A Five Day Training Course

Safety Design and Operational Practices for Streets and Highways

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Preface

Mobility, frequently acclaimed as our fifth freedom, is the basic fiber of modern society. Without mobility, progress in our community is stifled; with it, growth and prosperity prevail. Although mobility is basic to our society, it is also our nemesis. The price we pay for mobility is the hazard of personal injury and property damage due to collisions of errant vehicles. It may seem pessimistic to assume that accidents resulting from mobility are inevitable but, realistically, this is true. So long as people choose to operate their vehicle of mobility, Murphy's law prevails; when things can go wrong, they will. Only complete automation may eliminate accidents, but people build and operate automation systems!

As a people, we may not be able to change our final destiny, but through the application of technology we can alter the degree of its impact. Unscrambled, this means that we cannot eliminate traffic accidents; but we can reduce the number and severity of accidents through the application of recent developments in design criteria, design standards, design policy and practices, and safety devices for streets and highways. A training course has been prepared to present this technology, and this document is the participant's notebook for the training course.

This course was developed under the auspices of the Federal Highway Administration's Offices of Highway Safety, Engineering and Traffic Operations. It provides a practical approach to the application of traffic safety in the design and operation of streets and highways. Upon completion of the course, the participants should be qualified to:

- Effectively integrate safety as a principal criterion in planning, designing and operating traffic facilities.
- Identify hazardous conditions or situations, and
- Select and apply appropriate coutermeasures to eliminate or neutralize any hazard.

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INTRODUCTION

Objectives:

As a result of this session, the participant should:

1. Consider specific safety problems from a broader, more comprehensive point of view,

2. Approach the tasks of design/operations/management with an attitude more attuned to safety, and

3. Integrate safety as a principal criterion in the design/operations process.

1.1.1 DEFINING THE HIGHWAY SAFETY PROBLEM

As we begin a course on Safety Design and Operational Practices, we should perhaps attempt to resolve the question - What is the highway safety problem? It is not quite so easily defined because it is different things to different people, dependent upon each one's own perspective. It may be classified as an engineering problem, an education problem and as an enforcement problem, dependent upon one's point of view. Actually, it is a combination of all three, plus more.

Highway safety is obviously a social problem because of its profound effect on society. This point is illustrated by the National Safety Council statistics on motor vehicle accidents for 1976 (1).

General Statistics

Deaths	46,700
Disabling injuries	1,800,000
Costs	$24.7 Billion

Accident Totals	No. of Accidents	No. of Drivers Involved
Fatal	40,600	59,000
Disabling injury	1,200,000	2,000,000
Property damage and non-disabling injury	15,600,000	26,400,000
Total	16,800,000	28,400,000

Regardless of which formula one may use to compute the monetary loss and the physiological and psychological stress due to these accidents, the true cost to society is so great that it defies comprehension. Yet society seems to accept this nemesis as simply a part of life. Why? Is it simply the price we pay for the convenience of the highway mode? These questions certainly relate to the technological and scientific (engineering) aspects of highway safety, and we will try to resolve some of these questions.

At least part of the safety problem is explained, hypothetically, by stating that it is inherent in man's nature. What does that mean -- that man is bent upon killing himself? Indirectly, yes! Since the beginning of recorded history, man has desired mobility. Through desire, ambition and even restlessness, man has wished to travel from place to place for various reasons. In the 15th century, Martha "Mother" Shipton recorded in her poem "Prophesy" the following lines:

> "Carriages without horses shall go,
> And accidents will fill the world
> With woe."

Even though Mother Shipton was accused of being a witch, she could have drawn this conclusion simply as a student of human nature. She could have observed that man is inherently physically lazy; he will not walk when he can ride. He is impatient; he wants to travel quickly or rapidly. He is inventive; he will experiment and expand his knowledge and technology until he finds a better, faster way. And, he is daring - he will overextend himself on rare occasions and become the victim of the laws of chance -- in other words, he can have an accident. Perhaps this explains why we have accidents in highway travel as well as in every other mode. Does this mean that accidents -- death and destruction -- are inevitable, so why fight them? The response is, yes and no, for so long as we have humans at the control of the transport vehicle, there will be accidents; however, to accept this phenomenon as inevitable and not attempt to correct it would be against the natural laws of human nature. The inherent ambition of man presses him to more efficient transportation on the one hand, and this same compulsion pushes man to work and to devise methods of reducing the probability and severity of accidents. No, we will never eliminate accidents but yes, we will make significant reductions in accidents through technological advancement.

If traffic safety is a social problem like a disease, then why do we not attack it as we have polio, smallpox and other illnesses and eliminate the nemesis? We have done this to some extent; but, somehow, traffic accidents are more "accepted" than most diseases. It

is believed that the probability of occurrence has a great deal of influence. In an epidemic, people relate very closely because of the high probability of contracting the disease. But traffic accidents always "happen to the other guy" because of the infrequency of their occurrence to the individual. Through some simple calculations and typical accident rates, we may determine the following statistics:

- On the average, a driver will be involved in an accident every 55,000 miles. Even if he should drive 20,000 miles per year, he still averages an accident approximately every three years - and this includes all accidents - fender benders to fatals.

- Further, we may note that one fatal accident occurs every 22,000,000 miles. Again, for a driver who drives 20,000 miles per year, he may expect to be killed every 1100 years! Is it not natural that the driver relates an accident, particularly a fatal accident, as something that happens to the other driver?

As engineers we should consider yet another view of the highway safety problem and the high degree of emphasis that is currently placed upon it. This is reflected in where we are in the transportation mode life cycle. In theory, there are six basic steps or phases in the life cycle of any transportation mode. These include:

- Need
- Technological Breakthrough
- Developmental Phase
- Mature Phase
- Declining Phase
- Basic Service Phase

How does life cycle relate to traffic safety? Let us explain briefly each phase of the life cycle.

Need. There is always a need for a better mousetrap -- or a faster, more efficient transportation mode, whether it be actual or perceived. How else could we explain the current clamor over the SST?

Technological Breakthrough. In order for the need to be satisfied, a technological breakthrough, or an invention, is necessary. For example, the invention of the wheel and the domestication of the horse aided highway transportation greatly; however, the major breakthrough was the invention of the internal combustion engine.

Development Phase. The development phase is the period in which a transportation system is expanded to satisfy public need and desire. In highway transportation, the developmental phase encompasses the period from the 1920's to the present, or at least to the 1960's. During this period, we experienced the tremendous expansion of highway facilities -- the primary, secondary, urban and Interstate highways. In the automobile, we have experienced developments in propulsion, control, and construction methods that have made possible auto ownership by practically every American of driving age. We have transformed the embryonic T-model on the dusty trail (or sea of mud!) to a highly sophisticated transportation system that is an integral part of our independent lifestyle.

