aid the designer in determining whether the obstruction constitutes a threat to an errant motorist that is significant enough to warrant action. Most man-made objects incorporated into a highway project can be designed to minimize or eliminate the danger they present to a motorist and thus make shielding unnecessary. This is particularly true of drainage features such as small culverts and ditches.

5.2.3 Bystanders, Pedestrians, and Bicyclists

An area of concern to highway officials is what has been termed the "innocent bystander" problem. In most such

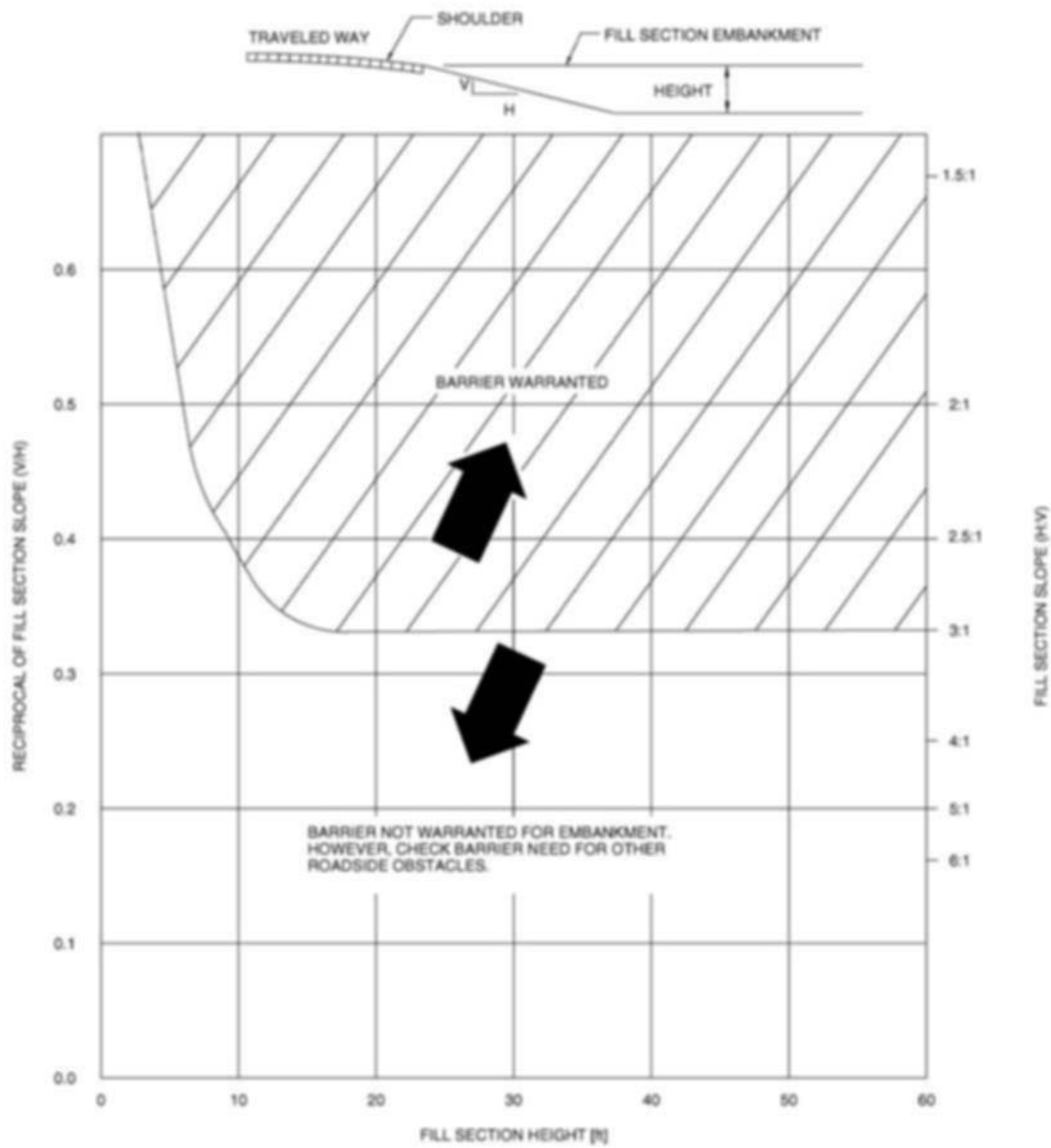


FIGURE 5.1b Comparative risk warrants for embankments [U.S. customary units]

cases, the conventional criteria presented in the previous sections cannot be used to establish barrier needs. For example, a major street, highway, or freeway may adjoin a schoolyard, but the boundaries are beyond the clear distance. There are no criteria that would require that a barrier be installed. If, however, a barrier is installed, it could be placed near the school boundary to minimize the potential for vehicle contact. Reference should be made to Section 5.6.1 for lateral placement criteria. Consideration might also be given to installing a barrier to shield businesses and residences that are near the right-of-way, particularly at locations having a history of run-off-the-road crashes.

Pedestrians and cyclists are another category of concern that should be given design consideration. The most desirable solution is to separate them from vehicular traffic. Since this solution is not always practical, alternate means of protecting them are sometimes necessary. As in the case of bystander warrants, there are no objective criteria to draw on for pedestrian and cyclist barrier warrants. On low-speed streets, a vertical faced curb will usually suffice to separate pedestrians and cyclists from vehicular traffic. However, at speeds over 40 km/h [25 mph], a vehicle may mount the curb for relatively flat approach angles. Hence, when sidewalks or bicycle paths are adja-

TABLE 5.1 Barrier warrants for non-traversable terrain and roadside obstacles[1,2]

Obstacle	Warrants
Bridge piers, abutments, and railing ends	Shielding generally required
Boulders	Judgment decision based on nature of fixed object and likelihood of impact
Culverts, pipes, headwalls	Judgment decision based on size, shape, and location of obstacle
Cut & fill slopes (smooth)	Shielding not generally required
Cut & fill slopes (rough)	Judgment decision based on likelihood of impact
Ditches (parallel)	Refer to Figures 3.6 and 3.7
Ditches (transverse)	Shielding generally required if likelihood of head-on impact is high
Embankment	Judgment decision based on fill height and slope (see Figure 5.1)
Retaining walls	Judgment decision based on relative smoothness of wall and anticipated maximum angle of impact
Sign/luminaire supports[3]	Shielding generally required for non-breakaway supports
Traffic signal supports[4]	Isolated traffic signals within clear zone on high-speed rural facilities may warrant shielding
Trees	Judgment decision based on site-specific circumstances
Utility poles	Shielding may be warranted on a case-by-case basis
Permanent bodies of water	Judgment decision based on location and depth of water and likelihood of encroachment

Notes:

1 Shielding non-traversable terrain or a roadside obstacle is usually warranted only when it is within the clear zone and cannot practically or economically be removed, relocated, or made breakaway, and it is determined that the barrier provides a safety improvement over the unshielded condition.

2 Marginal situations, with respect to placement or omission of a barrier, will usually be decided by crash experience, either at the site or at a comparable site.

3 Where feasible, all sign and luminaire supports should be a breakaway design regardless of their distance from the roadway if there is reasonable likelihood of their being hit by an errant motorist. The placement and locations for breakaway supports should also consider the safety of pedestrians from potential debris resulting from impacted systems.

4 In practice, relatively few traffic signal supports, including flashing light signals and gates used at railroad crossings, are shielded. If shielding is deemed necessary, however, crash cushions are sometimes used in lieu of a longitudinal barrier installation.

cent to the traveled way of high-speed facilities, some provision might be made for the safety of pedestrians and cyclists.

5.2.4 Motorcycles and Barrier Design

There have been numerous instances nationwide where roadside barriers have contributed to the severity of crashes involving motorcycles. Most commonly, motorcyclists have been seriously injured or killed after impacting some types of open-faced traffic barriers, particularly after contacting the edges of steel guardrail posts or the tops of these posts where they project above the rail element. Some European countries have attempted to address this concern at locations having both high motorcycle use and a high number of crashes by adding a lower rubrail to the design or by padding the posts with expanded foam. However, no systematic approach toward this issue has been developed because of the random nature of motorcycle crashes and the questionable effectiveness of modifications to existing barriers. Based on the experience of other countries and the lack of any cost-effective countermeasures or barrier designs, there appears to be little basis for developing guardrails designed for motorcyclists. There is some suggestion that a smooth,

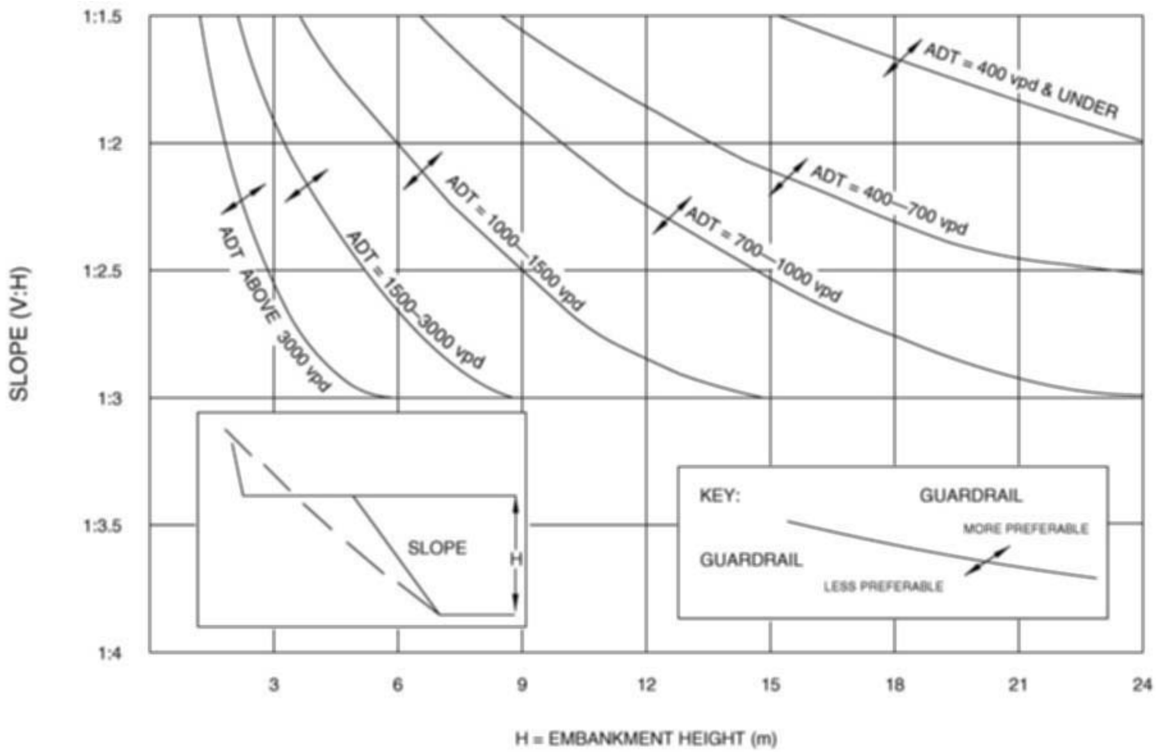


FIGURE 5.2a Example design chart for embankment warrants based on fill height, slope, and traffic volume (metric units)

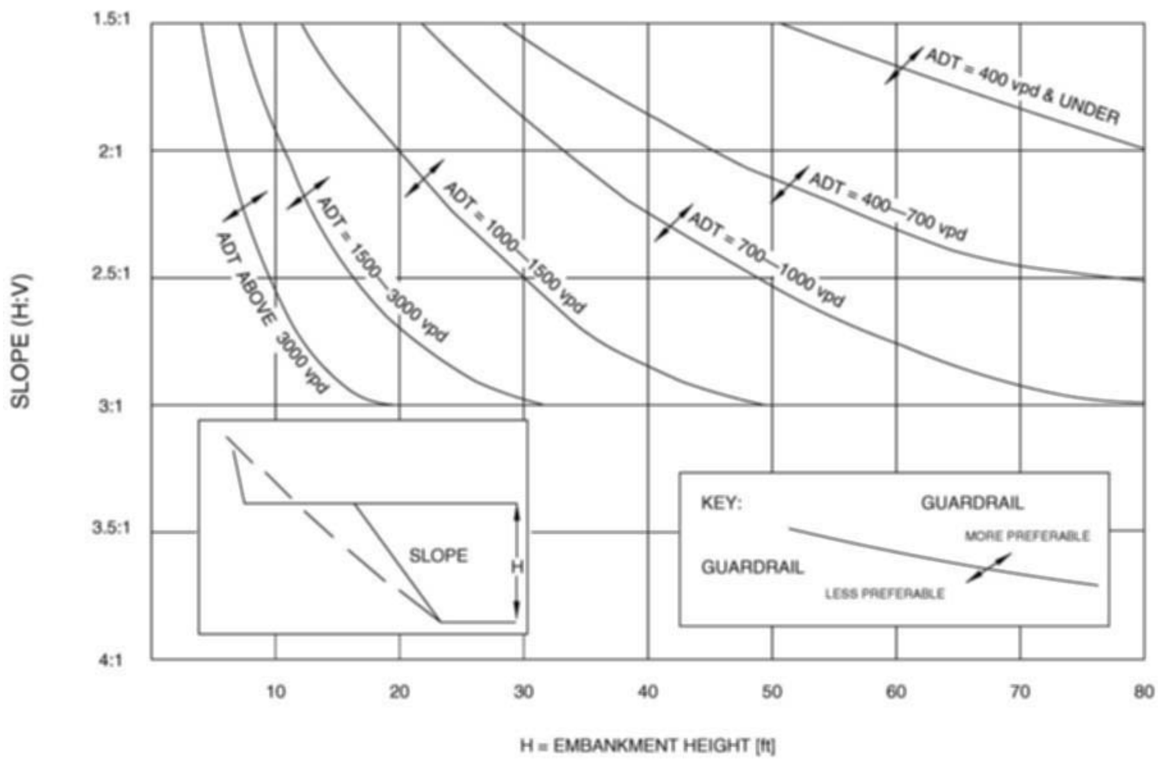


FIGURE 5.2b Example design chart for embankment warrants based on fill height, slope, and traffic volume [U.S. customary units]

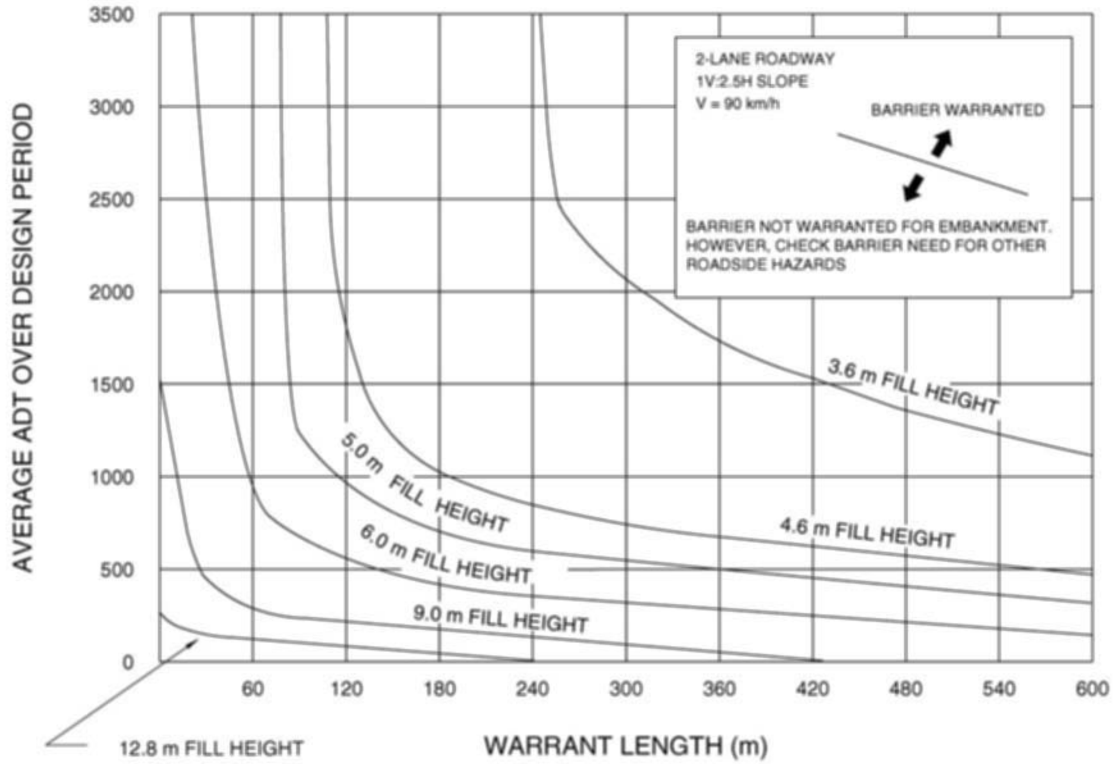


FIGURE 5.3a Example design chart for cost-effective embankment warrants based on traffic speeds and volumes, slope geometry and length of slope (metric units)

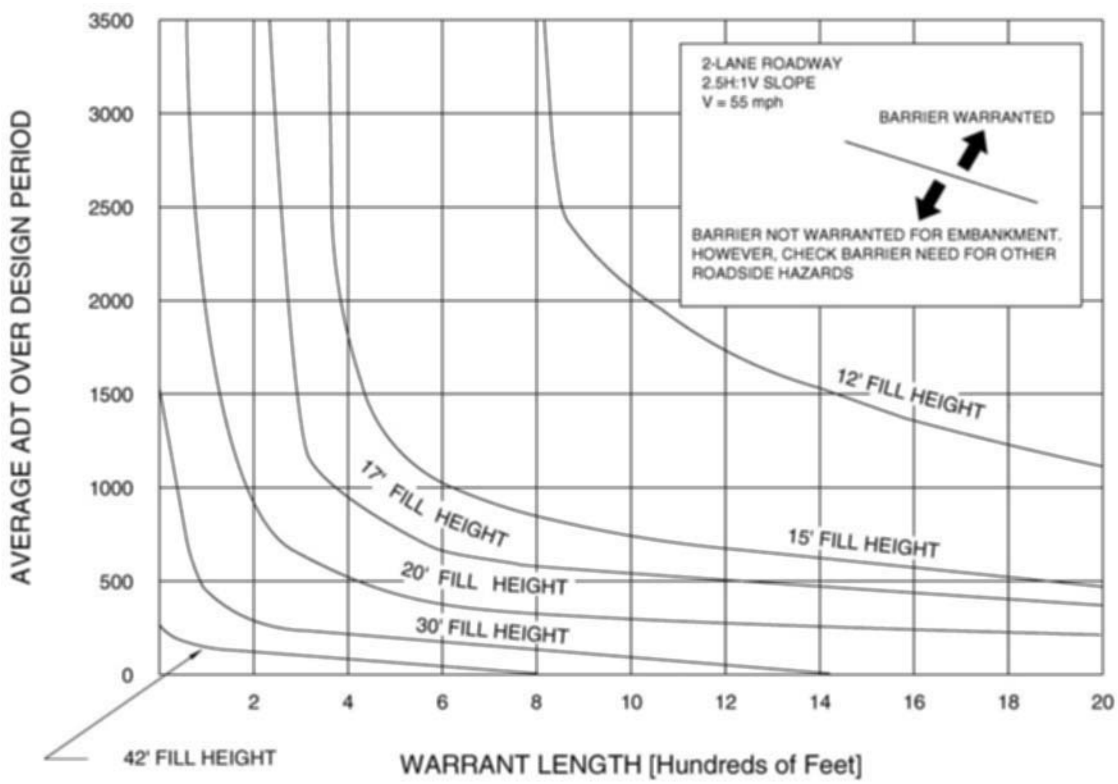


FIGURE 5.3b Example design chart for cost-effective embankment warrants based on traffic speeds and volumes, slope geometry and length of slope [U.S. customary units]

solid-faced barrier such as a concrete safety shape is least likely to cause traumatic injuries to cyclists upon contact.

5.3 PERFORMANCE LEVEL SELECTION FACTORS

Most roadside barriers were developed, tested, and installed with the intention of containing and redirecting passenger vehicles with masses up to 2000 kg [4,400 lb]. Properly designed and installed barrier systems have proven to be very effective in reducing the amount of damage and lessening the severity of personal injuries when struck by automobiles and similar-sized vehicles at relatively shallow angles (less than 25 degrees) and at reasonable impact speeds (less than 110 km/h [65 mph]). However, it has long been understood that barriers designed for cars should not be expected to perform equally well for larger vehicles, such as buses and trucks. Recognizing this fact, several highway agencies have developed and used barrier systems capable of redirecting vehicles as heavy as 36,000-kg [80,000-lb] tractor-trailer combination trucks. Although objective warrants for the use of higher performance traffic barriers do not presently exist, subjective factors most often considered for new construction or safety upgrading include:

- high percentage of heavy vehicles in traffic stream,
- adverse geometrics, such as sharp curvature, which are often combined with poor sight distance, and
- severe consequences associated with penetration of a barrier by a large vehicle.

These same factors apply on reconstruction or rehabilitation projects but, in these cases, the designer will usually have the added benefit of past crash history, the past performance of the system, and maintenance costs associated with the existing barrier. In addition, a higher performance barrier is likely to lessen the severity of future crashes or reduce maintenance costs significantly. Section 5.4 includes information on the size of vehicle for which each system has been successfully crash tested.

5.4 STRUCTURAL AND SAFETY CHARACTERISTICS OF ROADSIDE BARRIERS

This section includes information on the most commonly used operational roadside barriers as well as data on selected experimental systems. Separate subsections address standard sections of roadside barriers and transition sections. Figure 5.4 graphically depicts each of these elements for typical installations. Information on the structural and safety characteristics of each system is presented in a narrative format, and includes the following information:

- a photograph or sketch of the barrier.
- a barrier description showing the main elements of the barrier and post spacing. Prior to selection of a specific barrier system, the designer should

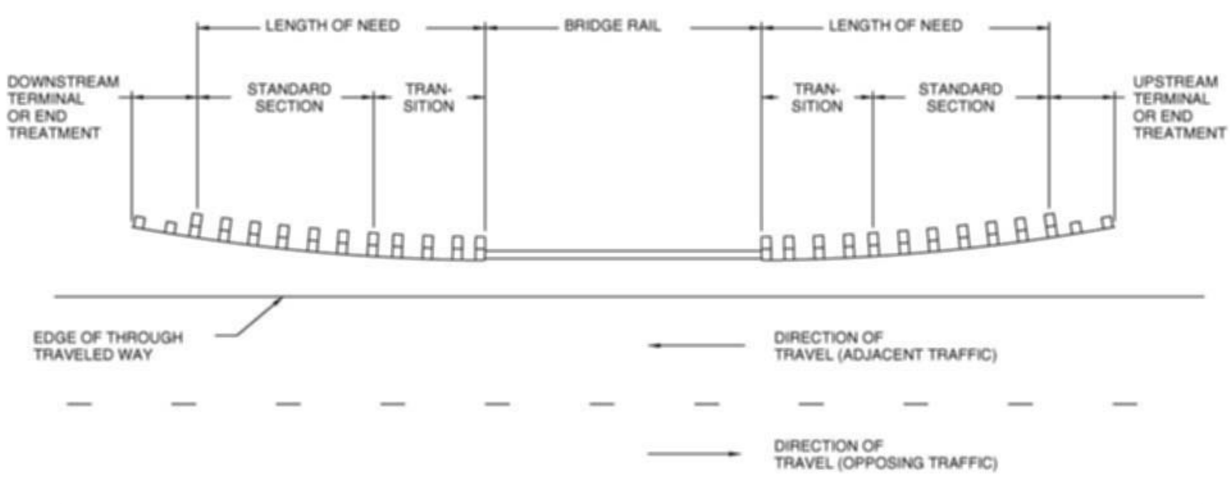


FIGURE 5.4 Definition of roadside barriers

obtain full details of the system from standard drawings or a similar source.

- a brief description of the impact performance of each system. This will describe the range of vehicles for which the system has been successfully crash tested. For standard sections, the dynamic deflection observed during the NCHRP Report 350 standard strength test for the 2000 kg [4,400 lb] pickup truck impacting a barrier at an angle of 25 degrees and at a velocity of 100 km/h [60 mph] is used.

- field performance data for experimental barriers are included when available. This provides the designer with in-service evaluation information and is intended to encourage the use and evaluation of additional pilot installations at appropriate locations.

Additional information on individual barrier systems, including design details and barrier damage resulting from tests, is presented in Appendix B.

5.4.1 Standard Sections of Roadside Barriers

Roadside barriers are usually categorized as flexible, semi-rigid, or rigid, depending on their deflection characteristics on impact. Flexible systems are generally more forgiving than the other categories since much of the impact energy is dissipated by the deflection of the barrier and lower impact forces are imposed upon the vehicle.

This section is not intended to be all-inclusive, but to cover the most widely used roadside barriers. The barriers and approved test levels included in the following subsections are listed in Table 5.2.

5.4.1.1 Three-Strand Cable

The barrier system shown in Figure 5.5 consists of steel cables mounted on weak posts. Several variations of this design (SGR01a and SGR01b) have been successfully crash tested. (See Appendix B for individual designs.)

Impact performance: This system, with a top cable height of 760 mm [30 in.], has been successfully tested to NCHRP Report 350, TL-3. The dynamic lateral deflection observed during strength testing with a 2000 kg [4,400 lb] pickup truck at 95.1 km/h [59.1 mph] and at an angle of 26.7 degrees was 2.4 m [7.8 ft]. Earlier testing with a 2040 kg [4,500 lb] passenger sedan resulted in a lateral deflection of 3.5 m [11.5 ft]. This system will generally redirect vehicles in the 820–2000 kg [1,800–4,400 lb] ranges, but some discussion is needed to distinguish between design variations of this system. The steel S75 x 8.5 [S3 x 5.7] post design with 700 mm [28 in.] top rail height has been tested the most extensively of all past designs. In addition to the vehicle range described above, this design has successfully contained and redirected a low front profile car and an 1800 kg [4,100 lb] van.

The cable barrier redirects impacting vehicles after sufficient tension is developed in the cable, with the posts in the impact area offering only slight resistance. However, testing on the S75 x 8.5 [S3 x 5.7] post design has shown that closer post spacing can reduce lateral deflection to some extent. (Prior testing with a 1600 kg [3,500 lb] car at 100 km/h [60 mph] on this design produced deflections of 2.1 m [7 ft] to 3.3 m [11 ft] for associated post spacing of 1.2–4.9 m [4–16 ft]). Several states with extensive experience using cable rail allow a down slope as steep as 1V:2H behind the rail.

Cable barriers placed on the inside of curves require additional deflection before tension develops in the cable. Among agencies using this barrier, guidelines vary regarding maximum curvature allowed. The State of New York installs the S75 x 8.5 [S3 x 5.7] post design on curves with radii up to 220 m [721.5 ft] with standard 4.9 m [16 ft] spacing and with radii up to 135 m [442.5 ft] with 3.8 m [12 foot] post spacing.

Primary advantages of cable guardrail include low initial cost, effective vehicle containment and redirection over a wide range of vehicle sizes and installation conditions, and low deceleration forces upon the vehicle occupants. It is also advantageous in snow or sand areas because its open design prevents drifting on or alongside the roadway. Major drawbacks to the use of cable guardrail include the comparatively long lengths of barrier which are non-functional and in need of repair following an impact, the clear area needed behind the barrier to accommodate the design deflection distance, its reduced effectiveness on the inside of curves, and its sensitivity to correct height installation and maintenance.

5.4.1.2 W-Beam (Weak Post)

The barrier system shown in Figure 5.6 behaves very much like a cable guardrail, i.e., the posts serve primarily to hold the rail at the proper elevation and they separate readily when struck. The W-beam rail then redirects impacting vehicles as it is placed in tension. Post size is identical to the cable system (S75 x 8.5 [S3 x 5.7]) design, but posts are

TABLE 5.2 Roadside barriers and approved test levels

BARRIER SYSTEM (with AASHTO-AGC-ARTBA designation)	TEST LEVEL
FLEXIBLE SYSTEMS	
• 3-Strand Cable (Weak Post) (SGR01a & b)	TL-3
• W-Beam (Weak Post) (SGR02)	TL-2
• Modified W-Beam (Weak Post) (SGR02)	TL-3
• Ironwood Aesthetic Barrier	TL-3
SEMI-RIGID SYSTEMS	
• Box Beam (Weak Post) (SGR03)	TL-3
• Blocked-out W-Beam (Strong Post)	
-Steel or Wood Post with Wood or Plastic Block (SGR04a & b)	TL-3
-Steel Post with Steel Block (SGR04a)	TL-2
• Blocked-out Thrie-Beam (Strong Post)	
-Wood or Steel Post with Wood or Plastic Block (SGR09a & c)	TL-3
• Modified Thrie-Beam (Strong Post) (SGR09b)	TL-4
• Merritt Parkway Aesthetic Guardrail	TL-3
• Steel-Backed Timber Guardrail	TL-3
RIGID SYSTEMS (CONCRETE & MASONRY):	
• New Jersey Concrete Safety Shape	
-810 mm [32 in.] tall (SGM11a)	TL-4
-1070 mm [42 in.] tall (SGM11b)	TL-5
• F-Shape Barrier	
-810 mm [32 in.] (SGM10a)	TL-4
-1070 mm [42 in.] (SGM10b)	TL-5
• Vertical Concrete Barrier	
-810 mm [32 in.]	TL-4
-1070 mm [42 in.]	TL-5
• Single Slope Barrier	
-810 mm [32 in.]	TL-4
-1070 mm [42 in.]	TL-5
• Ontario Tall Wall Median Barrier (SGM12)	TL-5
• Stone Masonry Wall/Precast Masonry Wall	TL-3

installed at 3.8 m [12 ft] centers to match the W-beam hole pattern. The suggested distance from the ground to the centerline of the rail is 550 mm [22 in.].

Impact performance: The W-beam weak-post system has been successfully tested to NCHRP Report 350, TL-2 with a 2000 kg [4,400 lb] pickup truck. The dynamic lateral deflection in the 2000 kg [4,400 lb] test (26.1-degree impact angle, 71 km/h [45 mph]) was 1.4 m [4.6 ft]. This barrier failed a 99.8 km/h [62 mph], 24.4-degree impact angle crash test; thus, it is not classified as a TL-3 barrier. However, the barrier is approved for a TL-2 barrier system with the mounting height of 550 mm [22 in.] to the center of the rail.

This system may retain some degree of effectiveness after minor hits due to the rigidity of the W-beam rail element and thus has some advantage over a cable system. As with the cable system, lateral deflection can be reduced to some extent by closer post spacing. This system, as with all barriers having a relatively narrow restraining width, is somewhat vulnerable to vaulting or vehicle underride caused by incorrect mounting height or irregularities in the approach terrain.

A modification to the standard weak post design was developed and successfully tested to NCHRP Report 350, TL-3. The modifications included raising the mounting height to 820 mm [32.3 in.] and adding W-beam back-up plates at each post. All rail splices were centered mid-span between posts rather than at a post location. The dynamic deflection was measured at 2.12 m [6 ft 11.5 in.] when the barrier was hit by a 2000 kg [4,400 lb] pickup truck at 102.4 km/h [63.6 mph] and a 26.5-degree impact angle.

FIGURE 5.5 Three-strand cable barrier

FIGURE 5.6 Weak post W-beam barrier

FIGURE 5.7 Ironwood aesthetic guardrail

5.4.1.3 Ironwood Aesthetic Guardrail

The barrier shown in Figure 5.7 is a weak-post system consisting of S75 x 8.5 [S3 x 5.7] steel posts on 2000 mm [6.5 ft] centers supporting a composite rail element. This composite rail consists of 203 mm [8 in.] diameter routed round-wood posts with a 6 mm [0.25 in.] thick steel channel embedded on the back side to provide the needed tensile strength of the system. The top height of the rail is 660 mm [26 in.]. The steel support posts are faced with 171 mm [6 .75 in.] diameter timber posts above the ground line to present an all-timber appearance from the roadway. The Ironwood guardrail, which is a proprietary design, was successfully tested to NCHRP Report 350, TL-3. Maximum dynamic deflection resulting from the 100 km/h [60 mph] impact with the 2000 kg [4,400 lb] pickup truck at an angle of 25 degrees was 1640 mm [5 ft 4.5 in.].

There are currently no crashworthy terminal designs for the Ironwood guardrail or any of the other aesthetic barriers described in this Chapter. Acceptable end treatments include anchoring any of these barriers in a backslope (see Section 8.2.3) or flaring the barrier to the edge of the clear zone established for a particular project.

5.4.1.4 Box Beam (Weak Post)

Figure 5.8 shows a typical installation of a box beam rail (SGR03 system). Resistance in this system is achieved through the combined flexure and tensile stiffness of the rail. Posts near the point of impact are designed to break or tear away, thereby distributing the impact force to adjacent posts.

Impact performance: This system was successfully crash tested to NHCRP 350, TL-3. Dynamic lateral deflection in the 2000 kg [4,400 lb] pickup truck test (25.5-degree impact angle, 95.2 km/h [59 mph]) was 1.15 m [45 in.].

This system shares the same sensitivities to mounting height and irregularities in terrain as the weak-post W-beam systems. The suggested distance from the ground to centerline of rail is 610 mm [24 in.].

5.4.1.5 Blocked-Out W-Beam (Strong Post)

Strong-post W-beam is the most common barrier system in use today. It consists of steel posts (SGRO4a) as shown in Figure 5.9 or wood posts (SGR04b) as shown in Figure 5.10 that support a W-beam rail element that is blocked

FIGURE 5.8 Weak post box beam barrier

out from the posts with routed timber, steel, or recycled plastic spacer blocks. These blocks minimize vehicle snagging on the posts and reduce the likelihood of a vehicle vaulting over the barrier by maintaining rail height during the initial stages of post deflection. Resistance in this and all strong post systems results from a combination of tensile and flexural stiffness of the rail and the bending or shearing resistance of the posts.

Several acceptable strong post W-beam designs are in use. The spacer blocks are typically timber or recycled plastic with a 150 mm [6 in.] width to match each post's dimensions. One of the most commonly used designs, the steel post guardrail system with steel blocks, failed to meet the NCHRP Report 350 evaluation criteria at TL-3 when the pickup truck snagged on a post and subsequently overturned. However, this system remains acceptable as a TL-2 barrier. In order to provide a TL-3 barrier with steel posts, 150 mm x 200 mm [6 in. x 8 in.] routed wood or plastic blocks of similar dimensions should be used as a substitute for the steel blocks. Steel post W-beam using 150 mm x 150 mm [6 in. x 6 in.] routed wood or plastic blocks also met Report 350 evaluation criteria but with some reduction in performance. Individual designs for these and other strong-post W-beam variations are shown in Appendix B.

The standard length for timber posts has been increased to 1830 mm [6 feet] to match the length of steel posts, however, the recent Report 350 tests used the original 1625 mm [5 ft 4 in.] posts and either length remains acceptable. The original height to the top of the rail for strong post W-beam was 685 mm [27 in.]. This was slightly modified when the height measurement was changed from the top of the rail to the center of the rail with the adoption of metric units. A 550 mm [21.5 in.] height to the center of the rail translated to a 706 mm [28 in.] top height. Either top rail height is considered acceptable. A design improvement suggested for new installations of this and other strong-post guardrail systems is deletion of the rectangular post bolt washers. These washers are not necessary for system strength over the normal range of expected impacts. Furthermore, during severe impacts producing large post deflections, it is desirable that the rail element separates from the posts as they rotate back and downward. This keeps the railing height relatively constant and

FIGURE 5.9 Steel post W-beam with wood block-outs

FIGURE 5.10 Wood post W-beam with wood block-outs

FIGURE 5.11 Wood post thrie-beam barrier

reduces the likelihood that an impacting vehicle will vault the barrier. Use of these washers in strong post transition sections is optional.

Impact performance: Based primarily on testing of the two common designs noted above, this system is effective at redirecting vehicles in the 820–2000 kg [1,800–4,400 lb] range. The wood post (SGR04b) system with wood blocks passed the NCHRP Report 350 TL-3 test with a 2000 kg [4,400 lb] pickup truck (24.3-degree impact angle, 100.8 km/h [62.5 mph]). The maximum lateral deflection was 0.8 m [31.5 in.]. A steel post system with a 150 mm x 200 mm [6 in. x 8 in.] routed wood block also passed the NCHRP Report 350 TL-3 test with the 2000 kg [4,400 lb] pickup truck (25.5-degree impact angle, 101.5 km/h [63 mph]). The maximum lateral deflection of this system was 1.0 m [3.3 ft].

Strong post barrier systems usually remain functional after moderate to low speed impacts, thereby minimizing the need for immediate repair.

5.4.1.6 Blocked-Out Thrie-Beams

There are three types of thrie-beam barriers that have been tested under NCHRP Report 350. These barriers are discussed in the following subsections.

5.4.1.6.1 Blocked-Out Thrie-Beam (Wood Strong Post)

The SGR09c thrie-beam system, shown in Figure 5.11 and in Appendix B, is a stronger version of the blocked-out W-beam rail. The additional corrugation in the thrie-beam rail element stiffens the system, making it less prone to damage during low- and moderate-speed vehicle impacts. It also allows higher mounting of the rail, which increases its ability to contain vehicles larger than standard passenger cars under some impact conditions. The SGR09c system, with wood posts and blockouts, has been success-

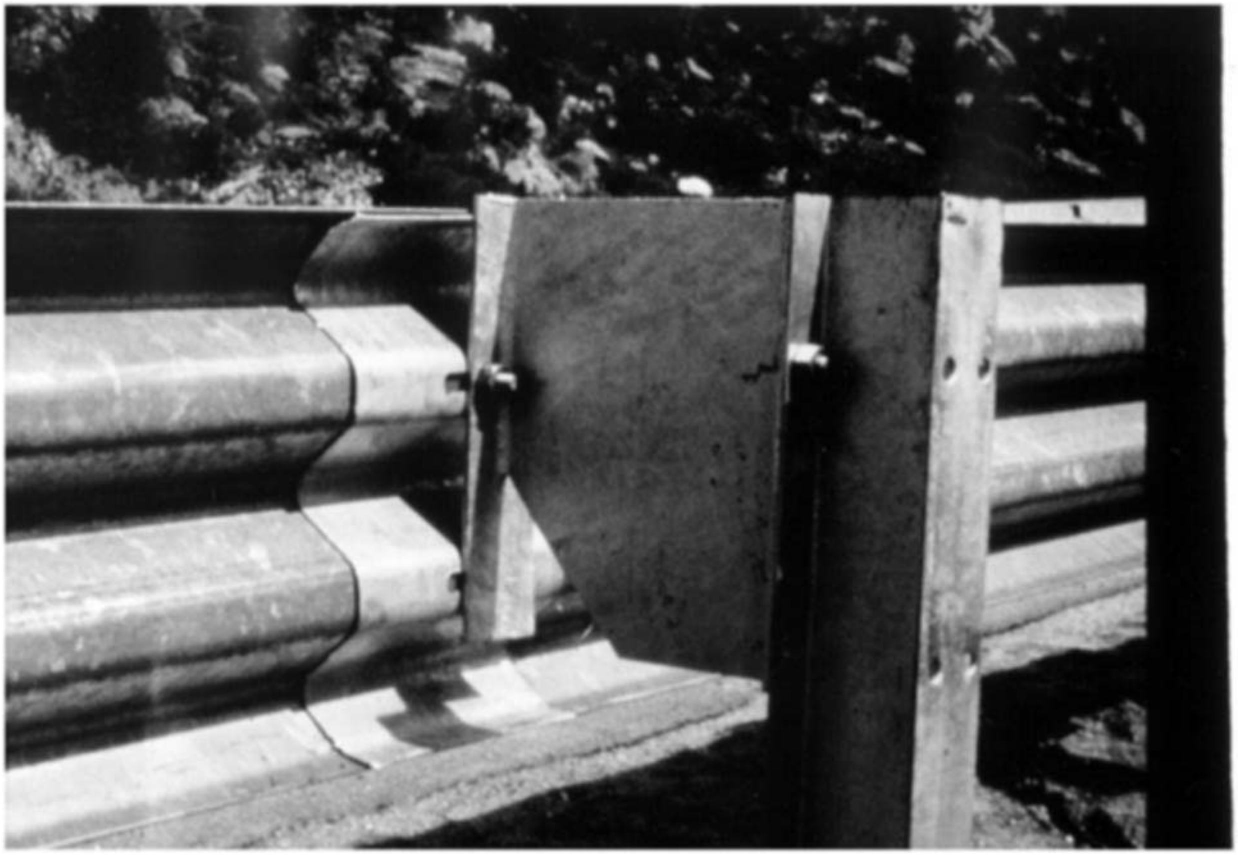


FIGURE 5.12 Modified thrie-beam guardrail

fully crash tested with a top railing height of 810 mm [32 in.].

Impact performance: The SGR09c thrie-beam system with wood posts and wood blocks was successfully crash tested to NHCRP 350, TL-3. The dynamic lateral deflection observed during strength testing with a 2000 kg [4,400 lb] pickup truck impacting at 99.6 km/h [61.9 mph] and at an angle of 23.6 degrees was 0.68 m [26.75 in.].

5.4.1.6.2 Blocked-Out Thrie-Beam (Steel Strong Post)

The original SGR09a system, which used a steel blockout, failed to pass NCHRP Report 350, TL-3. The original steel spacer blocks have been replaced with routed timber or routed, recycled plastic with a 150 mm [6 in.] width to match the post dimensions. This barrier, as with the thrie-beam wood-post system, has been successfully crash tested with a top railing height of 810 mm [32 in.].

Impact Performance: The SGR09a thrie-beam system with steel posts and wood blocks was successfully crash tested to NCHRP Report 350, TL-3. The dynamic lateral deflection observed during strength testing with a 2000 kg [4,400 lb] pickup truck impacting at 98.2 km/h [61.0 mph] and at an impact angle of 24.4 degrees was 0.58 m [23 in.]. In an earlier test to establish an upper performance limit, the original barrier (with steel offset blocks) contained and redirected a 9100 kg [20,000 lb] school bus, although it failed to keep the school bus upright during the test.

5.4.1.6.3 Modified Thrie-Beam

To improve the performance of thrie-beam guardrail for heavy vehicles, a steel block-out was developed. This 355 mm [14 in.] deep steel block-out has a triangular notch cut from its web (see SGR09b in Figure 5.12). This block-out design allows the lower portion of the thrie-beam and the flange of the steel block-out to bend inward during a crash, keeping the rail face nearly vertical in the impact zone as the posts are pushed backwards. This raises the height of the rail and further minimizes the likelihood of a vehicle rolling over the barrier. Other modifications to the standard thrie-beam design that have been incorporated

FIGURE 5.13 Merritt Parkway aesthetic guardrail

into this barrier include omitting rectangular post bolt washers and increasing the top of rail height to 860 mm [34 in.].

Impact performance: This system has been successfully crash tested to NHCRP 350, TL-4 with a 2000 kg [4,400 lb] pickup truck (100 km/h [60 mph], 25-degree impact angle) and an 8000 kg [18,000 lb] single unit truck. Earlier tests with a 9100 kg [20,000 lb] school bus (90 km/h [56 mph], 15-degree impact angle), and a 14,500 kg [32,000 lb] intercity bus (97 km/h [60 mph], 14-degree impact angle) were also successful. Dynamic lateral deflection in the 2000 kg [4,400 lb] pickup truck and the school bus test were 0.6 m [2 ft] and 0.9 m [3 ft], respectively.

Repair costs for all of the thrie-beam systems may be considerably less than other metal beam guardrail systems because the thrie-beam is not significantly damaged in shallow-angle impacts. Even for moderate to severe crashes, the barrier may remain partially functional and does not usually require immediate repair. Also, thrie-beam is generally easier to install and maintain than a W-beam/rubrail system, where a higher effective barrier height is the design goal.

5.4.1.7 Merritt Parkway Aesthetic Guardrail

The Connecticut Department of Transportation developed and tested an aesthetic steel-backed timber rail supported by W150 x 22.5 [W6 x 15] steel posts on 2896 mm [9.5 ft] centers. The rail element consists of 152 mm x 305 mm [6 in. x 12 in.] timber beams backed with 152 mm wide x 9.5 mm thick [6 in. x $^3/_8$ in.] steel plates and splices to provide tensile continuity. Height to the top of the rail is 762 mm [30 in.]. A wood block measuring 100 mm deep x 200 mm wide x 280 mm high [4 in. deep x 8 in. wide x 11 in. high] separates the rail element from the posts to minimize snagging. This barrier, shown in Figure 5.13, was tested to NCHRP Report 350, TL-3. Design deflection with the 2000 kg [4,400 lb] pickup truck was 1150 mm [46 in.] when the system was tested without a curb and 1020 mm [40 in.] when tested behind a 100 mm [4 in.] curb. Either option is acceptable for use.

FIGURE 5.14 Steel-backed timber guardrail

5.4.1.8 Steel-Backed Timber Guardrail

The semi-rigid barrier shown in Figure 5.14 was developed as an aesthetic alternative to conventional guardrail systems. The system consists of a 150 mm x 250 mm [6 in. x 10 in.] wood rail backed with a 10 mm [$^{3}/_{8}$ in.] thick steel plate and supported by 250 mm x 300 mm x 2100 mm [10 in. x 12 in. x 7 ft] timber posts. The rail is offset from the posts by 100 mm x 225 mm x 300 mm [4 in. x 9 in. x 12 in.] wooden spacer blocks. The steel plate provides needed tensile strength to the system. The wood members provide a more rustic appearance than the steel and concrete normally used in barriers. Thus, this railing is often specified for use along roads under the jurisdiction of the National Park Service and similar agencies.

Impact performance: This railing was originally crash tested under NCHRP Report 230 with an 820 kg [1,800 lb] vehicle at 81 km/h [50 mph], 20-degree impact angle, and with a 2040 kg [4,500 lb] vehicle at 81 km/h [50 mph], 25-degree impact angle. More recently, this design was tested to NCHRP Report 350 at TL-3 with a 2000 kg [4,400 lb] pickup truck impacting at 98.7 km/h [61.3 mph] at an impact angle of 24.5 degrees. The dynamic deflection of the barrier was reported to be 580 mm [23 in.]. Detailed design information on this barrier and on the rough masonry and precast concrete guardwalls can be found on the FHWA's Eastern Federal Lands web site at http://www.efl.fhwa.dot.gov/td.

5.4.1.9 Concrete Barriers

A number of rigid concrete systems have been developed that have varying shapes and heights of 810 mm [32 in.] and 1070 mm [42 in.]. The concrete barriers with a height of 810 mm [32 in.] passed NCHRP Report 350, TL-4, while taller barriers of similar shape with a height of 1070 mm [42 in.] passed NCHRP Report 350, TL-5.

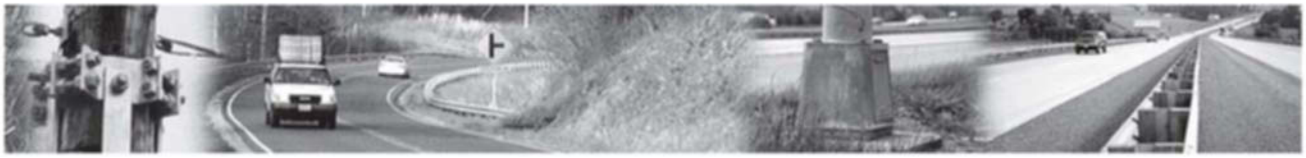
FIGURE 5.15 New Jersey safety-shape barrier

The New Jersey concrete safety shape roadside barrier is a rigid system having a sloped front face and a vertical back face. Except for the back face, the design details and performance of this barrier are identical to the concrete median barrier (CMB) and the reader is referred to Section 6.4.1.7 for a more complete discussion of this design. The New Jersey safety shape barrier (SGM11a and b) shown in Figure 5.15 and the F-shape (SGM10a and b) are both acceptable barrier profiles. Figure C.6 in Appendix C shows the differences between these similarly shaped barriers. The F-shape exhibited better performance in crash tests with 820 kg [1,800 lb] cars and 8000 kg [18,000 lb] single unit trucks. Constant slope concrete barriers (shown in Figure 6.8 as a median barrier), developed by the State of Texas and the State of California, have also been tested with pickup trucks and single unit trucks and found to perform satisfactorily. The reduced cross-section of this roadside barrier (as compared to the CMB) makes it more vulnerable to overturn; therefore, the roadside version usually contains more reinforcing steel and/or a more elaborate footing design unless earth support is available on the back side of the barrier.

Top of barrier height for the basic design is 810 mm [32 in.], but higher designs have been tested and built to obtain redirection of vehicles heavier than passenger cars.

Impact performance: Several of the semi-rigid concrete barriers such as the New Jersey concrete safety shape, F-shape, and constant slope face barriers with 810 mm [32 in.] height have been successfully tested to NHCRP 350, TL-4. The New Jersey safety shape has been the most commonly tested concrete barrier design in past years, and it has generally been tested in the median barrier configuration. A 1070 mm [42 in.] height modified New Jersey safety shape barrier, F-shape barrier, vertical concrete barrier, and the constant slope barrier have been successfully tested to NHCRP 350, TL-5. For example, the 1070 mm [42 in.] New Jersey safety shape barrier redirected a 36,300 kg [80,000 pound] tractor-trailer impacting at an angle of 15 degrees and a speed of 84 km/h [52 mph]. Another median type barrier that has been effectively used as a longitudinal system is the Ontario Tall Wall Median Barrier (SGM12) as shown in Figure 5.16. This 1070 mm [42 in.] New Jersey shape non-reinforced wall system is classified as a high-performance barrier and has TL-5 approval under NHCRP Report 350.

To counteract the overturning moment of trucks with higher centers of gravity or unrestrained loads, walls even higher than 1070 mm [42 in.] can be effective. Some significantly higher barriers have been constructed for special situations with satisfactory results in field application.



Chapter 6
2006 Update

Median Barriers

6.0 OVERVIEW

Median barriers are longitudinal barriers that are most commonly used to separate opposing traffic on a divided highway. They may also be used along heavily traveled roadways to separate through traffic from local traffic or to separate high occupancy vehicle (HOV) lanes from general purpose lanes. Most median barriers are similar to roadside barrier designs described in Chapter 5. However, median barriers, as discussed in this chapter, are those designed to redirect vehicles striking either side of the barrier.

This chapter references the performance requirements for median barriers and contains guidelines for selecting and installing an appropriate barrier system. The structural and safety characteristics of selected median barriers, including end treatments and transition sections, are presented. Finally, selection and placement guidelines are included for new construction, and methods are presented for identifying and upgrading existing systems that do not comply with current guidelines.

6.1 PERFORMANCE REQUIREMENTS

The performance requirements for median barriers are identical to those for roadside barriers as stated in Section 5.1. NCHRP Report 350 (10) contains detailed information on the required series of standard crash tests needed to evaluate the performance of longitudinal barriers.

6.2 GUIDELINES FOR MEDIAN BARRIER APPLICATION

Guidelines for the use of median barrier have evolved over the past 40 years. The primary guidance that has been used was based on a limited number of studies that examined vehicle encroachment paths on flat sideslopes. The basic premise in this guidance was that 80 percent of errant motorists were able to recover within 10 m [30 ft] of the traveled way. As a result, median barriers were not typically used in areas with medians that are more than 10 m [30 ft] wide. However, in the 1990s, several states noticed an increase in the number of cross-median crashes and developed new guidelines for their highways that expanded the use of median barrier. Some states adopted policies for installing median barrier based on median widths ranging from 10 m [30 ft] to 23 m [75 ft]. A nationwide survey of cross-median crashes in several states was conducted by FHWA in 2004, and based on responses received from over 25 states, it was found that there are a significant percentage of fatal cross-median crashes occurring where median widths exceed 10 m [30 ft]. While the survey found that some cross-median crashes occurred in medians in excess of 60 m [200 ft] wide, approximately two thirds of the crashes occurred where the median was less than 15 m [50 ft] in width.

It is recognized that the increased use of median barriers has some disadvantages. The initial costs of installing a barrier can be significant. In addition, the installation of a barrier will generally increase the number of reported

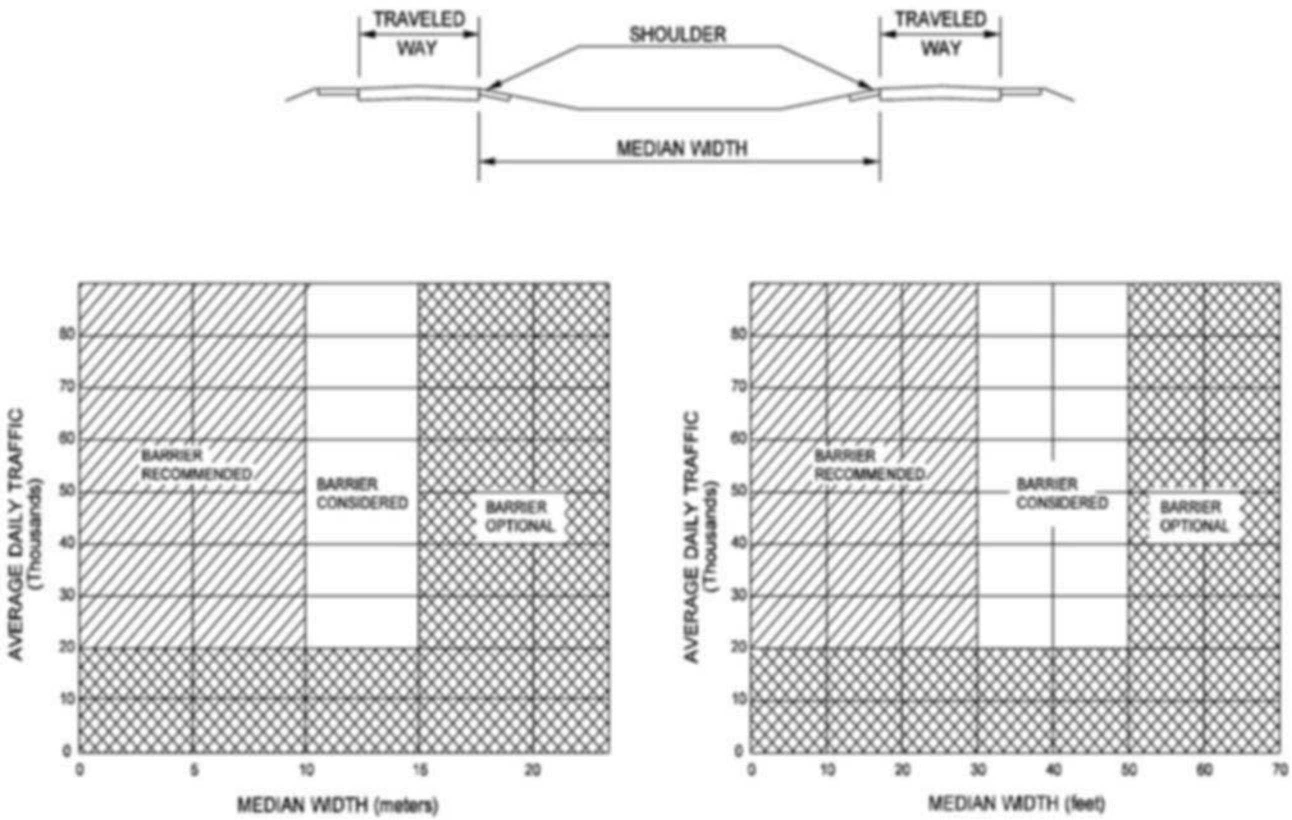


Figure 6.1 Guidelines for median barriers on high-speed, fully controlled-access roadways

crashes as it reduces the recovery area available. As a result, there could be increased maintenance costs to repair the barrier as well as increased exposure to the maintenance crews completing the repairs. Another concern associated with the installation of a median barrier is that it will limit the options of maintenance and emergency service vehicles to cross the median. In snowy climates, a median barrier may also affect the ability to store snow in the median. There may be other environmental impacts depending on the grading required to install the barrier. For these reasons, a one-size-fits-all recommendation for the use of median barrier is not appropriate.

Studies (6, 9) have shown that median barriers can significantly reduce the occurrence of cross-median crashes and the overall severity of median-related crashes. With the potential to reduce high-severity crashes, it is recommended that median barrier be considered for high-speed, fully controlled-access roadways that have traversable medians as shown in Figure 6.1.

Figure 6.1 recommends median barrier on high-speed, fully controlled-access roadways for locations where the median is 10 m [30 ft] in width or less and the average daily traffic (ADT) is greater than 20,000 vehicles per day. For locations with median widths less than 15 m [50 ft] and where the ADT is less than 20,000 vehicles per day, a median barrier is optional. However, the facility should be designed to facilitate future barrier placement if there are significant increases in average daily traffic and/or a history of cross-median crashes is experienced. For locations where median widths are greater than 10 m [30 ft] but less than 15 m [50 ft], and where the ADT is greater than 20,000 vehicles per day, a cost/benefit analysis or an engineering study evaluating such factors as traffic volumes, vehicle classifications, median crossover history, crash incidents, vertical and horizontal alignment relationships, and median/terrain configurations may be conducted at the discretion of the transportation agency to determine the appropriate application for median barrier installations. For locations with median widths equal to or greater than 15 m [50 ft], a barrier is not normally considered except in special circumstances such as a location with a significant history of cross-median crashes.

Each transportation agency has the flexibility to develop its particular median barrier guidelines. For example, California completed a detailed study in 1997 that suggested medians as wide as 23 m [75 ft] with traffic volumes in excess of 60,000 vehicles per day would be candidates for a median barrier study (3). California uses a crash study warrant to identify sections of freeways that may require the installation of a median barrier. This warrant requires a minimum of 0.31 cross-median crashes per kilometer [0.50 cross-median crashes per mile] of any severity per year, or 0.075 fatal crashes per kilometer [0.12 fatal crashes per mile] per year. The rate calculation requires a minimum of three crashes occurring within a five-year period.

In some cases, it may be determined that a median barrier is only necessary at locations where there are concentrations of cross-median crashes. For example, the Florida Department of Transportation found that 62 percent of all cross-median crashes occurred within one-half mile and 82 percent occurred within one mile of interchange ramp termini (1).

Median barriers are sometimes used on high-volume facilities, which do not have fully controlled access. As indicated in Figure 6.1, these median barrier guidelines were developed for use on high-speed, fully controlled-access roadways. Utilizing these guidelines on roadways that do not have full access control requires the need for engineering analyses and judgment, taking into consideration such items as, right-of-way constraints, property access needs, number of intersections and driveway openings, adjacent commercial development, sight distance at intersections, barrier end termination, etc. Therefore, trying to apply these guidelines to roadways that do not have full access control can be rather complex in many locations.

Special consideration should be given to barrier needs for medians separating roadways at different elevations. The ability of an errant driver leaving the higher roadway to return to the road or to stop diminishes as the difference in elevation increases. Thus, the potential for crossover crashes increases. For such sections, the clear-zone criteria given in Chapter 3 should be used as a guideline for establishing barrier need. Section 6.6.1 addresses the placement of barrier on sloped medians.

6.3 PERFORMANCE LEVEL SELECTION PROCEDURES

As with roadside barriers, most median barriers have been developed, tested, and installed with the intention of containing and redirecting passenger vehicles and pickup trucks. Some highway agencies have identified locations where heavy vehicle containment was considered necessary and have designed and installed high-performance median barriers having significantly greater capabilities than commonly used designs. Factors most often considered in reaching a decision on such barrier use include

- high percentage or large average daily number of heavy vehicles,
- adverse geometrics (horizontal curvature), and
- severe consequences of vehicular (or cargo) penetration into opposing traffic lanes.

Section 6.4 includes information on the maximum size of vehicle that has been successfully crash tested for each median barrier system described in that section.

6.4 STRUCTURAL AND SAFETY CHARACTERISTICS OF MEDIAN BARRIERS

This section identifies selected median barrier systems and summarizes the structural and safety characteristics of each. It is subdivided into length-of-need sections, transitions, and end treatments. Characteristics unique to each system are emphasized.

6.4.1 Crashworthy Median Barrier Systems

As with roadside barriers, median barriers can be categorized as flexible, semi-rigid, or rigid. This section includes descriptions and performance capabilities of crashworthy median barrier systems that have met the criteria of NCHRP Report 350 (10), beginning with flexible median barriers and ending with rigid systems. Also included is a discussion of a moveable barrier system that can be used for special traffic situations, such as reversible traffic lanes, where periodic relocation of the barrier is required. Some barriers that are designed to restrain and redirect large vehicles are also identified and included in this section. The barriers to be addressed and their corresponding test levels are:

- Weak-Post, W-Beam Guardrail — TL-3
- 3-Strand Cable, Weak Post — TL-3
- High-Tension Cable Barrier — TL-3 *
- Box-Beam Barrier — TL-3
- Blocked-Out W-Beam (Strong Post)
  - Steel or Wood Post with Wood or Plastic Block — TL-3
  - Steel Post with Steel Block — TL-2
- Blocked-Out Thrie-Beam (Strong Post)
  - Wood or Steel Post with Wood or Plastic Block — TL-3

*Several of the High-Tension Cable Barriers have versions that were successfully tested at TL-4.

Figure 6.2 Weak-post, W-beam median barrier

- Modified Thrie-Beam TL-4
- Concrete Barrier
  - Vertical Wall
    810 mm [32 in.] tall TL-4
    1070 mm [42 in.] tall TL-5
  - New Jersey Shape
    810 mm [32 in.] tall TL-4
    1070 mm [42 in.] tall TL-5
  - Single Slope
    810 mm [32 in.] tall TL-4
    1070 mm [42 in.] tall TL-5
  - F-Shape
    810 mm [32 in.] tall TL-4
    1070 mm [42 in.] tall TL-5
- Quickchange® Moveable Barrier (including SRTS and CRTS) TL-3

Each of these systems is described in the following subsections. The mounting heights included in these descriptions are measured from the ground to the top of the rail, cable, or barrier. The generally accepted variations from nominal heights are 75 mm [3 in.] for the rigid and semi-rigid systems and 50 mm [2 in.] for the flexible systems. Additional information on individual median barrier systems, including design details, is included in Appendix C.

6.4.1.1 Weak-Post, W-Beam

The weak-post, W-beam system, shown in Figure 6.2, is similar to the roadside weak-post system described in Chapter 5. The mounting height to the top of the W-beam is 840 mm [33 in.] and the design deflection ranges from 1.5 m to 2.1 m [5 ft to 7 ft]. The weak-post system is sensitive to height variations and should not be used as a median barrier where terrain irregularities exist. Because the W-beam does not interlock with a vehicle's sheet metal, the likelihood of going over or under the rail is increased if the bumper height at impact is in a range that is higher or lower than normal. Consequently, this system is recommended only in relatively flat, traversable medians without curbs or ditches that could affect vehicle trajectory. It should not be used where frost heave or erosion is likely to alter the beam mounting height relative to the shoulder beyond 50 mm [2 in.]. A proper anchorage at each end is critical.

6.4.1.2 Three-Strand Cable

This flexible barrier is similar to the roadside cable barrier described in Chapter 5 except, when used in a median, the middle cable is installed on the opposite side of each post from the other two and the spacing between the cables is different.

A cable barrier should be used only if adequate deflection distance exists to accommodate approximately 3.5 m [12 ft] of movement; i.e., the median width should be at least 7 m [24 ft] if the barrier is centered. Shortening the post spacing as discussed in Chapter 5 can reduce deflection distances. Proper anchorage at the ends is critical.

Cable systems must be installed and maintained as close to the design height as feasible in order to function properly. To accommodate both larger and smaller vehicles, the lower cable on the NCHRP Report 350 design is 530 mm [21 in.] and the top cable 770 mm [30 in.] above the ground. The center cable is 650 mm [25.5 in.] above the ground. There are several different designs of the three-strand cable median barrier in use throughout the country. When selecting one of these systems, the designer is encouraged to review and consider the compliance testing and/or in-service performance history.

Although the cable barrier is relatively inexpensive to install and performs well when hit, it must be repaired after each hit to maintain its effectiveness. Consequently, its use in areas where it is likely to be hit frequently, such as on the outside of sharp curves, is not recommended. A typical installation is shown in Figure 6.3.

6.4.1.3 High-Tension Cable Barrier

There are several proprietary, high-tension cable barrier systems that have been developed and are increasing in use. These systems are installed with a significantly greater tension in the cables than the generic three-cable system discussed in the previous section. There are several differences in the performance of the high-tension systems compared to the generic three-cable system. The deflection of these systems is reduced to 2 m [6.6 ft] to 2.8 m [9.2 ft] depending on the system and the post spacing. The high-tension systems also result in less damage to the barrier and in many cases, the cables remain at the proper height after an impact that damages several posts. The posts can be installed in sleeves in the ground to facilitate removal and replacement.

Figure 6.3 Three-strand cable median barrier

There are currently five high-tension cable barrier systems that have been accepted by FHWA as meeting NCHRP Report 350, Test Level 3 conditions. A modified version of the Brifen, CASS, and Gibraltar systems have been successfully tested at Test Level 4 conditions. All of these use weak posts to support the cables. However, they each utilize a unique post design. The following are the currently accepted high-tension cable barrier systems:

Brifen Wire Rope Safety Fence
The Brifen system, manufactured by Brifen USA, Inc., uses three or four cables, one placed in a slot on the post and the others intertwined between the posts. See Figure 6.4.

CASS
The Cable Safety System (CASS), manufactured by Trinity Industries, Inc., uses three cables that are placed in a slot on the posts and separated by spacer blocks. See Figure 6.5.

U.S. High-Tension Cable System
The U.S. High-Tension Cable System, manufactured by the Nucor Steel Marion Inc., uses three cables attached to U-channel steel posts by unique hook bolts. See Figure 6.6.

Blue Systems (Safence)
This system is a four-cable design. For a median barrier all four cables are centered within the top portion of slotted posts. Safence is a Swedish design that is not currently manufactured in the United States. Oklahoma DOT has installed and is evaluating a short section of this barrier. See Figure 6.7.

Gibraltar Cable Barrier System
The Gibraltar Cable Barrier System uses C-posts to support three cables. A steel hairpin and lock plate are used to attach the cables to the posts. See Figure 6.8.

6.4.1.4 Box-Beam Median Barrier

The box-beam median barrier shown in Figure 6.9 is considered a semi-rigid barrier and is similar to the roadside box beam described in Chapter 5. Its design deflection distance is approximately 1.7 m [5.5 ft]. As with the weak-

Figure 6.4 Brifen Wire Rope Safety Fence

Figure 6.5 The Cable Safety System (CASS)

Figure 6.6 U.S. High-Tension Cable System

Figure 6.7 Safence Cable Barrier System

post W-beam, this system is most suitable for use in traversable medians having no significant terrain irregularities.

6.4.1.5 Blocked-Out W-Beam (Strong Post)

Blocked-out W-beam median barrier may be installed using either wood or steel posts. When constructed with blocks made of either wood or one of several recycled plastics, either post design qualifies as meeting NCHRP Report 350, TL-3. A steel post design using steel blocks has been accepted as a TL-2 barrier. The strong-post, W-beam system, shown in Figure 6.10, has been extensively used to prevent crossover crashes in relatively narrow medians.

Since these systems are semi-rigid, meaning their design deflection distances are in the 0.6- to 1.2-m [2- to 4-ft] range, they have typically been used in medians approximately 3 m [10 ft] or more in width. The normal mounting height of the rail is 700 mm [27 in.]. However, in some locations, designers have specified 760-mm [30-in.] mounting heights in an attempt to better contain large vehicles. This mounting height is higher than their roadside barrier counterparts. This taller design has not been tested, but it falls within the generally accepted tolerance of 75 mm [3 in.] from nominal height for a W-beam, strong-post guardrail. To minimize post-snagging problems with the higher mounting heights, a separate rubrail has sometimes been added to the design. A rubrail has also been added when the W-beam is placed behind a curb, typically on structure approaches. Most state agencies have used a structural steel channel or tube for the rubrail, but occasionally a separate W-beam centered 250 mm [10 in.] above grade is specified.

Strong-post, W-beam median barriers generally cause higher forces on impacting vehicles and their occupants than do flexible systems, but they do not usually require immediate repair to remain functional except after very severe impacts.

6.4.1.6 Blocked-Out Thrie-Beam (Strong Post)

This NCHRP Report 350, TL-3 system is similar in most respects to the blocked-out W-beam median barrier but is capable of accommodating a larger range of vehicle sizes due to its increased beam depth. Posts may be either wood or steel with blocks of either wood or one of several approved recycled plastics. The use of thrie-beam also elimi-

Figure 6.8 Gibraltar Cable Barrier System

Figure 6.9 Box-beam median barrier

Figure 6.10 Strong-post W-beam median barrier

nates the need for a separate rubrail. Design deflection for this barrier is in the range of 0.3 to 0.9 m [1 to 3 ft], and its typical mounting height is 810 mm [32 in.].

6.4.1.7 Modified Thrie-Beam Median Barrier

Using the spacer blocks developed in conjunction with the modified thrie-beam roadside barrier described in Chapter 5 can significantly enhance performance of the thrie-beam median barrier. This barrier successfully contained and redirected an 8000-kg [18,000-lb] single-unit truck impacting at a nominal speed of 80 km/h [50 mph] and an impact angle of 15 degrees. The roadside version of this barrier also contained and redirected an 18000-kg [40,000-lb] intercity bus under the same conditions. Thus, both the single-faced roadside design and the double-faced median barrier design are considered to be TL-4 longitudinal barriers. The modified thrie-beam median barrier is shown in Figure 6.11.

6.4.1.8 Concrete Barrier

Concrete barrier is the most common rigid median barrier in use today. Its popularity is based on its relatively low life-cycle cost, generally effective performance, and its maintenance-free characteristics. Concrete barrier designs vary in shape, construction type, and reinforcement.

Research has shown that variations in the face of the concrete barrier can have a significant effect on barrier performance (4). Concrete barrier shapes that meet the NCHRP Report 350 criteria are the New Jersey and F-shapes, the single-slope barrier (two variations in slope), and the vertical wall. These shapes when adequately designed and reinforced may all be considered TL-4 designs at the standard height of 810 mm [32 in.] and TL-5 designs at heights of 1070 mm [42 in.] and higher.

The New Jersey shape and F-shape barriers are commonly referred to as "safety shapes." Figure C.6, Appendix C, compares dimensions of these two barriers. The safety-shape concrete barriers were designed to minimize damage to vehicles as a result of low-angle impacts and reduce the occupant impact forces as compared to a vertical wall. The critical variable for these barriers is the height above the road surface of the break between the upper and lower slope. If this break is higher than 330 mm [13 in.], the chances of a vehicle overturning are increased, particularly for compact and subcompact automobiles. Although both shapes are effective in safely redirecting impacting vehicles, research indicates that the F-shape, which has the slope break at 250 mm [10 in.], may perform better for small vehicles with respect to vehicle roll than the New Jersey shape.

The basic New Jersey and F-shape have an overall height of 810 mm [32 in.]; this includes provision for a 75-mm [3-in.] future pavement overlay, reducing the height to 735 mm [29 in.] minimum. When total overlay depths are expected to exceed 75 mm [3 in.] or when an 810-mm [32-in.] height is considered inadequate, the total height of the concrete must be adjusted. This adjustment must be made above the slope breakpoint. The height extension may follow the slope of the upper face if the barrier is thick enough or adequately reinforced at the top, or the extension may be vertical. A height extension may also be considered for use as a screen to block headlight glare from opposing traffic lanes.

There are several factors related to safety-shape concrete median barriers that are important to note. For high-angle, high-speed impacts, passenger size vehicles may become partially airborne and in some cases may reach the top of the barrier. Fixed objects, e.g., luminaire supports, on top of the wall may cause snagging or separate from the barrier and fly into opposing traffic lanes. An example of how one state addressed this concern is New York State; they designed and tested a box-beam retrofit that is installed near the top of the upper face of the barrier to limit vehicle climb and to improve performance under these conditions. See Figure 6.13.

Another factor to consider is that, even for shallow-angle impacts, the roll angle toward the barrier imparted to high-center-of-gravity vehicles may be enough to permit contact by the top portion of the cargo box with fixed objects on top of or immediately behind the wall. Bridge piers are one of the common obstacles typically shielded by a concrete safety shape. Unless the barrier is significantly higher than 810 mm [32 in.] or modified as noted above, a bus or tractor trailer is likely to lean enough to strike the pier even though it does not penetrate the barrier. Even the 1070-mm [42-in.] high concrete safety shapes

Figure 6.11 Modified thrie-beam median barrier

Figure 6.12 Concrete safety-shape median barrier

shown in Figures C.7 and C.8, Appendix C, produced significant roll when struck by a 36000-kg [80,000-lb] combination truck at an impact angle of 15 degrees and 80 km/h [50 mph]. This so-called "Tall Wall" barrier is classified as a high-performance barrier. It has been successfully used for many years by the New Jersey Turnpike Authority in its reinforced version and in Ontario without reinforcement (7).

Single-slope concrete barriers have been developed and tested (2). Slopes of 9.1 degrees and 10.8 degrees have been used successfully on these barriers. The primary advantage of this barrier shape is that the pavement adjacent to it can be overlaid several times without affecting the performance of the barrier. The original height of 1070 mm [42 in.] may thus be reduced to 760 mm [30 in.] and still perform as a TL-4 barrier.

Vertical concrete barrier wall can be an effective alternative to the wider safety-shape barriers and can preserve available median shoulder width at narrow locations such as in front of bridge piers. A study of rollovers that resulted from crashes with concrete barriers concluded that the vertical wall offers the greatest reduction in rollover potential. Vehicle damage resulting from the initial crash into a vertical wall may be more extensive. However, occupant risk measurements from full-scale crash testing are comparable and the preservation of shoulder width and reductions in rollover potential are important safety benefits (8).

Many variations exist between highway agencies regarding reinforcing and footing details for concrete median barriers; however, there have been few reported problems with any particular design and a need (or desirability) for a standard detail is not apparent. Research by the California Department of Transportation has shown that a concrete footing is not necessary; the concrete can be cast directly on asphaltic concrete, portland cement concrete, or a well-compacted aggregate base (5). This research also revealed no adverse effects to barrier performance when contraction joints were left to form uncontrolled in lightly reinforced concrete. Longitudinal reinforcement in the upper portion of the barrier stem serves to control the size and scatter of concrete fragments that may occur as a result of severe barrier impacts. Several states use non-reinforced concrete barrier. Shrinkage cracks of up to 20 mm [$^3/_4$ in.] have not affected the operational strength of concrete barriers, and no breakouts have been experienced where the top width is at least 300 mm [12 in.]. In general, if the in-service performance of a State's concrete barrier design reflects desired results, that design may be considered acceptable.

Figure 6.13 New York retrofit of concrete barrier

Figure 6.14 Single-slope concrete median barrier

Figure 6.15 Quickchange® moveable barrier system

Concrete median barrier may be slip-formed, precast, or cast-in-place. Slip-formed barriers are cost-effective where long lengths of barrier can be placed without interruption. Equipment is available to slip-form barriers to a variable height where necessary to fit a stepped-median cross section and where the elevations of adjacent roadways do not vary by more than 0.9 m [3 ft]. Precast construction is sometimes used as an alternate to slip-formed barrier and is frequently used where split median barriers are needed to shield objects such as bridge piers or overhead sign supports. Precast concrete barrier sections can be embedded in or anchored to the pavement to form a rigid barrier. However, several states use an unanchored precast concrete barrier for permanent installations. The unanchored barrier deflects when impacted, reducing the force of impact as compared to a rigid barrier. The deflected barrier requires repositioning, but the effort is less than the repair of any other semi-rigid barrier system. Cast-in-place construction is the most versatile method because forming can be varied to fit atypical situations.

Examples of concrete median barriers are shown in Figures 6.12 and 6.14.

6.4.1.9 Quickchange® Moveable Barrier System

This proprietary portable barrier system, shown in Figure 6.15, is composed of a chain of modified F-shape concrete barrier segments 940 mm [37 in.] in length that can be readily shifted laterally. Steel rods run the length of each segment, and specially designed hinges are attached to each end, which are then joined by pins. The top of each segment is T-shaped to allow pick up by a special vehicle and lateral movement from 1.2 to 5.5 m [4 to 18 ft]. The T slot is engaged by the vehicle conveyor system and the segment is lifted from the road. Continuous lengths of the barrier are transported on conveyor wheels through an elongated S curve, moved across the roadway, and set down to form a new parallel lane. Transfer speeds of 8 to 16 km/h [5 to 10 mph] are obtained depending on the lateral distance of movement. The design has met the crash test criteria of NCHRP Report 350, TL-3 with a deflection of 1.4 m [4.5 ft].

Several variations of the moveable barrier design have also been tested and approved as meeting NCHRP Report 350, TL-3. The Narrow Quickchange® Moveable Barrier

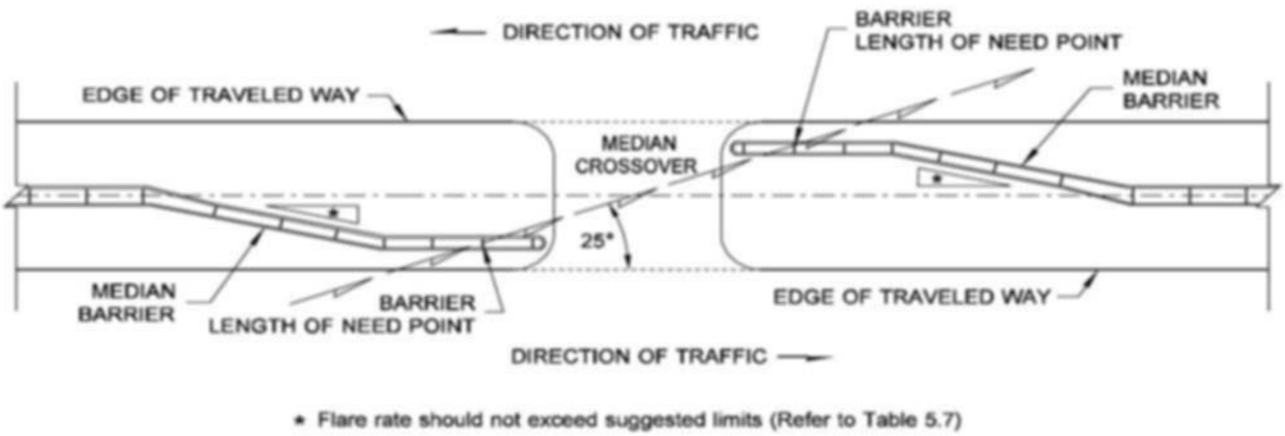


Figure 6.16 Barrier termination at permanent openings

consists of a steel casing that is filled with concrete and has a width of 305 mm [12 in.] as compared to a width of 457 mm [18 in.] for the standard Quickchange® Moveable Barrier. This system has a deflection of 0.9 m [3 ft]. Two other systems, known as the Steel Reactive Tension System (SRTS) and the Concrete Reactive Tension System (CRTS) are similar to the narrow and standard Quickchange® Moveable Barriers, respectively, except that an improved connection is used between modules. This connection contains spring-loaded hinges that keep the individual segments in tension and reduce the dynamic deflection of the system to 0.7 m [2.3 ft].

The Quickchange® Moveable Barriers may be used in construction zones on high-volume freeways where, due to construction operations and a desire to maintain traffic capacity, traffic lanes are opened and closed frequently. The system requires energy, time, and resources to set up the barriers initially; however, it allows a work zone to be quickly created and protected during periods of low traffic flow, and can be changed back to full lane utilization during the busy daytime period.

The system may also be used on roadways and bridges with unbalanced directional traffic, such as commuter or tourist routes. Once set up, the barrier can be moved rapidly to provide additional capacity in the direction of heavy traffic flow.

6.4.2 End Treatments

As with roadside barriers, median barriers must also be introduced and terminated safely. Therefore, all median barrier end treatments installed in locations where impacts are likely must be crashworthy. In addition, they generally must safely redirect vehicles impacting from the rear of the terminal or crash cushion where opposite direction hits are likely. See Chapter 8 for a discussion of the end treatments that are available.

Because of the more severe crashes that normally result from impacts with terminals and the cost of terminals when compared to the barrier itself, openings or breaks in median barriers should be kept to a minimum. Where permanent openings are required, the barrier ends should be shielded as described previously or, if the median is sufficiently wide, flared, or offset such that the upstream barrier effectively shields the end of the downstream section of barrier. The latter condition can be satisfied if the minimum angle (measured parallel to the roadway) from the upstream end to the offset downstream end is 25 degrees as shown in Figure 6.16.

If an emergency opening is required, for example, to route traffic around a crash that requires the roadway to be temporarily closed, there are proprietary devices that have been developed and tested to NCHRP Report 350, TL-3, that can be used to provide a temporary opening. The BarrierGate® manufactured by Energy Absorption Systems, Inc. and the SafeGuard® Gate system, manufactured by Barrier Systems, Inc. are used in conjunction with a concrete safety-shape median barrier to provide a temporary opening through the barrier when needed by emergency vehicles or to temporarily reroute traffic. The BarrierGate® system (see Figure 6.17) consists of two half-gates made from thrie-beam rail elements that slide along a steel track system. The BarrierGate® is opened and closed by an electronic control mechanism that can be manually overridden in the event of a power failure. The SafeGuard® system is a heavily reinforced steel barrier that can be disconnected from the concrete barrier. The system can be moved on wheels that are raised and lowered either manually or with compressed air.

Figure 6.17 BarrierGate®

6.4.3 Transitions

Transition sections are needed between adjoining median barriers having significantly different deflection characteristics, e.g., between a semi-rigid median barrier and a rigid median barrier, or when a median barrier must be stiffened to shield fixed objects in the median.

Impact performance requirements for median barrier transitions are essentially the same as those for the standard barrier transitions. Special emphasis must be placed on the avoidance of designs that may cause vehicle snagging or excessive deflection of the transition section. Detailed discussion of barrier transitions is included in Chapter 7, "Bridge Railings and Transitions."

6.5 SELECTION GUIDELINES

Once it has been determined that a median barrier is warranted, a specific barrier type must be selected. In general, the most desirable system is one that satisfies performance requirements at the least total life-cycle cost. Table 5.3 summarizes the major factors that should be considered before making a final selection. Each of these factors is described in the following sections.

6.5.1 Barrier Performance Capability

The first decision to be made when selecting an appropriate median barrier concerns the level of performance required. In most cases, a standard barrier capable of redirecting passenger cars and light vans and trucks will be adequate (NCHRP Report 350, TL-3). However, at locations with adverse geometrics, high traffic volumes and speeds, or a significant volume of heavy truck traffic, higher performance level median barriers may be considered, particularly since the result of a heavy vehicle penetrating a median barrier is likely to be catastrophic. The median barriers identified in Section 6.4.1 as TL-4 or higher have an increased capability to contain and redirect large vehicles.

6.5.2 Barrier Deflection Characteristics

Once the desired performance level has been determined, site characteristics often dictate the type of median barrier to install. Relatively wide, flat medians are suited for flexible or semi-rigid barriers, provided the design deflection distance is less than one-half the median width. Narrow medians within heavily traveled roadways usually require a rigid barrier having little or no deflection when hit. Deflection distances for each type of operational median barrier are discussed in Section 6.4.1.

Crash testing and field experience has shown that, during impact, a large truck or similar high-center-of-gravity vehicle will typically lean over and extend for some distance behind the barrier. The clear area that should be provided behind a barrier and beyond its dynamic deflection distance to account for this behavior is called "working width." The designer should consider the working width when locating a median barrier to shield a rigid object, such as a bridge pier or sign support. While it is desirable to avoid having any fixed objects within the working width of the barrier, it is understood that in some instances it will not be practical to provide a separation between the barrier and the object. In critical areas, it may then be desirable to use a higher performing barrier or, for a concrete barrier, to change the barrier height and shape to minimize vehicular overhang in a crash.

6.5.3 Compatibility

The specific type of median barrier selected will also depend to some extent upon its compatibility with other median features, such as luminaire and overhead sign supports and bridge piers. If a non-rigid barrier is used in such cases, crashworthy transition sections must be available to stiffen the barrier locally if the fixed object is within the design deflection distance of the barrier. In addition to acceptable transition designs, a crashworthy end treatment is also necessary if the barrier begins or terminates in a location where it is likely to be struck by an errant motorist. Detailed information on transition sections and end treatments is included in Chapters 7 and 8, respectively.

6.5.4 Costs

Initial costs, repair costs, and future maintenance costs of each candidate median barrier should be carefully evaluated. As a rule, the initial cost of a system increases as rigidity and strength increase, but repair and maintenance costs usually decrease with increased strength. Consideration should also be given to the costs incurred by the motorist as a result of a crash with the barrier. These costs include personal injuries to the driver and occupants as well as property damage to the impacting vehicle. If a barrier can be located in the center of a median where it is less likely to be hit, and repairs do not necessitate closing a lane of traffic, flexible or semi-rigid median barrier may be the best choice. However, if a barrier must be located immediately adjacent to a high-speed, high-volume traffic lane, a rigid barrier requiring no significant maintenance is recommended.

6.5.5 Maintenance

Although the same general maintenance considerations for the selection of a roadside barrier also apply to median barriers, crash maintenance is usually a more important factor. Because median barriers are typically installed closer to the traveled way, one or more high-speed lanes usually have to be closed to repair or replace damaged barriers. This creates a safety concern for both the repair crew and for motorists using the road. Consequently, a rigid barrier system (usually concrete) is the barrier of choice in many locations, particularly for high-volume urban freeways and expressways where the barrier must be located in close proximity to the traffic lane.

6.5.6 Aesthetic and Environmental Considerations

As with the roadside barriers, aesthetic concerns are seldom an overriding consideration in the selection of an appropriate median barrier. In those instances where a "natural" barrier is required, care must be exercised to ensure that structural and performance requirements remain adequate.

Environmental factors that warrant consideration are similar to those summarized in Chapter 5 for roadside barriers.

6.5.7 Field Experience

To make effective decisions regarding the type of barrier to install on new construction, each highway agency should have a process to monitor and evaluate the performance and maintenance characteristics of its existing installations. Information from maintenance personnel is essential for designers to select the most cost-effective system.

6.6 PLACEMENT RECOMMENDATIONS

All of the barriers included in Section 6.4.1 are capable of containing and redirecting their respective design vehicles if they are properly installed in the field. Without exception, all traffic barriers perform best when an impacting vehicle has all of its wheels on the ground at the time of impact, and its suspension system is neither compressed nor extended. Thus, a major factor to consider in the lateral placement of a median barrier is the effect of the terrain between the edge of the traveled way and the barrier on the vehicle's trajectory. Two other significant factors affecting barrier performance are the flare rate of the barrier, especially at transition sections, and the treatment of rigid objects in the median. A discussion of each of these factors follows.

6.6.1 Terrain Effects

Terrain conditions between the traveled way and the barrier can have a significant effect on the barrier's impact performance. Curbs and sloped medians (including superelevated sections) are two prominent features that deserve attention. See Chapter 5 for a discussion on the use of curbs with a barrier. The slopes in the median can affect the performance of the barrier as the vehicle suspension is compressed and the drainage swales can impart a roll moment on a traversing vehicle. A vehicle that traverses one of these features prior to impact may go over or under the barrier or snag on the support posts of a strong-post system.

The most desirable median is one that is relatively flat (slopes of 1V:10H or flatter) and free of rigid objects. If warranted, the barrier can then be placed at the center of the median. When these conditions cannot be met, placement guidelines are necessary.

Figure 6.18 shows three basic median sections for which placement guidelines are presented. In each section, it is assumed that a median barrier meets guidelines for installation. Section I applies to depressed medians or medians with a ditch section. Section II applies to stepped medians or medians that separate travel ways with significant differences in elevation. Section III applies to raised medians, or median berms.

Section I—The slopes and the ditch section should first be checked by the criteria in Chapter 3 to determine if the guidelines suggest the installation of a roadside barrier. If both slopes require shielding, i.e., the ditch is non-traversable (Illustration 1), a roadside barrier should be placed near the shoulder on each side of the median ("b" and "d"). If only one slope requires shielding, e.g., S2, a median barrier should be placed at "b." In this situation, a rigid or semi-rigid barrier is suggested, and a rubrail should be installed on the ditch side of the barrier to prevent vehicles that have crossed the ditch from snagging on a post-and-beam railing system. There has also been some anecdotal evidence that a vehicle traveling up a slope steeper than1V:6H before contacting the barrier may override it. Research is planned to quantify possible placement concerns when a rigid or semi-rigid barrier is located on one side of a traversable, sloped median. If neither slope requires shielding but either one or both are steeper than 1V:10H (Illustration 2), a median barrier should generally be placed on the side with the steeper slope. For example, if

S2 = 1V:6H and S3 = 1V:10H,

the barrier would be placed at "b." A rigid or semi-rigid system is suggested in this situation. If both slopes are relatively flat (Illustration 3), a median barrier may be placed at or near the center of the median (at "c") if vehicle override is not likely. Any type of median barrier having an appropriate test level for the application can be used provided its dynamic deflection is not greater than one-half the median width.

Although any median barrier is likely to perform best when it is installed on relatively flat terrain, cable barriers have been shown to perform effectively when placed on a 1V:6H sideslope when the vehicle travels down the slope prior to impact. However, based on recent crash reports, some vehicle types, when striking a cable barrier from behind after traveling across a ditch, can underride the barrier. Computer simulation and limited full-scale testing on 1V:6H slopes have shown that the barrier will redirect vehicles after traversing the ditch when it is placed within 0.3 m [1 ft] (either side) of the ditchline. However, when the current configuration of cable median barrier was placed 1.2 m [4 ft] from the ditchline, a test with a passenger sedan showed that after crossing the ditch the vehicle reached the cables with its suspension compressed; the bumper passed under the lowest cable, and the vehicle continued through the cable median barrier with no redirection. Computer simulation has predicted that when the barrier is placed eight feet from the ditch bottom, the vehicle will be contained. Based on this testing and more recent simulation studies, it appears that maximum redirection can be achieved with the current configuration if the area from 0.3 m [1 ft] to 2.4 m [8 ft] from the ditchline on 1V:6H slopes is avoided. Additional research is needed to further support the recommended offset distances for this and other slopes and to determine what practical modifications to the barrier can be developed to enhance its performance in locations that may be less than optimal. These placement guidelines apply to all cable barriers, including high-tension designs and four-cable systems.

Since most reported penetrations have involved passenger vehicles with relatively low front profiles impacting at high speeds and high angles, it is not considered cost-effective to reposition existing cable barrier that has been installed within this area unless a recurring crash problem is evident.

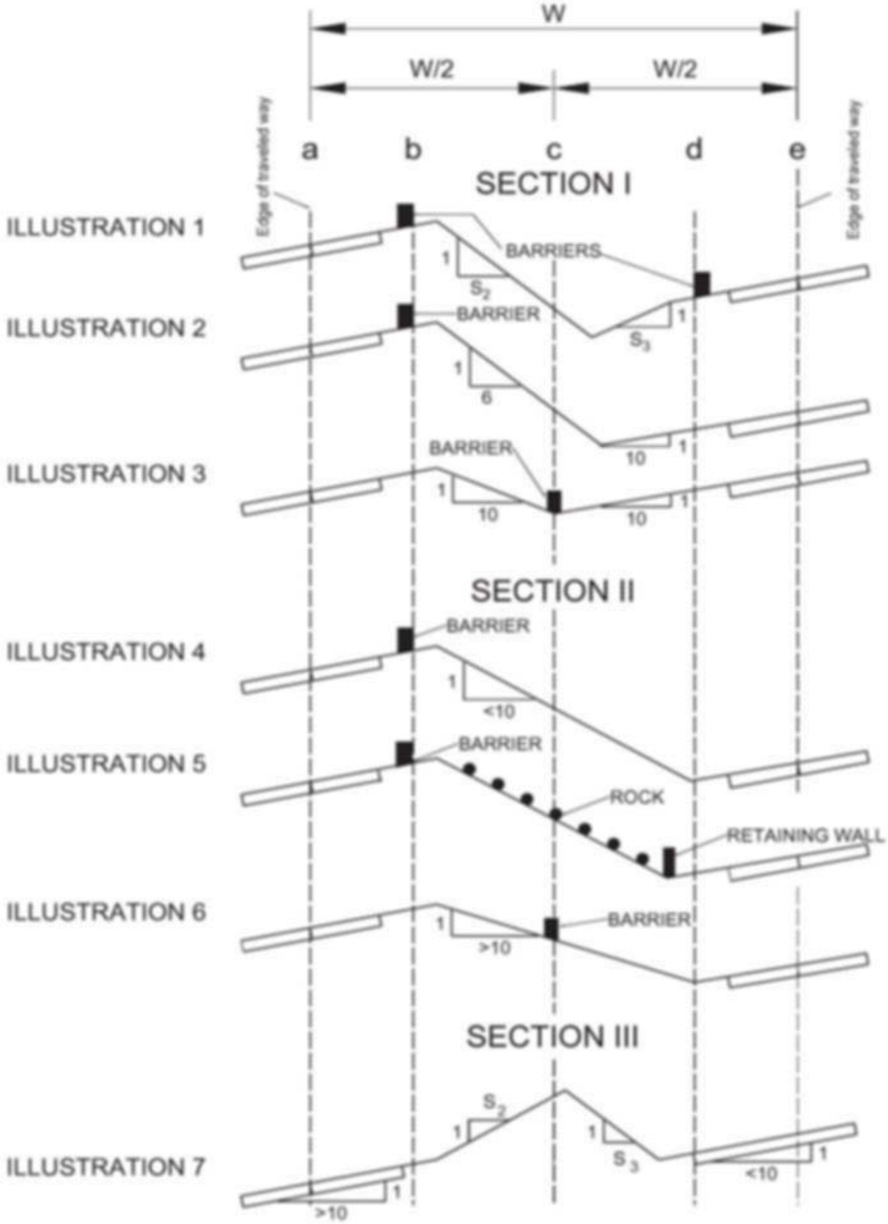


Figure 6.18 Recommended barrier placement in non-level medians

Section II—If the embankment slope is steeper than approximately 1V:10H (Illustration 4), a median barrier should be placed at "b." If the slope contains obstacles or consists of a rough rock cut (as discussed in Chapter 3) a roadside barrier should be placed at both "b" and "d" (Illustration 5). It is not unusual for this section to have a retaining wall at "d." If so, it is suggested that the base of the wall be contoured to the exterior shape of a concrete median barrier. If the cross slope is flatter than approximately 1V:10H, a barrier could be placed at or near the center of the median (Illustration 6).

Section III—Placement criteria for median barriers on this cross section (Illustration 7) are not clearly defined. Research has shown that such a cross section, if high enough and wide enough, can redirect vehicles impacting at relatively shallow angles. However, this type of median

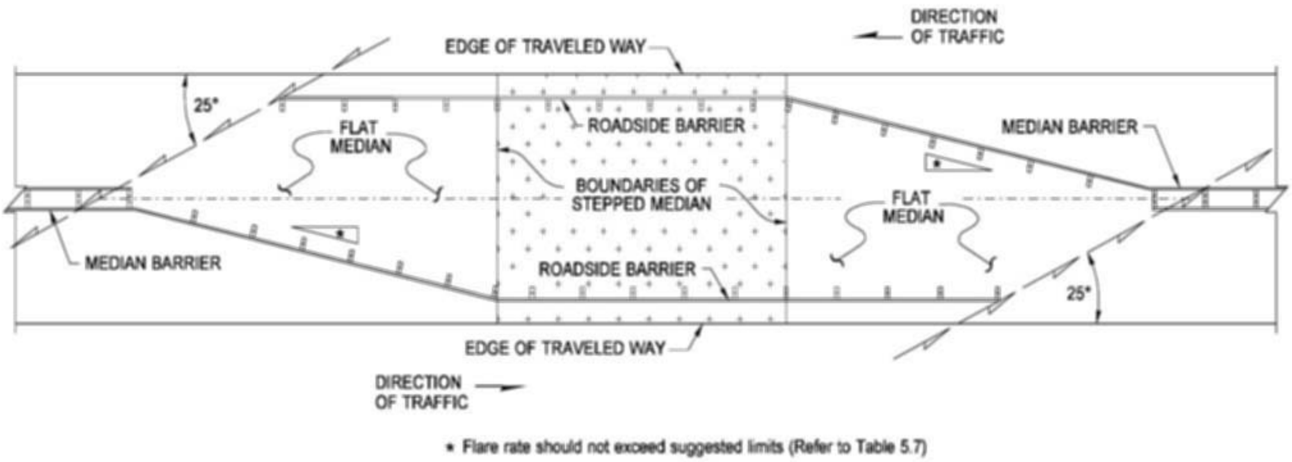


Figure 6.19 Example of a split median barrier layout

design should not generally be construed to be a barrier or to provide positive protection against crossover crashes.

If slopes are not traversable (rough rock cut, etc.), a roadside barrier should be placed at "b" and "d." If retaining walls are used at "b" and "d," it is recommended that the base of the wall be contoured to the exterior shape of a standard concrete barrier.

When the guidelines suggest installing a median barrier, it is desirable that the same barrier be used throughout the length of need, and that the barrier be placed in the middle of relatively flat medians that have slopes that are 1V:6H or flatter. However, it may be necessary to deviate from these guidelines in some cases. For example, the median in Section I of Figure 6.18, where the roadways are stepped (on significantly different elevations), may require a barrier on both sides of the median. If a single median barrier is installed upstream and downstream from the section, it may be necessary to "split" the median barrier as illustrated in Figure 6.19. Most of the operational median barriers can be split this way, especially box beams, W-beam types, and concrete barrier.

6.6.2 Fixed Objects within the Median

In many locations, an obstacle such as a rigid object may be located in a median. If a median barrier is not being installed and the object is outside of the clear zone for one direction of traffic, the barrier should be treated as a roadside barrier (see Chapter 5). Appropriate flare rates should be used for the approaching traffic side of the barrier and, if the deflection distance for the barrier cannot be provided, a transition may be necessary to stiffen the barrier in advance of the object. In addition, when the object is within the clear zone for both directions, the object and back side of the barrier needs to be shielded as well.

Typical examples of objects that are often located in a median are bridge piers and overhead sign support structures. If shielding for both directions of travel is necessary and if the median is flat (side slopes less than approximately 1V:10H), two means of protection are suggested. In the first case, the designer should investigate the possible use of a crash cushion to shield the object. A second suggestion is to employ either semi-rigid or rigid barriers with crash cushions or end treatments to shield the barrier ends as illustrated in Figure 6.20. If semi-rigid systems are used, the distance from the barrier to the obstruction should be greater than the dynamic deflection of the barrier. If a concrete barrier is used, the barrier can be placed adjacent to the obstruction unless there is a concern for a high-center-of-gravity vehicle striking the obstruction when contact with the barrier causes the top of the vehicle to lean over the railing.

6.7 UPGRADING SYSTEMS

Some existing median barriers do not meet suggested performance levels. Older barriers usually fall into one of two categories, namely, those that have structural inadequacies and those that are functionally inadequate.

Table 5.9 provides a checklist for evaluating the structural adequacy of roadside barriers. The same factors can be applied to median barriers. Persons inspecting existing installations should stay abreast of current traffic barrier designs and guidelines as well as promising new research findings. Of course, there is no substitute for field data or crash records to evaluate the performance of a system.

States are encouraged to adopt policies that consider modification or replacement of barrier systems that do not meet current guidelines. It is recognized that this action is not always cost-effective, therefore decisions regarding treatment of existing systems must be based on a case-

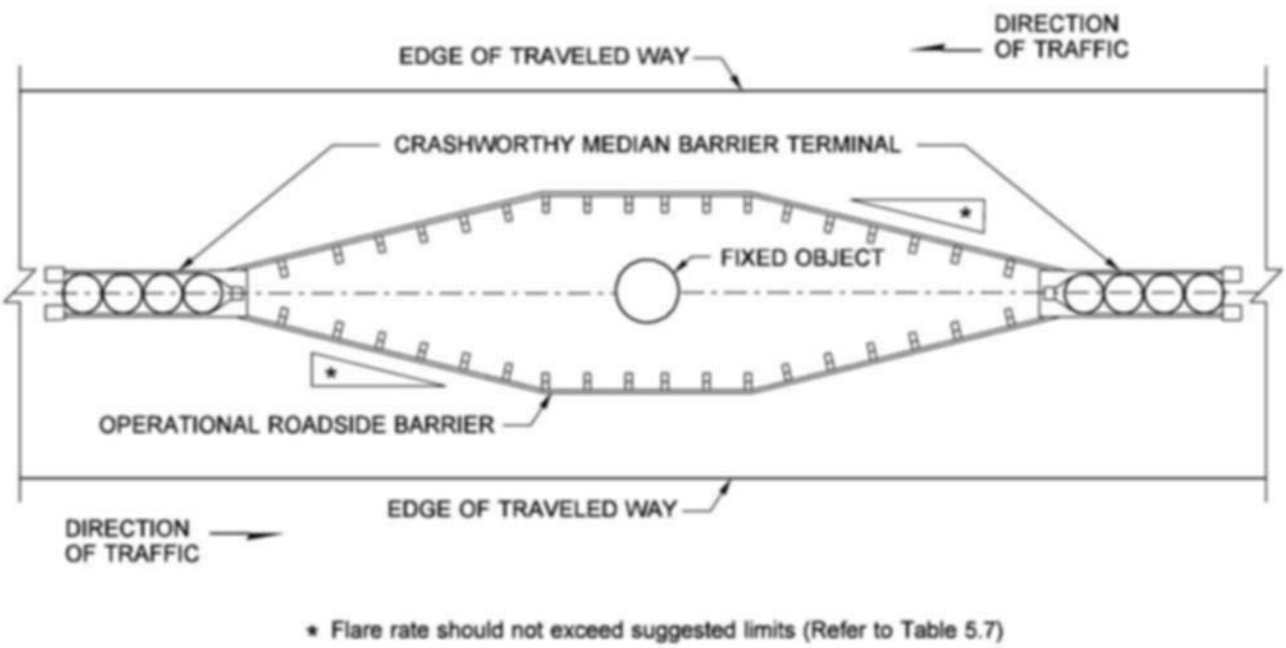


Figure 6.20 Suggested layout for shielding a rigid object in a median

by-case analysis considering upgrade costs, repair and maintenance costs, and potential crash frequency and severity. Table 5.9 may also be used to evaluate the functional adequacy of existing barriers. If the barrier is placed in a depressed median or a median with surface irregularities, it may not function properly. If improperly located, corrective measures should be considered. If necessary, the barrier can be moved near the shoulder's edge or returned to a position in which the approach terrain to the barrier is no steeper than the criteria suggest. Another possible solution would be to extend the shoulder to the lateral distance desired and place the barrier on the shoulder. Steep flare rates for approach and transition sections should be flattened to conform to the criteria recommended in Table 5.7.