Mature Phase. At some point in the most recent years, our highway system went into the Mature Phase. As in life, the mature phase is the time for maximum service. In highway transportation, it is a time to shift our priorities. Whereas in the developmental phase we concentrated on developing a complete system, we now must concern ourselves with operating this complete system. This change in priority brings several operational criteria to the forefront - environmental quality, social concern and safety. During the developmental phase we had neither the time, the technology, nor the inclination to develop all these qualities to their maximum extent. But now is the time of concern -- the time to fine-tune this fantastic system of streets and highways, and to make it serve mankind more safely and efficiently.

Declining Phase. The declining phase is that period when a technological breakthrough has facilitated the beginning of the development of a new transportation mode. The new transportation mode attracts users away from the mature mode and takes away the resources that must be available for maintenance and operation. The result of declining resources is declining service and thus the declining phase. Such a mode will go through various stages of disrepair until it is reduced to providing basic service.

Basic Service Phase. The basic service phase is that phase in which a given transportation mode provides the service or services that it can provide more effectively and economically than any other transportation mode. For example, the railroad has gone through the declining phase and is now operating in the basic service phase. It is handling those large commodities and providing the services that can be provided best and most economically by rail transportation.

Based on the premise that highway transportation is now in the mature phase, it is logical that our efforts should be devoted primarily to improving the system to make it operate more safely and more efficiently. This will be accomplished through the implementation of new technology gained from operating experience with the current system. Most of this implementation will result in

modifications of existing facilities rather than reconstruction. This current emphasis on implementation of improvements should not and cannot preclude the necessity for some new construction. As the population expands, it is necessary that we expand our system accordingly.

It is a gross error to claim the insensitivity of highway engineers as the root of the highway safety problem. It is true that highway engineers have in the past operated according to criteria that were not compatible with current thinking. The same can be said for planners, for administrators and even enforcement officials. But they were acting in good faith, serving the public as best they could with the technology available to them at that time. For example, the roadside sign supports used along our Interstate highways in the early 1960's are not acceptable according to today's standards; yet they were designed according to the accepted criteria of that time. These supports were serviceable; they maintained the sign in a position that it could be read by the driver. They were designed to be aesthetically pleasing, to require a minimum of maintenance, and to facilitate easy maintenance of the grass area around the sign. They were comparatively economical; thus, they possessed the attributes of good engineering. But... people ran off the road, struck these sign supports and suffered serious injury or death.

Safety was not a criterion for two reasons. First, we had not considered the negative alternative -- the paved roadway was built to drive on -- the roadside was built to drain surface water. In addition, the attitude of the populace until recently was that an individual who ran off the roadway was either drunk or otherwise reckless and irresponsible and probably deserved his fate. Since that time, however, we have come to realize that the unfortunate driver may not be guilty of some wrong and deserves every consideration for lessening the chances of misfortune. In fact, the cause of the errant driver's dilemma may be due to roadway or operational deficiencies that have become deficient because of the rapid advancements in technology.

Then, perhaps we can describe the highway safety problem in this manner: Safety problems exist because we must operate a highway system whose elements are in various stages of obsolescence due to changing technology, for drivers who demand the maximum state of mobility, using the most modern automobiles that are within their financial means. We will never catch up, but it is our duty to keep trying.

1.1.2 STATISTICAL INFERENCES OF TRAFFIC ACCIDENT DATA

Now that we have defined, at least to a degree, the safety problem, the question is, "What can we do about it? Where are the safety problems, and what countermeasures should we use to combat the problem?" As will be discussed within this course, there are a number of measures by which we may identify safety problems, but analysis of accident statistics is the most common method used today. Let us explore some inferences that may be drawn from accident statistics.

First, just how good is our track record in dealing with the safety problem? Fatality rates over the years can provide a realistic measure. In 1937, the fatality rate was 14.6, and in 1975 it was 3.3 per 100 million vehicle miles; thus, we have reduced the rate by 77%. Improved engineering practice shares this accomplishment with automakers, educators, and enforcement personnel.

Statistics can be used to identify areas of potential improvement. A survey of accident experience by accident type should give us at least a preliminary indication of where we should direct our resources. For illustrative purposes, 1976 data on fatal accidents for a representative state are compared as follows (2):

Type of Accident		Percent of Total Fatalities
Pedestrian-Vehicle Collisions		16%
Vehicle-Vehicle Collisions		37%
Head-on	17%	
Rear-end	4%	
Angle	15%	
Sideswipe	1%	
Train-Vehicle Collisions		2%
Fixed Object Collisions		25%
Run-off-the-Road; Overturn in Road (loss of control)		11%
Bicycle-Vehicle Collisions		2%
Others		7%
		100%

In reviewing these statistics, we quickly note that there are certain types of accidents that contribute substantially to the highway safety problem. It appears logical to consider the potential effect that may be realized through concerted efforts in these areas. These are listed in the numerical priority based on percentage of total fatalities.

1.	Fixed Object Collisions	25%
2.	Pedestrian-Vehicle Collisions	16%
3.	Head-on Collisions	17%
4.	Angle Collisions	15%
5.	Loss-of-Control Collisions	11%

Fixed object collisions and loss-of-control collisions can be classified under one general category as out-of-control collisions, and they will account for approximately one-third of all accidents. These collisions generally involved only one vehicle losing control or over-driving the situation and becoming involved with a hostile environment. This hostile environment may include the encountering of critical side slopes or skidding which would cause overturning on the roadway or roadside. If, however, a fixed object exists within the area where the vehicle is out of control, then the collision becomes a fixed object collision. The source of the trauma is the same; the environment may differ. This particular type of collision perhaps has a high probability of correction by engineering methods. There are three alternatives: (1) we may eliminate the fixed object or the hostile environment such as a steep side slope, (2) we may erect a barrier or a cushion that will redirect or intercept the errant vehicle to avoid the trauma; or (3) we may even be able to eliminate the conditions which caused the vehicle to go out of control. This is an area in which there is considerable potential and, of course, this is an area where a great deal of work already has been done. A substantial part of this course relates to the single-vehicle-accident situation.

The right-angle collision or typically the intersection collision constitutes a very large portion of the fatal accidents -- roughly one-sixth of the total. These statistics could be worse than they are if it were not for the extensive efforts that have been made in the area of improving the quality of traffic control, the design of intersections and the many grade separations on controlled-access facilities. Accidents in this category are attributable to several factors, not all of them deficiencies in design and operation. Education and enforcement must share the credit. Efforts must be intensified in programs oriented to the 3 E's - Engineering, Education, and Enforcement.

Another category of vehicle-to-vehicle collisions, the head-on collision, also constitutes 17%, or roughly one-sixth of the total fatal accidents. These are largely the product of our two-lane highway system. Many of these could be reduced by eliminating the two-lane highways; however, this is totally unfeasible from the economic standpoint. There are methods of improving the safety of two-lane highways, and we will touch upon some of these in this course. Included are better sight distance applications, climbing lane and passing lane applications, and improved highway cross sections.