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Chapter 7
Bridge Railings and Transitions

7.0 OVERVIEW

A bridge railing is a longitudinal barrier intended to prevent a vehicle from running off the edge of a bridge or culvert. Normally they are constructed of a metal or concrete post and railing system, a concrete safety shape, or a combination of metal and concrete. Most bridge railings differ from roadside barriers in that bridge railings are an integral part of the structure (physically connected) and are usually designed to have virtually no deflection when struck by an errant vehicle.

This chapter summarizes the performance and structural requirements for bridge railings and presents examples of each of the six test levels defined in NCHRP Report 350 (1) for longitudinal barriers. It also addresses selection and placement guidelines for new construction and includes examples of some typical retrofit designs for older bridges having substandard railings. Finally, it addresses bridge railings and roadside barriers as a complete system and provides general information on appropriate transition sections between the two barrier types.

The information presented here is intended only to summarize selected sections of the current AASHTO *Standard Specifications for Highway Bridges* (2) and the *AASHTO LRFD Bridge Design Specifications* (3). Detailed information on analytic design procedures, design loadings, and materials specifications can be found in those documents.

7.1 PERFORMANCE REQUIREMENTS

The AASHTO *Standard Specifications for Highway Bridges* requires that bridge railings meet specific geometric criteria and be capable of resisting applied static loads without exceeding allowable stresses in any of their component members. These specifications do not presently mandate that a bridge railing designed to AASHTO standards be crash tested prior to its use. However, the Federal Highway Administration (FHWA) requires all bridge railings used on the National Highway System to be a crash-tested design.

The *AASHTO LRFD Bridge Design Specifications* provide the most current guidance regarding performance requirements for railings for new bridges and for rehabilitated bridges to the extent that railing replacement is determined to be appropriate. NCHRP Report 350 crash test criteria were used to develop the design criteria contained in the *AASHTO LRFD Bridge Design Specifications*.

Existing bridge railings designed to criteria contained in the AASHTO *Standard Specifications for Highway Bridges* and that may have been crash tested under previous guidelines may be acceptable for use on new or reconstruction projects through evaluation of their in-service performance. For existing bridge rails, individual states should develop a guideline for retention, upgrading, or both retention and upgrading of the in-place rails based on a safe, cost-effective approach. See Section 7.7, Upgrading of Bridge Railings, for additional guidance.

7.2 WARRANTS

Virtually all structures require some type of railing; however, on many small structures on low-speed, low-volume roadways, a railing designed to full AASHTO standards may be neither necessary nor desirable. A rigid railing requires approach guardrail and a transition section between barrier types. This full treatment may not be cost-effective on bridge-length culverts, and alternate treatments should be considered. Such treatments could include extending the structure and leaving the edges unshielded or using a less expensive, semi-rigid type railing.

The owner should develop the warrants for the bridge site. A bridge railing should be chosen to satisfy the concerns of the warrants as completely as possible and practical. Refer to Section 13 of the *AASHTO LRFD Bridge Design Specifications* for guidance in the development of warrants.

When a bridge also serves pedestrians, cyclists, or both, a barrier to shield them from vehicular traffic may be warranted. The need for a pedestrian or cyclist railing should be based on the volume and speed of roadway traffic, the number of pedestrians or cyclists using the bridge, and conditions on either end of the structure.

7.3 TEST LEVEL SELECTION PROCEDURES

As with other traffic barriers, the current design criteria for bridge railings relate primarily to standard size automobiles and pickup trucks and result in the selection of a design meeting NCHRP Report 350, TL-3. Test requirements are the same for a bridge rail as those for a longitudinal barrier as described in Chapter 5.

Several state highway agencies and the FHWA have recognized that it may be desirable in certain situations to design and install railings which can contain and redirect heavy vehicles such as buses and trucks. Although penetration of any railing by a vehicle is potentially hazardous to its occupants, locations where vehicular penetration of a railing system could be particularly hazardous to others as well should be given careful evaluation before deciding on the type of railing to install.

A second concern that must be considered in selecting a high-performance railing is its effective height. A railing may have adequate strength to prevent physical penetration, but unless it also has adequate height, an impacting vehicle or its cargo may roll over the railing or may roll onto its side away from the railing after redirection.

In addition, the shape of the face of the railing will have a significant effect on its performance. Various safety shapes have been successfully tested according to NCHRP Report 350 criteria. However, a safety-shape concrete railing can cause a large vehicle to roll up to 24 degrees before it contacts the upper edge of the railing. Thus, a vertical face may be more desirable whenever heavy vehicle rollover is a primary concern.

At the other extreme, some bridges carry only low traffic volumes at greatly reduced speeds. Bridge railings for these and similar structures may not need to be designed to the same performance level as railings to be used on high-speed, high-volume facilities. Section 5.3 lists the subjective factors most often considered in the selection of an appropriate test level for traffic barriers, including bridge railings, at a specific location.

7.4 CRASH-TESTED RAILINGS

In the past, crash test matrices for bridge railings have differed from those used for other longitudinal barriers. All new tests for bridge railings should be in accordance with the guidelines in NCHRP Report 350. The FHWA maintains a listing of designs that have recently been tested to one of the test levels defined in NCHRP Report 350 and of designs previously tested under earlier guidelines that have been assigned an NCHRP Report 350 equivalent test level.

For illustrative purposes, this section contains photographs and brief descriptions of some of the bridge railings that have been successfully crash tested to one of the six test levels defined in NCHRP Report 350. A complete list of crash-tested bridge railings may be obtained from the FHWA's Office of Highway Safety through its web site: http://safety.fhwa.dot.gov/programs/roadside_hardware.htm.

7.4.1 Test Level 1 Bridge Railings

Since TL-1 designs are tested at impact speeds of only 50 km/h [30 mph], TL-1 bridge railings are not very practical because operating speeds nearly always exceed that level. As a result, there have been almost no bridge railings designed and tested to TL-1. The U.S. Forest Service has done some testing on timber railings for low-speed situations, but most of that effort has been directed towards TL-2 or higher designs.

7.4.2 Test Level 2 Bridge Railings

The side-mounted, thrie-beam bridge railing shown in Figure 7.1 is unique because it is presently the only non-rigid bridge railing that has been successfully crash tested to meet the lower service level performance criteria included in NCHRP Report 230 (4). Intended primarily for use on lower volume secondary roads, the thrie-beam system consists of a thrie-beam rail element, the center of which is mounted 550 mm [22 in.] above the deck on wood or steel posts. Since the thrie-beam railing is designed to deflect on impact, an approach rail transition is not needed be-

FIGURE 7.1 Side-mounted, thrie-beam bridge railing

cause there is not a hard point in the system. Tests with compact and full-sized automobiles at impact speeds of 100 km/h [60 mph] and impact angles of 15 degrees resulted in smooth redirection and no evidence of snagging. A 9000 kg [20,000 lb] school bus impacting at 70 km/h [45 mph] and at a 7-degree angle resulted in similar performance.

Although not tested to NCHRP Report 350 criteria, the side-mounted, thrie-beam bridge railing is considered equivalent to a TL-2 design. Primary advantages to using this system include its relative simplicity and low cost. The post attachment detail is designed to yield on impact rather than cause damage to the bridge deck. Thus, the thrie-beam railing is significantly more forgiving than a rigid design and is likely to be easier to repair after a hit.

7.4.3 Test Level 3 Bridge Railings

The Wyoming Two-Tube Bridge Railing, shown in Figure 7.2, consists of two horizontal rail elements of TS 152 mm x 51 mm x 6.4 mm [6 in. x 2 in. x $^{1}/_{4}$ in.] structural steel supported by fabricated steel plate posts on 3 m [10 ft] centers. The height to the top of the upper rail is 740 mm [29 in.] and the height to the bottom of the lower rail is 405 mm [16 in.]. The faces of the rail elements are flush with the 150 mm [6 in.] concrete curb on which the posts are mounted. This design was tested to NCHRP Report 350, TL-3. A similar design using larger steel tube rail elements and support posts was successfully tested to TL-4. Transition designs from a standard box beam approach rail to both of these bridge rail designs have been tested to TL-3.

7.4.4 Test Level 4 Bridge Railings

There are several bridge railings that have been tested successfully with a single-unit truck impacting at 80 km/h [50 mph] and at 15-degree angle. Four representative TL-4 designs are described in the next subsections.

7.4.4.1 Solid Concrete Bridge Railings

All of the current solid concrete barriers (New Jersey shape and F-shape, single slope and vertical wall) are considered to be TL-4 bridge railings when adequately reinforced and built to a minimum height of 810 mm [32 in.].

FIGURE 7.2 Wyoming two-tube bridge railing

7.4.4.2 Massachusetts S3 Steel Bridge Railing

The S3 Steel Bridge Railing, shown in Figure 7.3, is a beam and post system consisting of three tubular steel rail elements on W150 x 37 [W6 x 25] posts mounted flush on the outside edge of a sidewalk, as shown, or directly on a 200 mm [8 in.] curb when no sidewalk is present. The top rail element is a TS 127 mm x 102 mm x 6.4 mm [5 in. x 4 in. x $^1/_4$ in.] steel tube, the top of which is 1082 mm [42$^1/_2$ in.] above the deck. The lower two railings are TS 127 mm x 127 mm x 6.4 mm [5 in. x 5 in. x $^1/_4$ in.] steel tubes centered 380 mm [15 in.] and 710 mm [28 in.] above the deck, respectively. The S3 Railing also includes 38 mm x 38 mm x 1.6 mm [1$^1/_2$ in. x 1$^1/_2$ in. x $^1/_{16}$ in.] "pickets" bolted to the back of the rail elements on 150 mm [6 in.] centers. These steel tubes satisfy AASHTO pedestrian rail geometrics and provide an aesthetic look to the bridge rail.

7.4.4.3 Wyoming Two-Tube Bridge Railing

A version of the Wyoming Two-Tube Bridge Railing as described in Section 7.4.3 and shown in Figure 7.2 was tested to TL-4 criteria. This design consists of two horizontal rail elements supported by fabricated steel plate posts on 3 m [10 ft] centers. The top rail element is a TS 152 mm x 102 mm x 6.4 mm [6 in. x 4 in. x $^1/_4$ in.] structural steel tube. The bottom rail element is a TS 152 mm x 76 mm x 6.4 mm [6 in. x 3 in. x $^1/_4$ in.] structural steel tube. The height to the top of the upper rail is 830 mm [33 in.] and the height to the bottom of the lower rail is 480 mm [19 in.]. The face of the rail elements are flush with the 150 mm [6 in.] high concrete curb on which the posts are mounted.

7.4.4.4 BR27C

The BR27C, shown in Figure 7.4, is designed to be either sidewalk mounted on a 1.5 m [5 ft] sidewalk with a 200 mm [8 in.] curb or flush mounted on a bridge deck. The total rail height is 1067 mm [42 in.]. The lower portion of the railing consists of a 610 mm [24 in.] high concrete parapet that is a constant 250 mm [10 in.] thick. The upper portion of the railing consists of TS 102 mm x 102 mm x 4.8 mm [4 in. x 4 in. x $^3/_{16}$ in.] A500 grade B structural tubing used as vertical posts spaced at 2 m [6.5 ft] centers. One TS

FIGURE 7.3 Massachusetts S3 steel bridge railing

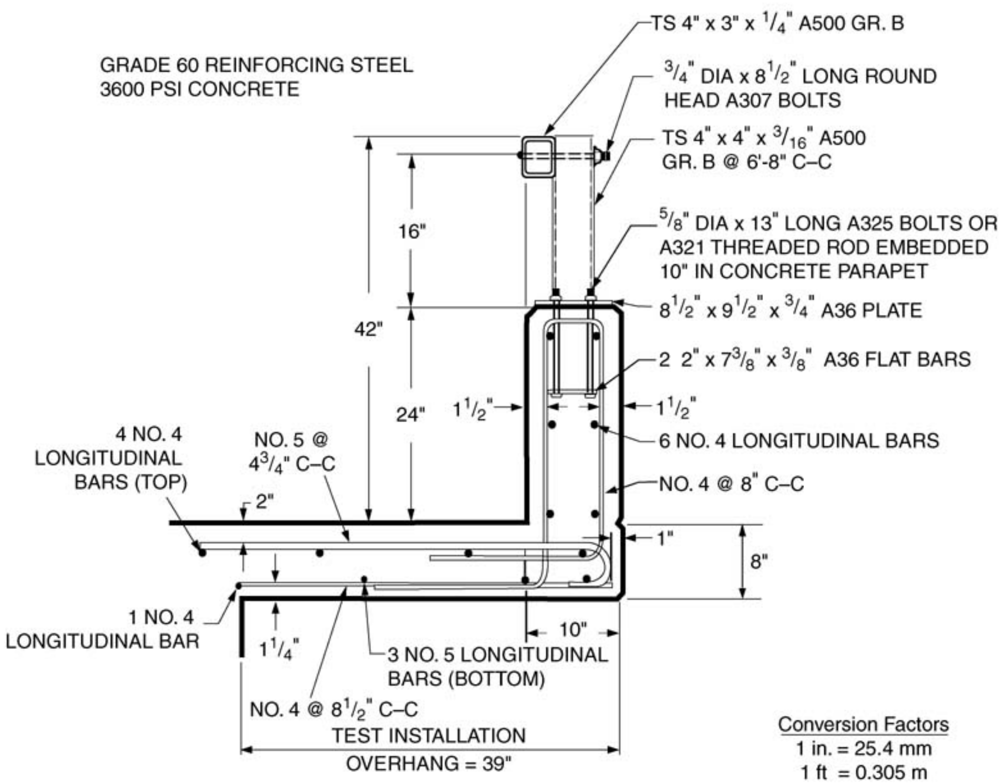


FIGURE 7.4 BR27C on sidewalk

FIGURE 7.5 Tall concrete safety-shape railing

102 mm x 76 mm x 6.4 mm [4 in. x 3 in. x $^{1}/_{4}$ in.] structural tube is used as a horizontal rail element mounted to each post with splices at low moment areas.

7.4.5 Test Level 5 Bridge Railings

All of the current solid concrete barriers (New Jersey and F-shapes, single-slope, and vertical wall) are considered to be TL-5 bridge railings when adequately reinforced and built to a minimum height of 1070 mm [42 in.]. The concrete safety shape shown in Figure 7.5 is one of the most common bridge railings used on new construction. Identical to concrete median barrier in the shape of its front face, the architectural treatment of the outside face may vary considerably, depending upon its location. Reinforcing of the shape when it is used as a bridge railing is significant. The concrete barrier requires virtually no maintenance for most hits.

7.4.6 Test Level 6 Bridge Railings

The Texas Type TT (Tank Truck) shown in Figure 7.6 is an extremely strong barrier railing that successfully contained and redirected a 36000 kg [80,000 lb] tractor-tank trailer impacting the barrier at 80 km/h [50 mph] at an angle of 15 degrees. This railing is warranted for use in only the most rare situations. The railing as tested consists of a very heavily reinforced and widened concrete safety shape with a massively reinforced continuous concrete member and post. Total railing height is 2290 mm [90 in.]. Although designed and tested as a bridge railing, this cross-section has also been used as a longitudinal barrier in some locations.

7.5 SELECTION GUIDELINES

There are five factors that should be considered in selecting a bridge railing: performance, compatibility, cost, field experience, and aesthetics. Despite the relative importance placed on these factors, the capability of a railing to contain and redirect the design vehicle should never be compromised.

FIGURE 7.6 Texas Type TT (Tank Truck) railing

7.5.1 Railing Performance

Generally, all bridge railings designed in accordance with AASHTO specifications since 1964 have adequate strength to prevent penetration by passenger cars. Many of these railings also provide smooth redirection, although full-scale crash testing has revealed poor performance in some railing designs. Post-crash evaluation of some of the failed systems revealed a lack of design capacity in a detail (such as base plate thickness or a post-to-base plate connection) that adversely affected the capacity of the railing. Bridge railings designed to the current *AASHTO LRFD Bridge Design Specifications* and crash tested to NCHRP Report 350 will provide the best performance.

7.5.2 Compatibility

When the approach roadside barrier significantly differs in strength, height, and deflection characteristics from a bridge railing, a crashworthy transition section, as defined in Section 7.8, is required. It is important to consider the selected bridge railing as a part of the total roadside barrier system that must function effectively as a unit. For urban/suburban roadways with speeds of 70 km/h [45 mph] or less and with continuous raised sidewalks on and off the bridge, bridge rail end treatments and stiffened transitions may not be warranted.

7.5.3 Costs

Costs generally fall into one of three categories: initial construction costs, long-term maintenance costs, and costs resulting from vehicle impacts with the railing. As a general rule, the initial cost of a system increases as its rigidity and strength increases, but it seldom becomes a significant portion of the total bridge construction cost except in the case of extremely long bridges or when a high-performance railing is used. In this case, the railing-to-bridge-deck anchorage requirements may significantly increase the total cost of the structure. This would be particularly true for a high-performance concrete railing that adds considerable dead load to the bridge.

Maintenance costs generally decrease significantly as the strength of railing increases. Some high-performance railings can be essentially maintenance-free unless they are struck by heavy vehicles. Railing designs that are susceptible to impact damage should be standardized to the extent possible so that the availability of replacement parts does not become a major problem. Railings that eliminate or minimize bridge deck damage are very desirable from a maintenance viewpoint.

Crash costs include both damages to vehicles and injury costs to motorists. Generally, more damage is inflicted upon the impacting vehicle and its occupants when the railing is hit if the vehicle is not redirected.

7.5.4 Field Experience

It is important that the in-service performance of any bridge railing that is widely used be evaluated to see if it is working as intended. By reviewing crashes involving bridge railings where available and by documenting damage and repair costs, highway agency personnel can readily determine if a specific design is performing well or if changes could be made to improve railing performance or significantly decrease repair costs.

7.5.5 Aesthetics

While there is no question that an aesthetic bridge railing may be particularly important in scenic areas or along park roads, the safety performance of a railing must not be sacrificed. Some rustic appearing railings have been developed and crash tested to be both effective and acceptable in appearance. Any non-standard bridge railing designed primarily for appearance should be crash tested before being used.

7.6 PLACEMENT RECOMMENDATIONS

A desirable feature of a bridge structure is a full, continuous shoulder so that the uniform clearance to roadside elements is maintained. However, there are many existing bridges that are narrower than the approach roadway and shoulder. When the bridge railing is located within the recommended shy distance (see Table 5.5), the approach railing should have the appropriate flare rate shown in Table 5.7.

Curbs higher than 200 mm [8 in.] in front of bridge railings are to be avoided. In low-speed situations with the bridge railing at the outer edge of the sidewalk, a raised sidewalk may provide some protection for pedestrians; however, a bridge railing between traffic and the sidewalk affords maximum pedestrian protection. A pedestrian railing would then be needed at the outer edge of the sidewalk.

Terminating the bridge railing requires special treatment considerations. Wherever possible, a crash tested transition from the approach guardrail should be attached to the end of the bridge rail. In some restricted, low-speed situations, a tapered end section parallel to the roadway may be used. The taper should be of sufficient length off the end of the bridge so that an impacting vehicle is ramped on and over the sloped end treatment before reaching the outside edge of the structure, yet not extend so far as to intrude on the sight distance of intersecting streets just off the end of the bridge. This method of terminating a railing in low-speed situations is shown in Figure 7.7.

Terminating a bridge railing in rural and high-speed urban areas also requires special treatment considerations. Flaring the end section and the sidewalk away from the roadway is sometimes possible. In instances where this is not practical, a crash cushion or a section of approach guardrail parallel to the roadway with a suitable end terminal may be used. The presence of a curb may adversely affect the performance of this type of end treatment. Termination using parallel approach rail with a suitable end terminal is shown in Figure 7.8.

7.7 UPGRADING OF BRIDGE RAILINGS

This section provides general guidelines for highway agency personnel responsible for identifying and correcting potentially deficient bridge railings.

7.7.1 Identification of Potentially Deficient Systems

Since the primary purpose of a bridge railing is to prevent penetration, it must be strong enough to redirect an impacting vehicle. Bridge railings designed to AASHTO specifications prior to 1964 may not meet current specifications. Most railings properly designed after 1964, if tested, will contain and redirect a 2040 kg [4,500 lb] passenger car impacting at 100 km/h [60 mph] at an angle of 25 degrees. If the capacity of a railing appears questionable, further evaluation should be made to verify critical design details (such as base plate connections, anchor bolts, material brittleness, welding details, and reinforcement development) to ensure that the design meets the intent of the current specifications.

FIGURE 7.7 End treatment for traffic railing on a bridge in low-speed situations

FIGURE 7.8 Terminating traffic barrier on bridge with end terminal

FIGURE 7.9 Inadequate railing strength

Occupant protection is also of considerable importance in a crash. Open-faced railings in particular may cause snagging, which produces high deceleration forces leading to occupant injuries. This type of deficiency can usually be detected best through full-scale crash testing or, in the case of an existing railing, through an analysis of available crash reports.

A third deficiency in many older railing systems is the presence of a curb or walkway between the driving lane and the bridge railing. The curb or the walkway may cause an impacting vehicle to go over the railing or at least strike it from an unstable position and subsequently roll over.

Finally, an adequate approach-rail to bridge-rail transition is essential as discussed in detail in Section 7.8. Figures 7.9 through 7.12 illustrate some of the more common deficiencies found in bridge railings designed before 1964. The next section identifies corrective measures that can be taken to improve the performance of these and similarly deficient systems.

7.7.2 Upgrading Systems

This section discusses only retrofit designs, i.e., changes, modifications, and additions to existing substandard railings that bring these railings up to acceptable performance levels. These retrofit designs may increase the strength of the railing, provide longitudinal continuity to the system, reduce or eliminate undesirable effects of curbs or narrow walkways in front of the bridge rail, and eliminate snagging potential. A retrofit design should also include an acceptable transition from the approach rail to the bridge rail itself.

One of the most common retrofit improvements consists of rebuilding the approach roadside barrier to current standards, including a transition section, and continuing the metal beam rail element across the structure to provide railing continuity. If the existing bridge has a safety curb, the retrofit railing can be blocked out to minimize the possibility of a vehicle ramping over the bridge railing. However, for most high-speed, high-volume roads, retro-

FIGURE 7.10 Lack of continuity in railing

FIGURE 7.11 Snagging potential

FIGURE 7.12 Presence of brush curb

fit designs should be crash tested before they are used. The next sections of this chapter provide information on tested designs that can be used once a determination is made that retrofitting a substandard bridge railing is a cost-effective alternative to leaving an existing railing as is or constructing a new crash-tested railing.

Existing railings that do not meet current standards may sometimes be left in place until the section of highway that includes the bridge is brought to full standards. Until a complete upgrading is done, each existing railing should be evaluated to determine the safest and most cost-effective treatment: retention of the rail, retrofit, or replacement. In general, existing concrete post and open railing systems that predate 1964 must be replaced or retrofitted. However, many existing safety curb and parapet railings are still performing well. Even though they do not meet current full railing strength, they remain functional because they can contain and redirect out-of-control vehicles in all but the most severe impacts.

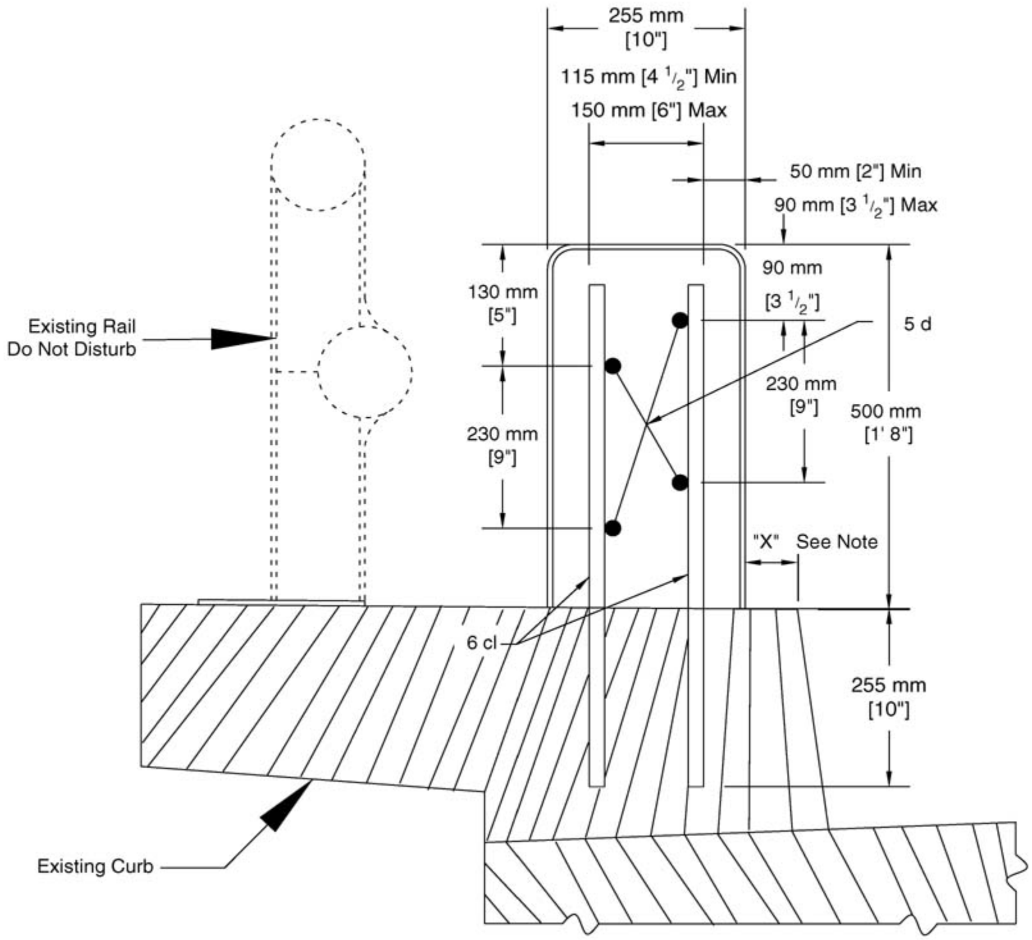
Some specific retrofit concepts that can be adapted to numerous types of deficient designs are:

- Concrete retrofit (safety shape or vertical)
- W-beam/thrie-beam retrofits
- Metal post and beam retrofits

These retrofits are illustrated in Figure 7.13 through Figure 7.15.

7.7.2.1 Concrete Retrofit (Safety Shape or Vertical)

The concrete safety shape that is commonly used for new construction can often be added to an existing substandard bridge railing as an economical retrofit design if the structure can carry the added dead load and if the existing curb and railing configuration can meet the anchorage and impact forces needed for the retrofit barrier. This design is most cost-effective when the existing railing can remain in place and does not require extensive modifica-



Note: On each side of bridge, dimension "X" can be a minimum of 1" and a maximum of 3", but must be constant for full length of bridge. However, approximately 10 linear feet at either end of rail length shall be transitioned to match existing beam guardrail attachment.

FIGURE 7.13 Iowa concrete block retrofit bridge railing

tions. Although a vertically faced retrofit can cause relatively high deceleration forces for sharp angle impacts, its addition to the top of an existing safety curb, as shown in Figure 7.13, creates an effective barrier. Care must be taken to avoid a protruding curb that can cause considerable wheel and suspension system damage and may contribute to vehicular vaulting in shallow angle impacts.

7.7.2.2 W-Beam/Thrie-Beam Retrofits

An inexpensive, short-term solution to the inadequacies of bridge railings designed before 1964 is to carry an approach roadside barrier (W-beam or thrie-beam) across the structure. While this treatment may not bring an existing bridge railing into full compliance with AASHTO design criteria, it can significantly improve the impact performance of a substandard railing. This treatment can be particularly cost-effective on low-volume roadways with structures having timber railings. Testing done in conjunction with the development of the side-mounted thrie-beam bridge railing (see Section 7.4.2) has shown that a bridge railing can be effective even if it deflects several feet upon impact. Continuous metal beam rail across a structure also eliminates one of the major problems of a bridge-rail/approach-rail transition, i.e., adequate anchorage to prevent the approach rail from pulling out on impact. By carrying the approach rail across the bridge, the only transition design elements that remain critical are gradual stiffening and elimination of a snagging potential. These concerns, too, become less critical if the bridge railing is not totally rigid, as is the case on some timber bridges.

FIGURE 7.14 Thrie-beam retrofit (New York)

Both Washington and New York States have successfully crash tested thrie-beam retrofits of existing substandard railings. The New York design is shown in Figure 7.14.

7.7.2.3 Metal Post and Beam Retrofits

A metal post and beam retrofit railing mounted at the curb edge, such as that shown in Figure 7.15, may be appropriate for use on an existing structure that has a relatively wide raised walkway. This design functions well as a traffic barrier separating motor vehicles from pedestrians using a sidewalk across a bridge. In many cases, the existing bridge railing can be used as, or converted to, a pedestrian railing.

The post attachment to the curb or bridge deck can be designed to withstand the design loads contained in the current *AASHTO LRFD Bridge Design Specifications* or can be a yielding design that eliminates bridge deck damage in high-angle, high-speed impacts. The metal rail elements should be in line with the face of the curb and spaced to minimize the likelihood of vehicle intrusion and subsequent snagging on the posts.

7.8 TRANSITIONS

A transition section is needed where a semi-rigid approach barrier joins a rigid bridge railing. Transitions may not be necessary when bridge railings with some flexibility (such as the TL-2 bridge rail described in Section 7.4.2) are used. The transition design should produce a gradual stiffening of the overall approach protection system so vehicular pocketing, snagging, or penetration can be reduced or avoided at any position along the transition. Details of special importance for transitions are as follows:

- The approach-rail/bridge-rail splice or connection must be as strong as the approach rail itself so it will not fail on impact by pulling out and allowing a vehicle to strike the end of the bridge railing. The use of a cast-in-place anchor or through-bolt connection is recommended. The transition must also be designed to minimize the likelihood of snagging an errant vehicle, as well as one from the opposing lane on a two-way facility.

- Strong post systems (usually blocked out) or combination normal post and strong beam systems can be used on transitions to rigid bridge

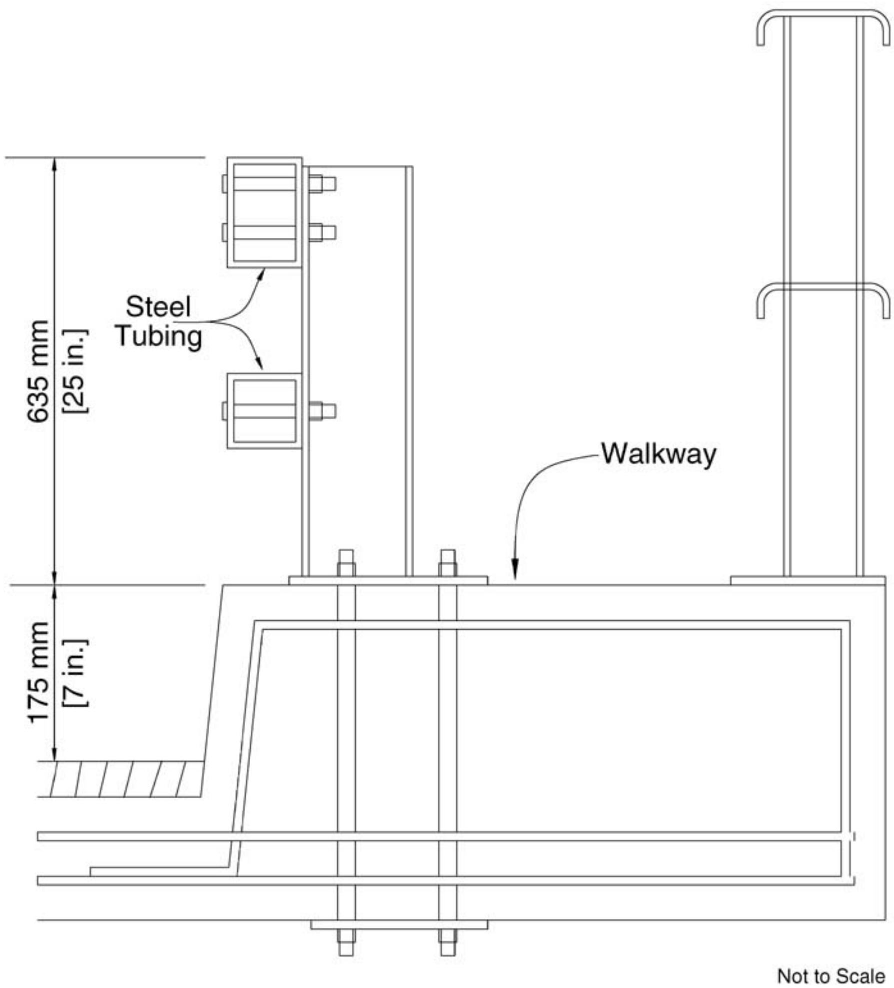


FIGURE 7.15 Metal post and beam retrofit

railings or other rigid objects. These systems should usually be blocked out from their posts unless the railing member is of sufficient width to prevent or reduce snagging to an acceptable level. However, block-outs or railing offsets alone may not be sufficient to prevent potential snagging at the immediate upstream end of the rigid bridge railing. A rubrail may be desirable in some designs using flexible W-beam or box-beam transition members. Tapering of the rigid bridge railing end behind the transition members at their connection point may also be desirable, especially when the approach transition is recessed into the concrete end of the bridge railing or other rigid object.

- The transition section should be long enough so that significant changes in deflection do not occur within a short distance. Generally, the transition length should be 10 to 12 times the difference in the lateral deflection of the two systems in question.

- The stiffness of the transition should increase smoothly and continuously from the less rigid to the more rigid system. This is usually accomplished by decreasing the post spacing, increasing the post size, or doing both, and by strengthening the rail element. W-beam or thrie-beam rail elements are typically strengthened by “nesting” two rails together.

- Drainage features such as curbs, raised inlets, curb inlets, ditches, or drainage swales, when constructed in front of barriers, especially in the transition area, may initiate vehicle instability that

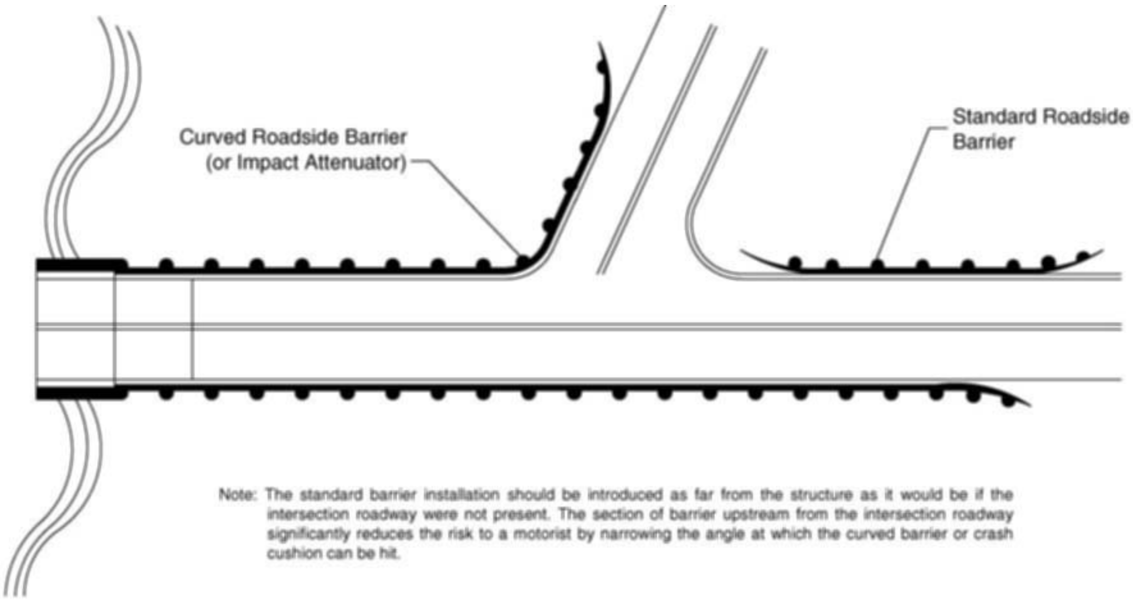


FIGURE 7.16 Possible solution to intersection side road near bridge

can, in some instances, adversely affect the crashworthiness of the transition. However, some transition designs incorporate a curb to reduce the probability of a vehicle snagging on the end of a rigid bridge railing. The slope between the edge of the driving lane and the barrier should be no steeper than 1V:10H.

When a minor road or driveway intersects a main road close to a bridge, it is often difficult to shield the bridge railing end adequately. The preferred solution is to close or relocate the intersecting road and install an approach railing with a standard transition section. If this solution cannot be done, curved guardrails that were crash tested to NCHRP Report 230 (4) can be used. An attempt should be made to ensure that errant vehicles do not go behind, through, or over the barrier. Some sacrifice in the crashworthiness of the barrier may be unavoidable in such circumstances, but the installation should be made as forgiving as possible. The use of appropriate crash cushions or other commercially available appurtenances may provide cost-effective solutions in some cases. Figure 7.16 depicts another possible solution using standard W-beam barrier that minimizes the risk to a motorist by shielding most of the object using a separate guardrail run. Because a motorist may hit the curved section of the rail at a very high angle, some states use weakened wood posts without offset blocks to support the curved section of rail. This design results in the posts breaking without significant leaning in the soil and permits capture of the impacting vehicle by the W-beam rail element. This curved guardrail design has been tested with 820 kg [1,800 lb] and 2040 kg [4,500 lb] passenger cars at 80 km/h [50 mph]. There is no curved guardrail design currently available that has met all NCHRP Report 350 evaluation criteria.

NCHRP Report 350 recommends that transitions be designed and crash tested to the test level appropriate for the intended application. Although the use of W-beam approach rail with neither an adequate blockout connection to the bridge rail nor a rubrail is relatively common, recent crash testing has shown that poor results are produced by allowing an impacting vehicle to snag on the end of the rigid bridge railing or concrete safety shape or parapet. These tests have also demonstrated that a more rigid guardrail transition to the bridge railing is necessary. This can be accomplished through: reduced post spacing; larger, longer, or both larger and longer posts; stronger rail elements (nested rail); and other special features.

Several new transition designs have been tested and proven satisfactory in accordance with NCHRP Report 350. Four of these designs are shown in Figures 7.17 through 7.20. The first two show transition details for a W-beam approach rail to a straight, vertical, concrete rail or end post and to a concrete safety-shape bridge rail, respectively. The third shows a thrie-beam transition to a modified safety-shape bridge railing. The fourth shows a thrie-beam transition to a curb-mounted steel post and beam bridge railing. Key design features of all these designs include:

FIGURE 7.17 W-beam transition to vertical concrete rail

FIGURE 7.18 W-beam transition to modified concrete safety shape

FIGURE 7.19 Thrie-beam transition to modified concrete safety shape

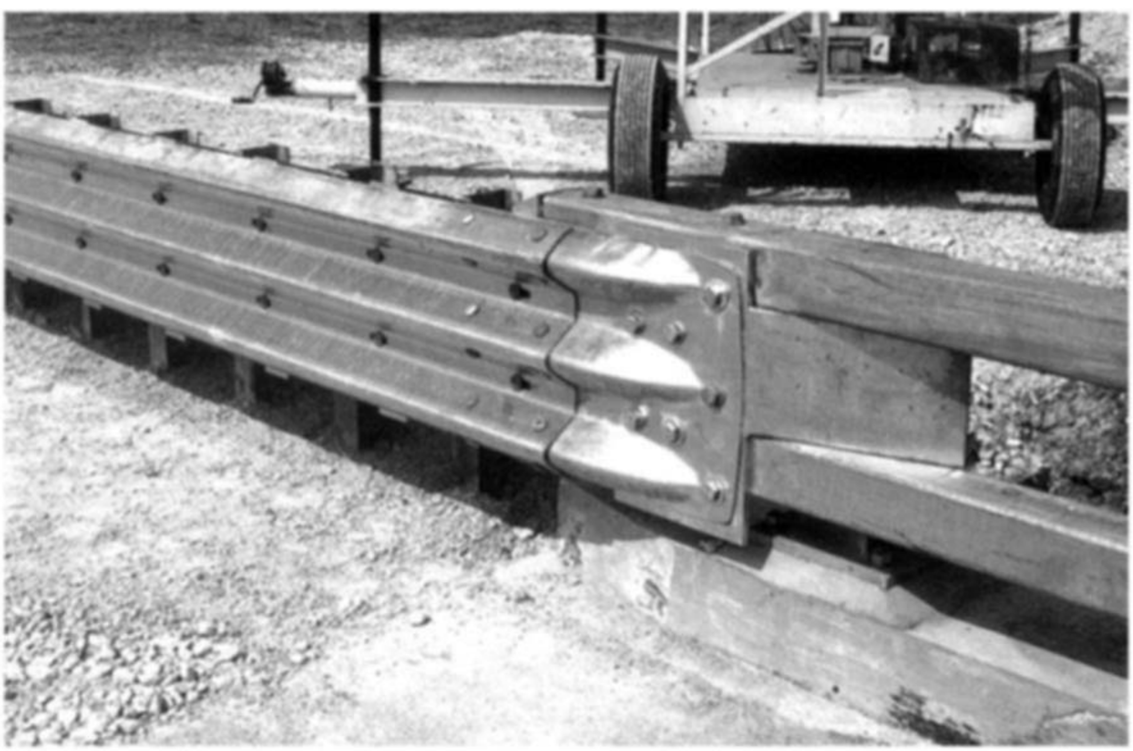


FIGURE 7.20a Thrie-beam transition to curb-mounted steel post and beam bridge railing

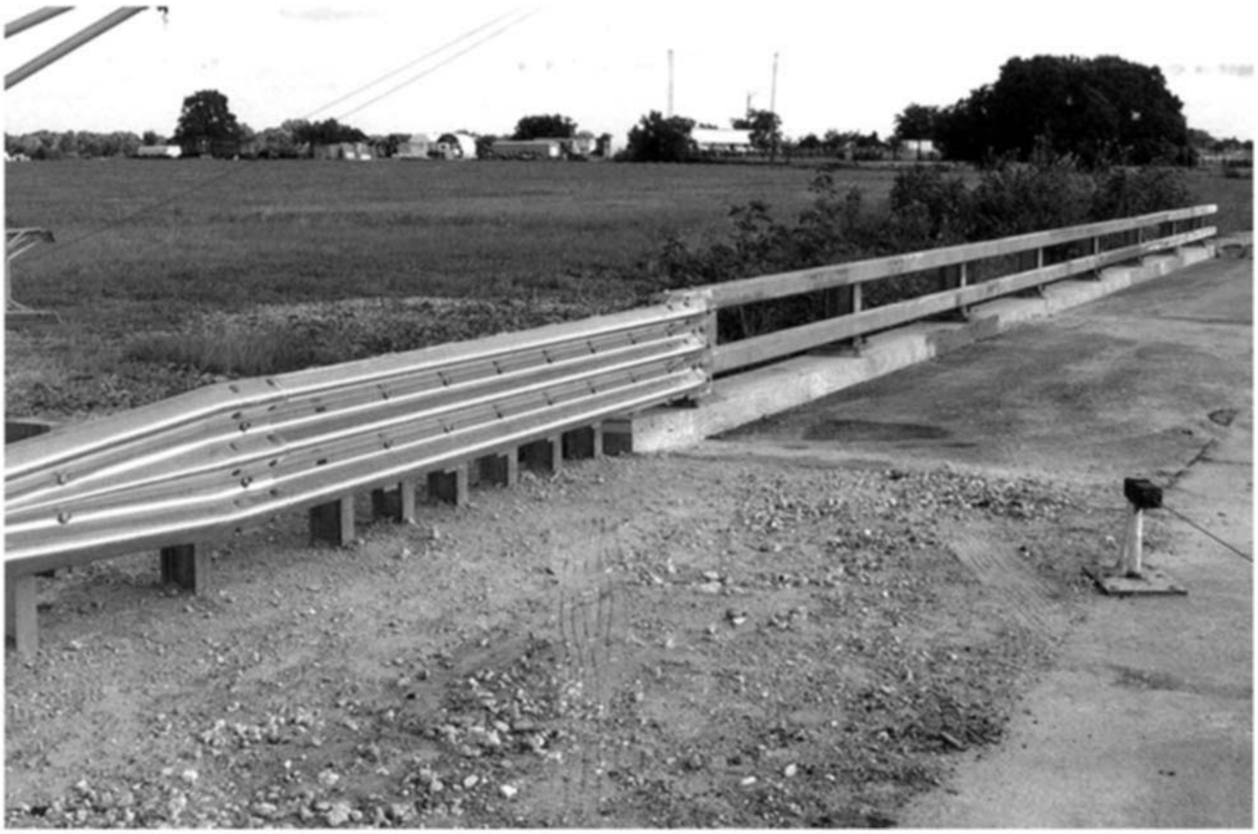


FIGURE 7.20b Thrie-beam transition to curb-mounted steel post and beam bridge railng

- larger, longer posts than were used in comparable NCHRP Report 230 (4) designs immediately adjacent to the parapet;
- nested W-beam or thrie-beam sections (one beam nested inside the other); and
- rubrail or tapered/flared concrete parapet (to minimize snagging at the bridge end).

Because relatively few transition designs have been tested to NCHRP Report 350, FHWA has agreed to the continued use of any transition design that was acceptable under NCHRP Report 230 (4) guidelines until October 2002 on the National Highway System. By then, it is anticipated that several NCHRP Report 350 designs will be available for use.

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2. AASHTO. *Standard Specifications for Highway Bridges*. American Association of State Highway and Transportation Officials, Washington, DC, 1996.
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Chapter 8
Barrier End Treatments and Crash Cushions

8.0 OVERVIEW

A crash involving a vehicle impacting an untreated end of a roadside barrier or a fixed object often results in serious consequences because vehicles are usually stopped abruptly. In addition, an impact with the untreated end of a longitudinal barrier can result in barrier elements penetrating the passenger compartment or causing vehicle instability and resulting in a roll over, thereby increasing the risk to the occupants. Barrier end treatments and crash cushions are frequently used to prevent impacts of this type by gradually decelerating an impacting vehicle to a stop or by redirecting it around the object of concern.

In very general terms, a barrier end treatment or terminal is normally used at the end of a roadside barrier where traffic passes on one side of the barrier and in one direction only. A crash cushion is normally used to shield the end of a median barrier or a fixed object located in a gore area. A crash cushion may also be used to shield a fixed object on either side of a roadway if a designer decides that a crash cushion is more cost-effective than a traffic barrier.

This chapter will explain the warrants for installation as well as the structural and performance requirements of barrier end treatments and crash cushions. Descriptions, selection guidelines, and placement recommendations for systems that have been successfully crash tested under current performance criteria are provided, except as noted.

8.1 PERFORMANCE REQUIREMENTS

NCHRP Report 350 (1) contains the current recommendations for testing and evaluating the performance of crash cushions and barrier end treatments. To be considered acceptable for installation on new or reconstruction projects, these devices must meet the evaluation criteria described in the report.

The criteria require that the impacting vehicle be gradually stopped or redirected by the crash cushion or end treatment when impacted end-on. In addition to end-on impacts, barrier end treatments and crash cushions must be capable of safely redirecting a vehicle that impacts the side of the device, both at mid-length and near the nose. Other criteria for these devices are outlined in the report. NCHRP Report 350 establishes three test levels (TLs) for barrier end treatments and crash cushions. All levels require impacts at specified locations and angles with both an 820 kg [1,800 lb] car and a 2000 kg [4,400 lb] pickup truck at impact speeds of 50, 70, and 100 km/h [30, 45, and 60 mph] for TL-1, 2, and 3, respectively. Although some devices can be modified to function acceptably under certain impact conditions at speeds higher than 100 km/h [60 mph], the additional physical space required and the increased costs associated with special designs have limited their use in the field.

All of the crashworthy end treatments and crash cushions discussed in this chapter have been successfully tested to NCHRP Report 350 TL- 2 or 3. Systems not shown to be crashworthy by either NCHRP Report 350 compliance testing or an in-service performance evaluation should be upgraded when extensive damage occurs or when major rehabilitation is conducted on the adjacent roadway pavement.

8.2 END TREATMENTS

A crashworthy end treatment is considered essential if a barrier terminates within the clear zone or is located in an area where it is likely to be struck by an errant motorist. To be crashworthy, the end treatment should not spear, vault, or roll a vehicle for head-on or angled impacts. For impacts within the length of need, the end treatment should have the same redirectional characteristics as the standard roadside barrier, which means that the end must be properly anchored. The end treatment for longitudinal barriers that rely on tensile strength for redirective capacity must be capable of developing the full tensile strength of the standard rail element whether a crashworthy end treatment is warranted or not.

End treatments can be classified as either gating or non-gating, depending on their behavior when impacted on the face near the end. A gating end treatment allows a vehicle impacting the nose or the side of the unit at an angle near the nose to pass through the device. Non-gating end treatments are capable of redirecting a vehicle impacting the nose or the side of the unit along the unit's entire length. For gating end treatments, the length of need usually starts at 3.81 m [12 ft 6 in.] from the impact head of the unit, but this can vary depending on the specific terminal used. Virtually all barrier terminals should be considered gating, i.e., a vehicle impacting at the end at an angle will proceed through and beyond the terminal. For this reason, the area behind and beyond all barrier terminals should be relatively traversable and free of significant fixed objects. The minimum recommended distance is a rectangular area approximately 23 m [75 ft] beyond the terminal parallel to the rail and 6 m [20 ft] behind and perpendicular to the rail. However, a runout area of that size cannot be expected to accommodate all impacts that might occur.

The grading between the traveled way and the terminal and the approach in front of any terminal should be essentially flat (slope no greater than 1V:10H in any direction) so that impacting vehicles will be relatively stable at the moment of contact. Typical grading layouts are shown in Figures 8.1 and 8.2 for a flared guardrail end terminal and a non-flared guardrail end terminal, respectively.

Table 8.1 summarizes the crashworthy end treatments discussed in the following subsections.

8.2.1 Three-Strand Cable Terminal

Several agencies that use the three-strand cable barrier have developed a terminal specific to their barrier design. The New York State Department of Transportation ran several tests on the design shown in the AASHTO-AGC-ARTBA Joint Committee Task Force 13 Report, *A Guide to Standardized Highway Barrier Hardware*, as Drawing SEC01 (2). One additional test was run by the FHWA to certify that this design has been successfully tested to NCHRP Report 350, TL-3. In the latest modification to this design, the cable barrier is flared back 1.2 m [4 ft] from the tangent barrier line to a concrete anchor, and the cable clamps shown in Drawing SEC01 are not used. This terminal is shown in Figure 8.3.

8.2.2 Wyoming Box Beam End Terminal (WYBET-350)

The Wyoming Box Beam End Terminal (WYBET-350) consists of a nosepiece welded to a short section of 150 mm x 150 mm [6 in. x 6 in.] box beam inserted into a 175 m x 175 mm [$6^3/_4$ in. x $6^3/_4$ in.] tube and held in place by a wood post. Inside the larger tube is a two-stage fiberglass composite tube. When impacted, the nosepiece telescopes into the larger tube. Crushing of the composite tube dissipates kinetic energy. This terminal, shown in Figure 8.4, is used with the box beam barrier discussed in Chapter 5. The WYBET-350 has been successfully tested to NCHRP Report 350, TL-3.

The terminal may be installed parallel to the roadway or flared out at a maximum rate of 1:10. Redirection of face impacts is considered to begin at the third post from the end of the terminal, allowing 10 m [32 ft] of the terminal to be included in the "length of need."

8.2.3 Barrier Anchored in Backslope

In areas of roadway cut section, or where the road is transitioning from cut to fill, it is sometimes possible to terminate a traffic barrier in the backslope, as shown in Figure 8.5. A W-beam guardrail anchored in the backslope, shown in Figure 8.6, has been successfully crash tested to NCHRP Report 350, TL-3. When properly designed and located, this type of anchor provides full shielding for the identified hazard, eliminates the possibility of an end-on impact with the barrier terminal, and minimizes the likelihood of the vehicle passing behind the rail. It is considered a non-gating terminal.

Key design considerations include (1) maintaining a uniform rail height relative to the roadway grade until the barrier crosses the ditch flow line, (2) using a flare rate within the clear zone that is appropriate for the design speed, (3) adding a rubrail for W-beam guardrail installations, and (4) using an anchor that is capable of developing the full tensile strength of the W-beam rail. Also, the foreslopes on the approach should be no greater than 1V:4H. If a barrier cannot be terminated in a backslope without violating any of these principles, a different type of end treatment may be more appropriate.

These design considerations also apply to terminating any of the aesthetic barriers identified in Chapter 5 in a backslope, including the Ironwood and Merritt Parkway guardrails, the steel-backed wood rail, and the stone masonry and precast masonry walls.

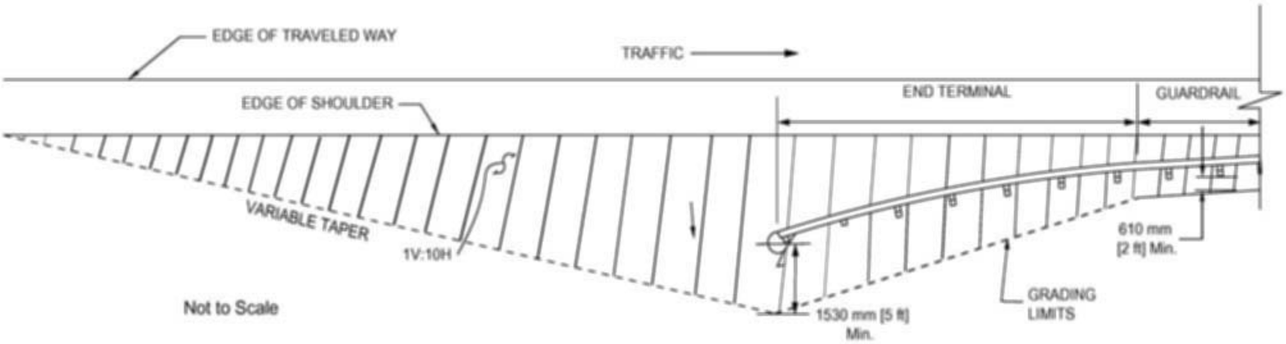

FIGURE 8.1 Grading for flared guardrail end treatment

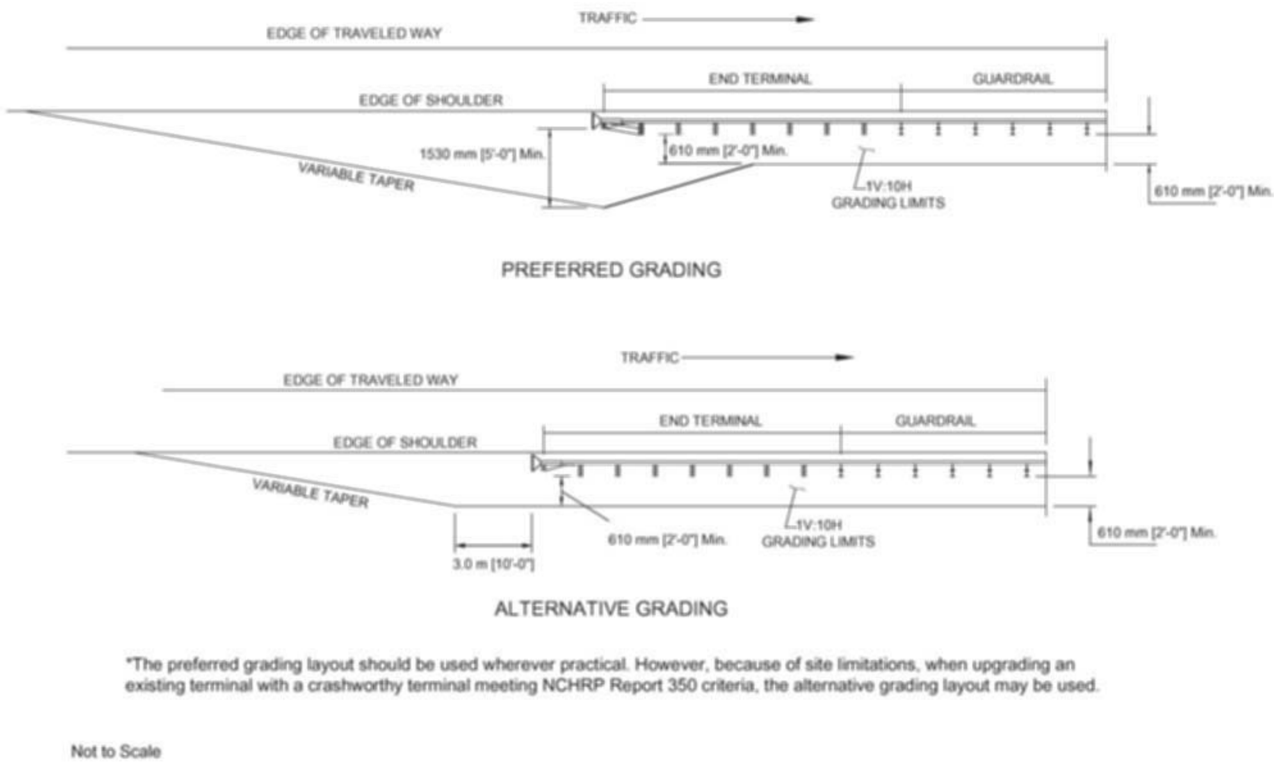

FIGURE 8.2 Grading for non-flared guardrail end treatment

TABLE 8.1 Crashworthy end treatments

System	NCHRP Report 350 Test Level	System Width	System Length
Three-Strand Cable Terminal	TL-3	1.2 m [4.0 ft] Flare	N/A
Wyoming Box Beam End Terminal (WYBET-350)	TL-3	0.6 m [2 ft]	15.2 m [50 ft]
Barrier Anchored in Backslope	TL-3	N/A	N/A
Eccentric Loader Terminal (ELT)	TL-3	0.5 m [1.6 ft] plus 1.2 m [4 ft] Flare	11.4 m [37.5 ft]
Slotted Rail Terminal (SRT-350)	TL-3	0.5 m [1.6 ft] plus 1.2 m [4 ft] Flare or 0.5 m [1.6 ft] plus 0.9 m [3 ft] Flare	11.4 m [37.5 ft]
REGENT	TL-3	0.5 m [1.6 ft] plus 1.3 m [4.3 ft] Flare	11.4 m [37.5 ft]
Vermont Low-Speed, W-Beam Guardrail End Terminal	TL-2	1.5 m [4.9 ft]	3.4 m [11.15 ft]
Flared Energy-Absorbing Terminal (FLEAT)	TL-2	0.5 m [1.6 ft] plus 0.51 m – 0.81 m [1.7 ft – 2.7 ft] Flare	7.62 m [25 ft]
	TL-3	0.5 m [1.6 ft] plus 0.76 m – 1.2 m [2.5 ft – 4 ft] Flare	11.4 m [37.5 ft]
Beam-Eating Steel Terminal (BEST)	TL-3	0.5 m [1.6 ft]	11.4 m [37.5 ft] or 15.2 m [50 ft]
Extruder Terminal (ET-2000)	TL-3	0.5 m [1.6 ft]	11.4 m [37.5 ft] or 15.2 m [50 ft]
Sequential Kinking Terminal (SKT-350)	TL-3	0.5 m [1.6 ft]	15.2 m [50 ft]
QuadTrend-350	TL-3	0.46 m [1.5 ft]	6.1 m [20 ft]
NEAT	TL-2	0.57 m [1.9 ft]	2.957 m [9.7 ft]
Sloped Concrete End Treatment	N/A	0.6 m [2 ft]	6 m to 12 m [20 ft to 40 ft]

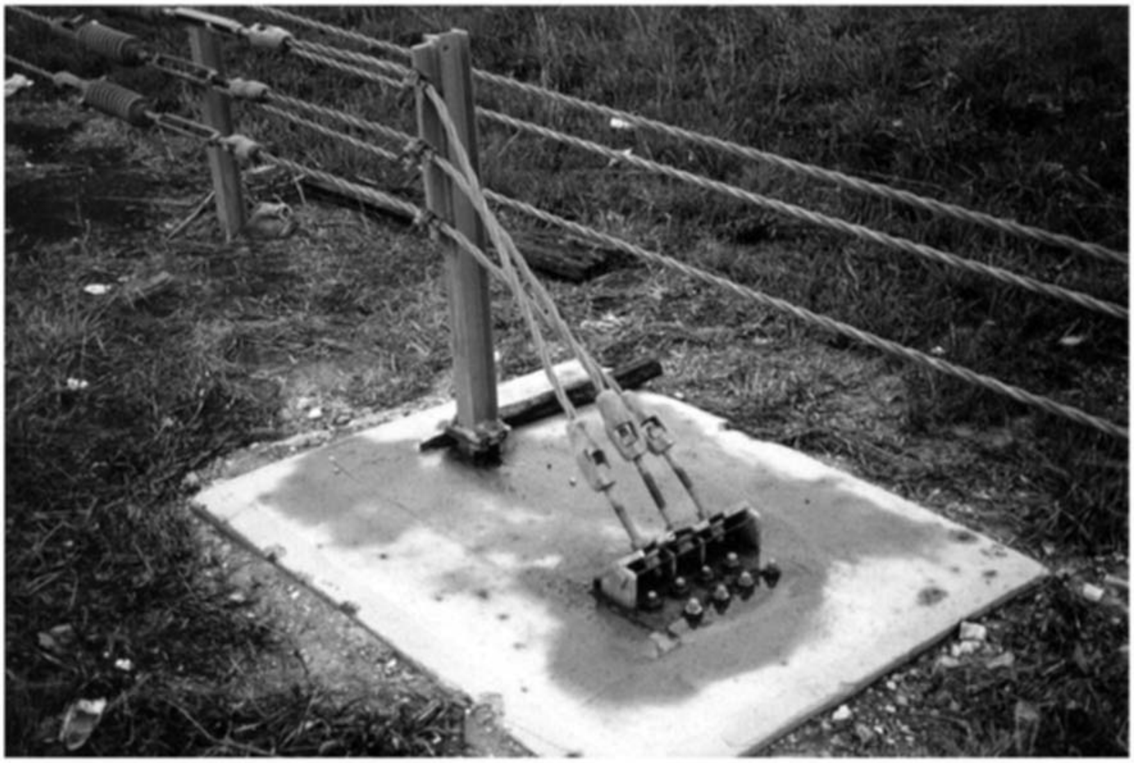

FIGURE 8.3 Three-strand cable terminal

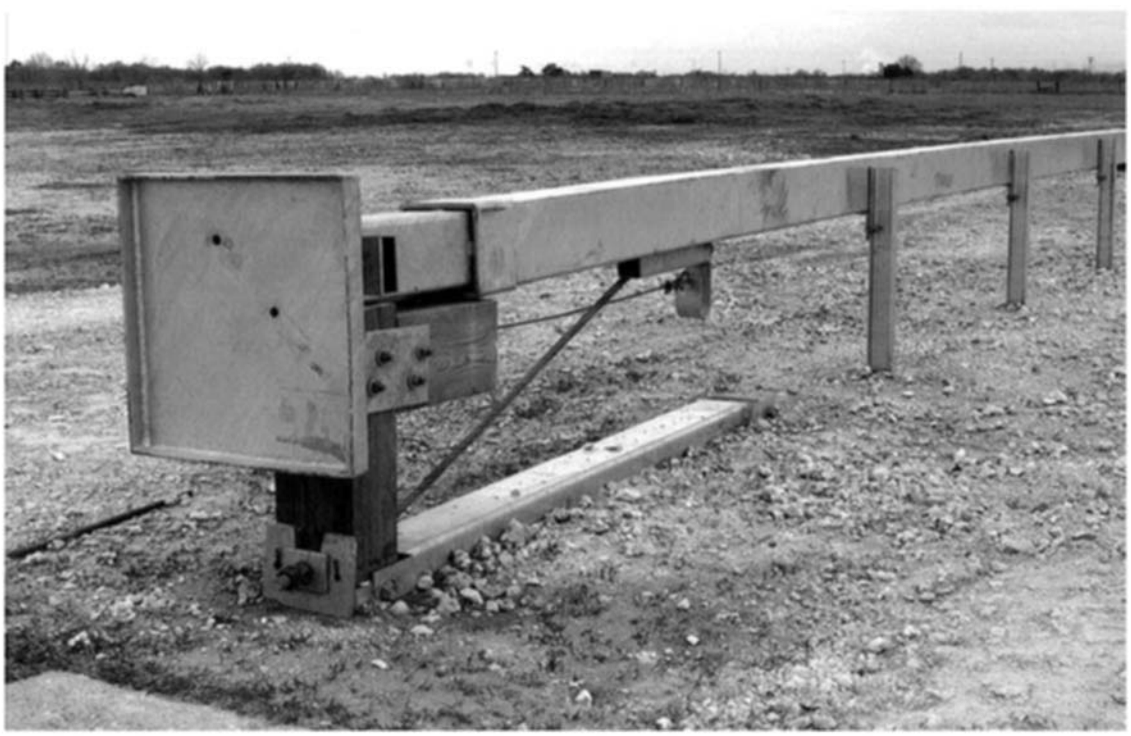

FIGURE 8.4 Wyoming box beam end terminal

FIGURE 8.5 Barrier anchored in backslope

FIGURE 8.6 W-beam guardrail anchored in backslope

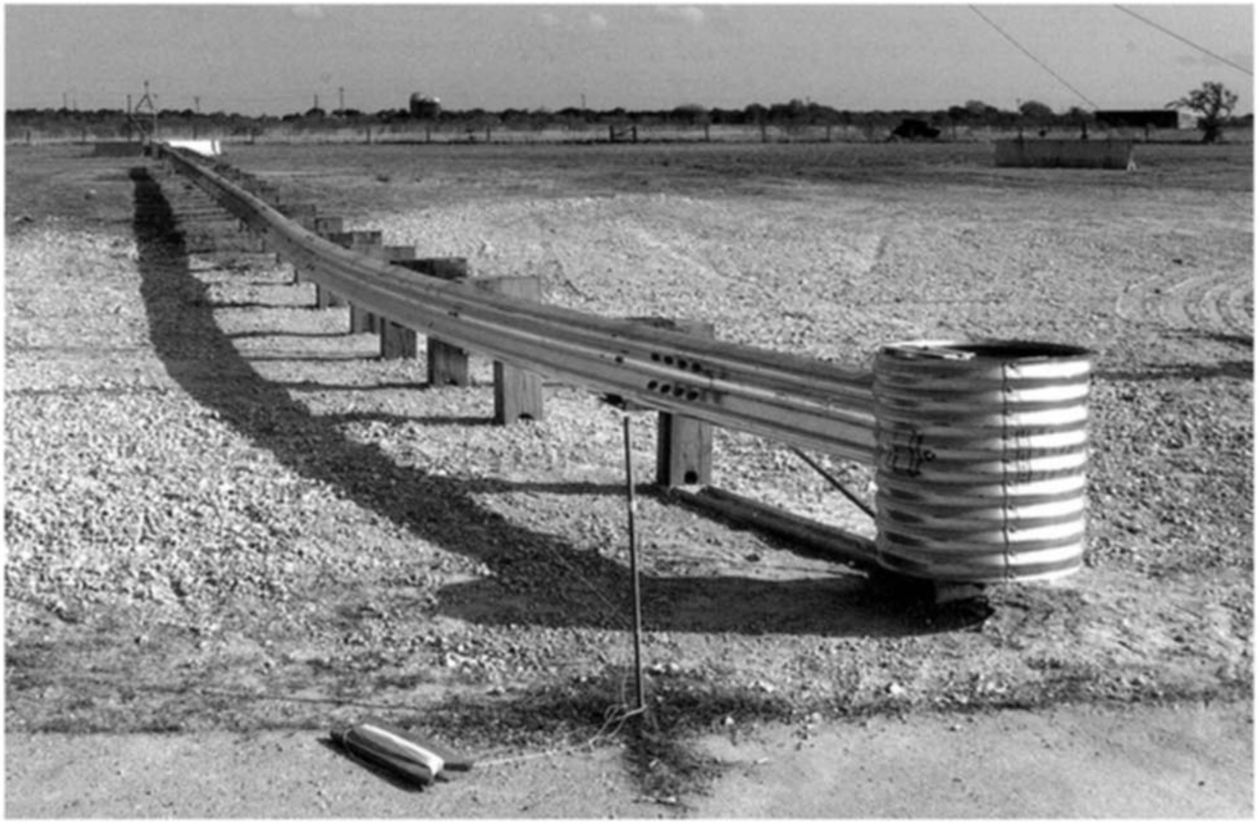

FIGURE 8.7 Eccentric loader terminal

8.2.4 Eccentric Loader Terminal (ELT)

Efforts to improve the performance of the Breakaway Cable Terminal (BCT) resulted in the development of the Eccentric Loader Terminal. This terminal is shown in Figures 8.7 and 8.8. On this terminal, the metal end of the standard BCT was replaced with a fabricated steel lever nose inside a section of corrugated steel pipe. The bolts were removed from all the posts in the terminal except the post where the curved flare and the tangent rail join as well as the adjacent post in the flared section. A strut between the steel tube foundations for the two end posts enables these posts to act together to resist cable loads resulting from downstream impacts. The next four posts are drilled with two holes, one at ground line and one below ground, to make them breakaway. A blockout is added to the second post to increase the curvature near the end of the rail reducing the column strength of the rail and reducing the likelihood of the rail penetrating an impacting vehicle.

This end treatment is designed with a 1.2 m [4 ft] offset to the end post. The rail element should be field bent, and all posts must be wood. The ELT has been tested successfully to NCHRP Report 350, TL-3.

The curved flare is critical for proper impact performance. Redirection begins 3.81 m [12 ft 6 in.] from the end of the terminal at the third post.

8.2.5 Slotted Rail Terminal (SRT-350)

The SRT-350 is a proprietary flared non-energy-absorbing terminal. There are two versions of the SRT-350, one with an offset of 1.2 m [4 ft] and another with an offset of 0.9 m [3 ft]. See Figures 8.9 and 8.10. These systems are designed to break away when impacted end-on. Both have been successfully tested to NCHRP Report 350, TL-3.

The two SRT-350 designs consist of a curved W-beam rail element in which longitudinal slots have been cut at specific locations to reduce its dynamic buckling strength to an acceptable level for end-on impacts and control the location of the buckling. As a result, the yaw of an impacting vehicle and the potential for secondary impacts with the bent rail are minimized. For downstream impacts, rail tension is developed through a cable anchor system. Length of need on this system begins 3.81 m [12 ft 6 in.] from the end (at the third post).

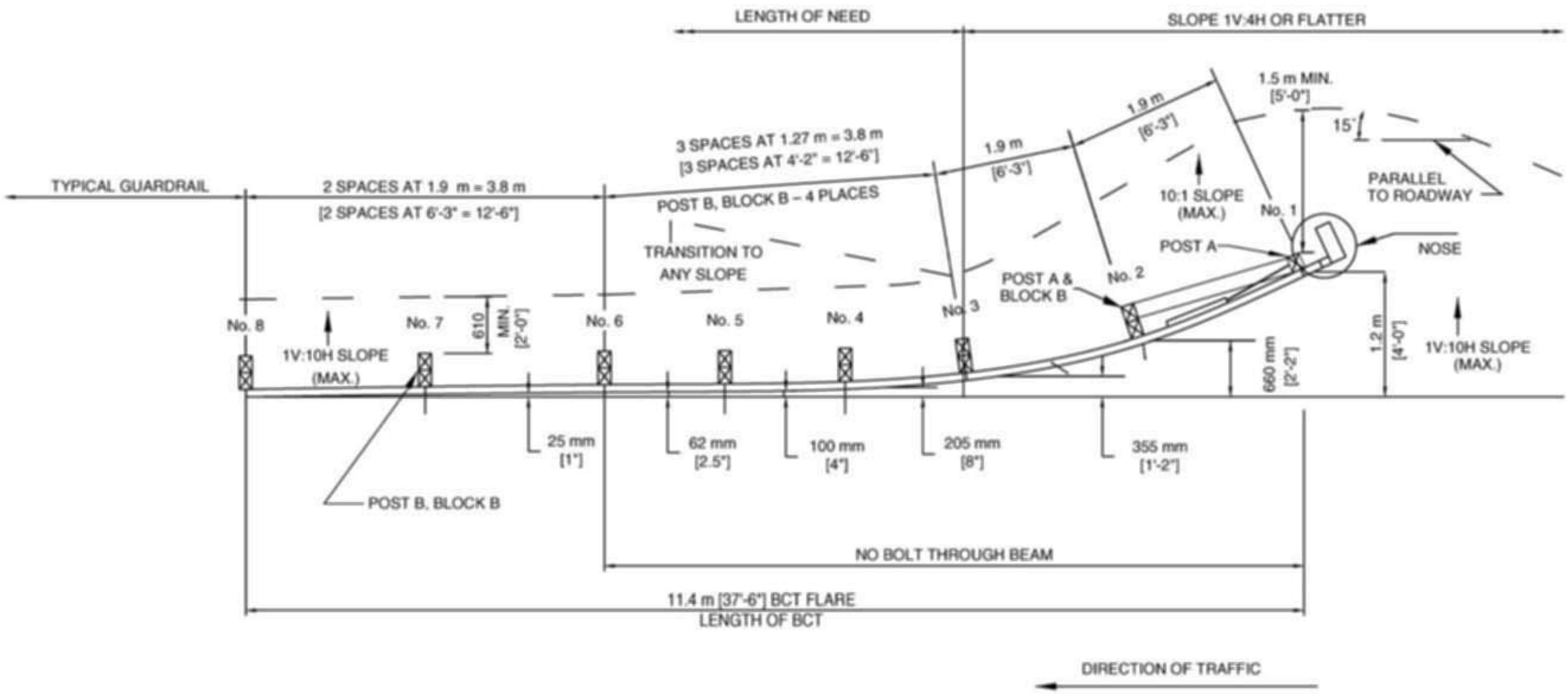

FIGURE 8.8 Plan layout for eccentric loader terminal

FIGURE 8.9 Slotted rail terminal (SRT-350) with 1.2 m [4 ft] flare

FIGURE 8.10 Slotted rail terminal (SRT-350) with 0.9 m [3 ft] flare

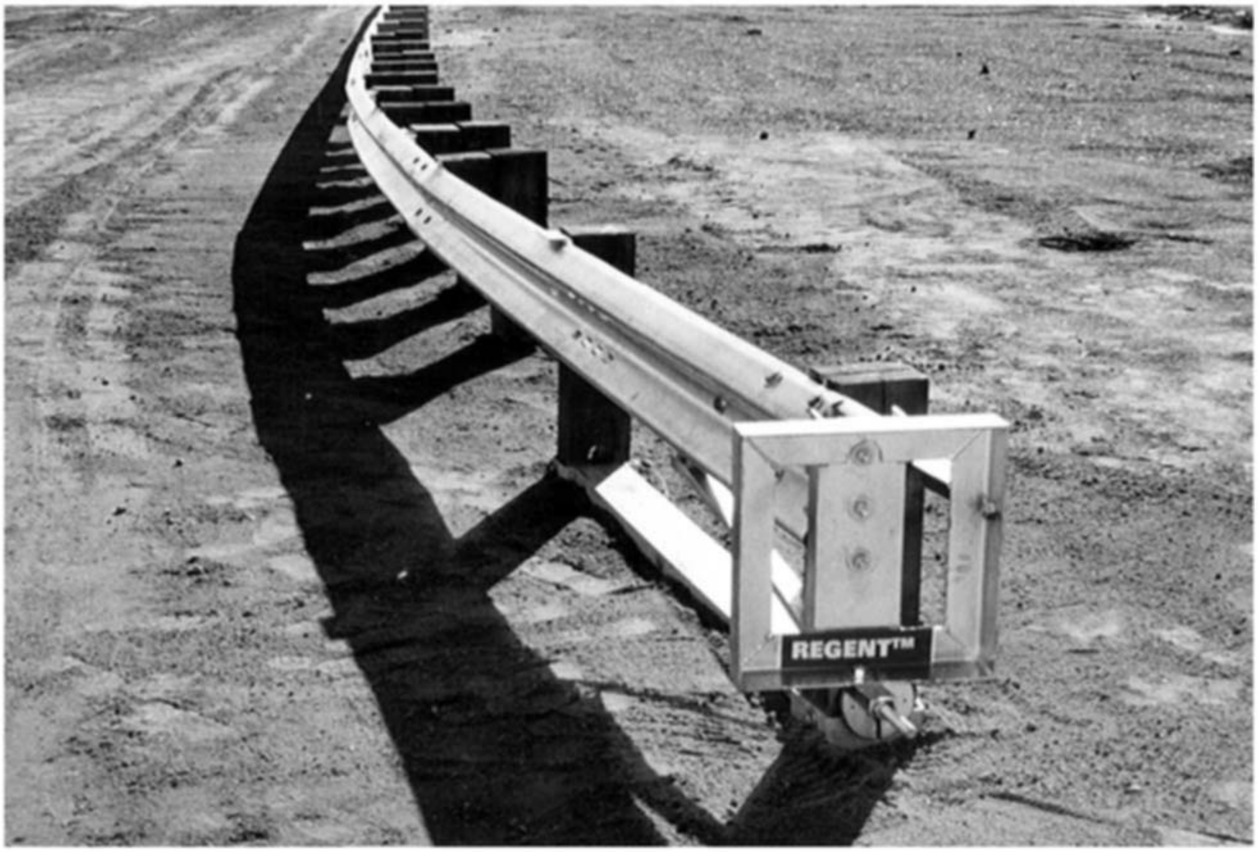

FIGURE 8.11 REGENT

It is critical that a traversable area, free of fixed objects, be provided behind the terminal since it is designed to break away when impacted, allowing the vehicle to travel behind the guardrail. The grading layout for the SRT-350 should be as shown in Figure 8.1.

8.2.6 REGENT Terminal

The REGENT, a proprietary energy-absorbing end treatment, is a flared W-beam terminal that consists of a slider head assembly, a cable anchor/strut and yoke assembly, modified W-beam rail panels, and special weakened wood posts at posts 1 and 2, and at posts 3 through 8. This terminal is shown in Figure 8.11. It meets the evaluation criteria of NCHRP Report 350 at TL-3.

The post offsets correspond to those of the BCT, except that the REGENT uses more posts to minimize deflection and the posts are unique in design. The modified rail elements are partially crushed at two locations to produce "upsets" designed to induce predictable kinks in the rail in end-on hits while maintaining most of the rail's bending strength to minimize deflection from side impacts. Redirection begins 3.81 m [12 ft 6 in.] from the end of the terminal (at the third post).

As with all gating terminals, it is critical that a traversable area, free of fixed objects, be provided behind the REGENT since it is designed to break away when impacted, allowing the vehicle to travel behind the guardrail. The grading layout for the REGENT should be as shown in Figure 8.1.

8.2.7 Vermont Low-Speed, W-Beam Guardrail End Terminal

The Vermont Low-Speed, W-Beam Guardrail End Terminal is a non-proprietary end treatment that is appropriate for use on roadways where anticipated impact speeds do not exceed 70 km/h [45 mph]. It has been successfully tested to NCHRP Report 350, TL-2. This terminal consists of a 3.8 m [12 ft 6 in.] W-beam rail section that is shop-bent to a 4.9 m [16 ft] radius mounted on W150 x 14 [W6 x 9] steel posts with steel blocks. An anchor consisting of a steel rod and buried concrete block is attached to the rail at the third post from the end. See Figure 8.12.

FIGURE 8.12 Vermont low-speed, W-beam guardrail end terminal

8.2.8 Flared Energy-Absorbing Terminal (FLEAT)

The FLEAT is a proprietary energy-absorbing end treatment that consists of an impact head installed at the end of a modified W-beam rail element. The terminal is shown in Figure 8.13. The FLEAT has been tested successfully to NCHRP Report 350, TL-3 with a total length of 11.4 m [37 ft 6 in.] and to TL-2 with a total length of 7.62 m [25 ft]. The TL-3 terminal is designed to be installed with a linear offset that can range from 0.76 m to 1.2 m [2 ft 6 in. to 4 ft]. The TL-2 design has an offset that can vary between 0.51 m and 0.81 m [1.7 ft and 2.7 ft].

The main components of the FLEAT are the impact head and guide tube assembly, a modified W-beam rail, a breakaway cable anchor assembly, and seven weakened posts (five for the TL-2 design). These posts may be wood or a welded breakaway design that may be used as an alternative to wood. The kinetic energy of a crash is absorbed by the head sliding along the rail element while bending it in a manner similar to the SKT-350, discussed in Section 8.2.11. The flattened rail exits the head on the traffic side and coils into a tight loop. For downstream impacts on the face of the rail, tension in the rail is developed through the cable anchor system. Redirection begins 3.81 m [12 ft 6 in.] from the end of the terminal (at the third post).

It is critical that a traversable area, free of fixed objects, be provided behind the terminal because it is designed to break away when impacted, allowing the vehicle to travel behind the guardrail. The grading layout for the FLEAT should be as shown in Figure 8.1.

8.2.9 Beam-Eating Steel Terminal (BEST)

The BEST is a proprietary energy-absorbing end treatment that consists of an impact head mounted on the end of a wood post W-beam guardrail system (Figure 8.14). The kinetic energy of a crash is absorbed by the head, which contains three cutter teeth that slide along the rail element and cut it into four relatively flat plates that are subsequently bent out of the path of the impacting vehicle. A quick release cable attachment is used which allows the W-beam to feed into the impact head during end-on impacts. This cable provides anchorage for downstream

FIGURE 8.13 Flared Energy-Absorbing Terminal (FLEAT)

FIGURE 8.14 Beam Eating Steel Terminal (BEST)

FIGURE 8.15 Extruder Terminal (ET-2000)

impacts. Redirection begins 3.81 m [12 ft 6 in.] from the end of the terminal (at the third post). No flare is required for this end treatment. However, to position the impact head entirely outside the shoulder, a 1:50 flare may be desirable. The BEST has been successfully tested to NCHRP Report 350, TL-3.

Typical layouts for grading around the BEST are shown in Figure 8.2.

8.2.10 Extruder Terminal (ET-2000)

The ET-2000 is a proprietary energy-absorbing end treatment, which consists of an extruder head installed over the end of a standard W-beam guardrail element. The kinetic energy of a crash is absorbed by the head sliding along the rail element while flattening it and bending it away from the traffic. The extruder head is made up of two sections, a squeezing section and a bending section. When the terminal is impacted end-on, the crash energy is dissipated as the extruder head travels along the rail. The W-beam is fed through the squeezing section, which reshapes the rail into a flat plate; then the bending section bends the rail around a small radius and directs it out to the side, away from the vehicle. A quick release cable attachment is used, which allows the W-beam to feed into the extruder during end-on impacts. This cable provides anchorage for downstream impacts. Redirection begins 3.81 m [12 ft 6 in.] from the end of the terminal (at the third post).

No flare is required for this end treatment. However, to position the impact head entirely outside the shoulder, a 1:50 flare may be desirable. The ET-2000 has been successfully tested to NCHRP Report 350, TL-3. Originally designed with breakaway timber posts, the ET-2000 has also been accepted for use with hinged breakaway steel posts. An alternate extruder head design, which weighs substantially less than the standard head and has been successfully tested to NCHRP Report 350, TL-3, is also available and is shown in Figure 8.15. Typical layouts for grading around the ET-2000 are shown in Figure 8.2.

FIGURE 8.16 Sequential Kinking Terminal (SKT-350)

8.2.11 Sequential Kinking Terminal (SKT-350)

The SKT-350 is a proprietary energy-absorbing end treatment that consists of an impact head mounted over the end of a W-beam guardrail element that has been modified by punching three slots in the valley of the rail at specific locations. The impact head being forced rearward, bending the W-beam rail element against the deflector plate, absorbs the kinetic energy of a crash, which, in conjunction with a "kinker" beam in the head, causes short segments of the rail to kink sequentially and bend away from the impacting vehicle. A cable anchorage system is provided to develop the tensile strength of the rail for downstream impacts. Redirection begins 3.81 m [12 ft 6 in.] from the end of the terminal (at the third post).

No flare is required for this end treatment, but some offset is recommended to locate the edge of the impact head farther from the traveled way. This terminal is shown in Figure 8.16. The SKT-350 has been successfully tested to NCHRP Report 350, TL-3. Like the ET-2000, it also has been accepted for use with breakaway steel posts as an alternative to timber posts.

Typical layouts for grading around the SKT-350 are shown in Figure 8.2.

8.2.12 QuadTrend-350

The QuadTrend-350 is a proprietary, unidirectional, gating barrier end treatment designed and tested for direct attachment to a vertical concrete barrier or to a vertical concrete bridge parapet without additional transition guardrail sections. It has been successfully tested to NCHRP Report 350, TL-3. A concrete pad is required. The sand-filled, energy-absorbing containers in this system are sacrificial and must be replaced following impact. Many of the other parts of the system can be reused. The QuadTrend-350 is shown in Figure 8.17. Grading around the QuadTrend-350 should conform to the recommended grading for non-flared gating terminals as shown in Figure 8.2.

FIGURE 8.17 QuadTrend-350

8.2.13 Narrow Energy-Absorbing Terminal (NEAT)

The NEAT is a proprietary, narrow, non-redirective, energy-absorbing terminal that has met the crash test criteria of NCHRP Report 350, TL-2. It is intended to shield the approach end of portable concrete safety-shaped barrier or the Quickchange® Moveable Barrier System. The NEAT has a mass of 130 kg [286 lbs] and consists of a 570 mm [22 in.] wide by 810 mm [32 in.] high by 2957 mm [9 ft 8 in.] long aluminum cartridge. Back-up attachments have been designed to attach the NEAT to the concrete safety shape or to the Quickchange System®. The NEAT is shown in Figure 8.18.

8.2.14 Sloped Concrete End Treatment

It is often appropriate to terminate a concrete barrier by tapering the end, although this end treatment has not met the crash testing criteria of NCHRP Report 350. This treatment should only be used in locations where the traffic speeds are low, 60 km/h [40 mph] or less, and space is limited by right-of-way constraints or presence of other roadside features that preclude the use of one of the tested end treatments. Other applications include locations where the barrier is flared out beyond the clear zone or where end-on impacts are not likely to occur. Recommended length of the taper is 6 m [20 ft] with 9 m to 12 m [30 ft to 40 ft] desirable. The height of the end of the taper should be no greater than 100 mm [4 in.]. Figure 8.19 shows a typical tapered end treatment on a concrete barrier.

8.3 CRASH CUSHIONS

Crash cushions or impact attenuators are protective devices that prevent errant vehicles from impacting fixed objects. This function is accomplished by gradually decelerating a vehicle to a safe stop for head-on impacts or, in most instances, by redirecting a vehicle away from the object for side impacts. Crash cushions are ideally suited for use at locations where fixed objects cannot be removed, relocated, or made breakaway, and cannot be adequately shielded by a longitudinal barrier.

Fixed objects that generally require shielding when located within the designated clear zone for a specific highway are listed in Table 5.1 in Chapter 5. Some of these

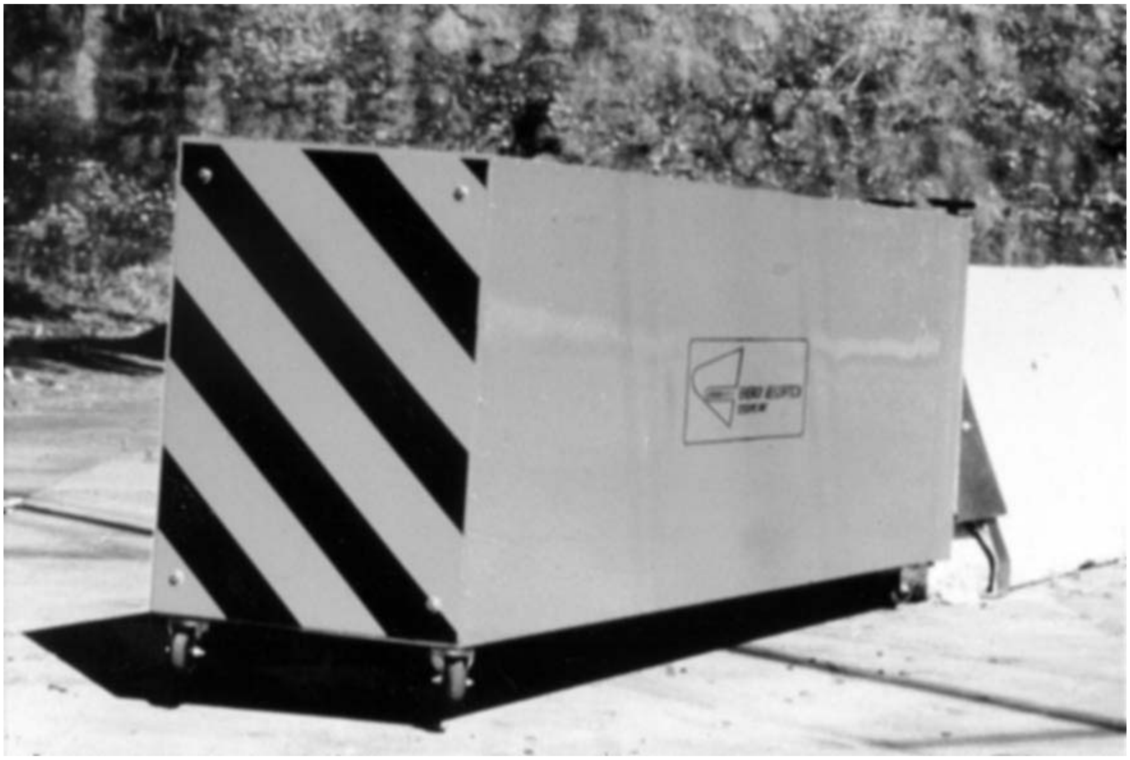

FIGURE 8.18 Narrow Energy-Absorbing Terminal (NEAT)

FIGURE 8.19 Sloped concrete end treatment

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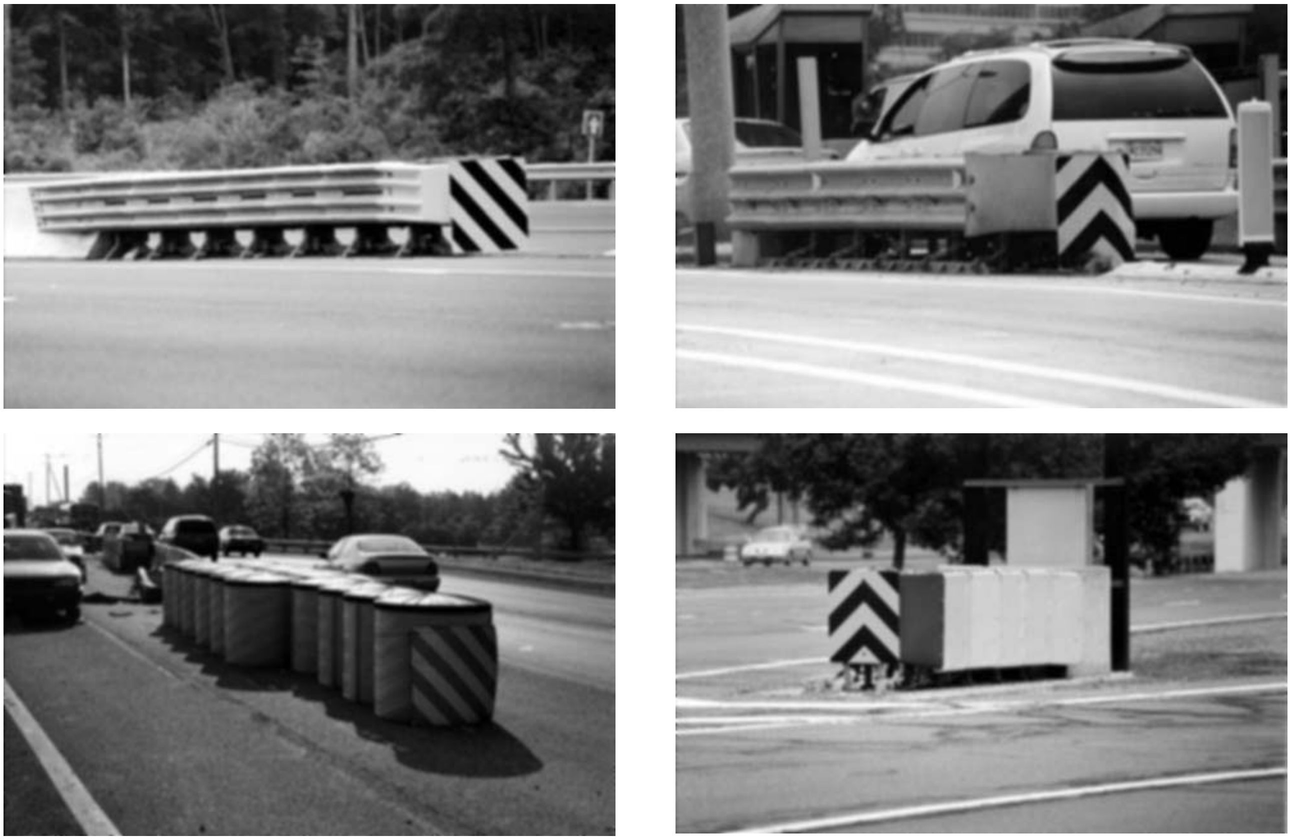


FIGURE 8.20 Crash cushion applications

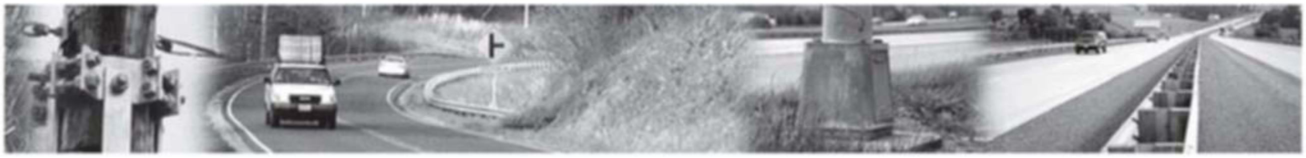
objects can be shielded only with a crash cushion, but most can also be shielded with a properly designed longitudinal barrier with crashworthy end terminals. A common application of a crash cushion is in an exit ramp gore on an elevated or depressed structure where a bridge rail end or a pier requires shielding. Crash cushions are also frequently used to shield the ends of median barriers. Typical applications are shown in Figure 8.20.

Long, steep downgrades on routes having high truck traffic present a unique type of problem with regard to highway safety. Loss of brakes on a vehicle on such a grade increases the potential for the vehicle to leave the roadway or impact other vehicles. Where such problems occur, special consideration should be given to the installation of a roadside deceleration device. One device that is commonly used is the gravel-bed attenuator. Some states have installed similar systems with good results, primarily to decelerate large vehicles safely. It should be noted that NCHRP Report 350 does not include specific test criteria for large-vehicle attenuation devices, but because they all are designed to stop vehicles impacting head on, a discussion of the gravel-bed attenuator and similar deceleration devices is included in Sections 8.3.2.11 through 8.3.2.13.

Another special condition for which crash cushions are applicable is the protection of construction and maintenance personnel as well as motorists in work zones. Portable and temporary crash cushions have been developed for use in such situations. In addition, several "truck mounted attenuators" (TMAs) are available for use in construction and maintenance zones. These types of crash cushions are discussed in detail in Chapter 9, Traffic Barriers, Traffic Control Devices, and Other Features for Work Zones.

Crash cushions have proven to be an effective and safe means of shielding particular types of roadside obstacles that cannot be shielded by other methods. Their prudent use has saved numerous lives by reducing severity of crashes. Their relatively low cost and potentially high safety payoff make them ideally suited for use at selected locations. Like other safety hardware, crash cushions primarily serve to lessen the severity of crashes rather than to prevent them from occurring.

This section briefly explains how crash cushions work and where their use may be warranted. Descriptions, design procedures, selection guidelines, and placement recommendations for systems that have been successfully crash tested are also provided. Most operational crash



Chapter 9
Traffic Barriers, Traffic Control Devices, and Other Safety Features for Work Zones

9.0 OVERVIEW

This chapter describes the safety, functional, and structural aspects of traffic barriers; traffic control devices; and safety features used in work zones; and provides guidance on their application.

The AASHTO *Summary Report on Work Zone Accidents* (1) contains several conclusions: (1) crashes that occur in work zones are generally more severe, producing more injuries and fatalities than the national average for all crashes; (2) fixed-object crashes in both rural and urban areas more frequently result in injuries and fatalities than vehicle-to-vehicle crashes; and (3) about half of all work-zone, fixed-object crashes occur in darkness. Tractor-trailer injury and fatality crashes in work zones are considerably higher than the national average for other types of crashes involving these vehicles.

Previous chapters in this Guide provide safety performance criteria for all types of safety features. Where warranted, this chapter adapts those criteria as necessary for application to work zones.

This chapter is not a stand-alone document on work-zone safety, but must be used in conjunction with traffic control guidance. *The Manual on Uniform Traffic Control Devices* (MUTCD) (2), Part VI, establishes the principles to be observed in the design, installation, and maintenance of traffic control devices in work zones and prescribes standards where possible. These principles and standards are aimed at the safe and efficient movement of traffic through work zones and the safety of the workers.

The design and selection of work-zone safety features should be based on expected operating speeds and proximity of vehicles to workers and pedestrians. Actual operating speeds may be considerably higher than posted speed limits and as much as 30 km/h to 40 km/h [20 mph to 25 mph] faster on freeways when temporary 60 km/h [40 mph] zones are established.

9.1 THE CLEAR-ZONE CONCEPT IN WORK ZONES

The forgiving roadside concept as promoted in earlier chapters should also be applied to all work zones as appropriate for the type of work being done and to the extent existing roadside conditions allow. This includes providing a clear recovery area for longer term projects and using traffic control devices and safety appurtenances that are crashworthy or shielded.

Additionally, work zones should be developed to provide a safe environment for pedestrians, bicyclists, and highway workers. This could mean providing safe pathways where pedestrians and bicyclists are allowed to traverse the work zone by shielding adjacent excavations or other unsafe areas.

9.1.1 Application of the Clear-Zone Concept in Work Zones

The work-zone "clear zone" is the unobstructed relatively flat area impacted by construction that extends outward from the edge of the traveled way. Because of the limited horizontal clearance available and the heightened awareness of motorists through work zones, the clear-zone requirements are less than the before-construction condition. The amount of available clear zone in a work zone affects the decision to delineate or shield exposed hazards such as Portable Concrete Barrier (PCB) ends, fixed objects, steep slopes, or drop-offs.

TABLE 9.1 Example of clear-zone widths for work zones

Speed (km/h)	Widths (m)	Speed [mph]	Widths [ft]
100 – 110	9	[60 – 70]	[30]
90	7	[55]	[23]
70 – 80	5	[45 – 50]	[16]
50 – 60	4	[30 – 40]	[13]

Engineering judgment must be used in applying the "clear zone" to work zones. Depending on site restrictions, it may only be feasible to provide an operational clearance. Some designers determine the width of a work-zone clear zone on a project-by-project basis, considering traffic speeds, volumes, roadway geometrics, available right-of-way width, and duration of work. Others, for ease of application, use a specified width.

Where roadside space is available, the width of commonly used work-zone clear zones ranges from 4 m to 6 m [12 ft to 18 ft]. The location of collateral hazards such as equipment and material storage can be controlled and should be subject to greater clear-zone widths such as 10 m [30 ft].

Generally, for ease of application of the clear zone in work zones, there is no adjustment made for horizontal curves.

Table 9.1 lists one State's clear-zone width guidance based on speed.

9.2 TRAFFIC BARRIERS

Work-zone traffic barriers are designed either as permanent barriers, as previously described in this Guide, or as temporary barriers that can be easily relocated. These barriers have several functions: (1) to protect traffic from entering work areas such as excavations or material storage sites; (2) to provide positive protection for workers; (3) to separate two-way traffic; (4) to protect construction such as falsework for bridges and other exposed objects; and (5) to separate pedestrians from vehicular traffic.

9.2.1 Temporary Longitudinal Barriers

Use of temporary longitudinal traffic barriers should be based on an engineering analysis. There are a number of factors such as traffic volume, traffic operating speed, offset, and duration that affect barrier need within work zones. However, improper use of temporary traffic barriers can provide a false sense of security for both the motorist and the worker. Therefore, care must be taken in their design, installation, and maintenance. The PCB is the option preferred by most state transportation agencies. Several other temporary traffic barrier designs are also available that may be appropriate for work-zone applications. Although no consensus on specific warrants exists, barriers are usually justified for bridge widening; shielding of roadside structures; roadway widening (especially with edge drop-off); and for separating two-lane, two-way traffic on one roadway of a normally divided facility (3). (See Table 9.2.)

9.2.1.1 Portable Concrete Safety-Shape Barriers

Portable concrete safety-shape barriers, also known as PCBs, are widely used in work zones to shield motorists as well as workers.

PCBs are free-standing, precast, concrete segments, 2.4 m to 9 m [8 ft to 30 ft] in length, with built-in connecting devices. Barrier weight varies from 600 kg/m to 750 kg/m [400 lb/ft to 500 lb/ft] depending on exact cross-section, geometry, and amount of reinforcement. The mass of individual segments can vary from 2000 kg to 7500 kg [4,500 lb to 16,500 lb], thus requiring heavy equipment for installation and removal. Adequate longitudinal reinforcement and positive connections ensure that the individual segments act as a smooth, continuous unit.

The impact performance of PCBs depends, among other factors, on segment length and mass, the manner in which segments are joined, the joint rotation, and the manner in which segments are anchored.

The acceptable cross sections are the same as those described in Chapter 6. Bottom corners of barrier segments may be beveled to minimize snagging of snow plows and to allow placement of the barrier segments in curves. A disadvantage is that, with the removal of the corners, resisting moment to lateral displacement is reduced.