In still another area, the vehicle-pedestrian collision, we have 16%, or again approximately one-sixth, of the total fatal accidents. In many states the pedestrian accident experience is even higher. For example, the national statistics show that roughly 20% of all fatal accidents are pedestrian types, and 30% of all urban fatal accidents are pedestrain accidents. These numbers become very significant, and a new dimension is added, by pointing out that only two and a half percent of all accidents involve pedestrians. This demonstrates the severity of the vehicle-pedestrian collision.

Pedestrian accidents are not easily dealt with because of the very nature of the conditions surrounding the accidents. The major contrast in operating characteristics is highly contributory. Pedestrians are slow and vulnerable, where vehicles are comparatively fast and invulnerable. When there is a conflict between vehicle and pedestrian, the vehicle wins. Pedestrian controls and safety provisions are not as effective as they should be because of a general disregard for laws pertaining to pedestrians and a general lack of enforcement. Enforcement officers claim that it is virtually impossible to enforce pedestrian laws.

We need to take a close look at our track record. Where is the greatest accident experience in our own locality, and which types offer the greatest potential for improvement? Portions of this course are directed toward the identification and correction of hazards based on accident experience.

1.1.3 WHOSE RESPONSIBILITY IS SAFETY?

Responsibility for safety is a rather nebulous charge. When we analyze the situation, we find that there are a large number of transportation-related disciplines that contribute to the safety or the lack of safety of a given highway system. This section will attempt to enumerate some of these disciplines and the manner in which they influence highway safety.

1.1.31 Administration

Persons responsible for the administration of a highway system, whether they be city, county, state or federal employees, have a very strong influence over the safety of that system. The administration makes decisions relative to the direction of programs, and these programs may be designed to accomplish a wide range of goals and objectives. If a high degree of emphasis is placed on safety within the direction of the programs, then certain levels of safety improvement will be achieved. If, on the other hand, little regard is given to safety, then little will be achieved.

Administrators (and politicians) have a great deal to do with the availability of funds for highway improvements. Further, administrators have a great deal of influence on the ways in which these funds will be spent. In the same manner as in the direction of programs, funds can be made available for safety improvements or they can be directed to other needs. Safety depends on administrative decision.

Administrators establish the order of priorities. The relative safety of a given highway system will depend to a great degree on the priority given to safety programs and projects.

It is important that the administrator recognize the value in establishing safety as a principal criterion in planning, design, operation and maintenance of highway systems and to utilize staff to the fullest extent to implement safety in all programs related to the highway system.

1.1.32 Planning

Perhaps very few people think in terms of the planner as having a direct or substantial influence on the safety of a highway system. On the contrary, safety may begin or end with the decisions made in the planning process. For example, the planner assumes responsibility for street network layout. We all know that intersection safety is dependent to a great degree on the pattern of the intersection; that is, how the various approach legs come together. Further, we all know that very little can be done to improve the safety of skewed intersections, offsets and other "oddball" situations. Thus, safety influences begin when the first line is laid on the paper; the orientation and relationship of these various lines representing streets determine essentially forevermore the efficiency and safety of operations.

Planners also assume a role of establishing and controlling land use policies. Certainly, land use has a great deal to do with the safety of a particular street or intersection. We may attempt to design a facility whereby land use and access will have a minimal effect on safety and operations. But that which we are able to do depends to a great degree on the policies that control land use and access. Thus, the planner has a very decided effect on safety through his actions with regard to access and land use.

1.1.33 Design

Geometric design is the implementation of the planning process. This is the point at which the layouts envisioned by the planner are given dimension. The requirements of design and ultimately of operation must be incorporated at the planning stage. The designer must employ established criteria that are based on operational requirements. The designer must evaluate trade-offs and make decisions as to which alternatives are the more utilitarian within the constraints imposed by administration policy and the basic plan established by the planner. The safety of the facility depends to a great degree on how well the designer utilizes existing technology and how high a priority he gives to safety criteria.

1.1.34 Construction

The construction of the highway is the implementation of planning and design decisions. It is important that construction carry out the intent of the planning and design process as nearly as possible, particularly in areas that relate to operational efficiency and safety. There should be minimal field changes, and these should be made to effect greater safety and operational efficiency rather than to sacrifice it for the benefit of other aspects.

Many times the construction agency has responsibility for operating traffic concurrently with the reconstruction of a facility. The manner in which detours are designed and operated determines to a great degree the level of safety that will be achieved. It is necessary that the construction agency consult those responsible for traffic operations and then carry out the directives of this group as closely as possible. Too frequently detours are thought of in terms of temporary situations. In extensive construction jobs, these should be viewed more as permanent arrangements as they can last for several years.

1.1.35 Operations

Traffic operations is the implementation of planning and design decisions. In another way, it can be viewed as the regulation of the use of a facility. The driver information and traffic control systems must insure that the safety benefits designed into the facility are achieved. Conversely, traffic control and information systems designed by the operations engineer cannot correct planning and design deficiencies. At best, the effects of a bad condition can only be minimized. To correct these deficiencies requires traffic operations input during the planning and design stages.

1.1.36 Maintenance

Safety should be a primary objective in all maintenance. Changing the basic design should not be undertaken without consulting the designer regarding changes, and obtaining his concurrence. Also, roadside appurtenances which must be maintained frequently due to collision damage should be brought to the attention of appropriate personnel to determine if a redesign would eliminate the maintenance problem and at the same time improve the safety of the system.

1.1.37 Enforcement

Enforcement should have as its basic objective the safety of the motoring public. Enforcing the regulations in accordance with operational intent of the regulation is most important. Also, in accident investigation, the officer's responsibility is to first obtain care for the injured and then to clear the roadway in order to protect the remainder of the motoring public. After this, the accident investigation should be accomplished. Enforcement should encourage motorists to never leave their damaged vehicles in the roadway if they can be moved safely.

Further, enforcement personnel can add greatly to the safety improvements by identifying trouble spots. Problems in interpreting and negotiating difficult roadway geometry frequently are first detected by enforcement. By having a good working relationship between engineering and enforcement, many of these problem areas can be located and corrected before major accidents occur.

1.1.38 Safety is a Cooperative Effort

With all of these different disciplines involved, highway safety is obviously a cooperative effort. Effective leadership must be exercised by management, and cooperation and effective communication must be achieved at all levels of activity. One method of achieving this cooperative effort is through safety operational and design review teams. By proper selection of review team structure, several different disciplines may be represented, and their input may be achieved through a structured and deliberate process.

1.1.4 OPTIMUM VERSUS MINIMUM DESIGN

The AASHTO Select Committee on Highway Safety addressed the subject of optimum versus minimum design in preparing the second edition of the Yellow Book, Highway Design and Operational Practices Related to Highway Safety. Because their comments were comprehensive and concise, they are quoted directly.