TABLE 9.2 Temporary longitudinal barriers

	PCB	Quickchange®	Low Profile	Plastic Shell/Steel
Structural Adequacy	Varies depending on the type of joint	TL-3*	TL-2*	TL-2 & TL-3*
Deflection	0 – 1.5 m [0 – 5 ft]	1.5 m [5 ft]	0.125 m [5 in]	3.8 & 3.9 – 6.9 m [22.6 ft]
Uses	Two-lane, two-way operation Shielding obstacles and falsework Shielding pavement edge drop-offs	Shielding for changeable lanes	Work sites in urban and suburban areas where sight distance is a problem	Shielding where high portability is desired; i.e., rapidly changing and emergency traffic control measures Protection in congested urban work sites

* NCHRP Report 350 Test Level

When impacted, the mass of the PCB and friction between the PCB and the underlying surface tend to limit movement and overturning. Each section should be properly connected to the adjacent section to provide barrier continuity to resist movement, snagging, and the instability of impacting vehicles. When lateral displacement of the barrier cannot be tolerated, it may be necessary to anchor the PCB to the underlying surface to prevent lateral movement. This can be done with drift pins or anchor bolts attached to the pavement or bridge deck. The pins or bolts should not protrude beyond the face of the PCB. (See Section 9.2.1.1.3.3.) A mechanism to limit sliding is to provide a mechanical interlock between the barrier and the pavement surface. Placing the PCB on a grout bed can provide this mechanical interlock. Through a research study underway at the time of the development of this edition of the Guide, researchers are developing methods to reduce lateral deflection by stiffening PCB joints. As the results of this research become available, they will be posted on the FHWA safety hardware website.

The designer should allow for adequate drainage through the PCB to prevent ponding.

9.2.1.1.1 Flare Rates

Flare rates for temporary barriers should be selected to provide the most cost-beneficial safety treatments possible. Low flare rates lead to longer flared sections and increase the number of impacts with the temporary barrier. Higher flare rates lead to shorter flared sections and fewer impacts but, for those impacts, increase the severity of redirection crashes and the number of barrier penetration crashes. Benefit/cost analyses of temporary concrete barriers indicate that total accident costs appear to be minimized for flare rates ranging from 4:1 to 8:1. A flare rate of 5:1 or 6:1 may be slightly more favorable for urban streets with high traffic volumes where speeds are lower and impact angles are higher (4).

9.2.1.1.2 Offset

A minimum offset of 0.6 m [2 ft] from the traveled lane to the PCB is desirable.

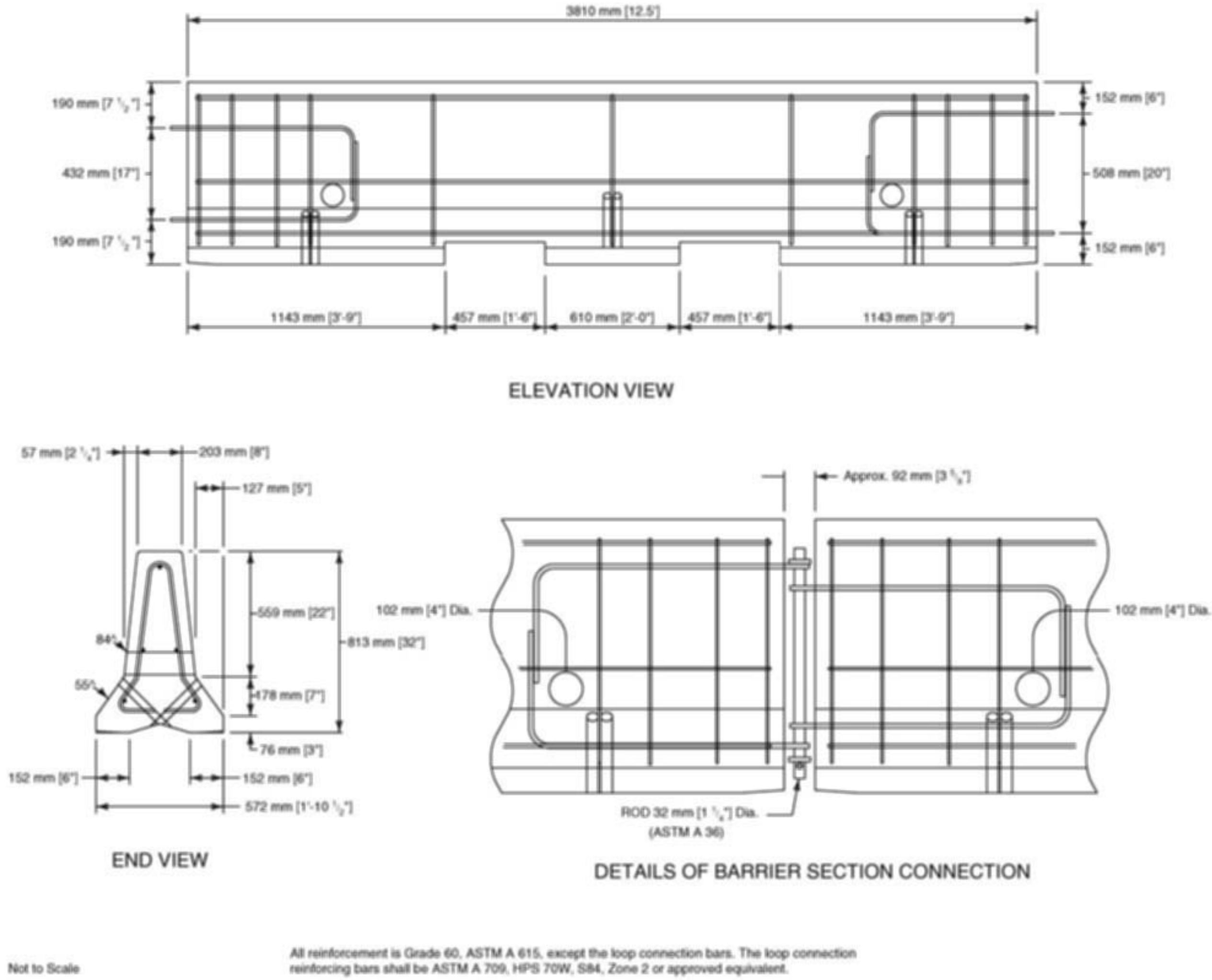


FIGURE 9.1 Iowa Temporary Concrete Barrier

9.2.1.1.3 Types of Portable Concrete Barrier (PCB) Systems

To perform properly and redirect vehicles, the PCB system should be capable of withstanding severe impacts. A PCB system's weakest point is its joint, which includes the physical connection and mating faces of adjoining segments. The methods for connecting PCB segments vary widely.

Many types of PCB connections have been crash tested and evaluated. (Refer to Chapter 5 for evaluation criteria.) Currently, the performance standards for temporary barriers are contained in NCHRP Report 350. Some versions of PCB connectors have been successfully crash tested with a 2000 kg [4,400 lb] pickup truck impacting at 100 km/h [60 mph] and at a 25-degree angle. Other connectors provide lower levels of performance (5). Depending on site conditions, a temporary installation may not necessarily need to meet the same performance level required of a permanently installed barrier system at the same site. Some existing joint connectors have provided adequate service when used at sites where the intent was to contain shallow impacts of passenger cars.

Satisfactory performance at the various test levels depends upon limiting the rotation of the individual segments by assuring that the connection is installed and maintained exactly as tested. (See Section 9.2.1.1.)

9.2.1.1.3.1 NCHRP Report 350 Tested PCB Systems

The July 1, 1998, *AASHTO-FHWA Agreement on NCHRP 350 Implementation* allowed continued use of any existing PCB until October 2000, but after that date requires the use of a design that transfers both moment and tension between segments and has been tested under NCHRP Report 230 guidelines. After October 2002, new barriers used on construction must meet NCHRP Report 350 guidelines, but previously existing barriers that met the earlier testing requirements can continue in use as long as they remain serviceable. Some state transportation agencies have established phase-out dates for non-Report 350 designs.

The systems listed below meet the TL-3 evaluation criteria. The maximum deflection listed below is for the Test 3-11, which is a pickup truck impacting at 100 km/h [60 mph] and at a 25-degree angle (6). Assuming that the barrier is not anchored at the ends, a similar impact nearer to either end than the impact location in the test condition would likely result in larger lateral deflections.

Iowa Temporary Concrete Barrier—This barrier consists of F-shaped concrete barrier segments 3.8 m [12.5 ft] long. Each barrier segment is 810 mm [32 in.] high with a top width of 200 mm [8 in.] and a base width of 570 mm [22$^{1}/_{2}$ in.]. The design is reinforced with five No. 16 [No. 5] longitudinal bars and fourteen No. 13 [No. 4] shear stirrup loops. (See Figure 9.1.)

Adjacent segments are connected by a pin and loop connection with a 32 mm [1$^{1}/_{4}$ in.] diameter ASTM A 36 steel pin. A top and bottom plate are used on the pin with a 13 mm [$^{1}/_{2}$ in.] bolt with nut through a hole 38 mm [1$^{1}/_{2}$ in.] from the bottom of the pin.

Each steel loop is manufactured from one No. 18 [No. 6] diameter grade 414 MPa [60 ksi] rebar.

The test installation was 81.5 m [267 ft] long with the impact point at approximately 45.5 m [150 ft] from the upstream end. The maximum permanent deflection at TL-3 was 1140 mm [45 in.].

Rockingham Precast Concrete Barrier—This proprietary barrier consists of 810 mm [32 in.] tall, F-shaped, concrete segments 3658 mm [12 ft] long. Each segment contains three No. 16 [No. 5] steel bars running the length of each segment and lapped with No. 18 [No. 6] steel bars at each end. (See Figure 9.2.)

Adjacent segments are connected with slotted tube/T-bar connections. One end of each unit has an integral "T" shape plate cast into the concrete and the opposite end has a slotted steel tube. Two units are connected by lifting one unit and lowering it so that the "T" in the end of one unit slides into the slot in the tube in the end of the other unit.

The test installation was 47.55 m [156 ft] long with the impact point 17.26 m [57 ft] from the upstream end, which resulted in a maximum barrier deflection of 1150 mm [45 in.].

J-J Hooks Portable Concrete Barrier—This proprietary barrier consists of 813 mm [32 in.] New Jersey shaped concrete segments 3658 mm [12 ft] long. Reinforcement in the barrier consists of welded wire fabric throughout its length. (See Figure 9.3.)

Adjacent segments are connected together by steel J-J hooks cast into each segment. These "hooks" are formed from 10 mm [$^{3}/_{8}$ in.] thick steel plates that are connected through the barrier by three No. 16 Grade 420 [No. 5 Grade 60] ASTM A 706/A 706M reinforcing bars.

This barrier meets the requirements for TL-3 when used with a 3658 mm [12 ft] long portable New Jersey shape concrete barrier or with an F-shape concrete barrier having the same base width (600 mm [2 ft]) as the tested New Jersey shaped design.

The free-standing installation comprised of 16 connected segments totaling 58.56 m [192 ft] in length. The impact point was approximately 21.2 m [70 ft] from the upstream end or 1.2 m [3 ft 11 in.] upstream from the joint between segment 7 and segment 8. The maximum deflection under this test set-up was reported as 1.3 m [4 ft 4 in.].

Modified Virginia DOT Portable Concrete Barrier—This barrier consists of 810 mm [32 in.] tall, F-shaped, concrete barrier segments 6100 mm [20 ft] long. Each segment contains three longitudinal No.19 [No. 6] bars and one longitudinal No. 13 [No. 4] bar. (See Figure 9.4.)

Adjacent segments are connected by 25 mm [1 in.] diameter ASTM A 36 steel pins 610 mm [24 in.] long, which pass through loops fabricated with 20 mm [$^{3}/_{4}$ in.] diameter steel bars. ASTM F 488 steel washers are used under the pinhead and above the 25 mm [1 in.] hex nut used to retain the pin at the bottom.

The test installation included five 6100 mm [20 ft] segments with two 3100 mm [10 ft] long segments added at each end of the installation, making a total length of approximately 43.3 m [142 ft]. The impact point was 1.3 m [4 ft 4 in.] upstream from the joint between segments 2 and 3, or approximately 17 m [56 ft] from the upstream end of the test installation. The maximum barrier deflection was 1830 mm [6 ft].

GPLINK® Pre-cast Temporary Concrete Barrier—This barrier consists of 870 mm [34 in.] tall concrete segments 6 m [20 ft] long. The width at the base supports is 440 mm [17 $^{5}/_{16}$ in.] and the barrier itself is 240 mm [9$^{1}/_{2}$ in.] thick with vertical sides. Steel reinforcing consists primarily of ten 16 mm [$^{5}/_{8}$ in.] diameter steel bars. (See Figure 9.5.)

Adjacent segments were connected with 680 mm [26$^{3}/_{4}$ in.] long, 22 mm [$^{7}/_{8}$ in.] diameter steel rods inserted through holes in steel plates, two of which are cast into each barrier segment.

The Test 3-11 installation was 96 m [315 ft] long consisting of 16 concrete segments of 6 m [20 ft]. It was impacted 35 m [115 ft] from the start of the barrier, resulting in a deflection by the pickup truck of 1760 mm [69 in.].

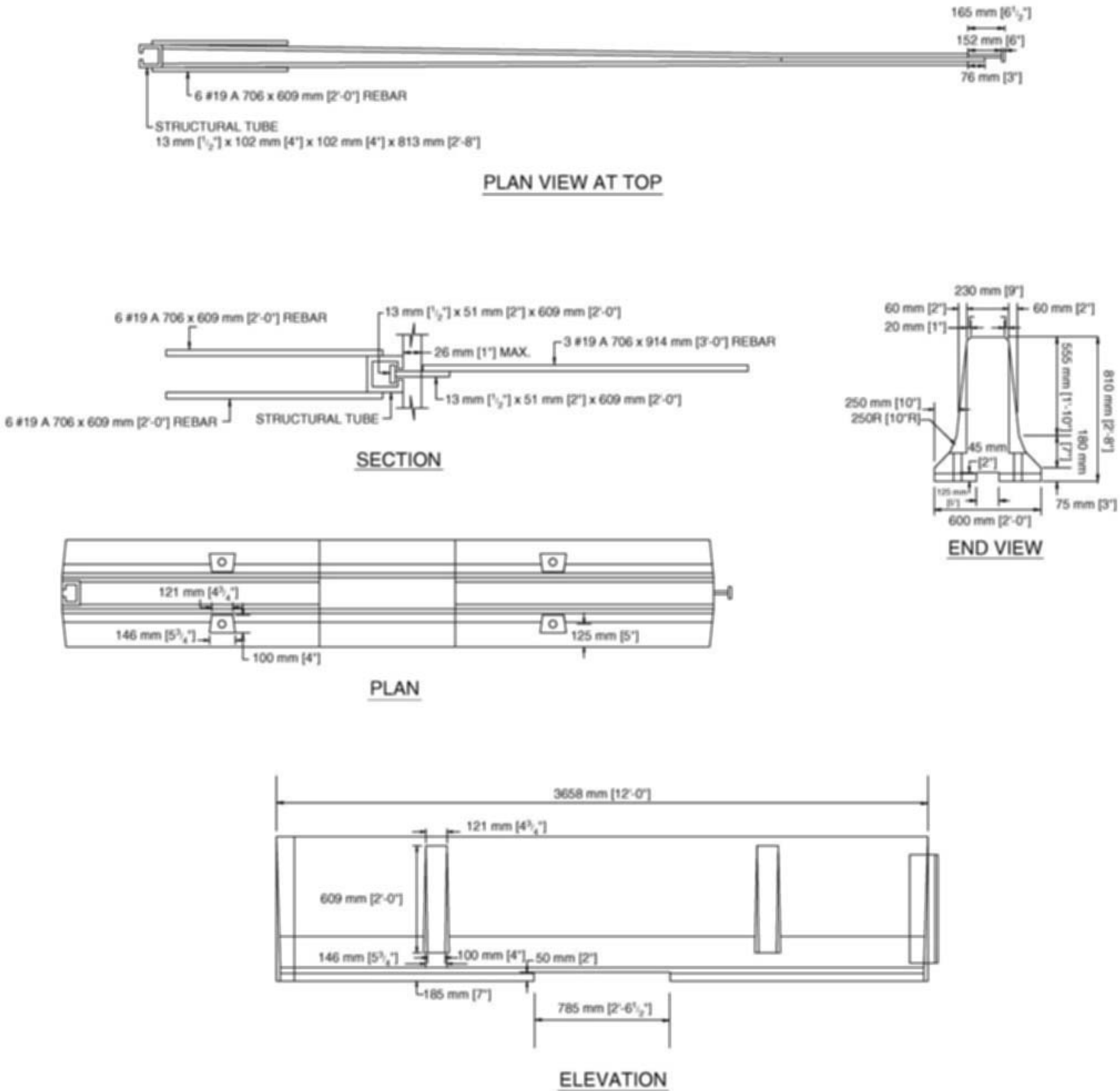


FIGURE 9.2 Rockingham Precast Concrete Barrier

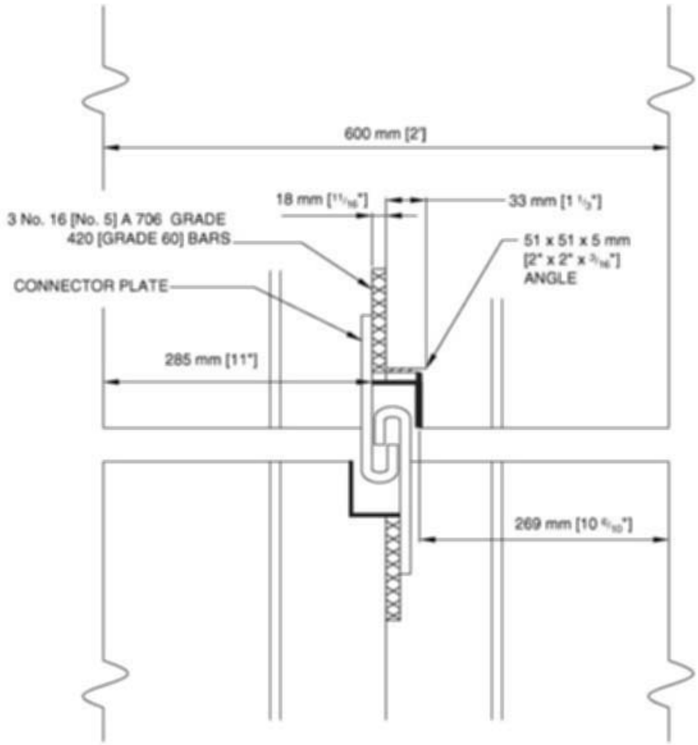


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FIGURE 9.3 J-J Hooks Portable Concrete Barrier

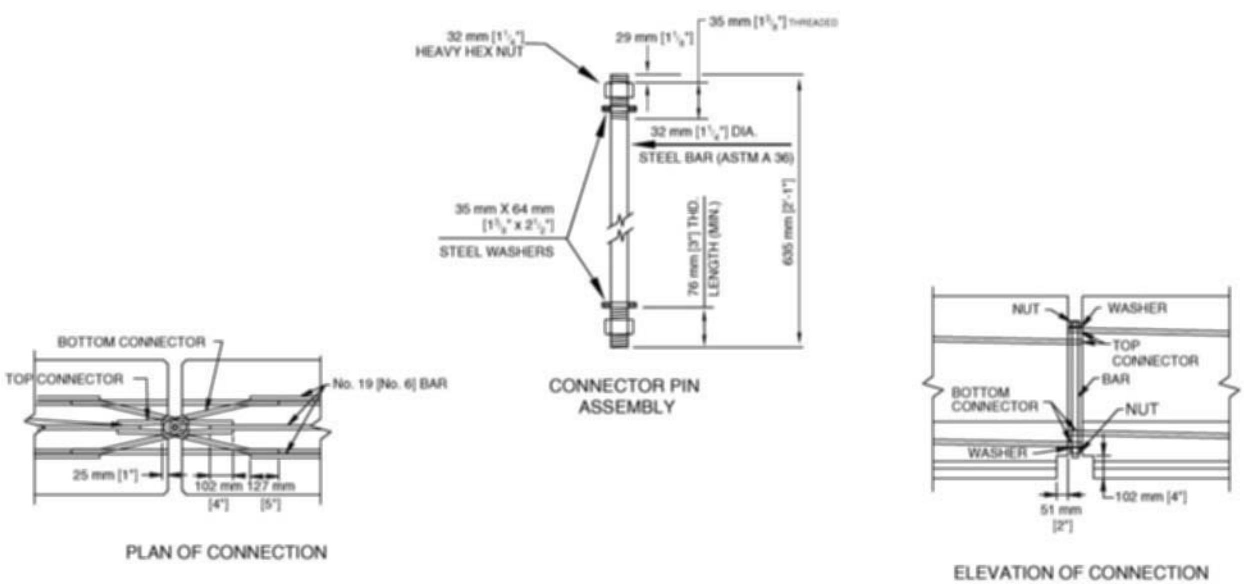

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FIGURE 9.4 Modified Virginia DOT Portable Concrete Barrier

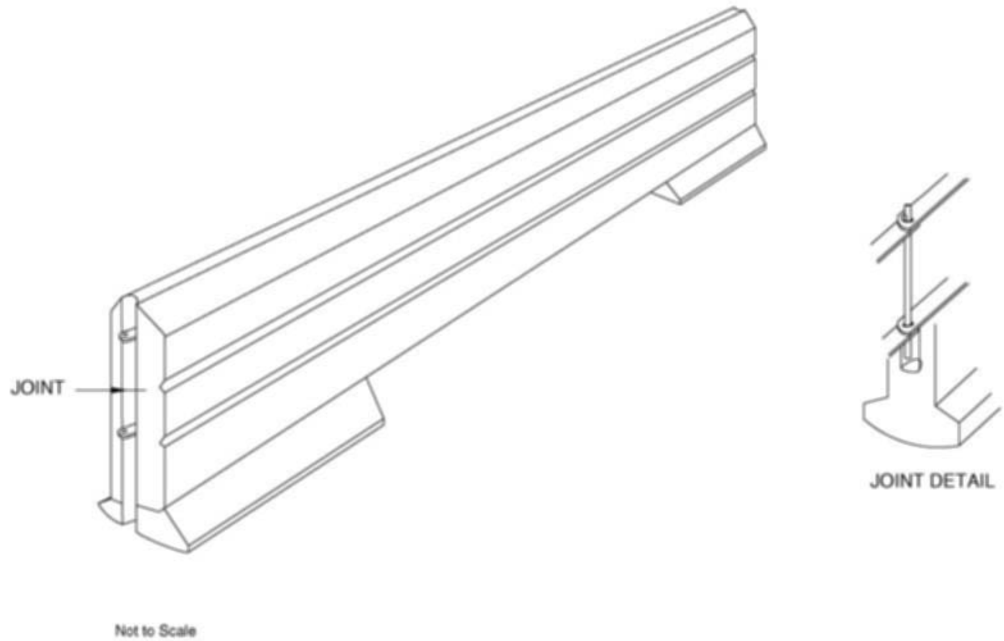

FIGURE 9.5 GPLINK® Precast Temporary Concrete Barrier

Georgia Temporary Concrete Barrier—This barrier consists of 810 mm [32 in.] tall New Jersey-shaped concrete segments 3 m [10 ft] long. Base width is 760 mm [2 ft 6 in.] and the barrier tapers to a 300 mm [12 in.] top width. Reinforcing consists primarily of six longitudinal No.13 [No. 4] bars with three bars located on each face of the barrier. Eleven V-shaped No.13 [No. 4] steel bars (four at each end on 200 mm [8 in.] centers and three evenly spaced between the ends) are used in each segment. (Figure 9.6.)

Adjacent segments are connected by a 638 mm [25 in.] long, 32 mm [$1^1/_4$ in.] diameter A 307 steel double hex bolt inserted through four loops (two at each end of each barrier segment) made from No. 16 [No. 5] steel bars and retained with a hex nut at its lower end. The use of larger washers than used in the crash tested design are suggested to strengthen the pin assembly.

The TL-3 test installation included eighteen barrier segments for a total installation length of 55.3 m [181 ft] (11). The impact point was 1.2 m [3 ft 11 in.] upstream from the connection between segments 7 and 8, or approximately 20 m [66 ft] from the upstream end of the test installation. The dynamic and permanent deflection of the barrier was reported to be 1930 mm [6 ft 4 in.] and 1880 mm [6 ft 2 in.] respectively.

Idaho 20-ft New Jersey Portable Barrier—This barrier consists of 810 mm [32 in.] tall New Jersey shaped concrete segments 6095 mm [20 ft] long. The base width is 610 mm [24 in.] and the top width is 150 mm [6 in.]. Each segment weighs approximately 3630 kg [8,000 pounds]. (See Figure 9.7.)

Adjacent segments are connected using 31.8 mm [$1^1/_4$ in.] diameter steel pins passed through four loops made from 19 mm [$^3/_4$ in.] diameter steel bars. Longitudinal reinforcement consists primarily of six No. 16 [No. 5] bars per segment. Two different connection designs were tested. The first consists of galvanized 32 mm diameter x 638 mm long [$1^1/_4$ in. x 25 in.] A 307 hex bolts secured by 32 mm [$1^1/_4$ in.] A 536 heavy hex nuts. Two ASTM F 844, Type A, wide washers were used, one under the bolt head and one above the nut. The connection in the second test was a 32 mm [$1^1/_4$ in.] diameter A 36 steel pin that was 660 mm [26 in.] long. No locking nut or other pin retention device was used in this design. The steel loops were identical in both tests.

The test installation was 73.2 m [240 ft] long and the pickup truck impacted the barrier 1.2 m [4 ft] from the midpoint. The maximum permanent deflection was 1 m [3 ft] with the bolted connection and 1.1 m [3 ft 7 in.] with the pinned connection.

California K-Rail Portable Concrete Barrier for Semi-Permanent Installations—A New Jersey shaped PCB pinned-with-stakes design that is compliant with the TL-3 criteria was developed by the California Department of Transportation. The PCB segment was 810 mm [32 in.] tall and 6.1 m [20 ft] long. The base width is 610 mm [24 in.]. Each segment weighs approximately 3630 kg [8,000

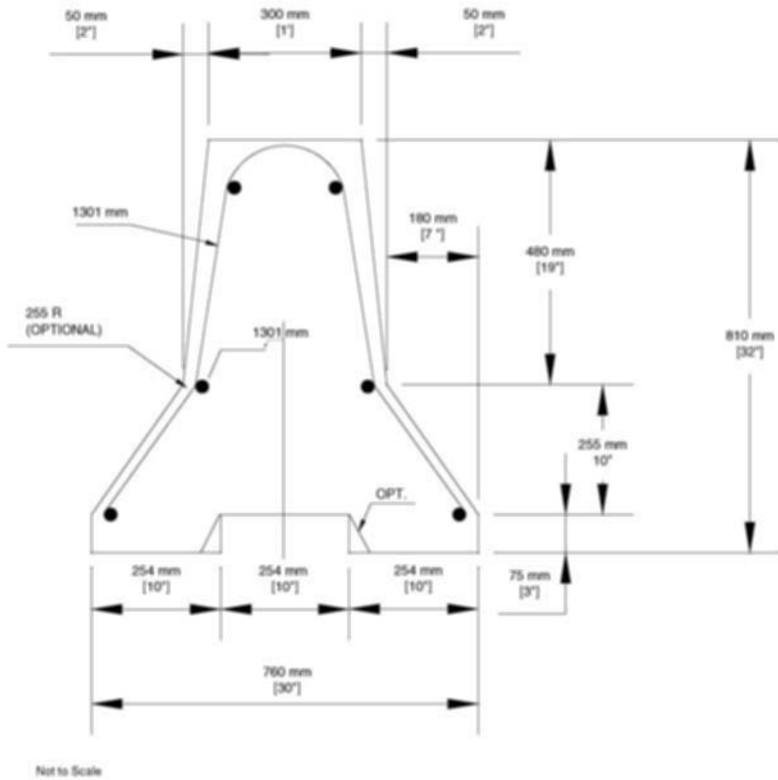

FIGURE 9.6 Georgia Temporary Concrete Barrier

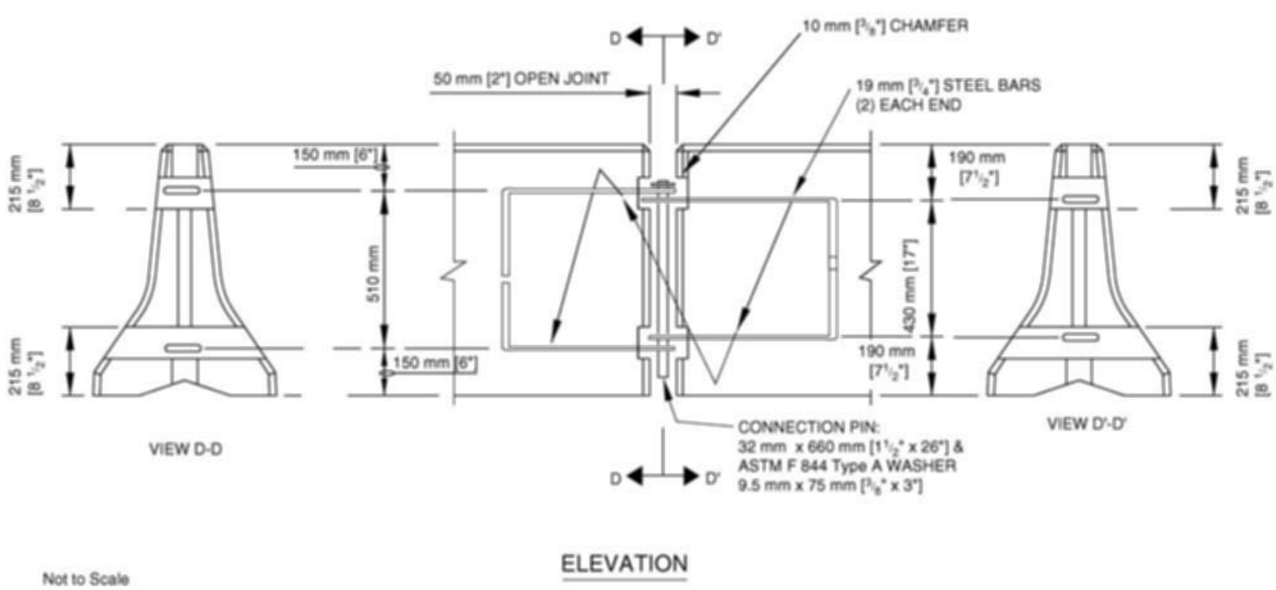

FIGURE 9.7 Idaho 20-ft New Jersey Portable Barrier

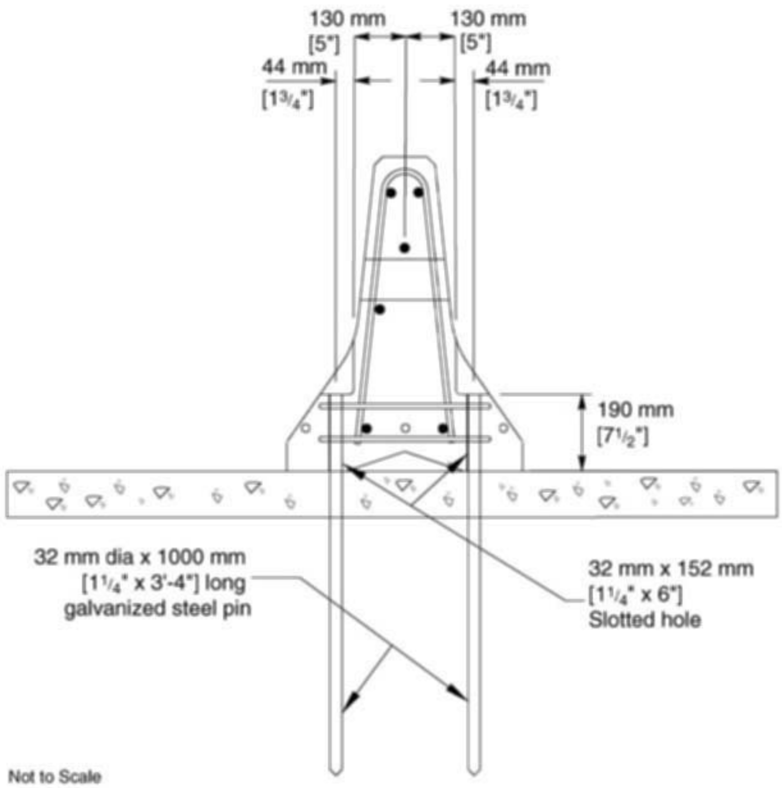

FIGURE 9.8 California K-Rail (PCB) for semi-permanent installations

pounds]. Adjacent segments are connected using 31.8 mm [1¹/₄ in.] diameter steel pins passed through four loops made from 19 mm [³/₄ in.] diameter steel bars. Longitudinal reinforcement consists primarily of six No. 16 [No. 5] bars per segment. Additionally, each segment is staked to an asphalt concrete pavement with four 25 mm [1 in.] diameter by 610 mm [24 in.] long steel stakes driven through holes cast in the lower sloped section of the PCB near each corner. The head of each stake is driven below the traffic face of the barrier to prevent snagging. A stake length of 1 m [3 ft 4 in.] and installation on an asphalt concrete pad having a minimum thickness of 50 mm [2 in.] and a minimum width of 1.2 m [4 ft] is recommended. As an alternative to an asphalt pad, the PCB may be installed on a compacted base material having a minimum thickness of 150 mm [6 in.] and a width of at least 1.2 m [4 ft]. (See Figure 9.8.) The test installation was 48.77 m [160 ft] long and consisted of eight segments.

The Test 3-11 of a pickup truck impacting at 100 km/h [60 mph] at a 25-degree angle resulted in a maximum permanent deflection of 70 mm [2³/₄ in.]. Maximum dynamic deflection at the top of the PCB was reported to be 260 mm [10 in.].

9.2.1.1.3.2 Tested and Operational Connections

Listed below are connections that have been crash tested under procedures in existence before NCHRP Report 350 (7). These connections are considered operational.

The AASHTO-FHWA Agreement on the NCHRP Report 350 Implementation states that a PCB will be considered crashworthy for use on the National Highway System if: (a) it has been crash tested and met the acceptance requirements proposed in either NCHRP Reports 230 or 350 (8 and 7, respectively); (b) it is a barrier with one of the five joints listed as "Tested and Operational Connections" in the 1996 AASHTO *Roadside Design Guide*; or (c) if an engineering study of in-service performance demonstrates that the barrier will provide the performance requirements of the site where it is to be used. A discussion of the five "tested and operational" joints follows.

Pin and Loop Joint—This joint is constructed by casting steel loops into each end of the barrier segments. The loops are then positioned so that they overlap and a steel pin is inserted through the loops. (See Figure 9.9.) There are several varieties of pin and loop connectors. They

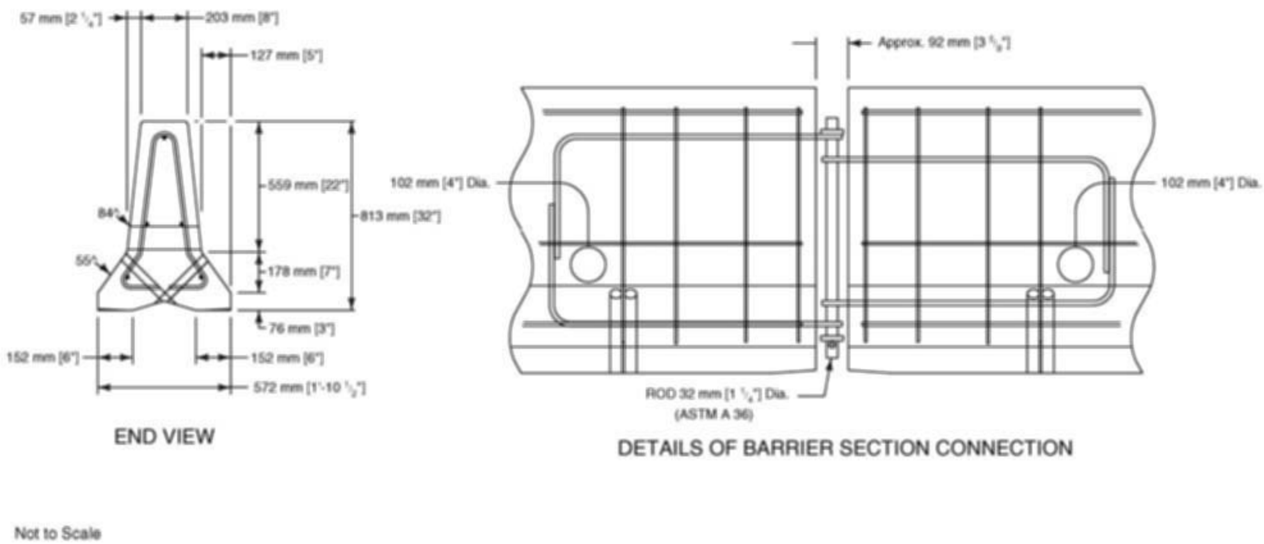

FIGURE 9.9 Pin and Loop Joint

differ according to gap width, pin diameter, manner in which the pin is secured, loop embedment length, and material used to form the loops. Such materials include steel eyebolts, smooth or deformed bars, and cable or wire rope. The wire rope may extend partially into the barrier or continue through the entire length of the segment. A trend is to use plain steel bars for loops, instead of wire rope and rebar loops, to obtain more consistent fracture toughness.

A successfully crash-tested version of this joint used reinforcing steel bar to form four loops (two loops in each barrier end). However, this joint design may allow significant lateral deflection before developing moment between two barrier segments. Therefore, this barrier design may allow large deflections under severe impact conditions, especially if short segments of barrier (less than 3 m [10 ft]) are used. It should be noted that such deflections could be reduced by measures such as placing a board on edge below the lower loops with the intent of removing the slack from the joint. The board should rest on the pavement surface with the ends of the board formed to follow the PCB profile 55-degree angle slope. Other mechanisms that can reduce deflection are referred to in Section 9.2.1.1.

Pins should be secured at both ends of the barrier segment. Securing a pin by drilling a hole and inserting a cotter pin just below the upper loops or slotting the end will retard the pin from jumping out on impact. A nut and washer will prevent a pin from being dislodged from the loops, although they may be difficult to install when the segments are in place and salt corrosion can make them difficult to remove. Capacity of the joint could be improved over the pin and loop joint that was satisfactorily crash tested by making a more positive pin connection (nuts and washers at the top and bottom on the pin), by increasing the size of the pin, and by making the pin from a higher strength steel.

Problems encountered in using these connectors include: the vertical steel pin for pin-and-loop connections may not remain installed since this pin is prone to removal by vandals; the loops may not be structurally adequate because of design deficiencies or previous damage; pin and loop connectors that are too close-fitting may restrict pin installation on curves or at angles. As a result of these problems, smaller pins may be used or pins may be left out, thus weakening the connection.

Channel Splice Joint—This joint is cast with two bolt holes at each end passing through the base of the barrier. Channel splice plates are then bolted to the sides of each adjoining segment. (See Figure 9.10.) Important factors for this connector are the type of channel, channel length, number of bolts, bolt diameter, bolt hole diameter, spacing between bolt holes, and segment length. This joint design can generate moderately high tensile, moment, and shear strength and does not allow significant joint deflection before the moment resistance is generated. This barrier system has been successfully crash tested with a full-sized passenger car impacting at 100 km/h [60 mph] and at a 25-degree angle.

This design has numerous parts and limited tolerances, thus requiring relatively accurate alignment during placement and limited flexibility in accommodating changes in alignment, such as curves or flares.

Vertical I-Beam Joint—This joint is constructed with a slotted steel tube cast into each end of the barrier. The segments are then linked by inserting a steel I-beam through the slotted tubes. (Figure 9.11.) This joint can develop

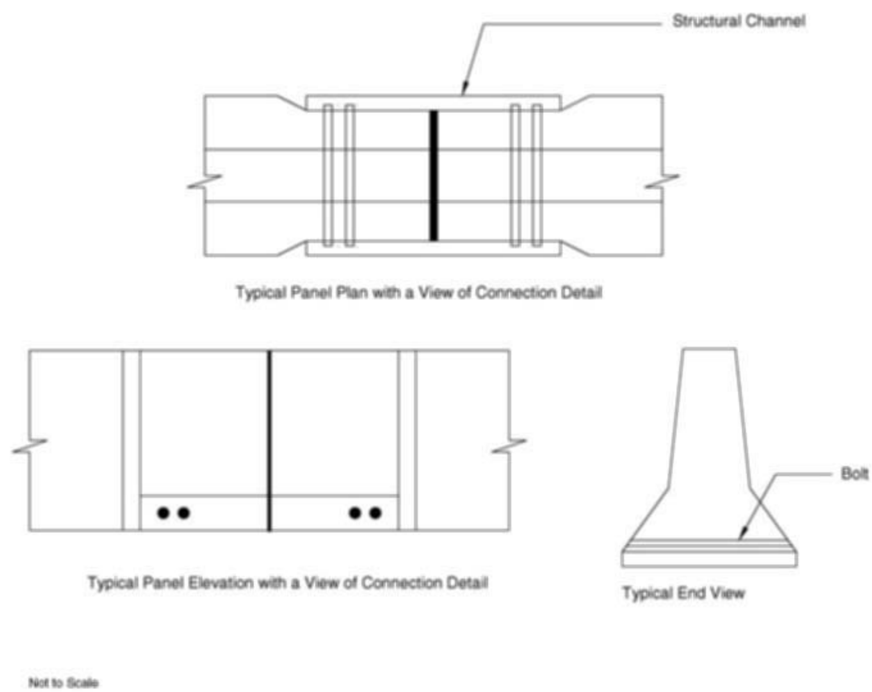

FIGURE 9.10 Channel Splice Joint

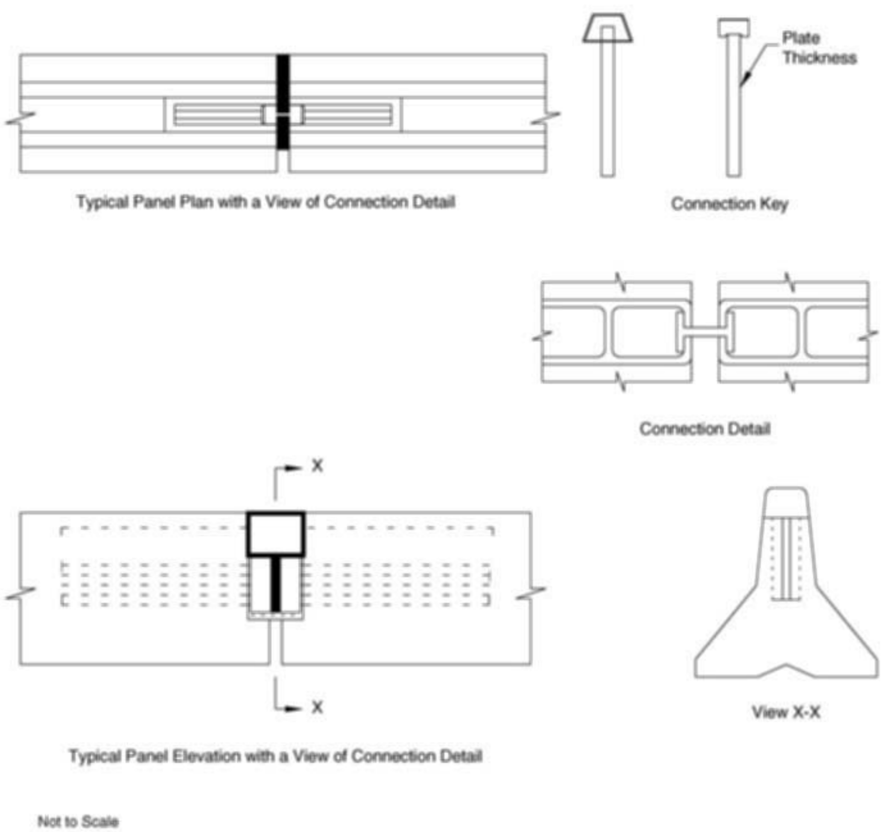

FIGURE 9.11 Vertical I-Beam Joint

very high tensile, moment, shear, and torsional strengths. It has been successfully crash tested with a 1980 kg [4,365 lb] passenger sedan impacting at 97 km/h [60 mph] at a 25-degree angle. The vertical I-beam joint also allows significant barrier movement before developing a restraining moment. Thus, to obtain optimal barrier performance, stepsshould be taken to reduce the amount of movement, such as removing slack from joints or using long barrier segments.

Lapped Joint—This joint is fabricated such that each segment overlaps the next in a vertical plane. The joint is secured with a single steel bolt that passes through the overlapping segments. This joint provides moderate moment and tensile capacity with relatively low shear and torsional strengths. (Figure 9.12.)

J-Hook Joint—This joint is a proprietary connection fabricated of two 254 mm [10 in.] high steel plates bent at the end in a J-hook. A version of this joint meets the NCHRP Report 230 criteria. (Figure 9.13.)

9.2.1.1.3.3 Securing PCBs to the Traveled Way

Bolting to Bridge Deck or Pavement—To ensure minimum or no deflection, PCBs are often bolted to a bridge deck or pavement during construction. While these installations are in common use, there has been only limited crash testing of these to date. Assuming that field performance of a design in use has been acceptable, it should be continued in use until such time as it (or a comparable design) has been successfully crash tested.

Using Staking—A maximum permanent deflection of 70 mm [$2^3/_4$ in.] at TL-3 test conditions has been achieved by a New Jersey shaped PCB pinned-with-stakes design that has been developed by the California Department of Transportation. Please see California K-Rail Portable Concrete Barrier for Semi-Permanent Installations (Section 9.2.1.1.3.1 and Figure 9.8) for a description of the barrier.

9.2.1.1.3.4 Special Cases

Strengthened Barriers (5)—Most catastrophic crashes with PCBs involve heavy trucks or vehicles at high speeds and at high angles. High-angle secondary impacts may occur when barriers are located on both sides of the road or on curves. For these conditions or where minimal deflection distances are available, strengthened, stiffened, or anchored barriers and connectors may be used. Candidate sites include bridges, bridge approaches, excavations, lateral shifts or crossovers, or any roadway where there are two or three parallel runs of barriers.

Restricted Sites—Because of restricted geometry, some sites may require the use of barriers where expected impacts could be at substantially greater than a 25-degree angle. One condition is where there are intersecting roadways that must be kept open, near, or within the work activity area. Detailed guidance to address this condition is found in the NCHRP Report 358, *Traffic Barriers and Control Treatments for Restricted Work Zones* (4). Another condition is where work within an intersection may need PCB to protect workers from an errant vehicle or to protect the public from an obstruction such as a deep excavation. If traffic must be maintained around the work site and the space is insufficient for a recommended PCB layout including end treatment, flare rates sharper than previously recommended for the layout may be justified at the work site.

The following criteria should be considered when required to deploy the PCB at restricted sites:

- PCBs should be used only at low speeds such as 60 km/h [40 mph] or less.
- All sections are to be adequately connected to adjacent sections.
- The end section must be anchored to prevent overturning and excessive sliding.
- Adequate clearance should be provided between the barrier and the work area to allow for sliding of the barrier. If adequate clearance is not available, the PCB should be anchored.
- Precautions must be taken to prevent the PCB from caving into an excavation. When placing a PCB around an excavation, the capability of the soil to withstand the load created by the PCB and any other objects near the cut face must be considered.

9.2.1.2 Other Concrete Barriers

Quickchange® Barrier System—This proprietary PCB system is composed of a chain of modified F-shaped concrete segments 1 m [3 ft] long which can be readily shifted laterally. Steel rods run the length of each segment and specially designed hinges are attached to each end, which are then joined by pins. The top of each system is T-shaped to allow the segment to be picked up by a special vehicle and moved laterally up to 5.5 m [18 ft]. (See Figure 9.14.) (See Chapter 6 for more detail.)

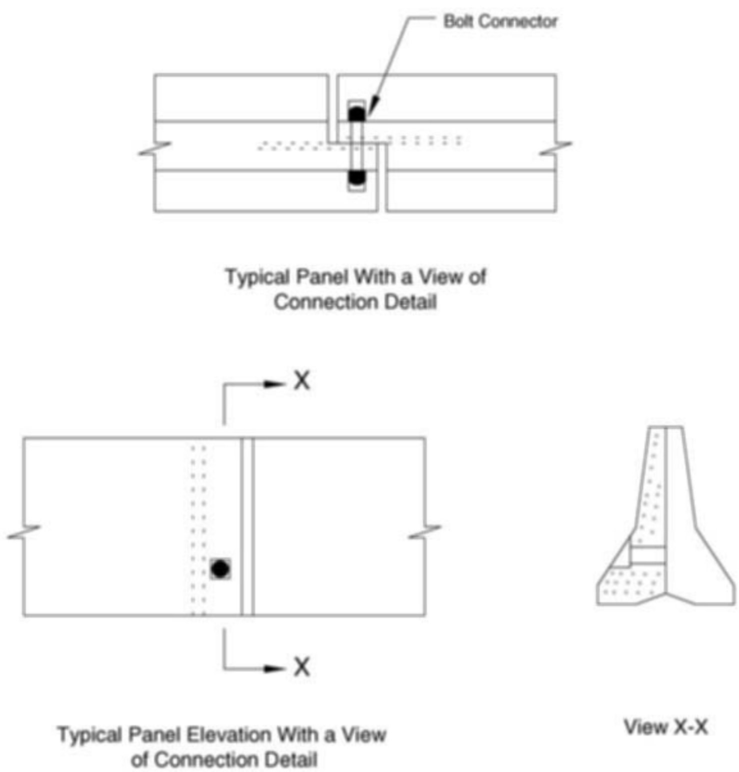

FIGURE 9.12 Lapped Joint

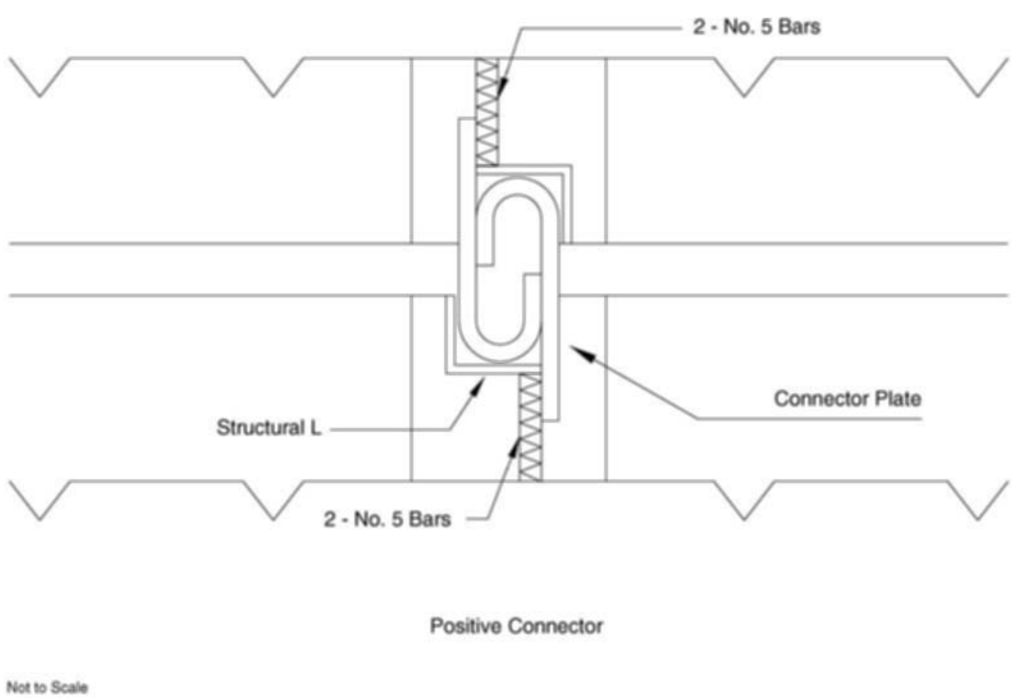

FIGURE 9.13 J-Hook Joint

Low-Profile Barrier System (9)—This proprietary portable precast concrete system is composed of 510 mm [20 in.] high barrier segments. Each segment is 660 mm [26 in.] wide at the base, with a reverse batter of the barrier face at a 20V:1H slope. The purpose of the barrier is to shield the work zone while improving the sight distance for drivers attempting to enter or exit the work zones from side roads or driveways. (See Figure 9.15.)

This barrier was satisfactorily crash tested to the NCHRP Report 350, TL-2 conditions with a 2040 kg [4,500 lb] pickup truck impacting at 70 km/h [45 mph] and at a 25-degree angle, and an 800 kg [1,800 lb] passenger car impacting at 70 km/h [45 mph] at a 20-degree angle. The result was a maximum deflection of 127 mm [5 in.]. It is being installed with a sloped end as the terminal.

9.2.1.3 Other Barriers

9.2.1.3.1 Water-Filled Plastic Shell with Steel Barriers

These are longitudinal barriers of segmented, polyethylene, plastic shells with a steel framework, designed for use with ballast, that have been successfully crash tested to NCHRP Report 350 requirements.

While there are a number of longitudinal plastic devices that have been used in work zones as channelizing devices, only those devices that have been successfully crash tested to the longitudinal barrier requirements of NCHRP Report 350 should be used as barriers.

9.2.1.3.1.1 Triton® Barrier

This proprietary barrier is composed of segments of lightweight polyethylene plastic shells 2134 mm long x 943 mm high x 533 mm wide [84 in. x $37^1/_8$ in. x 21 in.]. They are designed for use with water as ballast. The plastic barrier shell is supplemented by an internal steel framework to provide additional rigidity during handling and impacts. There is also a cable along the top connecting the joints between barrier segments. This cable provides the barrier's tensile capacity during impacts. Vertically aligned, interlocking knuckles at the end allow the sections to be joined with a pin. The pin connection allows the Triton® Barrier section to swivel and be positioned with an inside radius as small as 11.3 m [37 ft]. (See Figure 9.16.)

The Triton Barrier has been satisfactorily crash tested to the NCHRP Report 350, TL-2 conditions of a 2000 kg [4,400 lb] pickup truck traveling at 70 km/h [45 mph] and impacting at a 25-degree angle, and an 820 kg [1,800 lb] car traveling at 70 km/h [45 mph] and impacting at a 20-degree angle.

The test installation was composed of 45 segments that totaled 92 m [292 ft 6 in.] in length. The pickup truck impacted the test installation at mid-point. A total of 19 sections were deflected laterally ahead, 6 upstream and 13 downstream from the point of impact. The maximum lateral barrier deflection was 3.86 m [12 ft 8 in.] at a point 13.87 m [45 ft 6 in.] downstream from the point of attack.

A minimum of 15 barrier sections, totaling 29.7 m [97 ft 5 in.] in length, is needed for an installation when the barrier sections are used as the end treatment. Because of the characteristics of the barrier system, the length of need starts at the beginning of the fifth section although a total of ten sections is needed (with the first section empty) for the end treatment. A test under the conditions of NCHRP Report 350 (TL 2-11 used 15 sections pinned and free-standing with the endmost sections empty of water) resulted in a deflection of 5.5 m [18 ft].

The same Triton Barrier as that discussed above was modified with interior U-bolts at the ends of each module that were double nutted to the interior steel framework. In addition, each module set was strapped to two 178 mm [7 in.] high plastic pedestals to raise the module height. These modifications, accomplished with the use of a Triton TL-3 Kit, were tested with a length of 30 modules that resulted in a maximum deflection of 6.9 m [22 ft 7 in.]. Ten water-filled modules should be used in advance of the barrier length-of-need for expected TL-3 impact conditions. The Triton TL-3 meets NCHRP Report 350, TL-3 for longitudinal barriers.

9.2.1.3.1.2 GUARDIAN™ Safety Barrier

This is a proprietary, polyethylene plastic, longitudinal segmented barrier. Each segment is a plastic shell strengthened with internal baffles and gussets. Each is 1829 mm [72 in.] long, 1067 mm [42 in.] high, weighs 61.1 kg [134.7 lb] empty and nearly 770 kg [1,700 lb] when filled with water. The steel external frame weighs an additional 65 kg [143 lb]. (See Figure 9.17.)

To meet the requirements of NCHRP Report 350, TL-3, the system uses a fabricated steel assembly mounted to the GUARDIAN™ Safety Barrier and linking the segments to each other. This assembly consists of two inverted U-saddles that are placed over the top of each barrier. The saddles create an offset mounting point for the side panels, such that the upper longitudinal pipe is 127 mm [5 in.] from the vertical face of the barrier. The pipe assemblies consist of two formed flat bars with two steel pipes welded across at specific elevations. The pipe assemblies are then secured onto the saddles and secured to the barrier with steel cables that pass under the barrier. The cables prevent the system from lifting upon impact from a vehicle. The barrier is connected to the next in line by means of an external steel pipe sleeve splice element.

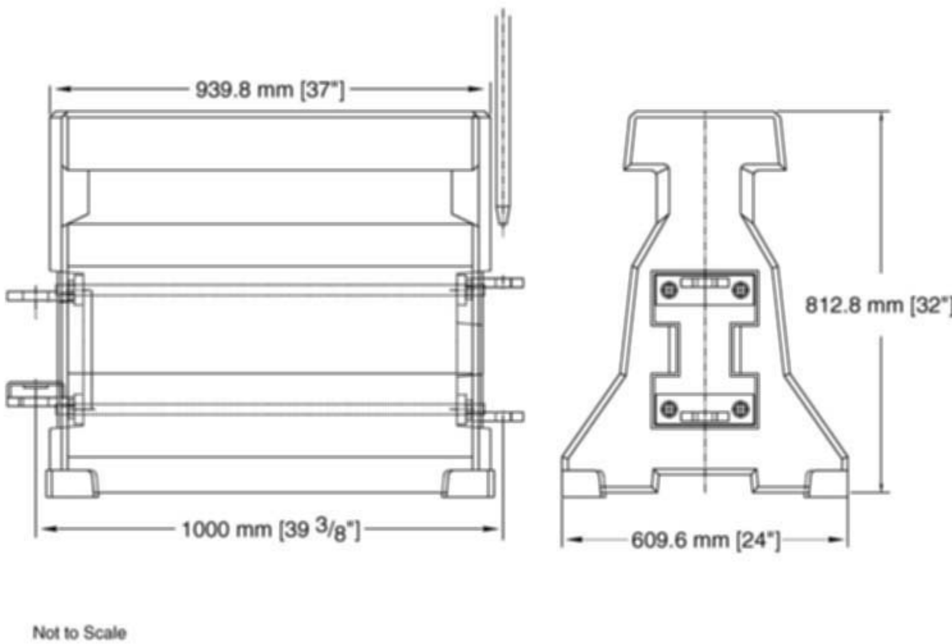

FIGURE 9.14 Quickchange® Barrier System

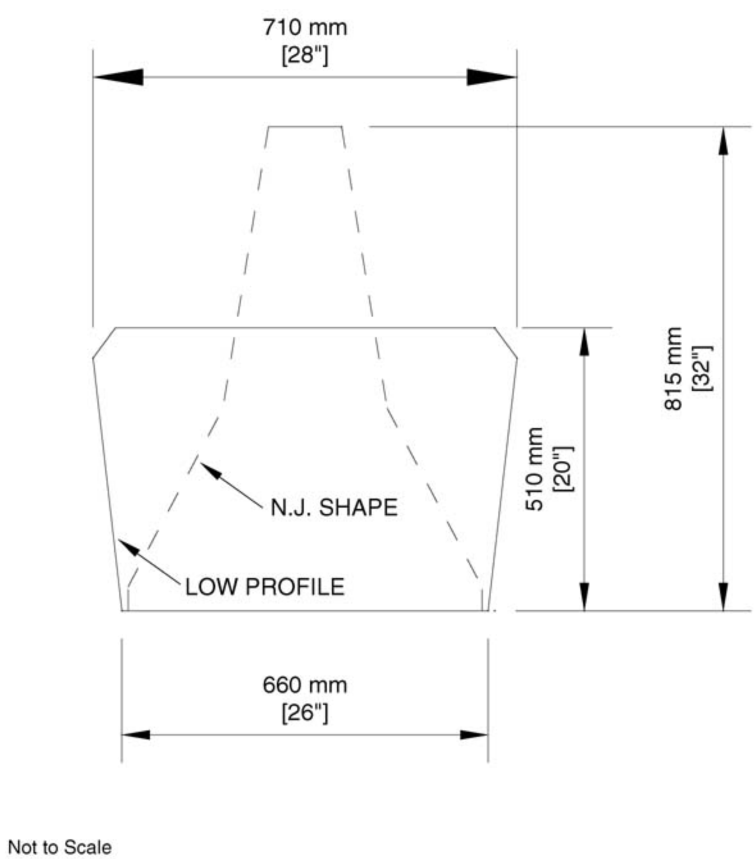

FIGURE 9.15 Low-Profile Barrier System

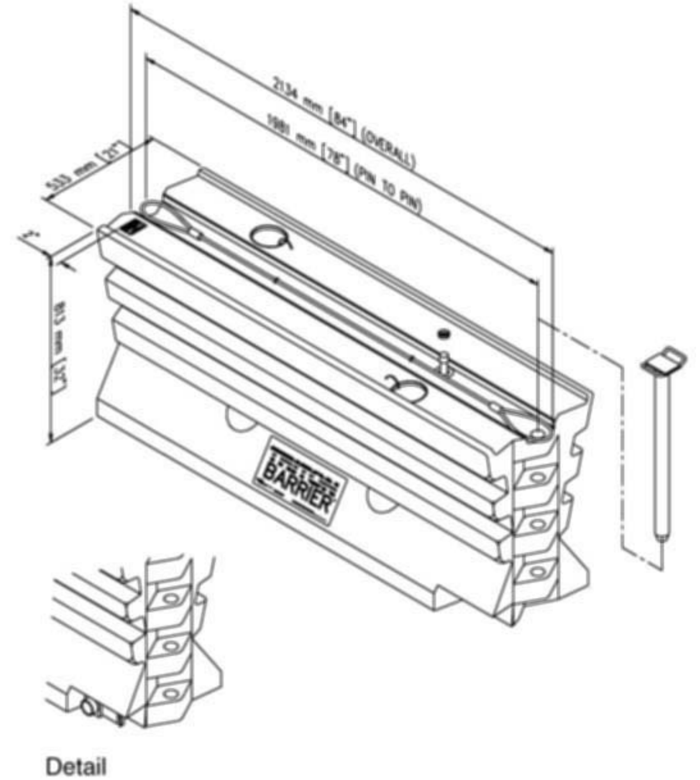

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FIGURE 9.16 Triton® Barrier

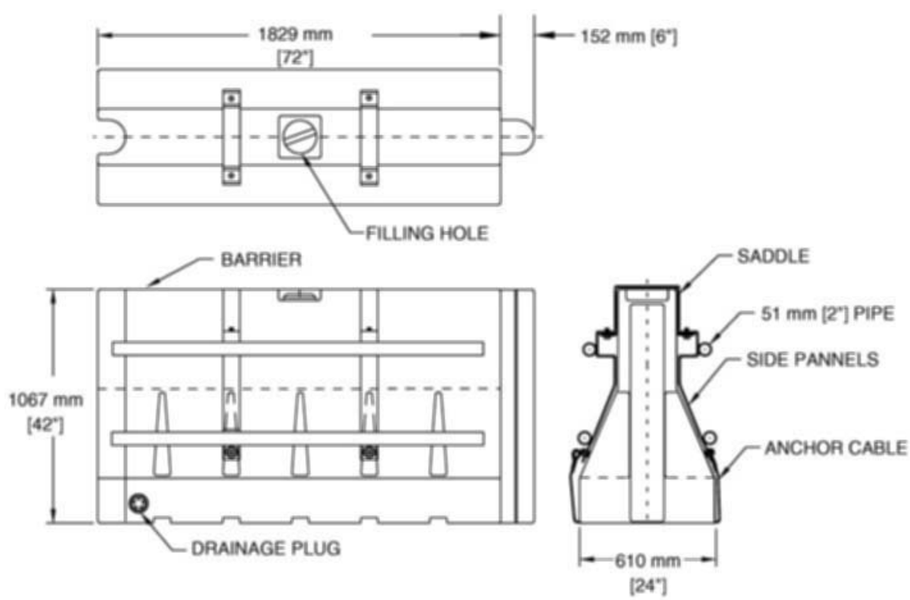

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FIGURE 9.17 GUARDIAN™ Safety Barrier System

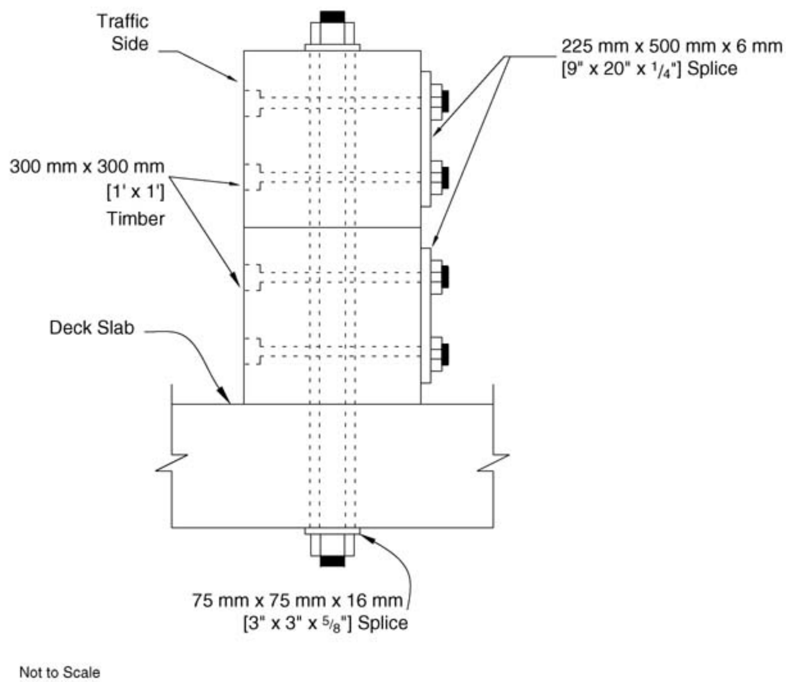

FIGURE 9.18 Timber Barrier Curb/Rail System

The minimum length of need for NCHRP Report 350, TL-3 compliance is 33 interconnected barriers or 60.4 m [198 ft 6 in.] when each segment is ballasted to capacity with a minimum total weight of 818 kg [1,800 lb]. Under these conditions, the system deflected 3.4 m [11 ft 2 in.].

The GUARDIAN Safety Barrier needs to be shielded with a conventional work-zone crash cushion or flared an appropriate distance from the approaching traffic.

9.2.1.3.2 Timber Barrier Curb/Rail

A 300 mm x 400 mm [12 in. x 16 in.] timber curb with a W-beam rail mounted on the 400 mm [16 in.] vertical face of the timber was tested (10). The curb redirected full-sized passenger cars that impacted at about 60 km/h [37 mph] and at a 15-degree angle, displaced less than 300 mm [12 in.]. It may be used where speeds are 60 km/h [37 mph] or less.

A stacked timber barrier for use on a bridge deck, consisting of two 300 mm x 300 mm [12 in. x 12 in.] timbers, has redirected a 2000 kg [4,400 lb] passenger car that impacted at 83 km/h [52 mph] and at a 13-degree angle. It may be used where speeds are 80 km/h [50 mph] or less and the expected impact angle will be shallow. (See Figure 9.18.)

No other timber barrier curb/rail should be used unless satisfactorily crash tested (11).

9.2.2 End Treatments

The desirable treatments for exposed ends of barriers are:

- connecting to an existing barrier (Chapter 5), or attaching a crashworthy end treatment such as a crash cushion (Section 9.3), or
- flaring away to the edge of the clear zone appropriate for construction traffic conditions as determined by the transportation agency (Section 9.1.1), or
- burying the end in the backslope.

For the PCB, either the buried-in-the-backslope or the sloped-end treatment may be used for lower speeds as follows:

- the buried-in-the-backslope treatment (Chapter 5) is recommended for 30 km/h [20 mph]or less with a 1.8 m to 3 m [6 ft to 10 ft] end taper, in case of soil settlement;
- the sloped-end treatment is recommended when other treatments are unfeasible. A sloped end may be used for speeds 45 km/h [30 mph] or less

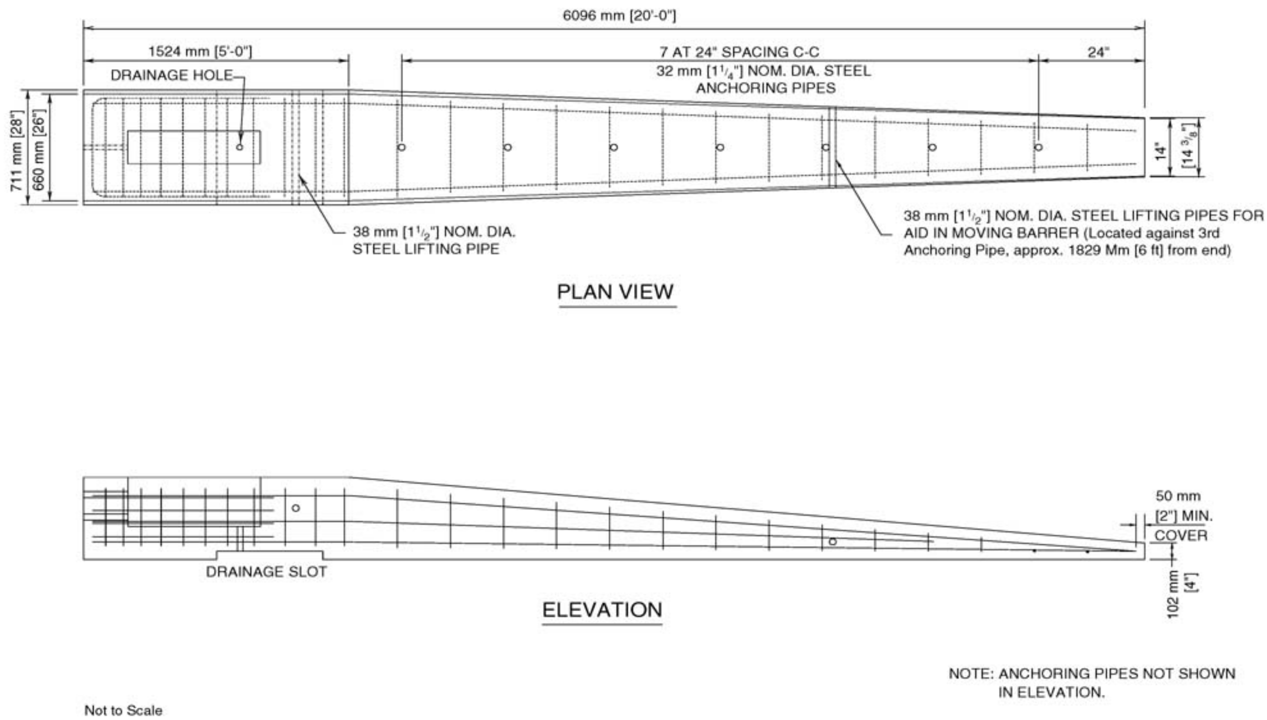

FIGURE 9.19 Low-Profile Barrier Sloped End

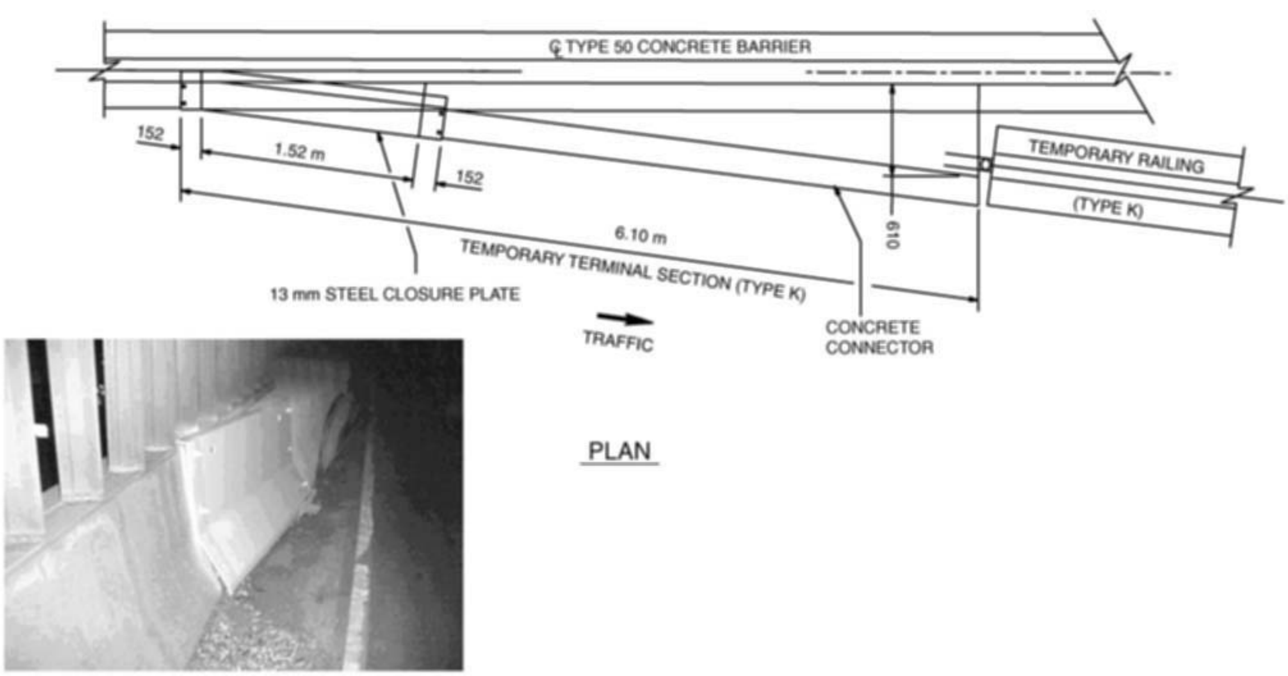

FIGURE 9.20 PCB Steel Plate Transition

> for conditions corresponding to TL-1 in NCHRP Report 350 (9). Generally, as the slope steepness increases, impact severity of this treatment will increase; but the probability of an impact in the sloped section will decrease as the slope increases.

For the Triton Barrier, the first 10 sections with an empty section on the beginning of a length of TL-2 Triton Barrier run has been found satisfactory for use as an end treatment by crash testing.

For the low-profile, portable, concrete barrier, a TL-2 sloped-end treatment consists of a 6.1 m [20 ft] precast concrete unit with a constant slope of the impact face (1V:20H) from the full barrier height of 508 mm [20 in.] to an end height of 102 mm [4 in.]. Lateral deflections are controlled by anchoring the end treatment to the pavement with steel pins inserted through precast holes at 610 mm [24 in.] centers. (See Figure 9.19.)

9.2.3 Transitions

As with permanent barriers, adequate transitions should be made between temporary barriers of differing flexibility or between temporary and permanent barriers.

9.2.3.1 PCB Steel Plate Transition

Often on a freeway, a shielded work zone includes a PCB butted up against an existing permanent concrete median barrier and flared out to the required width of the work zone. This leaves the "blunt-end" of the PCB rail section exposed unless shielded. A solution is to shield the blunt-end with a crash cushion or a transition consisting of a steel plate and a special precast concrete barrier transition section that connects to the permanent concrete median barrier. Versions of this transition are used in at least two states. Both state highway agencies report good experience with the transition.

The steel plate is 760 mm [30 in.] high, 13 mm [$^1/_2$ in.] thick and from 1.5 m to 2.0 m [5 ft to 6 ft 7 in.] long, conforms to the PCB barrier shape, and connects to a PCB transition section that, in turn, connects to a standard PCB segment. (See Figure 9.20.)

9.2.4 Applications

The length of a barrier affects its redirective capability. Shorter lengths may not effectively decrease the risk of injury because they introduce a barrier end that can be hazardous and they may not prevent penetration or provide adequate redirective capability. For a short section of barrier (under 30 m [100 ft]), a trade-off must be made as to which risk is greater—the risk that the obstacle or barrier presents to the motorist, or the risk of leaving an innocent bystander, such as a maintenance worker, unprotected.

Barriers may be used to channelize traffic, but should not be used as the primary tapering device except in low-speed urban conditions or otherwise constricted/restricted work or temporary traffic-control zones. (See reference 5 or the MUTCD for examples.) Lane tapers should be made of more forgiving channelizing devices such as barricades, barrels, or cones. Once the lane is closed, the barrier may be introduced. Barriers perform best when placed parallel to traffic flow.

When temporary barriers are installed on both sides of traffic, the beginnings of the barriers should be staggered to minimize the tendency of drivers to shy away from suddenly introduced objects near the traveled way.

Openings in barriers should be avoided if possible. Where necessary, the barrier ends should have an acceptable end treatment (Section 9.2.2) or offset.

For better night visibility, retroreflective devices or steady-burn warning lights may be mounted along the barrier. (See the MUTCD (2) for guidance.) Under some conditions when horizontal curves are present, the lights may appear as a solid line of lights across the roadway. Under these conditions, it may be better to put lights only on the barrier located on the outside of the curve or combine lights with chevrons or do both. To locate these conditions, a site-specific review may be necessary to determine the optimum lighting setup. Also, a solid edgeline may be placed on the pavement adjacent to the barrier to provide delineation.

9.3 CRASH CUSHIONS

Crash cushions are protective systems that prevent errant vehicles from impacting obstacles by either smoothly decelerating the vehicle to a stop when hit head-on or by redirecting it away from the obstacle for glancing impacts. Two types of crash cushions used in work zones are stationary and mobile (commonly called truck-mounted attenuators).

9.3.1 Stationary Crash Cushions

Crash cushions in work zones may be used in the same manner as at permanent highway installations, i.e., to protect the motorists from the exposed ends of barriers, fixed objects, or other obstacles. A number of stationary crash cushions are commonly used. Refer to Chapter 8 for detailed descriptions, installation requirements, and limita-

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tions. Delineation should be used on stationary crash cushions to make them conspicuous at night.

9.3.1.1 Sand-Filled Plastic Barrels

One type of stationary crash cushion is the sand-filled plastic barrel system. Configurations of sand barrels designed for permanent installations should be used, if space is available. Because of restricted work-zone site conditions and the lack of a feasible alternative in some instances, safety may still be improved by using sand barrels in configurations that are not recommended for permanent installations. Because the sand barrel system has virtually no redirective capability, this system should be 750 mm [30 in.] wider than the fixed object. Where there is inadequate clearance between the crash cushion and work-zone traffic, the following measures should be taken:

1. The barrier layout should be designed so that the approach ends of the temporary traffic barriers are offset to the edge of the clear zone that is appropriate for construction traffic conditions as determined by the transportation agency, or shielded according to the recommendations in Section 9.2.2.

2. The lateral offset between the back edge of a sand barrel crash cushion and the edge of the obstacle may be reduced to a minimum of 375 mm [15 in.] where a greater offset would cause unacceptable interference with traffic.

3. For ease of moving, barrels may be installed on pallets or a skid 100 mm [4 in.] or less in height. Barrels should be regularly inspected since they are susceptible to nuisance hits and provide little or no safety reserve after being hit.

9.3.1.2 QUADGUARD™ CZ SYSTEM

Another type of stationary crash cushion is the QuadGuard™ CZ. (See Figure 9.21.) It is a redirective crash cushion that is essentially identical to that used in a permanent installation mentioned in Chapter 8 except for its anchoring system. The QuadGuard CZ meets NCHRP Report 350, TL-3 criteria when anchored as tested. The specific anchoring system tested used a two-part polyester grout to anchor 20 mm diameter x 460 mm long [$^3/_4$ in. x 18 in.] threaded rods to a foundation of 150 mm [6 in.] deep asphalt over a 150 mm [6 in.] deep compacted subbase. The rods were embedded to a minimum depth of 410 mm [16 in.] in 22 mm [$^7/_8$ in.] diameter drilled holes. A total of 50 anchors are used. Standard installation details, detailed design guides, and installation procedures are available from the manufacturer.

9.3.1.3 TRACC

Another type of stationary crash cushion is the TRACC. (See Figure 9.22.) It is a redirective crash cushion that is identical to the product used in the permanent installation except that it rests on 200 mm [8 in.] of asphalt (or 150 mm [6 in.] of asphalt over 150 mm [6 in.] of compacted subbase). It is anchored with twenty-seven 460 mm [18 in.] long Grade 5 threaded studs set in drilled holes using a polyester resin meeting ACI 349 requirements. It meets NCHRP Report 350, TL-3 criteria for crash cushions. (See Chapter 8.)

9.3.1.4 REACT® 350 CZ

Another type of stationary crash cushion is the REACT® 350 CZ. (See Figure 9.23.) It is a redirective crash cushion which is essentially identical to that used in a permanent installation mentioned in Chapter 8 except for its anchoring system. The anchorage used is identical to that for a permanent installation except for the replacement of concrete expansion bolts with 19.1 mm x 203 mm [$^3/_4$ in. x 8 in.] American Railroad Engineering Association (AREA) Washer-Head Timber Drive spikes and the addition of twelve steel C channel anchors, 75 mm x 7.4 kg/m x 915 mm [3 in. x 5 lb/ft x 36 in.], driven adjacent to the front cable anchor plates. For the test, the REACT 350 unit was set on a 50 mm [2 in.] thick asphalt surface over a 254 mm [10 in.] thick base course. It met NCHRP Report 350, TL-3 criteria when anchored as tested. Standard installation details, detailed design guides, and installation procedures are available from the manufacturer.

9.3.1.5 Connecticut Impact Attenuation System (CIAS)

The Connecticut Impact Attenuation System (CIAS) has been modified for use in construction zones as a temporary installation by the Ontario Ministry of Transportation. (See Figure 9.24.) The modification is in the attachment to a freestanding, movable, reinforced concrete anchor block instead of the traditional cast-in-place concrete pad joined with the back wall. An advantage of this system when it is used in construction zones is that it can be separated easily into two or three component parts and relocated. It can also accommodate "nuisance" hits without requiring replacement. The system should be installed, however, on a rigid surface; when used as a temporary system, such as in construction zones, an asphalt surface is suitable.

Chapter 10
Roadside Safety in Urban or Restricted Environments

10.0 OVERVIEW

Generally, the principles and guidelines for roadside design presented in the previous chapters of this Guide discuss roadside safety considerations for rural highways, Interstates, and freeways where speeds are generally higher, approaching or exceeding 80 km/h [50 mph], and vehicles are operating under free-flow conditions. This chapter presents the designer with considerations to enhance safety on uncontrolled access highways in urban or restricted environments. The following conditions are typical for these types of roads or streets: lower or lowering speeds, dense abutting development, limited rights-of-way, closely spaced intersections and accesses to properties, higher traffic volumes, and the presence of special users including mass transit vehicles, delivery trucks, bicycles, and pedestrians including the disabled. These and other conditions influence the design and operation of highways in these areas.

Restricted environments are segments of roads and streets where conditions are different from adjacent sections of the road or street. These areas are not limited to urban environments, as they may also be found in rural or rural-urban transition areas. Examples include areas of restricted right of way, spot development, parks, playgrounds, or other facilities that increase or otherwise affect the vehicular or non-vehicular activity in the area.

Often there is no clear demarcation between rural and urban conditions. The rural-urban transition area where traffic is leaving a rural type setting and entering an urban type setting is commonly referred to as "suburban." Operating speeds reduce, but in many cases speeds tend to remain high, especially in off-peak hours. The number of abutting property access points and intersections becomes more frequent. Bicycle and pedestrian activity is also likely to increase. Generally, traffic volumes increase and the levels of service decrease. As one leaves an urban area, the process reverses. In major metropolitan centers, the area classified as suburban can radiate outward from the urban center for tens of kilometers [miles].

There may also be whole communities that are separated from the metropolitan center by rural-like conditions but function like a suburban area. Often these "bedroom communities" do not display many of the characteristics of a true urban area. The designer must be careful to design operational and safety treatments for highways in these communities and suburban areas based on their operating characteristics rather than blindly "force-fitting" safety and operational treatments used in urban areas.

Unlike rural areas, suburban areas may have more than occasional pedestrian and bicycle activity. Consequently, roadside safety for both motorists and non-motorists becomes more of a consideration.

In high-speed areas or on controlled-access facilities, protection for pedestrians from possible errant vehicles may be prudent as well as the placing of fencing or barrier to discourage pedestrians or bicyclists from entering the roadway.

Section 2.1.2 mentions that the highway designer has a significant degree of control over roadside geometry and appurtenances. This statement is more applicable for rural conditions and especially so for new rural highways. In urban or restricted conditions, however, the roadside environment (houses, businesses, trees, utility poles, signals, walkways, etc.) is already established and less flexible. Consequently, the designer has the challenge of providing roadside safety given the many pre-existing constraints at hand.

Existing road and street traffic volumes usually increase over the passage of time resulting in the need to make decisions regarding additional capacity. Designers must be cognizant that roadway widening may result in more potential conflicts for pedestrians and bicyclists that use the space both within and immediately adjacent to the facility. Appropriate measures should be considered to provide an adequate level of safety. A safe, efficient, and economical design is the goal.

The various appurtenances such as benches, trash barrels, and bike racks that accommodate pedestrians and bicyclists may be undesirable from the errant motorist's point of view. Ideally, appurtenances should be located where they are not likely to be hit by an errant vehicle. In situations where appurtenances are likely to be hit, they should be of a yielding nature, where practical, to minimize damage to the striking vehicle and its occupants. Breakaway supports for signs should be used unless an engineering study indicates otherwise. However, concern for pedestrians has led to the use of fixed supports in some urban areas. Examples of sites where breakaway supports may be imprudent are adjacent to bus shelters or in areas of extensive pedestrian concentrations. Many situations may need case-specific analysis. Consideration should be given to using breakaway supports for post-mounted signals installed in wide medians.

10.1 NEED FOR INDIVIDUAL STUDY OF SITES

While the clear roadside concept is still the goal of the designer, there are likely to be many compromises in the urban or suburban area. One misconception is that a curb with a 0.5 m [1.5 ft] offset behind it satisfies the clear roadside concept. Realistically, curbs have limited redirectional capabilities and only at low speeds, approximately 40 km/h [25 mph] or lower. Consequently, regardless of curbing, the designer must strive for a wider clear zone that is more reflective of the off-peak operating speed (85th percentile) or design speed, whichever is greater. At the higher speed end of the suburban area or on medium to high-speed urban facilities, serious consideration should be given to providing a full width paved shoulder and offsetting any curbing to the back of the paved shoulder. These shoulders can often be used to accommodate bicyclists and even the occasional pedestrian when sidewalks are not provided. The shoulder can be eliminated, if necessary, further into the suburban area where off-peak speeds are lower.

As always, for reconstruction or resurfacing projects, the crash history should be considered in determining the specific clear roadside treatment for each portion of a project.

The standard hierarchy of design options for the treatment of fixed objects should be considered for each location. They are, in order of preference:

- Remove the fixed object.
- Redesign the fixed object so it can be safely traversed.
- Relocate the fixed object to a point where it is less likely to be struck.
- Reduce impact severity by using an appropriate breakaway device or impact attenuator.
- Redirect a vehicle by shielding the obstacle with a longitudinal traffic barrier.
- Delineate the fixed object if the above alternatives are not appropriate.

10.2 DESIGN SPEED

Urban and suburban operating speeds vary by time of day more than rural operating speeds do. During free-flow conditions, and especially during late night, speeds are much higher; often well beyond the speed limit. Higher speeds result in the potential for more severe accidents, as indicated by the data shown in Table 10.1, which shows the percentage of single vehicle run-off-the-road crashes that occurred from 7:00 p.m. to 7:00 a.m. and from 7:00 a.m. to 7:00 p.m. on urban principal and minor arterials in one state. During the lower volume and higher speed period of 7:00 p.m. to 7:00 a.m., a greater percentage of injury and fatal crashes occur than during the other half of the day. While other factors such as alcohol, fatigue, and limited night-sight distance may contribute to this higher percentage, higher speeds and greater speed variance under free-flow conditions are likely to be significant contributing factors.

Consequently, roadside features need to be designed for the higher operating speeds that occur during free-flow conditions. This may mean that the estimated encroachment speed used to design for roadside features may be higher than the design speed for the roadway as a whole, especially if the off-peak operating speed (85th percentile) was not used to determine the project design speed. Also, as stated in the Preface, "since the design speed is often determined by the most restrictive physical features found on a specific project, there may be a significant percentage of a project length where that speed will be exceeded by a reasonable and prudent driver." Therefore, "the designer should consider the speed at which encroachments are most likely to occur when selecting an appropriate roadside design standard or feature."

TABLE 10.1 Percentage of single vehicle run-off-the-road crashes by severity and time period for urban principal and minor arterials in Illinois

Time Period	Property Damage Only Crashes	Possible Injury and Non-Incapacitating Injury Crashes	Incapacitating Injury and Fatal Crashes	Total
7 p.m.–7 a.m.	34.6%	13.6%	6.8%	55.0%
7 a.m.–7 p.m.	32.3%	8.8%	3.9%	45.0%
				100.0%

10.3 ROADSIDE BARRIERS IN URBAN AND RESTRICTED AREAS

A roadside barrier is a longitudinal barrier used to shield motorists from natural or synthetic obstacles located along either side of a roadway. The primary purpose of roadside barriers is to prevent a vehicle from striking a fixed object or roadside feature that is less forgiving than the barrier itself. This is accomplished by containing and redirecting the impacting vehicle. Barriers are also used to separate pedestrians and bicyclists from vehicular traffic when appropriate. Refer to Chapter 5 for a discussion of performance, structural, and safety characteristics of crashworthy roadside barriers.

An untreated end of a roadside barrier is not desirable since it may penetrate the passenger compartment or stop the vehicle too abruptly when hit. A crashworthy end treatment is therefore considered essential if the barrier terminates within the clear zone or in an area where the barrier is likely to be hit head-on by an errant vehicle. The selection of the proper treatment should be in accordance with the proposed test levels, warrants, and availability of maintenance. To be crashworthy, the end treatment should not spear, vault, or roll a vehicle for head-on or angled impacts.

Intersections and driveways complicate the selection and use of end treatments. A major factor in selecting and locating end treatments is obtaining the necessary corner sight distance at these locations. Refer to Chapter 8 for further guidance on this subject.

Aesthetic concerns can be a significant factor in the selection of a roadside barrier in environmentally sensitive locations such as recreational areas, parks, or many urban or suburban environments. In these instances, a natural-looking barrier that blends in with its surroundings is often selected. It is important that the systems used be crashworthy as well as visually acceptable to the highway agency.

Having decided that a roadside barrier is warranted at a given location and having selected the type of barrier to be used, the designer must specify the exact layout required. The major factors that must be considered include the following:

- lateral offset from the edge of pavement
- deflection distance of the barrier
- terrain effects
- flare rate
- length of need
- corner sight distance
- pedestrian activity including the needs of the disabled
- bicycle activity

Generally, a roadside barrier should be placed as far from the traveled way as conditions permit while ensuring that the system performs properly. Such placement gives an errant motorist the best chance of regaining control of the vehicle without striking the barrier. It also provides better sight distance, particularly at nearby intersections.

It is desirable that a uniform clearance be provided between traffic and roadside features such as bridge railings, retaining walls, roadside barriers, utility poles, and trees, particularly in urban areas where there is a preponderance of these elements. The placement of roadside barriers is covered in Chapter 5.

10.3.1 Barrier Warrants

Barrier warrants are based on the premise that a traffic barrier should be installed only if it reduces the severity of potential crashes. It is important to note that the probability or frequency of run-off-the-road crashes is not directly related to the severity of potential accidents.

Typically, barrier warrants have been based on a subjective analysis of certain roadside elements or conditions. If the consequences of a vehicle striking a fixed object or running off the road are believed to be more serious than hitting a traffic barrier, then the barrier is considered warranted. While this approach can be used often, there are instances where it is not immediately obvious whether the barrier or the unshielded condition presents the greater risk. Appendix A presents an analysis procedure that can be used to compare several alternative safety treatments and provides guidance to the designer.

A barrier may be warranted if:

1. there is a reasonable probability of a vehicle leaving the road in that location, and

2. the cumulative consequences of those departures significantly outweigh the cumulative consequences of impacts with the barrier.

Note that there will generally be many more impacts with a shielding barrier than there would otherwise be with the unshielded object.

Highway conditions that warrant shielding by a roadside barrier can be placed in one of two basic categories: embankments or roadside obstacles. Warrants for the first category are found in previous chapters. Low-profile barriers 600 mm [24 in] high for speeds of 70 km/h [45 mph] or less have been developed. They shield without obstructing visibility. The presence of pedestrians or other "bystanders" may justify protection from errant vehicular traffic.

10.3.2 Barriers to Protect Adjacent Land Use

In urban and suburban areas, more consideration should be given to protecting pedestrians who are using adjoining properties from risks posed by errant vehicles. Schools, playgrounds, and parks located on the outside of sharp curves or across T-intersections are examples of where barrier systems may be appropriate. At these locations, the probability of a vehicle leaving the roadway and striking a person or persons in these areas is greater than on tangent stretches of roadway. Because there aren't any specific warrants or guidelines for these situations, design judgment should be used.

Barriers intended to protect adjacent land use must prevent an errant vehicle from entering a specific area. A barrier that is not structurally adequate may be less desirable for the area it was intended to protect than having no barrier at all. Flying debris resulting from the impact of a vehicle into a deficient barrier can injure people in the area.

Consideration should also be given to installing a barrier to shield businesses and residences that are near the right-of-way, particularly at locations having a history of run-off-the-road accidents.

10.3.3 Guidelines for Pedestrian and Bicyclist Barriers

Pedestrians and bicyclists are another category of concern to highway designers. The most desirable solution to this problem is to separate them from vehicular traffic. Since this solution is not always practical, an alternative means of protecting them is sometimes necessary. Presently there are no objective criteria to draw on for installation of pedestrian or bicyclist barriers. On low-speed streets, a curb will usually suffice to delineate/separate pedestrians and cyclists from vehicular traffic. However, at speeds over 40 km/h [25 mph], a vehicle can mount the curb at relatively flat impact angles. Consequently, when sidewalks or bicycle paths are adjacent to the traveled way of high-speed facilities, some provision other than curbing may need to be made for the safety of pedestrians and bicyclists. For additional information concerning bicycle facilities, the reader is referred to AASHTO's *Guide for the Development of Bicycle Facilities* (1).

10.3.4 Pedestrian Restraint Systems

Accidents involving pedestrians account for almost one out of every five traffic fatalities. Pedestrian accidents in some cities have accounted for as many as one-half of the traffic fatalities.

A large percentage (almost 40 percent) of pedestrian deaths occur while crossing streets between intersections; the injury rate shows the same trend. A pedestrian barrier prevents these accidents. Fences or similar devices that separate pedestrian and vehicular traffic have been used successfully to channel pedestrians to safe crossing locations. It is critical when considering a pedestrian barrier that crossings be located within a reasonable walking distance. The feasibility of restricting pedestrian crossings should be determined on a case-by-case basis.

Sidewalk pedestrian barriers are located along or near the edge of a sidewalk to channel pedestrians to a crosswalk or grade-separated facility, or to impede their crossing at undesirable locations. Barriers may also be used outside school entrances and playgrounds. Often it is advisable to contain pedestrians at public transportation stops to prevent pedestrians from encroaching onto the roadway.

Common construction materials for pedestrian barriers include chain-link fencing, pipe and chain/cable, planters or other sidewalk furniture, and hedges. Planters are not recommended if they would be an additional fixed object in an otherwise clear zone. Planters are not recommended on narrow sidewalks where they may impede pedestrian circulation.

Median pedestrian barriers can significantly reduce the number of midblock crossings. Median barriers are frequently chain-link fences located along a median, which prevent pedestrians from crossing at non-intersection locations. They can be installed exclusively as pedestrian barriers or be incorporated with vehicle-separating median barriers. Intersection sight distance should be considered when designing a barrier.

Roadside pedestrian barriers are generally high chain-link fences located along a highway or freeway to prevent pedestrians from crossing the road. Pedestrian barriers should be crashworthy designs. For example, top longitudinal pipe cross bracing should not be used on chain-link fence.

Useful guidance may be found in the latest version of the *Uniform Federal Accessibility Standards* (2). Additional guidance may also be found in the *British Standard Specification for Pedestrian Restraint Systems* (3).

10.4 MEDIAN BARRIERS IN URBAN AREAS

A median barrier is a longitudinal barrier most commonly used to separate opposing traffic on a divided highway. It is also used along heavily traveled roadways to separate through traffic from local traffic or to separate special use lanes from other highway users. By definition, any longitudinal barrier placed on the left side of a divided roadway may be considered a median barrier. For median barriers on high-speed, controlled-access roadways that have relatively flat and traversable medians, refer to Chapter 6.

The use of standard highway median barriers on urban facilities with a design speed of 70 km/h [45 mph] or less with street intersections, regardless of access control, generally is not warranted. Alternate methods of separating opposing traffic are encouraged, such as the use of medians (in some cases raised medians). Flush medians are preferred over raised medians on highways with design speeds greater than 70 km/h [45 mph], since raised medians can cause errant vehicles to vault. Intersection sight distance should be considered when designing a raised median with plantings or barrier.

10.5 BRIDGE RAILINGS

The local variables regarding the placement of urban guardrail, bridge railing, and other barriers become more challenging. The primary reasons are the need to design these features around intersecting ramps and streets, to provide access to properties, and to maintain access for pedestrians, including persons with disabilities.

As detailed in Chapter 7, appropriate bridge railings need to be selected by considering roadway design, traffic volumes, percent of heavy vehicles in the traffic stream, and the volume of pedestrian traffic. The performance requirements of bridge railings for urban areas are no different from any other highway system. However, bridges carrying low traffic volumes at greatly reduced speeds may not need bridge railings designed to the same standard as railings used on high-speed, high-volume facilities. The railing should have adequate strength to prevent penetration by passenger vehicles, while the transition rail section approaching the bridge should be considered with the same selection considerations discussed in previous sections. Transitions that meet Test Levels 1 and 2 in accordance with NCHRP Report 350 are generally acceptable for cases with low roadway speeds. The bridge rail and transition section, nevertheless, must function effectively for the location and conditions selected. Standardization of urban bridge rail systems improves availability of replacement parts for maintenance departments. The FHWA requires a minimum TL-3 bridge railing on NHS projects unless supported by another rational selection procedure.

Highway structures, regardless of location and traffic volume, normally warrant rigid railing. A rigid bridge railing may require an approach guardrail and transition section. When a bridge also serves pedestrians, a barrier to shield them from vehicular traffic may be warranted. Placement of the bridge railing between traffic and the sidewalk affords maximum pedestrian protection. A pedestrian railing would then be needed at the outer edge of the bridge structure. The need for a bridge railing adjacent to the pedestrian walkway should be based upon the volume and speed of the roadway traffic, lane width, curb offset, and alignment. Other considerations include the number of pedestrians crossing the bridge, the crash statistics (if available), and the conditions on either end of the structure. The use of a bridge railing may create a problem unless the railing is terminated in an acceptable manner. Flaring the end section away from the roadway is often not practical because it would encroach upon the sidewalk, requiring the walkway to meander around the transition section and terminal unit.

In some instances, a crash cushion or metal beam barrier terminal can be used to shield the end of a barrier at the edge of a curve. However, the presence of a raised curb may adversely affect the performance of this type of end treatment. In low-speed situations, a concrete tapered end section parallel to the roadway may be the best compromise. Concrete bridge railing should be extended a sufficient length beyond the end of the bridge to protect drop-offs, yet not extend so far as to intrude on the sight distance of adjacent street intersections. Recommended taper lengths are 6 m [20 ft] minimum, with 10 to 13 m [30 to 45 ft] desirable.

Retrofitting existing bridge railings is a challenge. Typically, bridges designed to AASHTO specifications prior to 1964 may have deficient railings (based on current criteria). If the adequacy of a railing appears questionable, further evaluation should be made to ensure that the design meets the current specifications. In many older railing systems, the presence of curbs defines the walkway between the driving lane and the bridge railing. This curb may cause an impacting vehicle to go over the railing or to strike it from an unstable position contributing to the possibility of roll over. Several concrete railings installed on raised sidewalks have been successfully crash tested.

While some retrofit designs for a bridge railing not meeting current guidelines may not bring the railing to full AASHTO specifications, significant improvements can nevertheless be obtained. Chapter 7 outlines a number of retrofit concepts that can be adapted to different types of deficient railings. The metal post and beam retrofit functions well as a traffic barrier separating vehicles from pedestrians that are using an adjacent sidewalk on a bridge (Figure 7.16). In most cases, the metal post and beam system allows the existing bridge railing on a wide raised walkway to be used or converted to a pedestrian rail. Other retrofit means are also available and should be reviewed to determine their appropriateness for the conditions that exist.

10.5.1 Protective Screening at Overpasses

An object or debris that is thrown, dropped, or discharged from an overpass structure can cause significant damage and injuries. Protective screening might reduce the number of these incidents; however, it should be noted that screening will not stop a determined individual. In many cases, increased enforcement may provide a more effective deterrent.

While the most common protective screening in use is for pedestrian type overpasses, other types of screening are used, such as glare screens, to protect oncoming traffic on overpasses. Splash or debris screens are used to protect commercial or residential properties that are beneath or adjacent to the structure.

At present, it is not possible to establish absolute warrants as to when, where, or what type of barriers or screens should be installed. The general need for economy of design and desire to preserve the clean lines of the structures, unencumbered by screens, must be carefully balanced against the requirement that the highway traveler, overpass pedestrian, and property be provided maximum protection.

Various types and configurations of screens, usually of a chain-link fence type, have been installed on overpasses throughout the country in areas where it has been determined that the problem of throwing or dropping objects exists.

The simplest design for use on pedestrian overpasses is a vertical fence erected on the bridge railing of the structure. While this type of design has been effective in keeping children from playing on the railing, the design has proven somewhat ineffective in combating the problem of objects being thrown from the structure. Objects large enough to cause serious damage to passing vehicles can still be thrown over a vertical structure with some degree of accuracy. On pedestrian bridges, a semicircular enclosure has been placed on top of the two vertical walls to discourage this type of vandalism. This design has further evolved into a design with a partially enclosed curved top, which is used in some areas. Objects generally cannot be thrown over the top of a partially enclosed screen with any degree of accuracy.