"There has been a tendency to base highway design upon minimum standards rather than to adopt an optimum design. While minimum standards may have been adequate for the existing or even the assumed conditions, they may be inadequate if speeds and traffic volumes increase more rapidly than anticipated. As a result, more often than not, a new facility, expected to reach capacity years in the future, has become overloaded and less than satisfactory almost immediately upon being opened to traffic."

"The acceptance of minimum standards as the criteria for design too often occurred for reasons of economy. Frequently, a more liberal design would have cost little more over the life of the project and would increase its safety and usefulness substantially. Although much progress has been noted in the use of optimum design in many areas since 1967, the importance of the message cannot be overemphasized."

"Typical examples of the application of improved design include: sight distances greater than the minimum, flattening side slopes sufficiently to eliminate the need for guardrail, provision of increased right-of-way widths to avoid steep side slopes, and use of drainage pipe in lieu of an open steep-sided ditch. The Handbook of Highway Safety Design and Operating Practices, issued by the Federal Highway Administration in revised form in 1973, describes and illustrates a number of these problems."

"All too often, minimum design standards attempt to rely on the use of warning signs or other added roadside appurtenances for the safe operation that could have been built in by the use of higher standards in the new project at a minimal cost. Often the safety deficiencies generated by minimum design are impossible to correct by any known traffic device or appurtenance. A warning sign is a poor substitute for adequate geometric design."

"Consistency in design standards is desirable on any section of road, because problem locations are generally at the point where minimum design treatment is used. Highways built with high design standards put the traveler in an environment which is fundamentally safer because it is more likely to

compensate for the driving errors he eventually will make."

1.1.5 DEFINITION OF A SAFE ROADWAY

In keeping with the optimum design philosophy, a definition of a safe roadway is offered as food for thought.

"A safe roadway is one in which none of the driver-vehicle-roadway interactions approaches the critical level at any point along its length."

Safety is an attitude -- the attitude of all concerned with highways -- the administrator, designer, builder, the operator and the user.

REFERENCES

1 Accident Facts: 1977. National Safety Council. Chicago, Illinois, 1977.

2. Motor Vehicle Traffic Accidents. Texas Department of Public Safety. 1976.

3. Highway Design and Operational Practices Related to Highway Safety. AASHTO. Second Edition, 1974.

TOPIC 2 SESSION 1
MODEL SAFETY IMPROVEMENT PROGRAMS

Objectives:

With further study of the available documents, the participant should be able to:

1. Organize a safety improvement program to include the technical elements recommended in the presentation,

2. Identify hazardous locations based on analysis of accident data, and non-accident measures,

3. Establish a priority list of locations to be treated,

4. Analyze proposed alternative treatments and

5. Analyze the effectiveness of improvements.

2.1.1 INTRODUCTION

The Spot Safety Improvement Program for the Federal Aid Highway system was outlined in the FHWA Policies and Procedures Memorandum 21-16, dated August 30, 1965. This PPM was superseded by FHPM 6-8-2-1. Later, in conformance with the requirement of the Highway Safety Act of 1966, sixteen standards for education, enforcement, engineering and judicial practices were adopted in an effort to improve highway safety. Two additional standards have since been adopted. Of these 18 standards, 3 and part of another pertained to engineering improvements. These standards provide for identification of problem locations (Standard 9), design improvements (Standard 12), traffic operations improvements (standard 13) and pedestrian facility improvements (standard 14). This session pertains to the implementation of the 3 + standards, with particular emphasis on Standard 9.

For many years, city and state agencies had practiced safety improvement programs under the name of "Traffic Engineering Analysis of Accident Data." Traffic engineers utilized accident spot maps to assist them in visually identifying problem locations, specifically locations with high accident frequencies or high severity of accidents. Also, spot maps were used to identify pedestrian accidents, night accidents, and accidents involving drinking drivers. Spot maps were supplemented by high accident frequency location lists, and some agencies with statistical expertise employed accident rates and other statistical measures to refine the process of identifying and assigning priority to high accident locations.

Methods of analysis included the development of a collision diagram whereby the traffic engineer could identify accident characteristics that may yield suspect causative factors. Collision diagrams were supplemented with condition diagrams which showed the physical features of the accident locations. The traffic engineer could use the collision diagram, condition diagram, field studies, and his experience to identify deficiencies in the design of the facility or control strategy used to regulate and inform traffic on the facility.

These safety improvement programs were generally a part of the routine functions of the traffic engineering department and were carried out by a traffic technician under the direction of the traffic engineer. Guidelines for the conduct of these programs were minimal, generally limited to information available in the Traffic Engineering Handbook and the experience of the traffic engineer.

As indicated previously, the FHWA Policy and Procedures Memorandum 21-16 gave positive identification to the procedures that were to be followed in reducing or eliminating safety hazards at locations on Federal Aid Highway Systems. This memorandum clearly defined the spot safety improvement program as a program completely separate from the regular construction program. Projects under this new program must satisfy two basic criteria: 1) They must be identified as needing improvement based on accident analysis, and 2) They must be expected to produce a measureable reduction in the number and/or severity of accidents.

Procedures outlined in PPM 21-16 (now FHPM 6-8-2-1) include the following:

- Establishment of a field reference system for identifying location of individual occurrences. This might be in the form of reference mileposts, coordinates, etc.

- There must be maintained a traffic records system with the capability to correlate collision data with vehicle driver and highway data, including the capability to correlate accident experience with existing geometric features and traffic characteristics at specified locations. The ultimate objective of the system should be to identify causative factors of highway collisions.

- A procedure for identifying and reporting hazardous elements and locations based on accident analysis.

- A system for ranking proposed safety projects based on the potential for reducing the number and/or severity of accidents to provide for high-yield improvements to be programmed first.

- A regularly scheduled review and revision of rankings or priorities.

- A "before and after" accident evaluation program for analyzing types of hazardous conditions in relation to types of improvements to better define identification, analysis and improvement scheduling techniques.

2.1.2 A MODEL SPOT IMPROVEMENT PROGRAM

The safety improvement program outlined in PPM 21-16 is an excellent safety improvement program, but it is felt that a greater return can be realized, particularly in the urban areas, by establishing a broader spot improvement program. A spot improvement program should be responsive to operational problems in general whether they be congestion or safety. Further, problem locations should not be classified as design or operational problems until the source of the problem is analyzed. Many times it is found that both design and operational deficiencies contribute to the problem. Presented in subsequent paragraphs are steps that constitute a model spot improvement program. This is a six-step program identical in many ways to the program presented in NCHRP Report #162 (1), Methods for Evaluating Highway Safety Improvements, except that a much broader base of measures is suggested in dealing with a broader scope of problems. The steps are identified as follows:

- Identifying Problem Locations
- Selecting Alternative Improvements
- Evaluating Improvement Alternatives
- Programming and Implementing Improvements
- Evaluating Effectiveness of Improvements
- Evaluating the Program

2.1.3 IDENTIFYING PROBLEM LOCATIONS

A systematic method of identifying problem locations is most important. This is the point where the total program is enhanced or limited, based on the objective of the program. A rather broad objective can be achieved by setting up a broadly-based identification procedure. The elements of the identification procedure should include:

2.1.31 Traffic Measures

Speed and Delay Studies. Even limited studies on the arterial system will pinpoint problem locations and will contribute greatly to the formulation of long-range programs.

Volume/Capacity Studies. Most cities maintain a volume count program. If capacity computations are made for the system, and the V/C ratio is determined, an overall evaluation of the system operation can be made.

With computer technology available today, this could be a routine operation.

2.1.32 Field Observations

This is one of the principal methods of identification now in use, because of our natural exposure through driving. The field observation technique can be improved greatly by making aerial observations. A helicopter is better, but a light plane will perhaps suffice if a helicopter is not available.

2.1.33 Diagnostic Team Study

A specialized approach has been used to identify problems over a selected section of an urban system. The method was developed and used in a multi-state sponsored research project entitled, "Diagnostic Studies of Highway Visual Communications." It was used on rural and urban sections of freeways and arterials, and is now being adopted for use in certain agencies. The team study approach involves the forming of a multi-discipline team (design, operations, maintenance, and lay persons) to drive the route in question and comment spontaneously on their difficulty in carrying out assigned driving tasks. Comments are tape-recorded for evaluation later. The team is brought together for a critique; this is also recorded. It is important that the team concentrate on identification of problems rather than seeking alternative solutions.

2.1.34 Citizen Input

Citizens are a constant source of problem identification, but it would be desirable to detect problems by other methods before the citizen feels compelled to call.

2.1.35 Enforcement Input

Patrolmen who drive the system observe many problems. This input should be organized so that it is effective and uniform.

2.1.36 Use of Accident Data

Currently, federal policy requires that identification of hazardous locations be based on analysis of accident experience. Four different analysis techniques commonly are used:

- Number of accidents method (frequency)
- Rate of accidents method
- Number-rate method
- Rate-quality control method

The first two methods are quite simple and readily adaptable to the smaller highway and street systems. The latter two listed above are recommended for larger systems with higher traffic volumes and wider variations of traffic.

Table 2.1.1 shows the basic data requirements for each of the four methods of analysis.

The data described in Table 2.1.1 are sufficient for the purpose of identifying hazardous locations. However, additional information will be needed later for evaluating alternative safety improvements and preparing program information such as:

- Type of accident
- Severity of accident
- Time of day
- Lighting conditions
- Weather conditions
- Type of traffic control

TABLE 2.1.1

ACCIDENT DATA REQUIREMENTS

Basic Data Requirements	Number of Accidents Method	Accident Rate Method	Number-Rate Method	Rate-Quality Control Method
Time period	X	X	X	X
Accident locations	X	X	X	X
Section lengths		X	X	X
Traffic volumes		X	X	X
Average accident rates		X	X	X
Categories of highways			X	X

TABLE 2.1.2

ACCIDENT CRITERIA MEASUREMENT UNITS

Criteria	Number of Accidents Method	Accident Rate Method	Number-Rate Method	Rate-Quality Control Method
Sections:				
Accidents per mile			X	
Accidents per MVM		X	X	X
Intersections and Spots:				
Number of Accidents	X		X	
Accidents per MV		X	X	X

This type of information is included in standard accident reports.

Table 2.1.2 shows which of the criteria measurement units are applicable to each of the alternative methods of analysis.

Number of Accidents Method. This system can be used effectively for small town street systems, local street systems in larger cities and low volume county roads. Consideration of the exposure factor is not as significant as on systems with higher traffic volumes or wider ranges of traffic volumes.

This is the simplest and most direct approach. All accidents are recorded by location and by the time period during which they occurred (usually months). Use of an accident spot map has proven to be one of the best ways to document the information.

The simplicity of this approach is justified because of low traffic volumes. There will not be many accidents, and few clusters of accidents will be found. Where clusters do appear, there will be an objective basis for investigation to determine if some element of roadway facility may be contributing to the accidents.

Accident rate method. Analysis by number of accidents alone can result in misleading conclusions when there is considerable variation in traffic volumes throughout the road or street system. Two locations having the same number of accidents should not reflect the same degree of hazard potential if one carries twice as much traffic as the other. The accident rate method considers this variable.

In addition to the basic information on accidents and their locations, we must also know the traffic volumes at all locations--and we must be able to compute systemwide accident rates for comparison with specific locations.

The accident-rate method involves the steps described below. With relatively small systems, the processes and calculations can be performed manually. With larger systems a computer should be used for calculations and processing of data.

1. Locate all accidents in accordance with accepted coding practices.

2. Identify number of accidents in each established section and at individual intersections and spots.

3. Calculate the actual accident rate for each established section during the study period.

$$\text{Rate/MVM} = \frac{(\text{no. of accidents on section})(10^6)}{(\text{ADT})(\text{no. of days})(\text{section length})}$$

4. Calculate the actual accident rate for each intersection or spot during the study period.

$$\text{Rate/MV} = \frac{(\text{no. of accidents at intersection or spot})(10^6)}{(\text{ADT at location})(\text{no. of days})}$$

5. For the same period, calculate the systemwide average accident rates for sections, intersections and spots--using the formulas above and the summation of total accidents, total vehicle miles and total vehicles, respectively, for each category of location.

6. Select accident rate cut-off values as criteria for identifying hazardous locations. A value about twice the systemwide rate usually is realistic and practical.

7. If actual rates exceed the minimum established criteria, the location is identified as hazardous and placed on the list for investigation and analysis.

Selection of the cut-off value (step 6) is not as critical as it might appear. The principal purpose is to control the size of the list of locations to be investigated--a shorter list with high values, a longer list with low values. Experience will disclose the proper level for a particular agency.

The accident rate method is more complex than the accident numbers method--and usually gives better results. But compromises are made in detail of specific and overall statistical reliability. Some of these limitations are overcome by the rate quality control method and the number-rate method. Most agencies with large complex systems should adopt one of these latter two methods.

Number-Rate Method. The number-rate method is applicable to all highway or street systems--regardless of size of system or variations in traffic volumes.

A location with relatively high numbers of accidents per mile may appear to be quite hazardous. But if the traffic volume is exceptionally high at the location, the accident rate may not be abnormal--and the situation may not be as bad as it appears.

On the other hand, a location with relatively few accidents may show a very high accident rate because of extremely low traffic volumes. And again, the situation may not be as abnormal as it appears.

If both the number of accidents and accident rate at a location greatly exceed the average, we can be reasonably sure that the

accident record is abnormal--and that conditions should be examined. The number-rate method is based on this concept. Additionally, this method considers variables related to categories of highways and types of intersections--categories differentiating between rural and urban locations, number of lanes, divided or undivided and access control.