Care should be taken in the design of chain-link type screens to ensure that the opening at the bottom of the side screens, through which objects can be pushed or dropped, is eliminated or kept to a minimum. Where aesthetics are important, decorative type screening has been used.

Installation of protective screening should be analyzed on a case-by-case basis at the following locations:

- on existing structures where there have been incidents of objects being dropped or thrown from the overpass and where increased surveillance, warning signs, or apprehension of a few individuals has not effectively alleviated the problem;
- on an overpass near a school, playground, or other locations where it would be expected that the overpass would be frequently used by children not accompanied by adults;
- on all overpasses in urban areas used exclusively by pedestrians and not easily kept under surveillance by law enforcement personnel;

- on overpasses with walkways where experience on similar structures indicates a need for such screens; and

- on overpasses where private property that is subject to damage, such as buildings or power stations, is located beneath the structure.

In most cases, the erection of a protective screen on a new structure can be postponed until such time as there are indications of need.

10.6 IMPACT ATTENUATORS

Impact attenuators are ideally suited for use at urban locations when fixed objects cannot be removed, relocated, or made breakaway, and cannot be adequately shielded by a longitudinal barrier. In urban situations, the increase in roadway maintenance mileage, the tight right-of-way constraints, and the varying traffic flow conditions create situations that limit available options for removing or relocating fixed objects. The use of impact attenuators, as opposed to longitudinal barriers, becomes more appropriate to shield fixed objects such as those at exit ramp gores, ends of median barriers, and bridge piers and abutments, to name only a few.

The width available for the placement of impact attenuators can be restricted in urban areas. However, a number of impact attenuators are available for narrow width conditions. The systems outlined in Chapter 8 should be reviewed to determine the appropriateness of the system for the proposed site location.

A curb's tendency to cause vaulting can reduce the effectiveness of an impact attenuator. Therefore, impact attenuators should not normally be installed behind curbs. Where necessary for drainage, an existing curb no higher than 100 mm [4 in.] can be left in place, unless it has contributed to poor performance in the past.

Impact attenuators are not intended to reduce accidents, but to lessen the severity of the impact. If a particular crash cushion is struck frequently, it is important to determine why the collisions are occurring. Improved use of signs, pavement markings, delineation, reflectors, and luminaires may help to reduce the number of occurrences.

10.7 CURBS

Curbed sections are generally restricted to design speeds of 70 km/h [45 mph] or less on roadways in urban or highly developed areas. Items that need to be considered are: delineation of the pavement edge, delineation of pedestrian walkways, control of access points, retention of water on the roadway, and vaulting or destabilization of vehicles.

When a vehicle strikes a curb, the trajectory of that vehicle depends upon several variables: the size and suspension characteristics of the vehicle, its impact speed and angle, and the height and shape of the curb itself. Crash tests have shown that the use of guardrail with 150 mm [6 in.] curb should not be considered where high-speed, high-angle impacts are likely to occur. Where curb is needed for drainage, the use of a curb no higher than 100 mm [4 in.] is satisfactory. On low-speed facilities, a vaulting potential still exists; however, since the risk of such an occurrence is lessened, the use of 150 mm [6 in.] curb in combination with guardrail can be tolerated. Each situation should be considered individually, taking into account anticipated speeds and consequences of vehicular penetration of the barrier. Section 3.4.1 provides additional guidance for the use of curbs.

The common practice in urban settings is to use curbs adjacent to the highway shoulders to provide separation of pedestrians from the traffic flow. Realistically, curbs have limited redirectional capabilities and only at low speeds of approximately 40 km/h [25 mph] or lower. Curbs alone may not be adequate protection for pedestrians on adjacent sidewalks or for shielding utility poles. In some cases, other measures may need to be considered.

In urban conditions, from an operational standpoint, a minimum horizontal clearance of 0.5 m [1.5 ft] should be provided beyond the face of curbs to any obstructions. Designers should strive for horizontal clearances more appropriate for the off-peak operating speeds. At the higher speed end of the suburban or urban facilities, consideration should be given to providing a shoulder and offsetting any curbing to the back of the shoulder. The shoulders may be used to accommodate bicyclists and pedestrians where sidewalks are not provided.

10.8 DRAINAGE

On those urban or suburban roadways where operating speeds are generally lower, ditches are less of a safety problem to the errant motorist. Where practical, a closed drainage system should be considered. Curbs and drop inlets are common drainage elements in these cases.

Drainage inlets, grates, and similar devices should be placed flush with the ground surface and must be capable of supporting vehicle wheel loads. In addition, slots should be spaced and oriented so they will not be an obstacle to pedestrians or bicyclists.

Even though drainage ditches may be located outside the nominal clear zones in suburban areas, there may be a likelihood that errant vehicles that reach the ditch could be led down the ditch and could strike parallel culvert ends at driveways or intersecting roads. Traversable designs should be considered at these locations. Section 3.4.3.2 provides information on traversable designs.

10.9 LANDSCAPING

Along most urban streets, some type of landscaping exists. Trees, shrubs, lawns, decorative rock, and other materials are used to provide a pleasing setting for drivers, pedestrians, bicyclists, and abutting land owners.

The designer should always be consulted in the decisions regarding landscaping, particularly as they relate to sight distance and possible future lane needs. Considerations in the design of landscaping include:

- the mature size of trees and shrubs, and how this will affect safety, visibility, and maintenance cost.

- sufficient border area to accommodate the type of landscaping planned. If parking is allowed along the curb, will the landscaping allow curbside access to parked vehicles?

- potential future changes in roadway cross section. For example, the addition of a second left-turn lane at major intersections by taking approximately 3 m [10 ft] of additional space from the median island is becoming a common practice. Landscaping in the affected area should be minimal or should not be included in the plan.

Visibility restrictions resulting from landscaping are of principle concern to the designer. Points that must be considered include the following:

- border area landscaping should allow full visibility at driveways for drivers and pedestrians.

- a clear vision space from 1 m to 3 m [3 ft to 10 ft] above grade is desirable along all streets and at all intersections. This allows drivers in cars, trucks, and buses to have good sight distance. Many cities have ordinances regarding sight restrictions at corners which incorporate this "clear space" idea.

- landscaping very small islands should be avoided to reduce maintenance needs.

- large trees or rocks should not be used at decision points (e.g., gore areas, island noses) to "protect" poles and other appurtenances. Rather, each of the design options stated in Section 10.1 (in the order listed) should be considered to improve safety.

With respect to pedestrians, it is desirable to have a grass strip separating the sidewalk from the curb, thus further separating the pedestrian from vehicular traffic. The strip also provides room for snow storage and trash collection.

10.10 WORK ZONES

Construction work zones in urban areas have varying degrees of traffic control and work-zone protection needs. Conditions can vary from low-speed, low-volume urban streets to highway construction zones in high-volume arterial and interstate locations. The type of traffic control under consideration needs to be reviewed for the site conditions, operating speeds, and traffic flows within the construction zone. The *Manual on Uniform Traffic Control Devices* (4) establishes the principles to be observed in traffic control, design, installation, and maintenance of traffic control devices in work zones.

Chapter 9 details a number of available traffic barriers and traffic control devices for work zones. Effective use and implementation of these barriers and devices in urban conditions remains extremely important and must be given full consideration on an individual project basis, including provisions for bicyclists and pedestrians.

REFERENCES

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Chapter 11
Erecting Mailboxes on Streets and Highways

11.0 OVERVIEW

This chapter replaces the 1994 AASHTO publication *A Guide for Erecting Mailboxes on Highways* and deals with mailboxes and mailbox turnout design. Highway safety is the primary reason for a transportation agency to become involved in this type of design. Limited data exist for vehicle/mailbox collisions since most record systems do not specifically isolate these types of crashes. However, the data that are available suggest that as many as 70 to 100 people die annually in the United States when colliding with improperly designed mailboxes and their supports. While this number is low, it is significant because it is associated with an unnecessary hazard.

A point that makes this a sensitive issue is that postal patrons may view the mailbox as an extension of themselves and part of their domain. They may resent and even resist design directions concerning their mailboxes. An extra measure of diplomacy and public relations may be needed to effect changes in the design and location of mailbox installations.

11.1 MAILBOXES

The typical single mailbox installation, shown in Figure 11.1, consists of a light, sheet-metal box mounted on a 100 mm x 100 mm [4 in. x 4 in.] wooden post or a 38 mm [1 $^1/_2$ in.] diameter light gage pipe and is not a serious threat to motorists. Improvements to strengthen typical post-to-box mounting details, as discussed in Section 11.2.4, would further reduce its threat. Mailboxes supported by structures such as masonry columns, railroad rails and ties, tractor wheels, plow blades, and concrete-filled barrels, as

FIGURE 11.1 Typical single mailbox installations

FIGURE 11.2 Examples of hazardous single mailbox installations

shown in Figure 11.2, sometimes turn a single mailbox installation into a roadside obstacle that should be eliminated.

The typical grouped or multiple mailbox installation, shown in Figure 11.3, is also a serious hazard to the motorist who strikes it. This installation consists of two or more posts supporting a horizontal member, usually a timber plank, which supports the group of mailboxes. The horizontal members in these installations are poised at windshield height and when struck have impaled or decapitated motorists. For safe alternative designs for grouped mailbox installations, see Section 11.2.4.

Injury from striking a mailbox is not the only risk associated with mailboxes. The mail carrier's maneuvers in collecting and delivering mail and the patron's activities, either as pedestrian or motorist in collecting and depositing mail, create opportunities for traffic conflict and human error. Reducing the number and severity of these conflicts is an important objective of this chapter.

Only by banishing mailboxes from our highways can mailbox-related traffic accidents be eliminated. However, while elimination is impractical, many identifiable problems can be corrected. Through cooperation among transportation agencies, the U.S. Postal Service, and postal patrons, good design practices in mailbox installation and location can be implemented when mailboxes are installed or replaced. This should incur little or no cost increase with a typical mailbox lasting an average of about 10 years. Furthermore, when highways are rebuilt or undergo significant upgrading, there may be opportunities to incorporate relatively inexpensive mailbox improvements.

The general principles and guidelines contained in this chapter are also applicable to newspaper delivery boxes and similar devices located along public highways. These guidelines are compatible with the requirements of the U.S. Postal Service (see Appendix D) and are presented in the interest of providing the highest degree of safety practicable for the motoring public, mail carriers, and postal


FIGURE 11.3 Examples of hazardous multiple mailbox installations

patrons. Highway agencies and local entities are encouraged to use these guidelines in developing their own mailbox and installation policies and standards. It should be understood that these are general guidelines and that local conditions such as legal institutions and practices, population densities, topography, highway characteristics, snowfall, prevailing vehicle characteristics, etc., are factors to consider in developing regulations and standards.

11.2 GENERAL PRINCIPLES AND GUIDELINES

This section deals with regulations and design. Regulations are needed to establish consistency in acceptable mailbox turnouts and design.

11.2.1 Regulations

It is recommended that each highway agency adopt regulations for the placement of mailboxes and newspaper boxes within the right-of-way of public highways. Correlation of these regulations with those for the granting of driveway entrance permits should be considered. Mailbox and newspaper box control regulations should follow the principles and guidance contained in this document and include the following:

- a reference to pertinent statutes
- a statement that all mailbox installations must meet the requirements of the U.S. Postal Service
- a requirement that all mailbox and newspaper box installations conform to the current policies and standards of the highway agency regarding location, geometry, and structure of such installations
- information on where one can obtain copies of the current policies and standards
- a statement on permits, if required
- a statement on how approval of exceptions can be obtained
- a description of the highway agency's and the postal patron's responsibilities regarding new and replacement installations
- a description of the distribution of responsibilities and the procedures to be followed in removing unsafe or nonconforming installations

11.2.2 Mail Stop and Mailbox Location

Mailboxes should be placed for maximum convenience to the patron, consistent with safety considerations for highway traffic, the carrier, and the patron. Consideration should be given to: (1) minimum walking distance within the roadway for the patron, (2) available stopping sight distance in advance of the mailbox site, and (3) possible restrictions to corner sight distance at intersections and driveway entrances. Where feasible, new installations should be located on the far right side of an intersection with a road or driveway entrance.

Boxes should be placed only on the right-hand side of the highway in the direction of travel of the carrier. An exception is one-way streets where they may be placed on either side. It is undesirable to require pedestrian travel along the shoulder to access the mailbox. However, this may be the preferred solution when compared to alternatives such as constructing a turnout in a deep cut, placing a mailbox just beyond a sharp crest vertical curve, or constructing two or more closely spaced turnouts.

The placing of mailboxes along high-speed and/or high-volume highways should be avoided if other practical locations are available. Mailboxes should not be located where access is from the lanes of an expressway or where access, stopping, or parking is otherwise prohibited by law or regulation. Where there are frontage roads, the abutting property owners may be served by boxes located along the frontage roads. It is undesirable to locate a mailbox that would require a patron to cross the lanes of an expressway to deposit or retrieve mail. Where the U.S. Postal Service deems that service is not warranted on both frontage roads, or where there is a frontage road only on one side, patrons not served directly should be accommodated by mailboxes at a suitable and safe location in the vicinity of the crossroad nearest the patron's property.

Placing a mail stop near an intersection could have an effect on the operation of the intersection. The nature and magnitude of this effect depends on traffic speeds and volumes on each of the intersecting roadways, the number of mailboxes at the stop, extent of traffic control, how the stop is located relative to the traffic control, and the distance the stop is from the intersection.

At intersections where one roadway is given the right-of-way and the other is stop controlled, a vehicle at a mail stop on the through roadway approach may restrict the view from a vehicle entering the intersection from the right to through traffic behind the mail stop. A mail stop on the through road on the far side of the crossroad increases the chance the crossroad driver will pull into the path of the vehicle on the through road that is headed for the mail stop. A mail stop in advance of a stop sign creates the potential for a vehicle at the mail stop to block the view of the stop sign. The least troublesome location for a mail stop at these intersections is adjacent to a crossroad lane leaving the intersection. Nevertheless, there is still a chance that a driver re-entering traffic from the mail stop will not see or be seen from a vehicle turning onto the crossroad. Figure 11.4 shows suggested minimum clearance distance to nearest maibox in mailstops at intersection. Using the mail stop location dimensions in the figure will minimize the effect a stop will have on an intersection's operation and minimize the hazard to persons using the mail stop.

Mailbox heights are usually set to accommodate the mail carrier. Typically, the bottom of the mailbox is located 1.0 m to 1.2 m [39 in. to 47 in.] above the mail stop surface.

Mailboxes should be located so that a vehicle stopped at a mailbox is clear of the adjacent traveled way. The higher the traffic volume or speed, the greater the clearance should be. A reasonable exception to this principle may be on low-volume and low-speed streets and roads.

Most vehicles stopped at a mailbox will be clear of the traveled way when the mailbox is placed outside a 2.4 m [8 ft] wide usable shoulder or turnout. This location is recommended for most rural highways. Although a 2.8 m [9 ft] minimum shoulder is acceptable, a minimum 3 m [10 ft] turn out should be provided when practical. Where conditions justify, 3.6 m [12 ft] turnouts should be provided. However, it may not be reasonable to require even a 2.4 m [8 ft] shoulder or turnout on very low-volume, low-speed roads or streets. To provide space outside the all-weather surface for opening the mailbox door, it is recommended that the roadside face of a mailbox be set 200 mm to 300 mm [8 in. to 12 in.] outside the all-weather surface of the shoulder or turnout. Suggested guidelines for the placement of mailboxes are shown in Table 11.1. These are based on experience and design judgment.

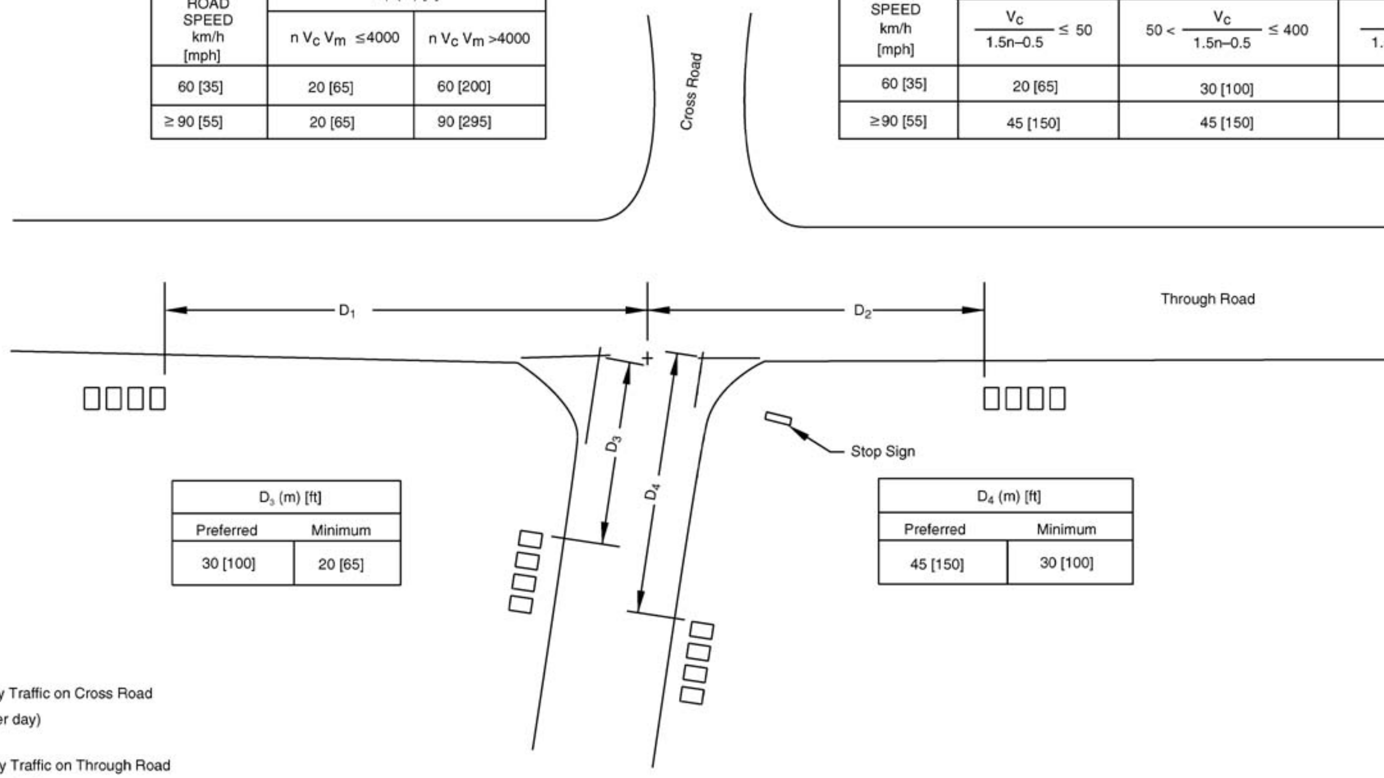
When a mailbox is installed in the vicinity of an existing guardrail, it should, wherever practical, be placed behind the guardrail.

11.2.3 Mailbox Turnout Design

Shoulder or turnout widths suitable to safely accommodate vehicles stopped at mailboxes are discussed in Section 11.2.2 and shown in Table 11.1.

The surface over which a vehicle is maneuvered to and from a mailbox must be sufficiently stable to support passenger cars stopping regularly during all-weather conditions. Where shoulder surface strength or width is not sufficient for this purpose, the shoulder should be modified to provide a suitable all-weather mailbox turnout. In most instances, adequate surface stabilization can be obtained by the addition of select materials to the in-place soils. A mailbox turnout for grouped mailboxes may require greater stabilization or possibly a surface treatment course to accommodate multiple patron use. Special measures may also be needed where highway traffic conditions encourage hard braking or high acceleration of vehicles in a mailbox turnout.

Drivers are usually required to slow their vehicles in traffic, which increases the risk of a crash. The ideal way to minimize this risk is to provide a speed change lane. A wide surface-treated shoulder is ideal for this purpose. Unfortunately, suitable shoulders are not available at most mailbox turnout locations and it would be far too expen-



THROUGH ROAD SPEED km/h [mph]	D_1 (m) [ft]	
	$n V_c V_m \le 4000$	$n V_c V_m > 4000$
60 [35]	20 [65]	60 [200]
≥ 90 [55]	20 [65]	90 [295]

THROUGH ROAD SPEED km/h [mph]	D_2 (m) [ft]		
	$\frac{V_c}{1.5n-0.5} \le 50$	$50 < \frac{V_c}{1.5n-0.5} \le 400$	$\frac{V_c}{1.5n-0.5} > 400$
60 [35]	20 [65]	30 [100]	30 [100]
≥90 [55]	45 [150]	45 [150]	60 [200]

D_3 (m) [ft]	
Preferred	Minimum
30 [100]	20 [65]

D_4 (m) [ft]	
Preferred	Minimum
45 [150]	30 [100]

V_c = Average Daily Traffic on Cross Road (vehicles per day)

V_m = Average Daily Traffic on Through Road (vehicles per day)

n = Number of Mailboxes at Mail Stop

FIGURE 11.4 Suggested minimum clearance distance to nearest mailbox in mailstops at intersections

TABLE 11.1 Suggested guidelines for lateral placement of mailboxes

<table>
<tr><th rowspan="2">Highway Type and ADT, (vpd)</th><th colspan="2">Width of All-Weather Surface Turnout or Available Shoulder at Mailbox,[1] (m) [ft]</th><th colspan="2">Distance Roadside Face of Mailbox Is to Be Offset Behind Edge of Turnout or Usable Shoulder, (mm) [in.]</th></tr>
<tr><th>Preferred</th><th>Minimum</th><th>Preferred</th><th>Minimum</th></tr>
<tr><td>Rural Highway
Over 10,000</td><td>> 3.6
[12]</td><td>2.4
[8]</td><td rowspan="5">200 to 300
[8 to 12]</td><td rowspan="3">0</td></tr>
<tr><td>Rural Highway
1,500 to 10,000</td><td>3.6
[12]</td><td>2.4
[8]</td></tr>
<tr><td>Rural Highway
400 to 1,500</td><td>3.0
[10]</td><td>2.4
[8]</td></tr>
<tr><td>Rural Road
Under 400</td><td>2.4
[8]</td><td>1.8
[6][2]</td><td rowspan="2">200
[10][3]</td></tr>
<tr><td>Residential Street Without Curb or All-Weather Shoulder</td><td>1.8
[6]</td><td>0.00</td></tr>
<tr><td>Curbed Residential Street</td><td colspan="2">Not Applicable</td><td>200 to 300
[8 to 12][4]</td><td>150 [6][4]</td></tr>
</table>

ADT = Average Daily Traffic

vpd = vehicles per day

1 If there is a need to provide for increased access, the following may be considered in conjunction with the local Postmaster:

a. Provide a level clear floor space 0.75 m x 1.2 m [30 in. x 48 in.] centered on the box for either side or forward approach.

b. Provide an accessible passage to and from the mailbox and projection into a circulation route (no more than 100 mm [4 in.] if between 0.7 m [28 in.] and 2.0 m [80 in.] AFF) so that the mailbox does not become a protruding object for pedestrians with impaired vision.

2 Strive for a 1.8 m [6 ft] minimum; however, in some situations this may not be practical. In those cases, provide as much as possible.

3 If a turnout is provided, this may reduce to zero.

4 Behind traffic-face of curb.

sive to provide shoulders or turnouts that would allow a speed change outside the traveled way. Figure 11.5 shows a mailbox turnout layout considered appropriate for different traffic conditions.

The minimum space needed for maneuvering to a parallel position in and out of traffic is also shown in Figure 11.5. The typical driver would probably slow considerably before starting into the low-speed turnout. This tendency makes it unsuitable for high-speed highways where driver expectancy does not include such slow-moving traffic.

Before entering a 2.4 m [8 ft] wide turnout with a 20:1 taper for high-speed traffic as shown in Figure 11.5, a driver would probably not slow as much before clearing the traveled way. While this is not an ideal exit maneuver, it would probably not create an unacceptable hazard on most rural highways for the few stops generated by a single mailbox.

Increasing the width of the turnout to 3.6 m [12 ft] and maintaining the 20:1 taper rate suggested in Figure 11.5 would induce a driver using the turnout to enter it at a fair rate of speed, but it will not be as fast as the through speed. While this is still not ideal, it should be quite acceptable for most sites. The exception may be found on highways operating at high speeds and carrying over 3,000 to 4,000 vehicles per day and with a high percentage of vehicles on long trips. For these conditions, consideration should be given to providing shoulders or turnouts at unavoidable mail stops that will provide for greater speed change opportunity outside the traffic stream.

The tapers shown in Figure 11.5 represent theoretical layouts. It may be more practical to square the ends of the turnout or to provide a stepped layout by strengthening and widening the shoulder to the full width of the turnout for the entire length of the taper. It may also be simpler to construct a continuous turnout-width shoulder rather than individual turnouts where mailbox turnouts are closely spaced.

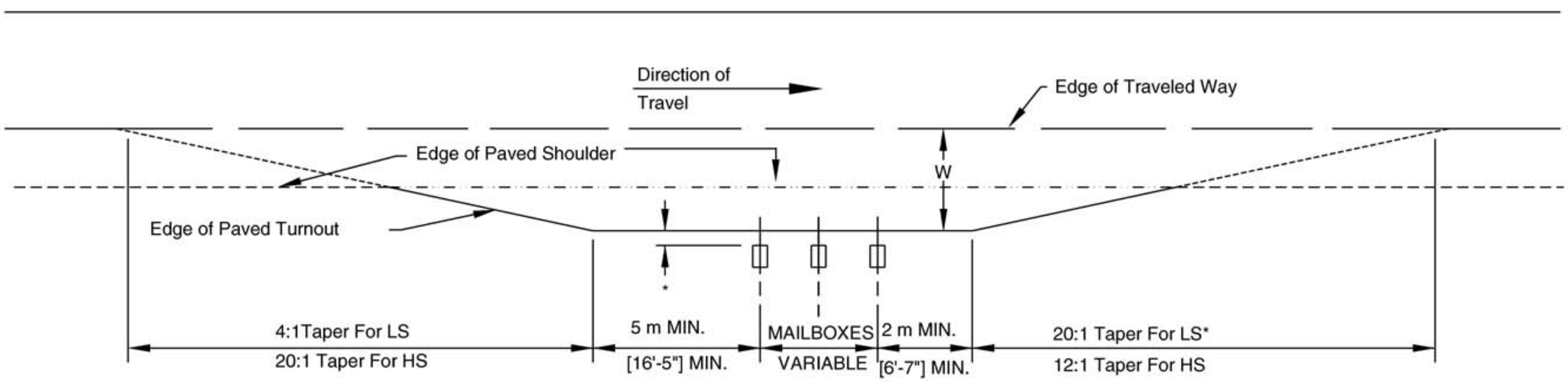
11.2.4 Mailbox Support and Attachment Design

All exposed mailboxes should be firmly attached to supports that yield or break away safely if struck by a vehicle. The NCHRP Report 350 contains performance criteria for mailbox supports when subjected to crash testing with an automobile. The criteria can be summarized as follows:

- Mailbox supports should, with a minor qualification, be no more substantial than required to resist service loads and to reasonably minimize vandalism. Nominal 100 mm x 100 mm [4 in. x 4 in.] or 100 mm [4 in.] diameter wood posts or 38 mm to 50 mm [1.5 in. to 2 in.]) diameter standard steel or aluminum pipe posts, embedded no more than 600 mm [24 in.] into the ground, should be the maximum strength supports considered. Lower strength supports, such as lightweight flanged channel steel posts, have provided satisfactory service in most environments. A metal post should not be fitted with an anchor plate. However, an anti-twist device that extends no more than 250 mm [10 in.] below the ground surface is acceptable. The minor qualification to the criterion of minimizing post strength is for the support to break rather than to bend under impact, and for the support to have sufficient strength to accelerate the box to a speed approaching that of the impacting vehicle so the chances of the box penetrating the vehicle's windshield are minimized. Test results indicate 100 mm x 100 mm [4 in. x 4 in.] or 100 mm [4 in.] diameter wood supports should be both minimum and maximum post dimensions.

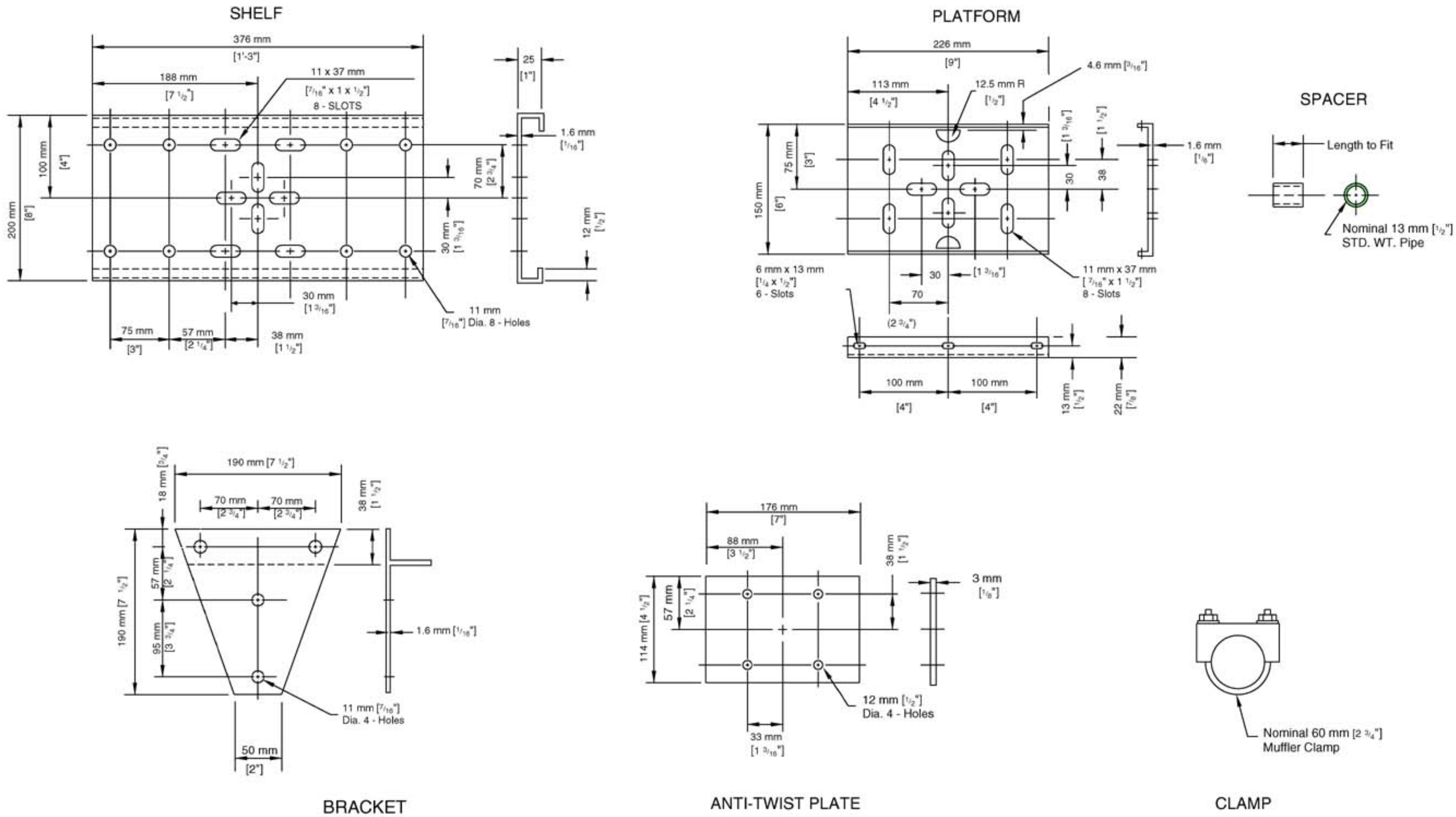
- Mailbox to post attachments should prevent mailboxes from separating from their supports under vehicle impacts. The lighter the mailbox, the easier it will be to meet this criterion or, conversely, given sufficient post attachment strength, the less sensitive the safety of an installation will be to the mass of the mailbox. Figures 11.6 through 11.10 show acceptable attachment and support details. The exact support hardware dimensions and design may vary, such as having a two-piece platform bracket or alternative slot and hole locations. However, the product must result in a satisfactory attachment of the mailbox to the post, and all components must fit together properly.

- Multiple mailbox installations must meet the same criteria as single mailbox installations. This requirement precludes the use of a heavy horizontal support member such as the one shown in Figure 11.3. Figures 11.7, 11.9, and 11.10 show acceptable multiple mailbox support systems. The use of a series of such installations or of individually supported boxes is acceptable. However, vehicle rollover occurred when crash tested with a small car at high speed impacting off-center of a row of eight closely spaced mailboxes individually supported with 3 kg/m [2 lb/ft] channel post supports. Review of a film from this test and results from other tests suggest that the reason for this performance was a ramping caused by the closely spaced mailboxes piling up. To avoid this problem, it is recommended the mailbox supports be separated a distance at least equal to three-



LS = A Minimum Design for Roads Carrying Low-Speed Traffic and for Local and Collector Roads.
HS = For Roads Carrying High-Speed Traffic.
W = For Suggested Widths, see Table 11.1.
MAILBOXES = For Mailbox Spacing and Variable Length, see Section 11.2.4, Mailbox Support and Attachment Design.
* = For Mailbox Face Offset, see Table 11.1, 0 mm to 300 mm [0" to 12"].

FIGURE 11.5 Mailbox turnout



SEE ALTERNATE BRACKET DESIGN
IN FIGURES 11.8 & 11.9.

NOTE: ALL DIMENSIONS IN MILLIMETERS UNLESS OTHERWISE INDICATED.
ALL DIMENSIONS IN BRACKETS ARE IN U.S. CUSTOMARY UNITS.

FIGURE 11.6 Mailbox support hardware, Series A

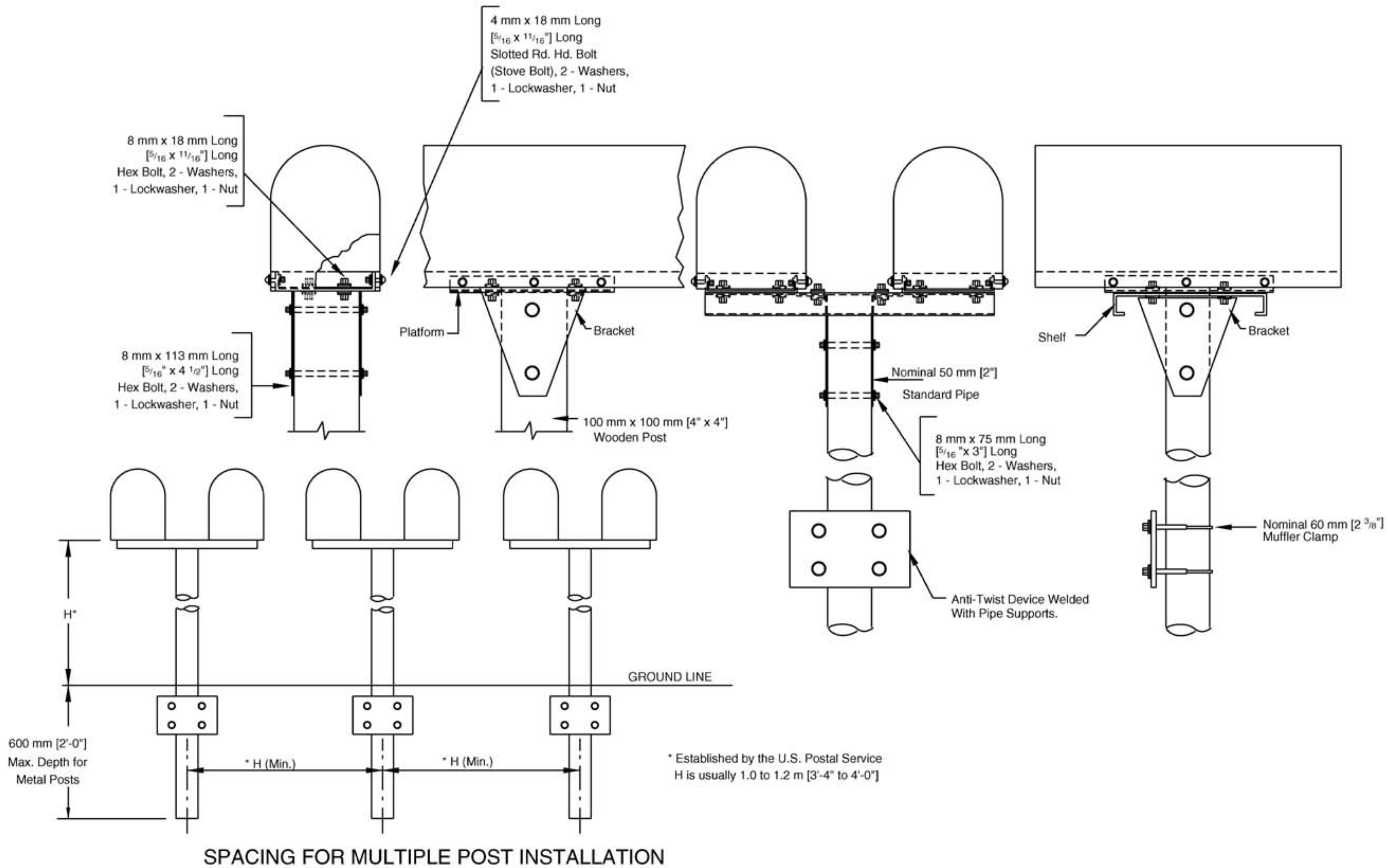
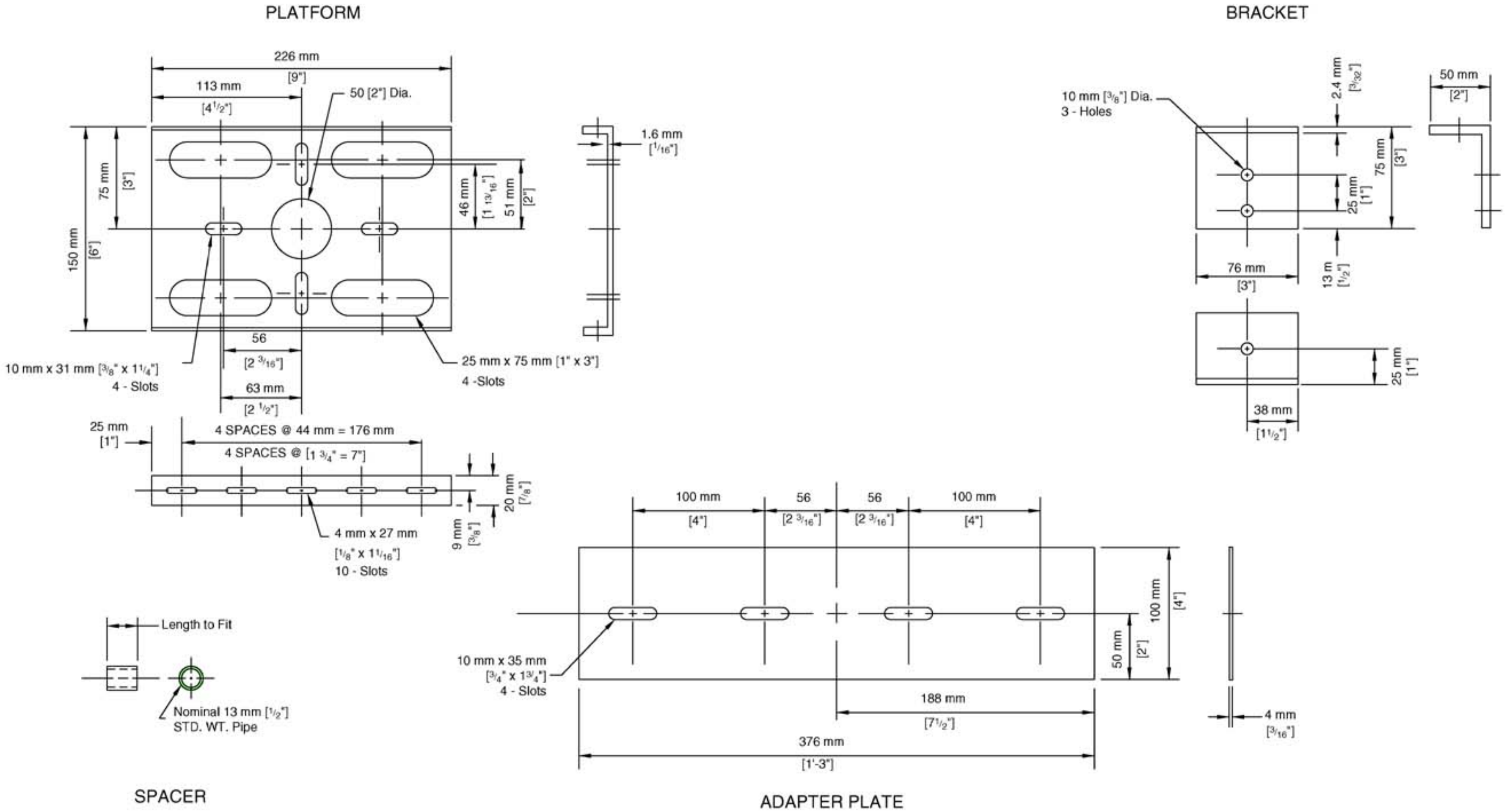


FIGURE 11.7 Single and double mailbox assemblies, Series A



NOTE: ALL DIMENSIONS IN MILLIMETERS UNLESS OTHERWISE INDICATED.
ALL DIMENSIONS IN BRACKETS ARE IN U.S. CUSTOMARY UNITS.

FIGURE 11.8 Mailbox support hardware, Series B

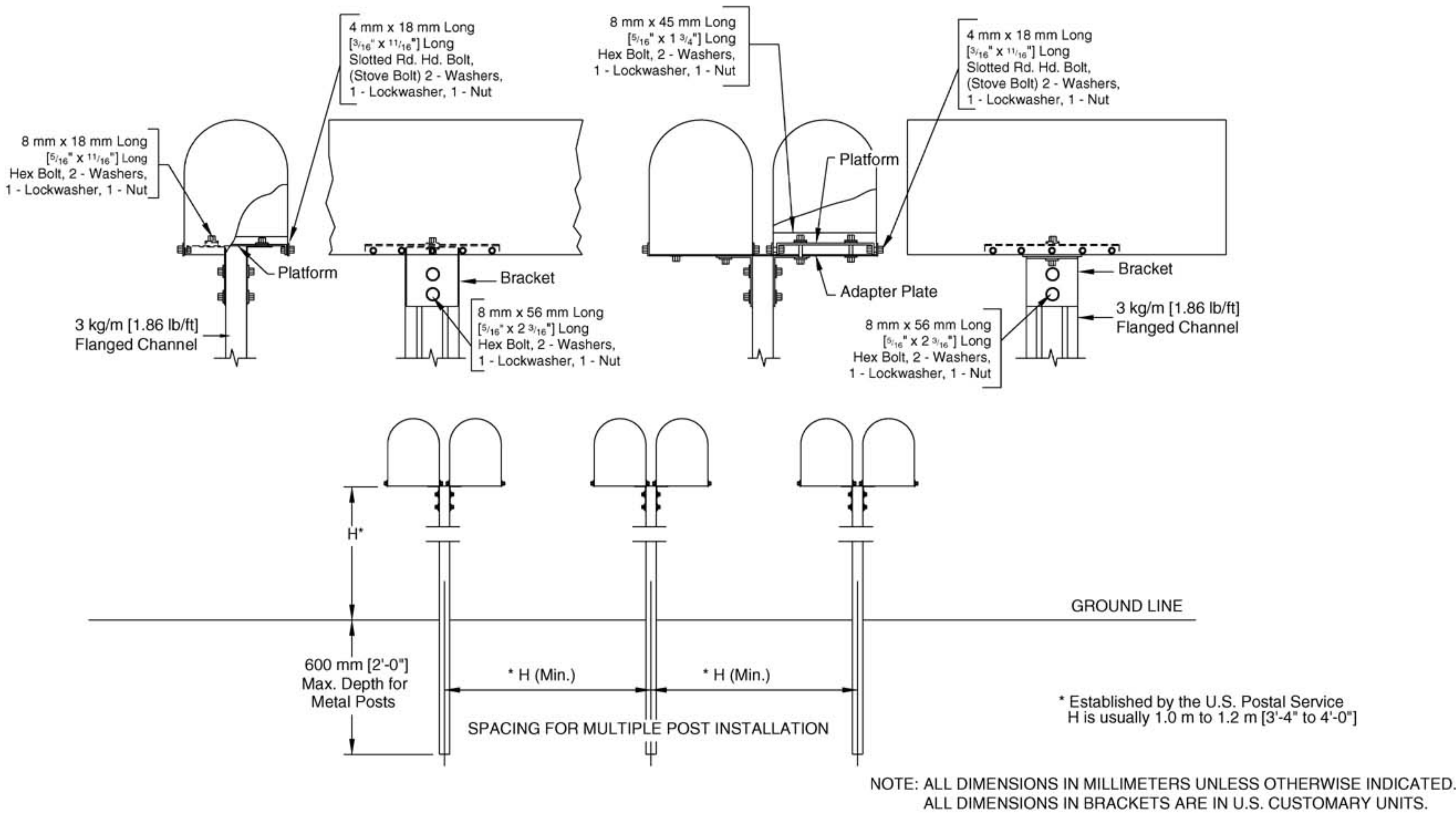


FIGURE 11.9 Single and double mailbox assemblies, Series B

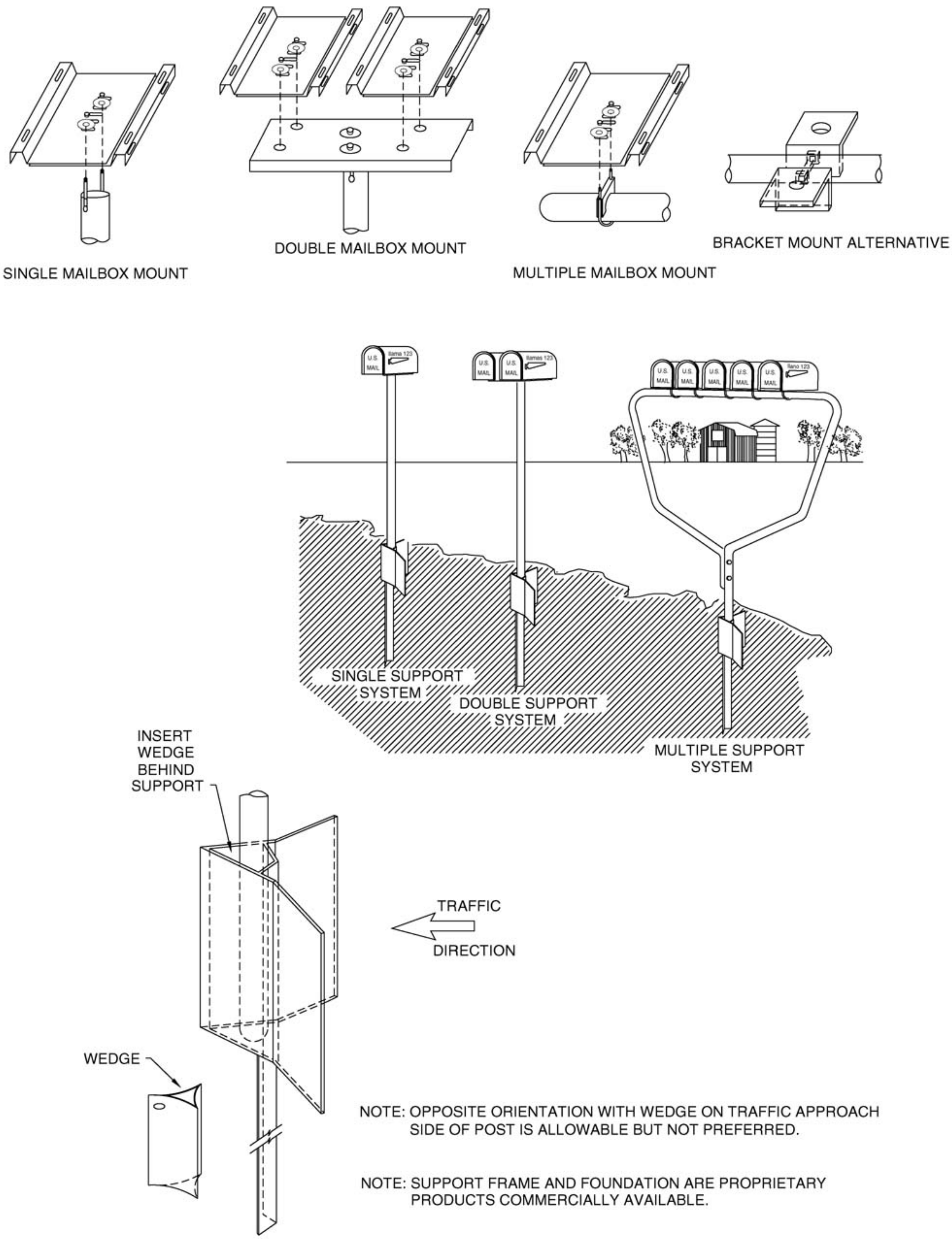


FIGURE 11.10 Single and double mailbox assemblies, Series C

FIGURE 11.11 Collection unit on auxillary lane (top) and neighborhood delivery and collection box units

fourths of their heights and preferably their full heights above ground. It is also preferred that multiple mailbox installations be located outside the highway clear zone, such as on a service road or a minor intersecting road.

- The Neighborhood Delivery and Collection Box Unit (NDCBU) is a specialized type of multiple mailbox installation, as shown in Figure 11.11. The NDCBU is a cluster of 8 to 16 locked boxes mounted on a pedestal or within a framework, the combination of which generally has a mass between 45 kg and 90 kg [100 lb and 200 lb]. While the NDCBU usually serves a limited number of single-family residences in urban areas, their use has been observed in rural areas. A crash test of one of these units at 100 km/h [60 mph] showed that it failed to meet safety requirements. Therefore, an NDCBU should be located outside the clear zone to allow for safe recovery of errant vehicles and for safe access by postal patrons and carriers. Postmasters and designers responsible for the location of an NDCBU should be instructed to contact local government authorities, including the appropriate highway officials (state, county, township, municipal, etc.) prior to installation. This communication will help to ensure the safe location of the NDCBU.

In areas of high snowfall, some highway agencies have found cantilever mailbox supports advantageous. While such designs do permit windshield contact with the box without the vehicle first contacting the support, tests of the design shown in Figures 11.12 and 11.13 did not reveal serious consequences. The operational advantage of these supports is that snow can be plowed close to the mailbox without the snow windrow pushing the support over.

The state of Minnesota has developed and tested a swing-away mailbox that is not patented and will not penetrate a vehicle windshield. This type of a mailbox support is designed to swing back out of the way when a snowplow truck goes by. (See Figure 11.14.)

Lightweight newspaper boxes may be mounted below the mailbox on the mailbox support.

Recently, mailboxes of heavy gage steel or other substantial materials have been designed and sold as deterrents to vandalism. These massive boxes, over 5 kg [11 lb], meet U.S. Postal Service requirements for minimum size, material durability, ease of access, etc., and are quite resistant to deformation. However, these boxes are potentially hazardous to occupants of errant vehicles regardless of the support used. They should be restricted to use only along low-speed, low-volume streets in residential areas.

11.3 MODEL MAILBOX REGULATION

A generic model regulation for mailboxes and newspaper delivery boxes on public highway rights-of-way is provided in Appendix E. The model is intended only as an example. States and municipalities can and should tailor the model to fit their own particular needs.

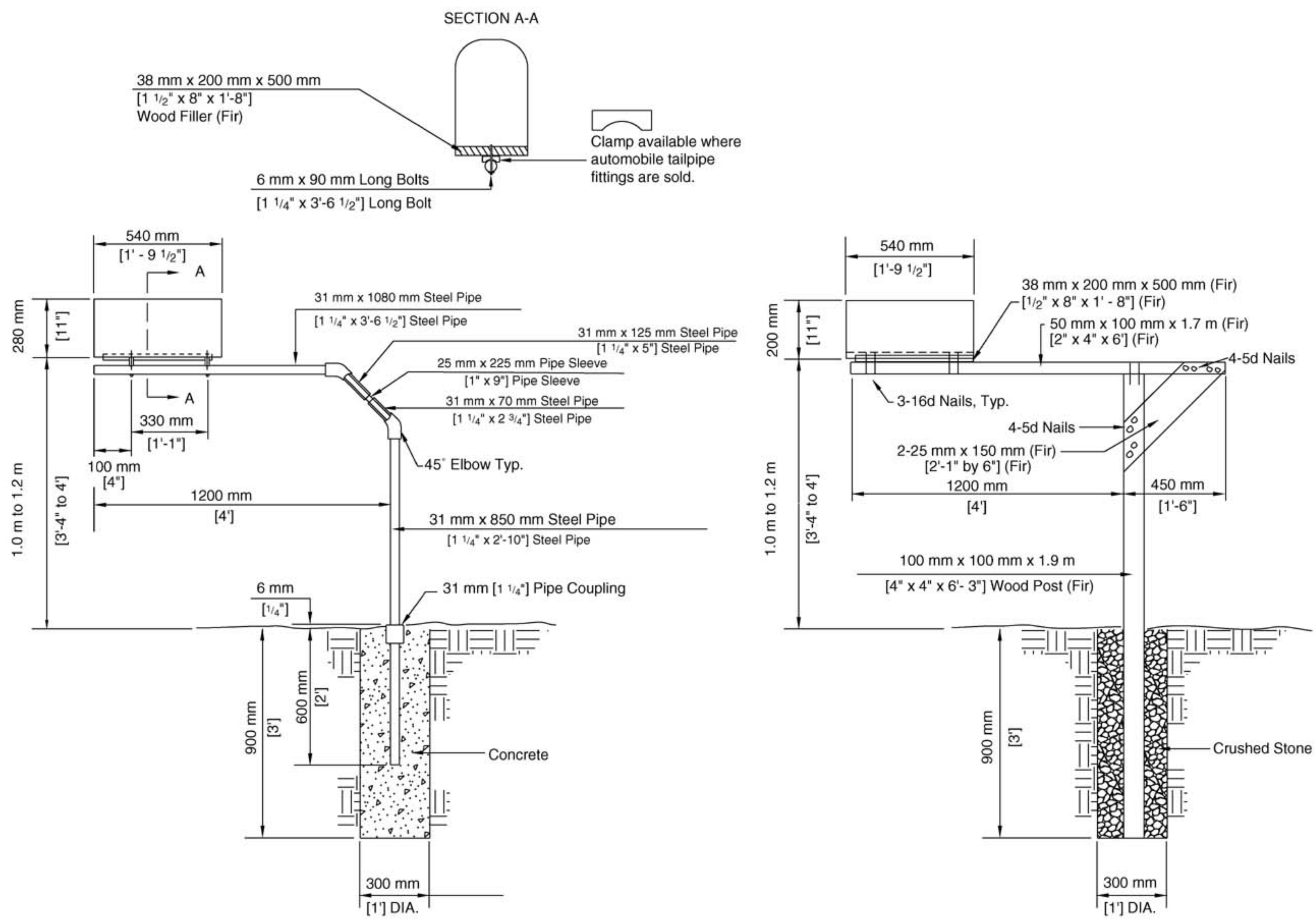


FIGURE 11.12 Cantilever mailbox supports

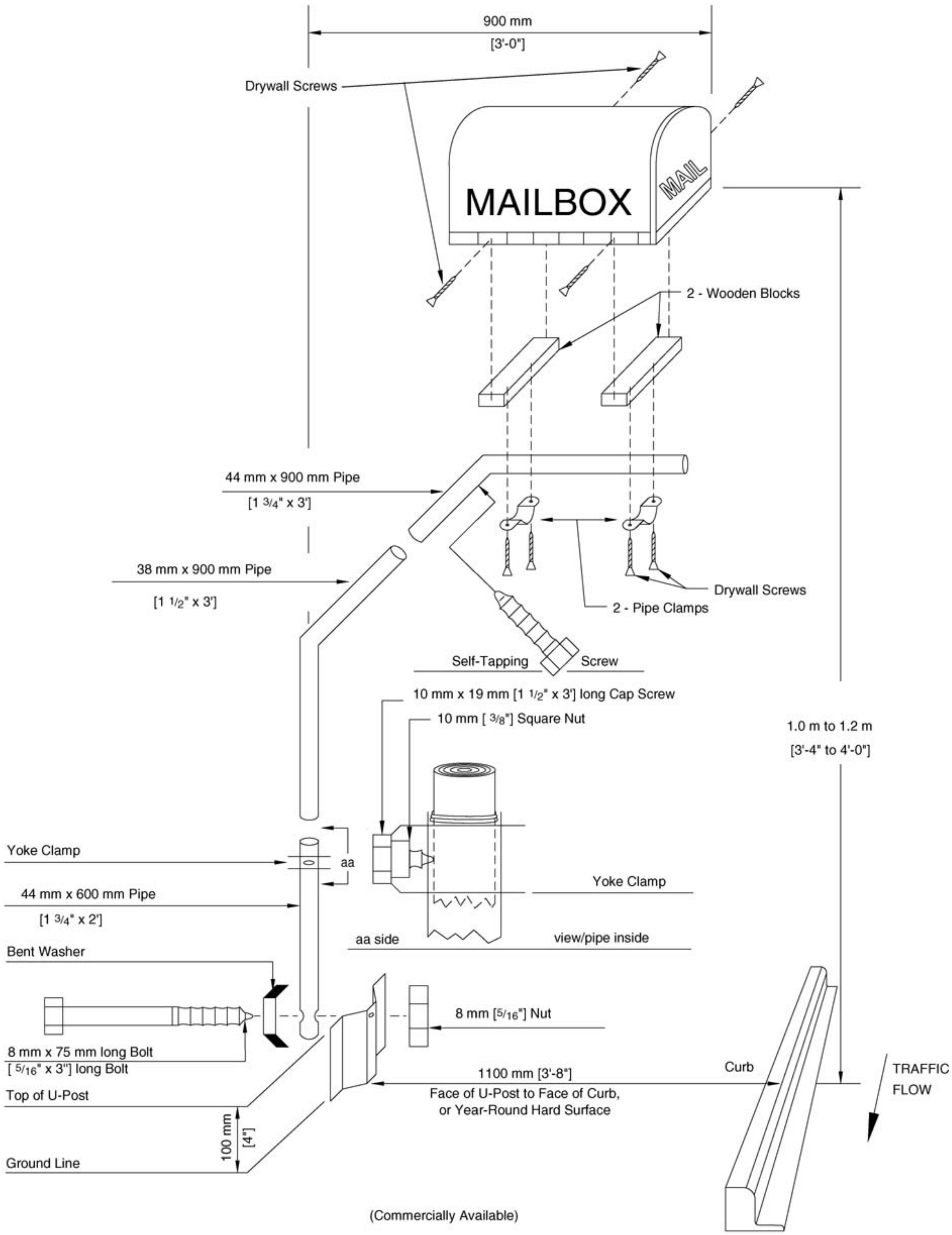


FIGURE 11.13 Breakaway cantilever mailbox supports

FIGURE 11.14 Minnesota swing-away mailbox

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APPENDIX A

A Cost-Effectiveness Selection Procedure

The Roaside Safety Analysis Program (RSAP) software was developed under National Cooperative Highway Research Program (NCHRP) Project 22-9 and represents one approach to using the *Roadside Design Guide*, as described in this appendix. It carries no guarantees or warranties from the American Association of State Highway and Transportation Officials (AASHTO). The RSAP program is intended as a tool for economic analysis and should not supersede the guidelines presented in the *Roadside Design Guide* or sound engineering judgment.

A1.0 OVERVIEW

This appendix provides a summary of the conceptual framework and algorithms contained in the RSAP computer program, which has been created to assist in the economic analysis of existing or proposed roadside conditions. The RSAP program has its genesis in the analysis procedure presented in Chapter VII of AASHTO's 1977 *Guide for Selecting, Locating, and Designing Traffic Barriers* (1), the ROADSIDE computer program presented in previous versions of the *Roadside Design Guide* (2) (3), and the Benefit-Cost Analysis Program (BCAP) (4) used in the development of the AASHTO's 1989 *Guide Specifications for Bridge Railings* (5).

Additional copies of the RSAP program and the associated documentation, *Engineer's Manual* and *User's Manual*, may be obtained from:

Transportation Research Board
National Cooperative Highway Research Program
2101 Constitution Avenue, NW
Washington, DC 20418

or ordered through the internet at http://www.nationalacademies.org/trb/bookstore.

The *Engineer's Manual* contains detailed descriptions of the conceptual framework and algorithms used with the cost-effectiveness analysis procedure. The *User's Manual* contains detailed descriptions of the operations of the RSAP program and is also available through the on-line help of the program.

The RSAP program is comprised of two separate but integrated programs: the User Interface Program and the Main Analysis Program. The Main Analysis Program contains the cost-effectiveness procedure itself and performs all the necessary calculations. The Main Analysis Program is written in the FORTRAN language because of its efficiency in performing scientific calculations. The User Interface Program is written in the C++ language, which is more adept at providing a user-friendly environment through the use of windows, screens, and menus.

The User Interface Program provides the users with a simple and structured means to input data into the RSAP program. The program generates input data files and transfer data files, which, together with the default and temporary data files, serve as inputs to the Main Analysis Program. After processing by the Main Analysis Program, the User Interface Program takes the outputs from the Main Analysis Program and presents the results to the user. The transfer of data files between the User Interface Program and the Main Analysis Program is conducted in ASCII format for simplicity and ease of file transfer.

To install and run the RSAP program, your computer must be an IBM PC or 100 percent compatible, and have the following:

1. A Pentium III or equivalent based platform
2. Memory—128 MB minimum
3. Disk space
    - Hard disk with at least 8.5 MB free for the program files
    - Additional disk space of at least 1 MB for storage of project input and output files, and temporary data files
4. Mouse—Recommended
5. Operating system—WINDOWS 98, NT, Me, 2000, or XP.

The RSAP installation program and associated documentation is provided on a CD-ROM. Users should have a good understanding of the WINDOWS operating environment and general mouse and keyboard techniques. On most PCs, the installation program can be initiated by inserting the CD-ROM into the drive and following the instructions on the screen. The program will automatically complete the installation of the RSAP program. For PCs with Autorun disabled, users must open the CD-ROM and double-click on the setup program.

A1.1 COST-EFFECTIVENESS ANALYSIS PROCEDURE

The cost-effectiveness analysis procedure in the RSAP program is based on benefit/cost (B/C) analysis. The basic concept behind benefit/cost analysis is that public funds should be invested only in projects where the expected benefits exceed the expected direct costs of the project. Benefits are measured in terms of reductions in crash or societal costs due to decreases in the number and/or severity of crashes. Direct highway agency costs are comprised of initial installation, maintenance, and crash repair costs. An incremental benefit/cost ratio between the incremental benefits and costs associated with an improvement option over the existing conditions or another improvement option is normally used as the primary measure of whether or not a safety improvement investment is appropriate. The incremental benefit/cost ratio is expressed as follows:

$$\text{B/C Ratio}_{2\text{-}1} = (AC_1 - AC_2)/(DC_2 - DC_1)$$

where:

B/C $\text{Ratio}_{2\text{-}1}$	=	incremental benefit/cost ratio of Alternative 2 compared to Alternative 1
AC_1, AC_2	=	annualized crash or societal cost of Alternatives 1 and 2
DC_1, DC_2	=	annualized direct cost of Alternatives 1 and 2

When the incremental benefit/cost ratio comparing safety improvement Alternative 2 to Alternative 1 is greater than 1, the analysis indicates that the increased benefits of Alternative 2 over Alternative 1, i.e., reduction in the crash and societal costs, are greater than the increased costs associated with Alternative 2 over Alternative 1.

Crash cost is estimated using an encroachment probability model, which is unique to roadside safety cost-effectiveness procedures. It is based on the concept that the run-off-the-road crash frequency can be directly related to the encroachment frequency, i.e., the number of vehicles inadvertently leaving the traveled portion of the roadway. The severity of run-off-the-road crashes is related to encroachment characteristics, such as the speed and angle of encroachment. The basic formulation of the encroachment model is expressed by the following equation:

$$E(AC) = \sum_{i=1}^{n} V * P(E) * P(A|E) * P(I_i|A) * C(I_i)$$

where:

E(AC)	=	Expected crash cost
V	=	Traffic volume, ADT
P(E)	=	Probability of an encroachment
P(A|E)	=	Probability of a crash given an encroachment
$P(I_i\|A)$	=	Probability of injury severity level "i", given a crash
$C(I_i)$	=	Cost associated with injury severity level "i"
n	=	Number of injury severity levels

A2.0 RSAP PROGRAM

There are four major modules to the encroachment probability-based cost-effectiveness analysis procedure in the RSAP program:

- Encroachment module,
- Crash prediction module,
- Severity prediction module, and
- Benefit/cost module.

Brief descriptions of each of these modules are presented as follows.

A2.1 ENCROACHMENT MODULE

The encroachment module uses roadway and traffic information to estimate the expected encroachment frequency, V*P(E), along a highway segment. A two-step process is used to estimate encroachment frequencies. The first step involves using highway type and traffic volume to estimate a base or average encroachment frequency. There are two available sources of encroachment data: a study by Hutchinson and Kennedy (6) in the mid-1960s and a study by Cooper (7) in the late 1970s. Both studies involved observations of tire tracks in the medians or on roadsides. The Cooper encroachment data were selected for use in the RSAP program for the encroachment rate-traffic volume relationships because they are more recent, have a larger sample size, and include data from two-lane and other non-freeway facilities as well as from controlled-access highways. Figure A.1 shows the encroachment frequency curves used by the RSAP program. Encroachment rates are expressed as the number of encroachments per km [mi] per year per ADT, broken down by undivided and divided highways.

Two adjustments are made to these encroachment frequency curves:

1. The encroachment frequency is adjusted upward by a ratio of 2.466 for two-lane undivided highways and 1.878 for multi-lane divided highways to account for under-reporting of encroachments due to paved shoulders.

2. The encroachment frequency is multiplied by a factor of 0.6 to account for the lack of ability to detect the difference between controlled and uncontrolled encroachments. The percentage of uncontrolled encroachments is assumed to be 60 percent based on a study of reported vs. unreported crashes involving longitudinal barriers (8).

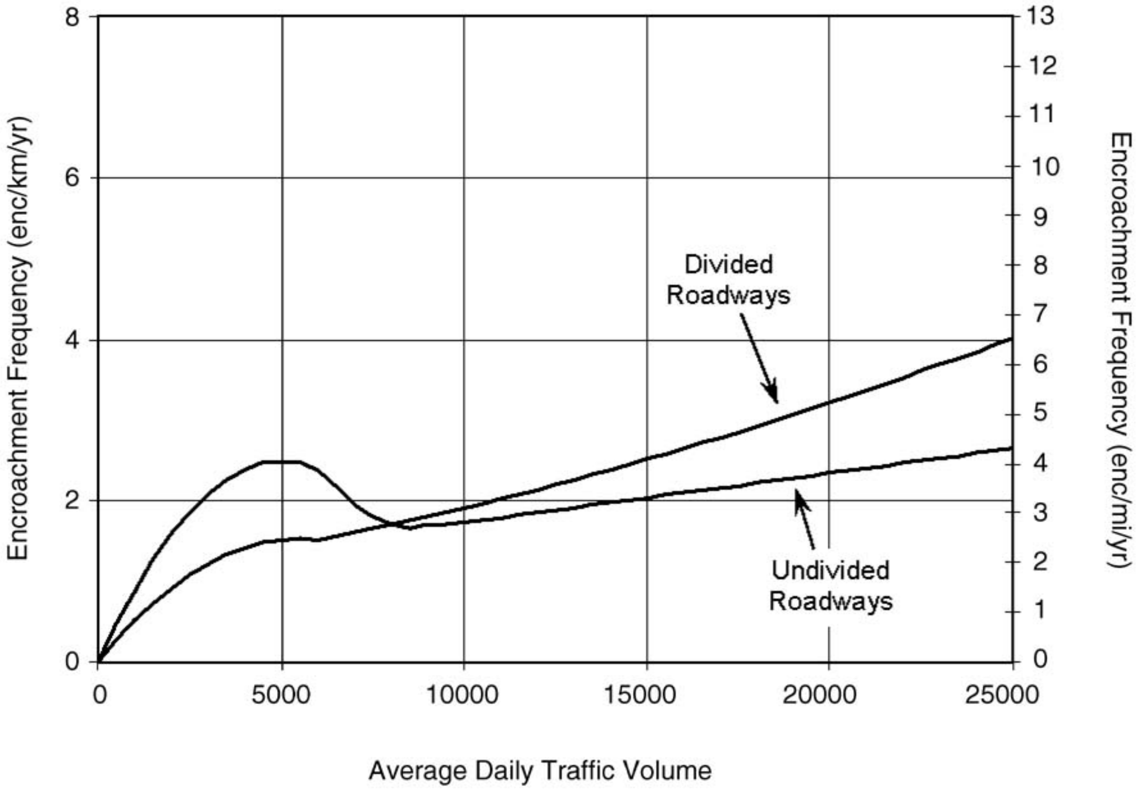

FIGURE A.1 Encroachment frequency curves used by the RSAP Program

Base encroachment rates are then modified to account for specific highway characteristics, including horizontal and vertical alignment and the annual traffic growth factor. The rationale for these adjustment factors is that encroachment rates are affected by these characteristics and the base encroachment rates should therefore be adjusted accordingly.

Crash data studies have indicated that crash rates on horizontal curves and vertical grades are significantly higher than those on tangent sections (9, 10). It is logical to assume that encroachment rates would also be similarly affected by horizontal curves and vertical grades. Thus, the RSAP program incorporates adjustment factors to increase encroachment rates on horizontal curves and vertical grades, as shown in Figure A.2. The adjustment factors are based on research conducted by Wright and Robertson (9).

Note that the adjustment factors for horizontal curvature and vertical grade are determined in relation to the direction of travel and the direction the vehicle ran off the road. A downgrade for one direction of travel would become an upgrade for the opposing direction of travel. Similarly, a vehicle running off to the right would be on the outside of a curve for one direction of travel and on the inside of a curve for the opposing direction of travel.

The traffic volume (ADT) entered into the RSAP program applies to the current year or construction year. To allow for future increases in traffic volume, the RSAP program allows users to input an annual traffic growth in percent. For a given year, n, in the future, the traffic volume is calculated as follows:

$$ADT_n = ADT_1 * (1 + g/100)^n$$

where:

ADT_n = traffic volume in year "n"
ADT_1 = current or base year traffic volume
g = annual percent traffic growth

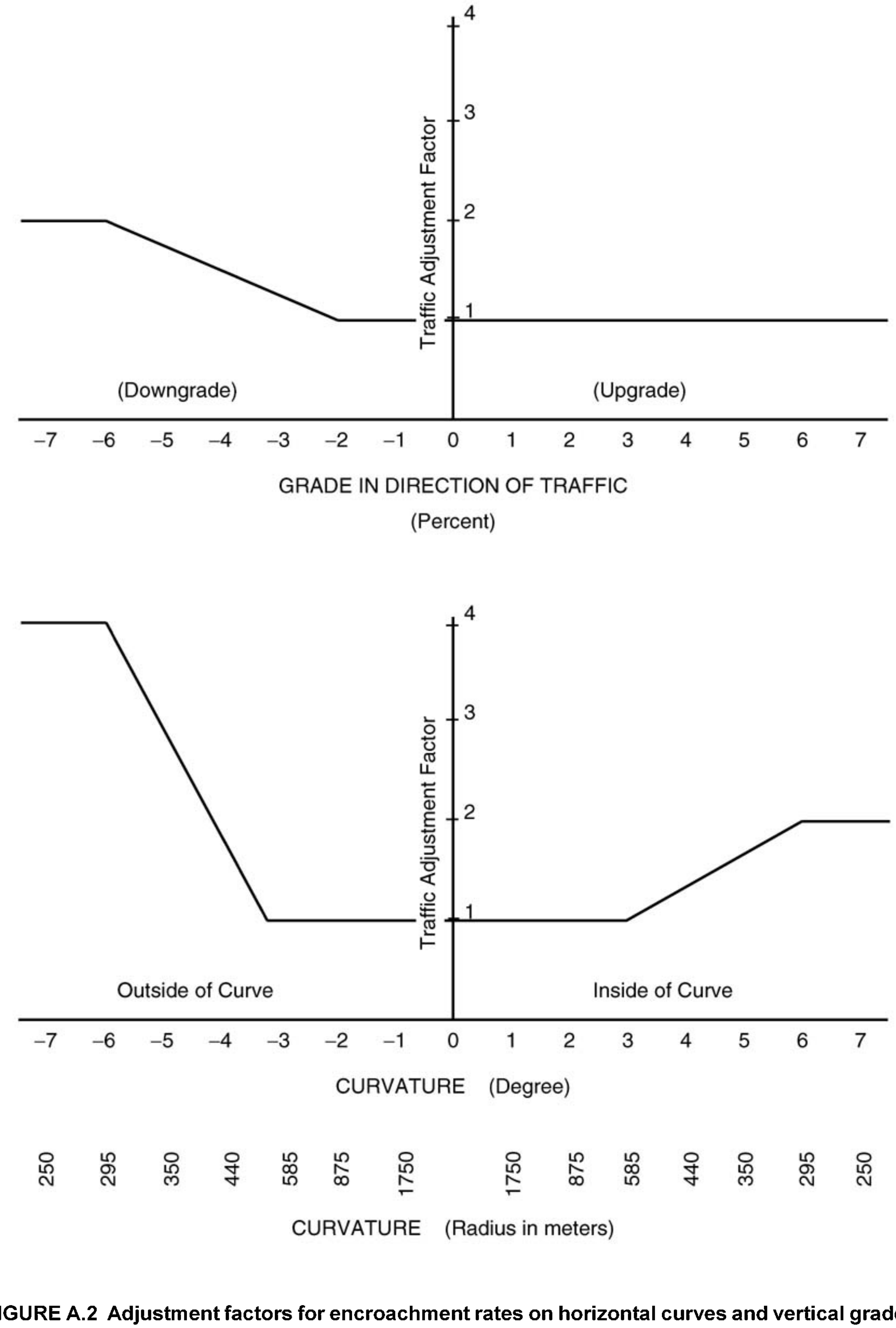

FIGURE A.2 Adjustment factors for encroachment rates on horizontal curves and vertical grades

The traffic growth adjustment factor averages the traffic volume over the life of the project and is calculated as follows:

$$\text{Traffic growth adjustment factor} = \sum_{n=1}^{N} (1+i/100)^n/N$$

where:

N = project life in years

The RSAP program allows the input of a user-defined adjustment factor to account for special or unusual situations that could affect encroachment frequencies beyond the parameters incorporated into the program. For example, an adjustment factor of greater than 1.0 may be appropriate if the highway section under consideration has a higher than average crash history or encroachment frequencies at night. An adjustment factor of less than 1.0 may be appropriate for a highway section with special safety countermeasures, such as rumble strips on the shoulder or increased law enforcement activities.

The encroachment module will then combine base encroachment rates and adjustment factors to determine encroachment frequencies for the highway section under study.

A2.2 CRASH PREDICTION MODULE

The crash prediction module assesses if an encroachment would result in a crash, P(A|E). A stochastic process using the Monte Carlo simulation technique is used for the crash prediction module, which involves using random selection processes to simulate vehicles running off the road within the highway section under study. One encroachment is simulated each time with the following characteristics randomly assigned to the encroachment: location along the highway, lane of origination, direction of encroachment, vehicle type, vehicle speed and angle, and vehicle orientation.

The random assignment of characteristics is based on distributions built into the program. For example, encroachments are assumed to be evenly distributed within a homogeneous roadway section and are a function of the encroachment frequency (i.e., the encroachments vary among roadway sections with different geometrics and encroachment frequencies). The lane of origination and direction of encroachment are a function of traffic volume distribution by lane. Vehicle type, which has 12 categories ranging from a small passenger car to a tractor-trailer, is a function of the vehicle mix calculated from the nominal truck percentage (user input item). Vehicle speed, angle, and orientation are determined from distributions estimated from real-world crash data (11).

A weighting scheme is used with the random encroachment assignments to ensure that rare events, i.e., combinations of distributions with low probabilities such as a tractor-trailer impact with high impact speed at an angle, will be properly represented in the distributions.

For each encroachment, the path (assumed to be a straight line) and the impact envelope of the vehicle are a function of the encroachment angle and the physical dimensions and orientation of the vehicle, as shown in Figure A.3. The presence of roadside features within the impact envelope of the vehicle is then checked. If there is no roadside feature within the impact envelope, the encroachment would not result in a crash. If there is a roadside feature within the impact envelope, then a crash would occur with the probability determined by the lateral extent of encroachment for the vehicle. The severity of the crash and the associated crash cost are then estimated by the severity prediction module. The crash frequency and crash cost are then multiplied by the probability that the vehicle would travel far enough laterally to reach the hazard. The lateral extent of encroachment distribution, shown in Figure A.4, is based on the Cooper encroachment data (7).

A new encroachment is then randomly generated and the process is repeated. After every 10,000 encroachments, the convergence of the solution is examined. The distributions of the encroachment characteristics for the simulated encroachments are compared to the pre-established distributions to check if they are within the specified level of convergence, which can be set by the user to high (1 percent), medium (5 percent), or low (10 percent). If all of these distributions are within the specified level of convergence, the simulation is terminated and the results are saved in the output files. Otherwise, another 10,000 iterations will be undertaken and the convergence checks outlined above will be repeated.

A2.3 SEVERITY PREDICTION MODULE

After a crash is predicted to occur, the next step is to estimate the severity of the impact using the crash severity prediction module. Crash severity estimation is perhaps the most important step of this cost-effectiveness analysis procedure. For most roadside safety improvements, the benefit, or reduction in crash cost, is derived from lower crash severity with little or no change, and sometimes even an increase, in the crash frequency. The crash cost is principally a function of the crash severity, i.e., the probability of injury and/or fatality, since the associated crash cost is highly non-linear.

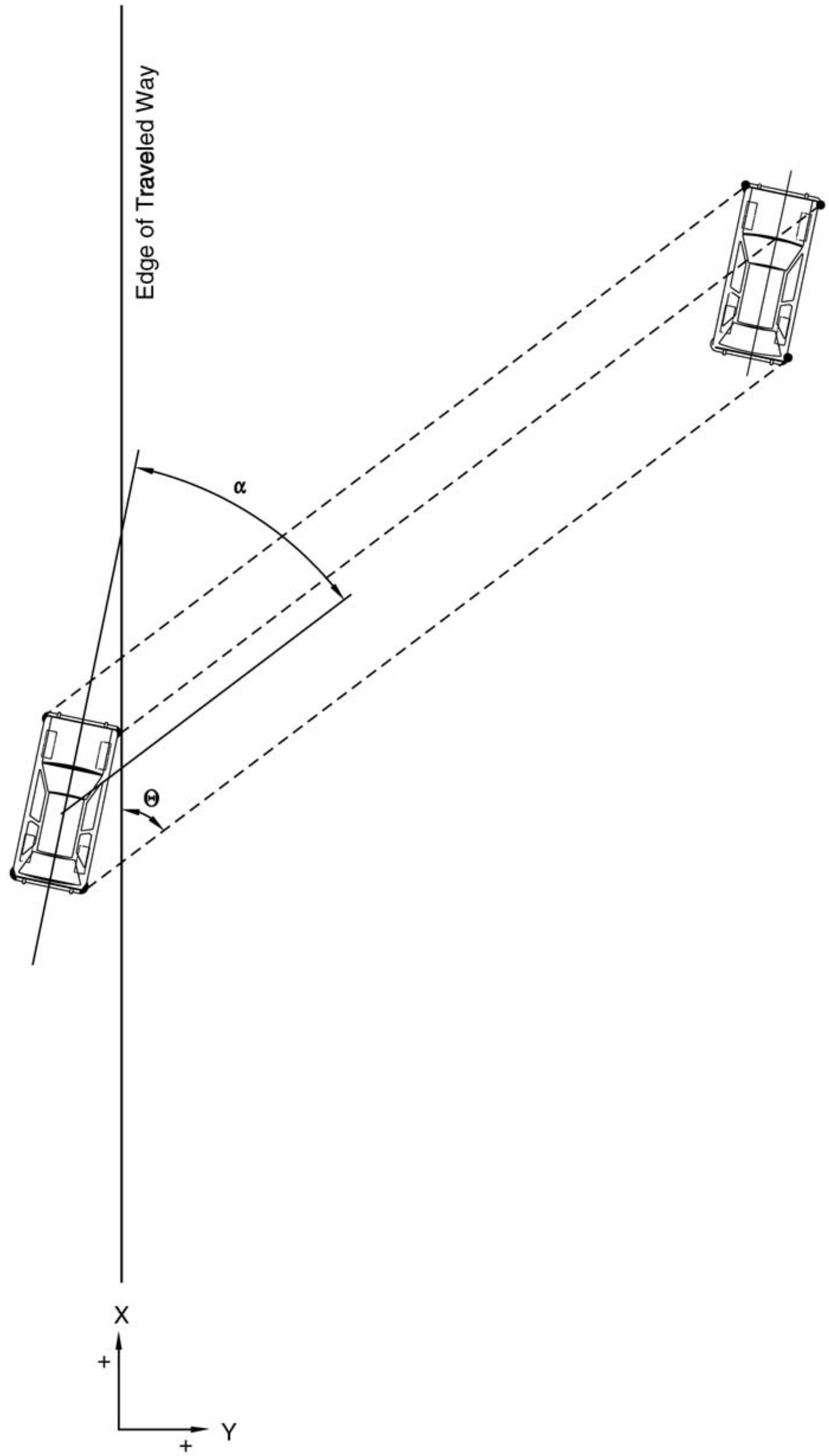

FIGURE A.3 Vehicle path and impact envelope

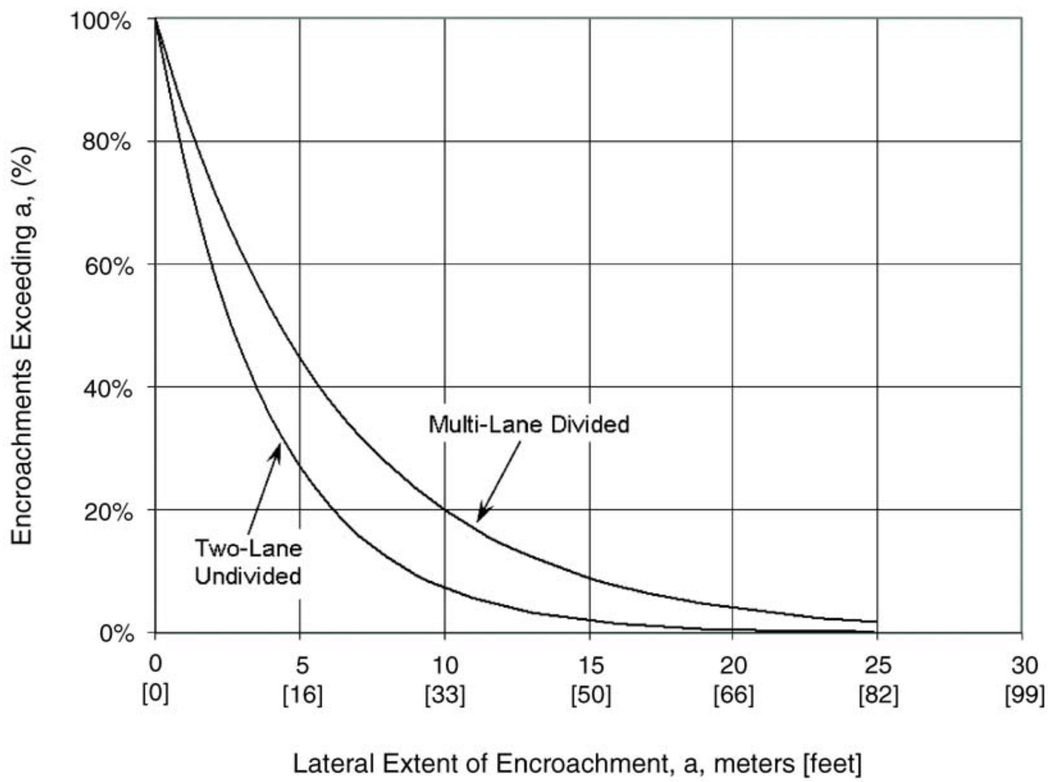

FIGURE A.4 Lateral extent of encroachment distribution

Each crash predicted by the crash prediction module is associated with a particular roadside feature or hazard, vehicle type, impact speed, impact angle, and vehicle orientation. This information is then used by the severity prediction module to estimate the severity, $P(I_i|A)$, and associated costs, E(AC), for the crash. For a given roadside object or feature and impacting vehicle, the conditions under which the vehicle impacts the roadside feature, i.e., speed, angle and vehicle orientation, determine the outcome and severity of the crash. In the case of a roadside safety device, e.g., a guardrail, crash cushion, etc., the performance limit of the safety device should also be taken into account. For example, the severity of an impact involving a longitudinal barrier is much different if the vehicle is successfully redirected than if the vehicle penetrates the barrier or rolls over. Separate procedures for determining impact performance are developed for each roadside hardware feature included in the model. Also, these procedures are vehicle dependent, i.e., different techniques are used to estimate rollover or vaulting potentials for passenger cars and trucks.

Crash severity is expressed in terms of a severity index (SI), which is a surrogate measure for injury probability and severity. Table A.1 illustrates the relationships of severity indices and probability of injury for various injury levels.

The severity indices used in the RSAP program are basically those used in the ROADSIDE program with some modifications. Specifically, severity indices are expressed as a function of impact speed instead of roadway design speed. In the *Roadside Design Guide* (2), average severities or SI values are provided for the various roadside objects and features for design speeds of 50, 70, 90, and 110 km/h [30, 45, 55, and 70 mph], which were assumed to be the design speeds for urban collectors, rural collectors and urban arterials, rural arterials, and freeways and interstate highways, respectively. For each roadside object or feature, a linear regression line was fitted through these SI values as a function of speed. Note that these regression lines would almost always originate from the zero point since an impact speed of zero (0) km/h [0 mph] should not produce any damage to the vehicle or injury to the occupants. Figure A.5 shows an example of this linear relationship between SI and impact speed.

TABLE A.1 Relationship of Severity Indices (SI) and probability of injury

Severity	Injury Level (%)						
Index (SI)	None	PDO1	PDO2	C	B	A	K
0.0	100.0	—	—	—	—	—	—
0.5	—	100.0	—	—	—	—	—
1.0	—	66.7	23.7	7.3	2.3	—	—
2.0	—	—	71.0	22.0	7.0	—	—
3.0	—	—	43.0	34.0	21.0	1.0	1.0
4.0	—	—	30.0	30.0	32.0	5.0	3.0
5.0	—	—	15.0	22.0	45.0	10.0	8.0
6.0	—	—	7.0	16.0	39.0	20.0	18.0
7.0	—	—	2.0	10.0	28.0	30.0	30.0
8.0	—	—	—	4.0	19.0	27.0	50.0
9.0	—	—	—	—	7.0	18.0	75.0
10.0	—	—	—	—	—	—	100.0

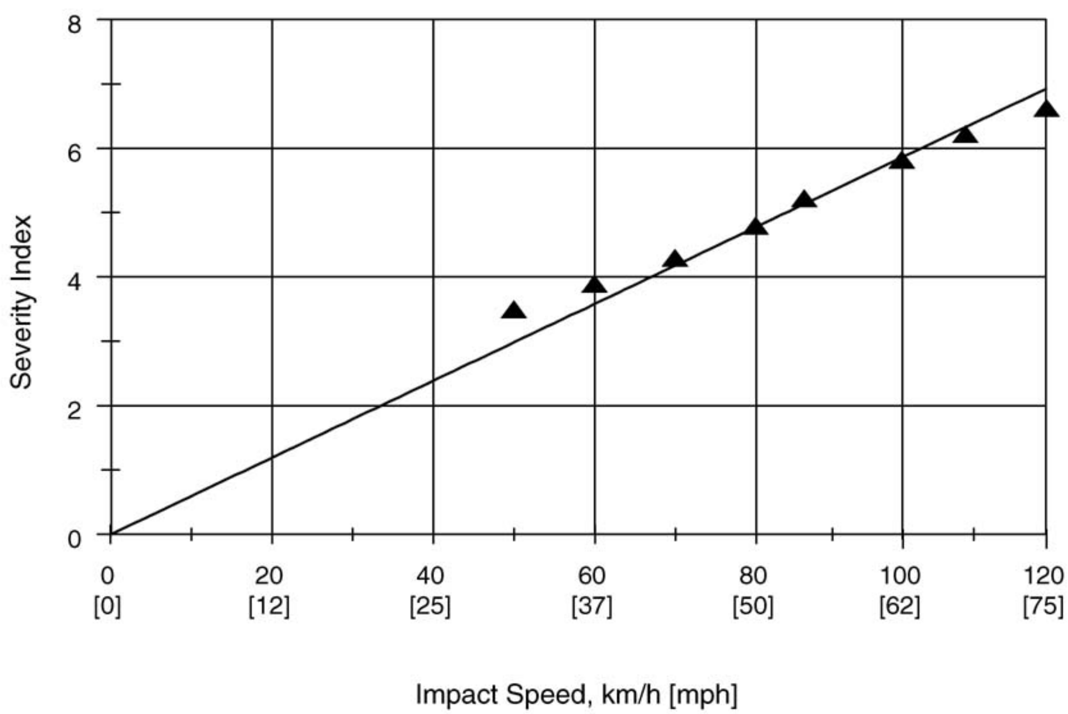

FIGURE A.5 Example of relationship between Severity Index (SI) and impact speed

This simplistic calibration method removed some of the inconsistencies in the earlier ROADSIDE SI tables. More importantly, it relates SI values to specific impact speeds for each roadside object or feature instead of average SI values. There are, however, two exceptions to this procedure. First, large vertical drops would not necessarily have an SI value of zero for an impact speed of zero because gravity would also play a large role in the probability of vehicle damage and occupant injury. Therefore, the regression lines for vertical drops were not fitted through the zero point. Second, lateral speed, V_{lat} , was used instead of impact speed for the SI relationships of longitudinal barriers since the severity of a longitudinal barrier impact is a function of both the impact speed and the impact angle. ($V_{lat} = V*\sin\theta$, where V is the impact speed and θ is the impact angle.)

A2.4 BENEFIT/COST MODULE

After the severity of a crash is estimated by the crash severity prediction module, the crash or societal costs associated with the crash are then calculated by multiplying the probability of each level of injury by the cost associated with that level of injury.

$$AC = \sum_{i=1}^{n} P(I_i) * C(I_i)$$

where:

AC = crash cost
$P(I_i)$ = probability of injury severity level "i"
$C(I_i)$ = cost associated with injury severity level "i"
n = total number of injury severity levels

As previously shown in Table A.1, the severity index (SI) is associated with six injury levels: fatality (K), severe injury (A), moderate injury (B), slight injury (C), property-damage-only level 2 (PDO2), and property-damage-only level 1 (PDO1). The severity estimate of the crash is then converted to crash costs using crash cost figures selected by the user. The program offers the choice of crash cost figures from the AASHTO *Roadside Design Guide* (2) or the FHWA comprehensive cost figures based on the willingness to pay approach, as shown in Table A.2. Alternatively, the user can input values for crash costs for various injury severity levels to suit the particular needs of the agency.

The crash costs are normalized to an annual basis. The normalization process involves two steps:

1. The crash cost is divided by the weighted number of encroachments and then multiplied by the expected number of encroachments per year to convert to an annual basis.

2. The crash cost is unweighted to arrive at the true crash cost. As discussed previously, the probability distributions for various encroachment characteristics are weighted to ensure proper sampling of conditions with very low probabilities to improve the accuracy of the analysis results and the speed at which the RSAP program arrives at a solution.

The direct costs, which include the costs for initial installation of the safety feature, normal maintenance, and repair of damages from crashes, are also normalized to an annual basis. The initial installation is converted to an annual basis using the project life and the discount rate. The normal maintenance cost is already entered on an annual basis. The cost of repairing roadside safety hardware is estimated by correlating repair costs to impact energy terms. For example, results from full-scale crash testing and computer simulations are used to determine the relationship between impact energy terms and length of guardrail damage. The unit repair cost for a typical guardrail, e.g., $50.00 per meter [$15.24 per foot] is then estimated. The total repair cost is therefore the product of the length of damaged rail and the unit cost for repair. Procedures for estimating the extent of hardware damage are developed for each longitudinal barrier design, as well as most common crash cushions, barrier terminals, and other roadside safety devices.

Incremental benefit/cost ratios are then calculated for all alternatives in a pairwise manner. As shown previously, the expression for calculating the incremental benefit/cost ratios is as follows:

$$B/C\ Ratio_{2-1} = (AC_1 - AC_2)/(DC_2 - DC_1)$$

where:

$B/C\ Ratio_{2-1}$ = incremental benefit/cost ratio of Alternative 2 compared to Alternative 1
AC_1, AC_2 = annualized crash or societal cost of Alternatives 1 and 2
DC_1, DC_2 = annualized direct cost of Alternatives 1 and 2

The numerator of this equation is the difference in crash or societal costs between the two alternatives. Since Alternative 2 is being evaluated as a potential safety

TABLE A.2 Crash cost figures*

Crash Severity	*Roadside Design Guide*	FHWA Comprehensive Cost
Fatal Crash	$1,000,000	$2,600,000
Severe Injury Crash	200,000	180,000
Moderate Injury Crash	12,500	36,000
Slight Injury Crash	3,750	19,000
PDO Crash Level 2	3,125	2,000
PDO Crash Level 1	625	2,000

* Crash cost figures are based upon the 1996 edition of the *Roadside Design Guide* and a 1994 FHWA memorandum entitled "Update of Value of Life and Injuries for Use in Preparing Economic Evaluations."

improvement over Alternative 1, the societal or crash costs of Alternative 1 would be expected to be higher than those of Alternative 2. Thus, the numerator is expressed as (AC_1–AC_2). The denominator of the equation represents the differences in direct costs to the transportation agency associated with implementing the safety improvement of Alternative 2 in relation to Alternative 1. Again, since Alternative 2 is being evaluated as a potential safety improvement over Alternative 1, the direct costs of Alternative 2 would be expected to be higher than those of Alternative 1. Hence, the denominator is expressed as (DC_2–DC_1).

A3.0 COMPARISON WITH ROADSIDE PROGRAM

Table A.3 presents the major differences between the RSAP program and the ROADSIDE program, which is the cost-effectiveness analysis procedure presented in previous versions of the *Roadside Design Guide*(2,3). ROADSIDE uses a constant encroachment rate of 0.0003 encroachment per km [0.0005 encroachment per mile] per year per ADT. The lateral extent of encroachment distribution is based on a constant deceleration rate of 3.66 m/sec/sec [12 ft/sec/sec], or 0.4 g, and a sine curve density function for steer back. In comparison, the RSAP program uses the Cooper encroachment data. Adjustments were made to account for encroachments with 4 m [13.1 ft] or less of lateral extent which might not have been detected due to presence of paved shoulders.

ROADSIDE uses a hypothetical distribution for encroachment speed based on design speed and an average encroachment angle based on the point-mass model. A constant deceleration rate of 3.66 m/sec/sec [12 ft/sec/sec], or 0.4 g, is assumed for calculating the impact speed from the encroachment speed. A straight path is assumed so that the impact angle is the same as the encroachment angle. In comparison, RSAP uses impact speed and angle distributions from real-world crash data. A straight path with no braking is assumed so that the encroachment speed and angle are the same as the impact speed and angle.

ROADSIDE uses only a single vehicle type and an average encroachment angle for the hazard imaging. Vehicle orientation is not taken into account. The program can handle only one hazard at a time and shielding of one hazard by another is not incorporated. For multiple hazards, each hazard has to be analyzed individually and the crash costs summed manually. In comparison, RSAP allows for 12 vehicle types. Vehicle orientation is incorporated into the program based on real-world crash data. Hazard imaging is based on the size of the vehicle, encroachment angle, and vehicle orientation. The program can handle multiple hazards with algorithms to account for shielding of one hazard by another and multiple impacts.

ROADSIDE uses an average severity index without accounting for speed. RSAP estimates severity as a function of impact speed instead of an average value. These improvements incorporated into RSAP provide better severity estimates, which is perhaps the most critical element for estimating crash costs. Further, ROADSIDE assumes that all impacts with a hazard shielded by a barrier are eliminated, regardless of barrier length. RSAP allows for impact with a hazard shielded by barrier if the vehicle encroaches upstream of the barrier.

A4.0 SUMMARY

This appendix provides a summary of the conceptual framework and algorithms contained in the RSAP computer program, which has been created to assist in the economic analysis of existing or proposed roadside conditions. For users desiring more detailed information, please refer to the *Engineer's Manual*. Also, for detailed descriptions on the operation of the RSAP program, please refer to the *User's Manual*.

TABLE A.3 Comparison between RSAP and ROADSIDE programs

Data Element	ROADSIDE	RSAP
Encroachment Rate	A constant of 0.0003 encroachments per km per year [0.0005 encroachments per mi per year] per vehicle per day	Cooper encroachment data, adjusted for encroachments with lateral extent ≤ 4 m [13.1 ft]
Encroachment Speed	Function of design speed	Same as impact speed
Encroachment Angle	Average angle based on point-mass model	Same as impact angle
Impact Speed	= Encroachment speed–speed loss with 3.66 m/sec/sec [12 ft/sec/sec] [0.4 g] deceleration rate	Based on real-world crash data
Impact Angle	Same as encroachment angle	Based on real-world crash data
Lateral Extent of Encroachment	Assumes 3.66 m/sec/sec [12 ft/sec/sec] [0.4 g] deceleration rate and sine curve density function for steer back	Cooper encroachment data, lateral extent ≤ 4 m [13.1 ft]
Vehicle Type	One	12 vehicle types, based on nominal percent trucks
Vehicle Orientation	None	Based on real-world crash data
Shielding of One Hazard by Another	No	Yes
Multiple Hazards	Each hazard has to be analyzed individually and the crash costs summed manually	Yes
Effect of Barrier Protection	All impacts with hazard shielded by barrier eliminated, regardless of barrier length	Vehicles encroaching upstream of barrier could impact hazard shielded by barrier
Severity (SI)	Average values only	Function of impact speed
Incremental B/C Ratios for Multiple Alternatives	Have to be calculated manually	Yes
Solution Method	Deterministic	Stochastic using the Monte Carlo simulation technique

The RSAP program presents many new advances and features over its predecessors. Highlights of the improvements incorporated into the RSAP program are summarized as follows:

- A user-friendly interface with WINDOWS-like screens and menus to facilitate easier use of the program by inexperienced users. Features of the User Interface Program include:

  –simplified data input process with multiple choice entries where appropriate,

  –numerous built-in default values to reduce data entry requirements,

  –on-screen instructions and help,

  –built-in edit and consistency checks,

  –options to choose built-in default values or input user-defined values for crash cost figures and vehicle mix, and

  –choice of reports to preview or print to hard copies or electronic files.

- Capability to handle evaluations of projects with a maximum of 20 different safety improvement alternatives, 20 consecutive roadway segments for roadways of up to 16 lanes, and 1,000 roadside features. The program is capable of simultaneously analyzing hazards on either or both sides of the roadway as well as in the median for a divided roadway.

- Use of a stochastic solution method with the Monte Carlo simulation technique to allow for modular design of the Main Analysis Program. The program can be updated in the future without a major rewrite of the source code to incorporate such new features as curvilinear vehicle path, driver inputs, side impacts, etc.

- Use of re-analyzed Cooper encroachment data for encroachment rates and lateral extent of encroachment distributions with adjustments for under-reporting of encroachments with small lateral extent due to presence of paved shoulders and controlled versus uncontrolled encroachments.

- Use of real-world crash data for impact speed and angle distributions instead of theoretical distributions.

- Incorporation of vehicle orientation into the analysis code to better define vehicle swath or impact envelope. More importantly, this would allow for future consideration of non-tracking crashes and side impacts, which account for a significant percentage of run-off-the-road crashes and have been shown to result in higher severities than tracking crashes.

While the RSAP program is an improvement over existing procedures, it also has drawbacks and limitations, most of which are the result of lack of available data or which require a level of effort beyond that available for this study. Some of the limitations and future modifications and refinements are as follows:

- **Applications.** The RSAP program is intended for the evaluation of safety treatments for hazards/features along the roadside or in the median and cannot handle other applications, such as crossover crashes at narrow median sites.

- **Computational time.** The use of the Monte Carlo simulation technique requires longer computational time.

- **Multiple solutions.** Due to the nature of the stochastic process, the solutions or answers will vary from run to run within a range as determined by the convergence criteria. This variation for a given project can be eliminated by using the same seed number for all the runs, which is an option provided in the program.

- **Encroachment data.** The Cooper encroachment data are almost 30 years old and many improvements to vehicle and highway designs have been implemented in the interim. The encroachment probability model can greatly benefit from better encroachment data.

- **Vehicle path.** The RSAP program does not currently take into account vehicle and driver behavior during encroachments due to lack of available data. The incorporation of curvilinear vehicle paths, vehicle orientation, and slope effects would significantly improve crash prediction and impact severity estimation.

- **Extent of lateral encroachment distributions.** The effects of roadside slopes and geometrics are not adequately addressed in the current distributions for the extent of lateral encroachment.

- **Crash severity.** The severity index approach currently incorporated in the RSAP program has many limitations. A better approach to estimate severity, such as the probability of injury approach, would be highly desirable. In the interim, the severity indices of individual roadside objects or features could benefit from a critical review and then revised as appropriate.

- **Impact models.** The impact models for predicting vehicle penetration and rollover that are incorporated into the RSAP program are relatively simplistic in nature and could benefit from more sophisticated and better validated models.

Finally, it should again be emphasized that the RSAP program is intended as a tool for economic analysis and should not supersede the guidelines presented in the *Roadside Design Guide* or sound engineering judgment.

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