The number-rate method involves the following steps in addition to the basic recording of accidents and their locations:

1. For sections of highway, compute average accidents per mile for each category of highway--based on total data for all sections of each category.

Av. accidents per mile =

$$\frac{\Sigma(\text{number of accidents})}{\Sigma(\text{miles of category})}$$

Av. accidents per MVM =

$$\frac{\Sigma(\text{number of accidents})(10^6)}{\Sigma(\text{section ADT})(\text{no. of days})(\text{section length})}$$

2. Identify all clusters of accidents (2 or more within 0.10 mile) at spots and intersections, and compute average accidents per location and per million vehicles for each category of highway.

Av. accidents per location =

$$\frac{\text{total number of accidents}}{\text{total number of locations}}$$

Av. accidents per MV =

$$\frac{(\text{total number of accidents})(10^6)}{\Sigma(\text{location ADT})(\text{no. of days})}$$

3. Select cut-off values for each of the criteria above--start with values about twice the systemwide average for each highway category.

4. For each section, calculate both the actual number of accidents per mile and per vehicle mile.

5. For each cluster of accidents (spot or intersection) calculate both the number of accidents and the accidents per million vehicles passing the location.

6. All locations with numbers of accidents and accident rates both higher than the critical cut-off values should be placed on the hazardous location list. Comparisons must be made with criteria for the particular category of highway being analyzed.

Rate Quality Control Method. The rate quality control method is applicable to systems of all sizes and ranges of traffic volumes. As with the number-rate method, consideration is made of various categories of highway--rural, urban, 2-lane, 4-lane, etc. But the rate quality control method assures control of the quality of the analyses by applying a statistical test to determine whether a particular accident rate is unusual, as related to a predetermined average accident rate for locations having similar characteristics. The tests applied are based on the commonly accepted assumptions that accidents fit the Poisson distribution.

The critical rate is determined statistically as a function of the systemwide average accident rate for the category of highway and the vehicle exposure (vehicles or vehicle miles) at the location being studied.

Critical rates are computed by the following formula:

$$Rc = Ra + K\sqrt{\frac{Ra}{m}} - \frac{0.5}{m}$$

Where: Rc = Critical accident rate
(For sections-accidents per MVM)
(For intersections or spots-accidents per MV)
Ra = Systemwide average accident rate by highway category
(For sections-accidents per MVM)
(For intersections or spots-accidents per MV)
m = Vehicle exposure during study period (MV or MVM)
K = Constant

The value of K determines the level of confidence that accident rates above the critical rate are significant and have not resulted by chance. A 95% level of confidence is desirable. Example values of K for various levels of confidence are shown below.

Level of Confidence	K
0.995	2.576
0.95	1.645
0.90	1.282

The rate quality control method involves the following steps in addition to the basic recording of accidents and their locations.

1. Compute systemwide average number of accidents per MVM for each category of highway--based on total data for all sections of each category.

Av. accidents per MVM =

$$\frac{(\text{no. of accidents})(10^6)}{(\text{section ADT})(\text{no. of days})(\text{section length})}$$

2. Identify all clusters of accidents (2 or more within 0.1 mile) at spots and intersections, and compute systemwide average accidents per MV at such locations by categories of highways.

Av. accidents per MV =

$$\frac{(\text{no. of cluster accidents})(10^6)}{(\text{ADT at clusters})(\text{no. of days})}$$

3. For each individual location, determine the vehicle exposure, m, during the study period.

For sections:

m =

$$\frac{(\text{section ADT})(\text{no. of days})(\text{section length})}{(10^6)} = \text{MVM}$$

For intersections and spots:

$$m = \frac{(\text{location ADT})(\text{no. of days})}{(10^6)} = \text{MV}$$

4. For each location, compute the critical accident rate, Rc, by the formula:

$$Rc = Ra + K\sqrt{\frac{Ra}{m}} - \frac{.05}{m}$$

Where: Ra = Average accident rate for category of highway being studied (MVM for sections--MV for intersections and spots)
m = Vehicle exposure at location (MVM for sections--MV for intersections and spots)
K = Constant for probability level

Start with a value of K = 1.5. A larger value of K will reduce the length of the hazardous location listing--but will increase the level of confidence that the locations truly are hazardous. A smaller value of K will produce a longer list with a lower level of confidence.

5. Compute the actual observed accident rate at each location for the same time period.

For sections:

Accidents per MVM =

$$\frac{\text{no. of accidents}}{\text{millions of vehicle miles}}$$

For intersections and spots:

Accidents per MV =

$$\frac{\text{no. of accidents}}{\text{millions of vehicles}}$$

6. Compare the actual accident rate with the critical rate at each location and prepare a list of all locations (sectors, intersections and spots) with rates exceeding the critical value.

An Example Application. The Georgia DOT has developed a computer record analysis procedure that establishes priorities for spot and section improvements. The major features of the analysis procedure are outlined below.

- Computation of accident rates. The accident file is linked to the state volume file so that rates may be determined automatically.

- Computation of accident severity. To provide a measure of accident severity at a given location or section, Georgia has assigned an empirical weighting system to Fatal, Personal Injury (class A, B, and C), and Property Damage accidents, as follows:

I_F	- Fatal accident	- 5
I_A	- Personal injury accident, Class A	- 4
I_B	- Personal injury accident, Class B	- 3
I_C	- Personal injury accident, Class C	- 2
P_D	- Property damage only	- 1

Therefore, the average severity becomes:

S =

$$(I_F \times 5 + I_A \times 4 + I_B \times 3 + I_C \times 2 + P_D)/F$$

- Significance of accident statistics. Once the data for a location have been determined, the question is, are the larger statistics, frequency, rate, and severity, due to chance or a real difference in conditions? This question is resolved by determining the critical level of each of the statistics as discussed earlier.

Critical frequency =

$$F_c = F_a + K\sqrt{\frac{F_a}{m}} - \frac{1}{2m}$$

Where: F_c = Critical frequency
F_a = Average frequency for all roadways of same class for state or area
K = Constant for probability level
m = Average exposure of traffic during study

Similarly, critical rate and critical severity also can be computed.

Critical rate =

$$R_c = R_a + K\sqrt{\frac{R_a}{m}} - \frac{1}{2m}$$

Where r_a = average rate for all roadways of same class for state or area

Critical severity =

$$S_c = S_a + K\sqrt{\frac{S_a}{m}} - \frac{1}{2m}$$

Where S_a = average severity for all roadways of same class for state or area.

Safety Indices. A comparison is then made of actual statistics with critical statistics to determine if the location is to go on the priority listing. This is accomplished by computing a Safety Index for each of the statistics.

Frequency Index:

$$FI = \frac{F}{Fc}$$

Rate Index:

$$RI = \frac{R}{R_c}$$

Severity Index:

$$SI = \frac{S}{S_c}$$

If none of the three indices is greater than 1.0, this would indicate that the safety problem at the location is not critical and, therefore, it would be dropped from further consideration.

Final Ranking of Accident Locations For Priority Purposes. The spot and section locations are ranked on the basis of each of the indices' being weighted empirically as follows:

Index Points =

FI(0.70) + RI(0.50) + SI(2.0)

Therefore, to summarize, the Georgia method includes the following steps:

- Calculate accident rates based on frequency and severity
- Compute critical values for frequency, rate and severity
- Compute safety indices by comparing actual to average frequency, rate and severity
- Rank the locations according to index points using an empirical weighting scheme

2.1.37 The Hazardousness Rating Formula

In research conducted by Taylor and Thompson (2), a Hazardousness Rating formula was developed which incorporates both accident and non-accident measures or predictors. This formula is intended to be a supplement rather than an alternative to accident record systems in the identification and ranking of problem locations. The following paragraphs briefly describe the procedure developed by Taylor and Thompson.

Indicators Used in the Formula. From the initial list of indicators, the following were selected for inclusion in the final Hazardousness Index:

- Accident Measures
  - + Accident Rates
  - + Accident Numbers
  - + Accident Severity
- Non-Accident Measures
  - + Traffic Conflicts
  - + Erratic Maneuvers
  - + Sight Distance Ratio
  - + Volume/Capacity Ratio
  - + Driver Expectancy
  - + Information System Deficiencies

Scaling and Weighting Factors. Having selected these nine indicators, Taylor and Thompson established a scaling system for transforming the raw data for each indicator into a form which would be compatible with other indicators in the final Hazardousness Index. A scale of 0 to 100 was used for the Indicator Value where 33 designates the separation of "normal" and "hazardous," and 67 designates the separation of "hazardous" and "very hazardous." As a next step, four control values were established for each indicator which would designate limits for each level of hazardousness. For example, the indicator values for number of accidents is shown in Figure 2.1.1. Similar relationships between raw data and indicator values were also developed for the other eight indicators.

Once the scaling factors used to reduce the raw data were identified, the next step was to determine how much weight each of the indicators should carry in the final Hazardousness Index formula. Based on workshops attended by various highway officials from around the country, the following weighting factors were determined:

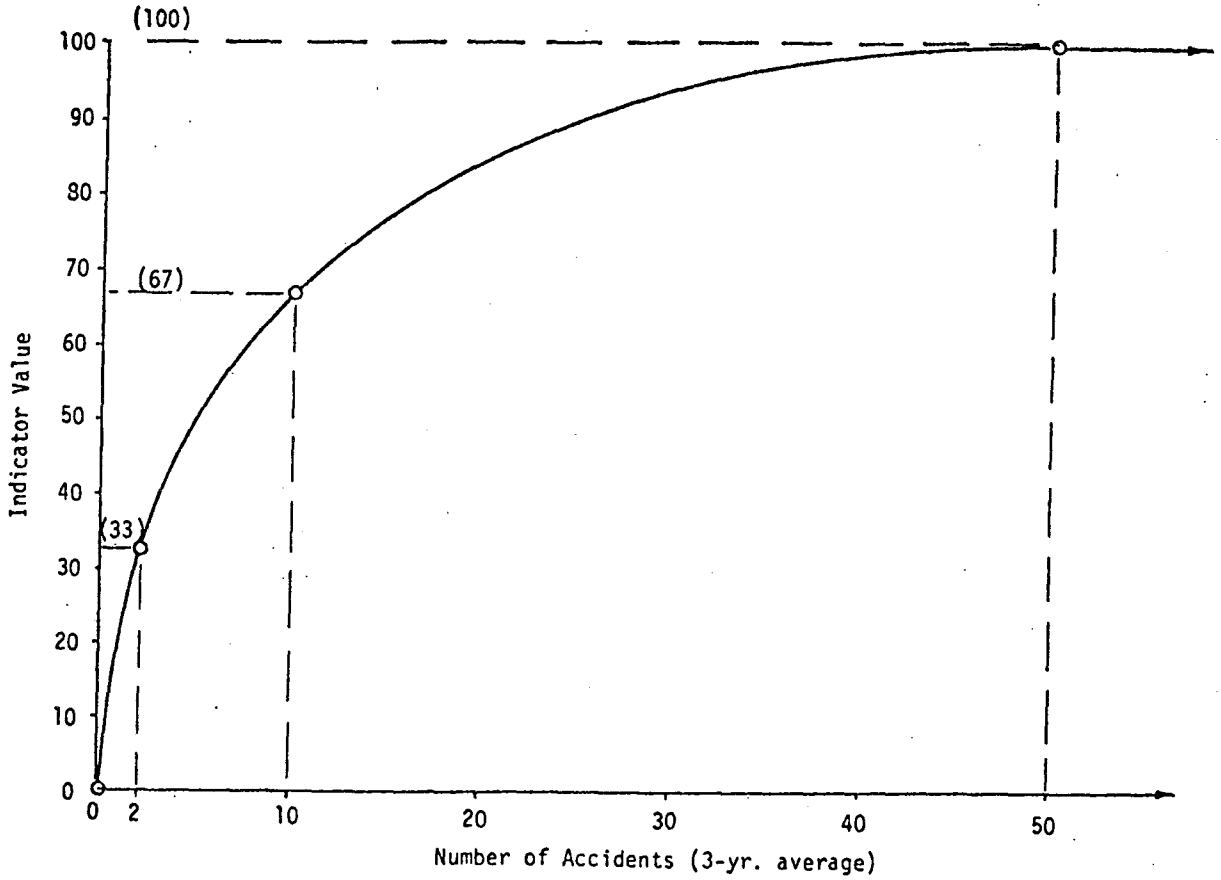

Figure 2.1.1 Indicator Values for Number of Accidents

Indicator	Final Weight
Number of Accidents	14.5
Accident Rate	19.9
Accident Severity	16.9
Volume/Capacity Ratio	7.3
Sight Distance	6.6
Traffic Conflicts	5.3
Erratic Manuevers	6.1
Driver Expectancy	13.2
Information System Deficiencies	10.2
	100.0

Determination of the Hazardousness Rating Formula. Using the weighting factors from above and applying them to the scaling factors related to the raw data, the Hazardousness Index is used in the following general form:

$$H.I. = \frac{\Sigma[W_i \ (I.V.)_i]}{\Sigma \ W_i}$$

where:

H.I. is the Hazardous Index for the site under study

W_i is the weighting factor for indicator i

$(I.V.)_i$ is the Indicator Value for indicator i

ΣW_i is the sum of the weighting factors for all the indicators used at the site under study

The form used for the actual calculation of the Hazardousness Index is shown in Figure 2.1.2. Note that the weights for indicators not used or for which data are not available, are not included in the calculations.

Application of the Hazardousness Index. It is not practical to collect all the indicator data for all spot locations within a particular jurisdiction. Some of the indicators require extensive data collection while others require at least a visit to the site. Therefore, it is not feasible to utilize the Hazardousness Index as a screening process. Rather, its value lies in comparing the relative hazardousness of various sites already under consideration. Taylor and Thompson suggest the following procedure for identifying hazardous locations:

1. Select the top "20" sites (arbitrary number, but perhaps twice the number for which treatment funds are likely to be

DETERMINATION OF HAZARDOUSNESS INDEX

Site Number_____ Date_____

Type_____

Indicator	Data Value	Indicator Value		Height		Partial H.I.'s
Number of Accidents	____acc/yr	____	x	0.145	=	____
Accident Rate	____acc/MEV	____	x	0.199	=	____
Accident Severity	____dollars	____	x	0.169	=	____
Volume/Capacity Ratio	____	____	x	0.073	=	____
Sight Distance Ratio	____(wt.avg.)	____	x	0.066	=	____
Traffic Conflict	____conf/hr.	____	x	0.053	=	____
Erratic Maneuvers	____e.m./hr.	____	x	0.061	=	____
Driver Expectancy	____(wt.avg.)	____	x	0.132	=	____
Info. System Deficiencies	____(wt.avg.)	____	x	0.102	=	____
Sums:				____*		____

H.I. = $\frac{\text{Sum of Partial H.I.'s}}{\text{Sum of Applicable Weights}}$ = ____ = ____

Relative Strength of Evaluation:

Sum of Applicable Weights x 100 = ____%

*Do not include weights for indicators not used at this site.

Figure 2.1.2 Form for Determination of the Hazardous Index

available) on the basis of the accident records system alone. (It is suggested that this screening process be accomplished by developing a computer program and format to provide partial Hazardousness Indices on the basis of the first three terms of the HRF.)

2. Add "5" sites for which a number of citizen complaints have been registered.

3. Add "5" sites which the safety officials know to be hazardous even though a few or no accidents have occurred (perhaps by chance, because of new construction or a major change in operational characteristics, etc.).

4. Collect the non-accident indicator data for these 30 sites.

5. Compute the relative hazardousness of the 30 sites on the basis of the comprehensive Hazardousness Rating Formula.

2.1.38 Sensitivity of Analysis Methods

Several methods of identifying high hazard locations are available for implementation, and the method chosen by an agency must be suited to the qualifications and needs of the agency. Certainly each agency should strive to use the method of identification that is most effective for the given application. The agency should periodically evaluate the effectiveness of method employed to insure that the method is, in fact, identifying those locations that constitute the greatest safety hazards. The sensitivity of the method used is very important and should be checked periodically.

2.1.4 SELECTING ALTERNATIVE IMPROVEMENTS

The purpose of this section is to set forth some orderly systematic procedures for investigating the hazardous locations and for identifying possible solutions.

The Manual of Traffic Engineering Studies, published by the Institute of Transportation Engineers, sets forth a basic approach for analyzing hazardous locations. This involves four basic steps:

- Prepare collision diagrams
- Summarize accident characteristics
- Conduct field observations
- Prescribe improvements

Suggested procedures to be followed are described in the following sections.

2.1.41 Prepare Collision Diagrams

Most accident reporting systems provide for fairly comprehensive information on accident occurrence and related conditions--information such as:

- Location description
- General geometric layout of location
- Time and date of each accident
- Severity of accident
- Pavement condition for each accident
- Weather at each accident
- Intended paths of vehicles involved
- Intersection control devices

Graphical presentation of such information usually is more effective than statistical summaries. A simple collision diagram will facilitate analyses--particularly at intersections. Strip maps or multi-page maps are necessary to illustrate accidents on long sections of highway.

2.1.42 Summarize Accident Characteristics

By summarizing factual data related to accidents, it frequently is possible to

identify relationships between accidents and possible contributing factors. When data for all accidents at a particular location consistently show common characteristics of time of day, weather, type of accident, or other conditions, some of the mystery is removed, and these indicators can be used to point the direction of further investigations.

The importance of good factual reporting is obvious. Unreliable summarized data can lead to erroneous conclusions. But reliable data, summarized in meaningful ways, can be one of the most valuable tools for evaluating hazardous locations.

2.1.43 Field Observations

It may be necessary to observe hazardous locations in the field if causes of accidents cannot be determined from diagram and statistical summaries. Observations also may be needed to verify assumptions made in the office.

Procedures for field observation normally should include the following steps:

1. Review of existing available data--accident reports, accident summaries, diagrams, traffic counts, violation summaries and other operational data.

2. Schedule the observations according to apparent significant characteristics--nighttime, wet pavement, etc.

3. Select several good vantage points and observe drivers to identify unusual behavior and, if possible, determine the cause of the behavior.

4. Drive through the location several times from different directions--with particular attention to how the driver might see the environment.

5. Inventory unique features not included in existing records.

6. Discuss the situation with persons residing in or working near the locations --for the personal insight.

7. Document the findings and conclusions.

The ITE Manual of Traffic Studies lists eleven questions that the analyst should consider during a field observation:

1. Are the accidents caused by physical conditions of the road or adjacent property, and can the conditions be eliminated or corrected?

2. Is a blind corner responsible? Can it be eliminated? If not, can adequate measures be taken to warn the motorists?

3. Are the existing signs, signals, and pavement markings doing the job for which they were intended? Is it possible they are, in any way, contributing causes of accidents rather than contributions to accident prevention?

4. Is traffic properly channelized to minimize the occurrence of accidents?

5. Would accidents be prevented by the prohibition of any single traffic movement, such as a minor left-turn movement?

6. Can part of the traffic be diverted to other thoroughfares where the accident potentialities are not as great?

7. Are night accidents far out of proportion to daytime accidents, based on traffic volume, indicating need for special nighttime protection, such as street lighting, signal control or reflectorized signs or markings?

8. Do conditions show that additional traffic laws or selective enforcement are required?

9. Is there a need for supplemental studies of traffic movement, such as driver observance of existing control devices, speed studies of vehicles approaching the accident location and others?

10. Is parking in the area contributing to accidents? If so, perhaps reduction of the width of approach lanes or sight obstructions in advance of the intersection resulting from the parking are causing the accidents?

11. Are there adequate advance warning signs of route changes so that the proper lanes may be chosen by approaching motorists well in advance of the area, thus minimizing the need for lane changing near the accident location?

2.1.44 Prescribe Improvements

After identifying hazardous locations and the related circumstances that probably have contributed to accidents, the next step obviously is to determine what action would be most effective to correct, or at least to improve the situation.

Unfortunately, this decision frequently is not as clear cut as it might appear. Often we jump to an apparent obvious conclusion without considering all alternatives--or the ultimate effect that a particular action might precipitate. Someday we may be able to feed a computer with data on all circumstances and conditions and receive back a 100 percent foolproof solution. Until that time comes, however, there is no substitute for careful, comprehensive and logical analysis by an experienced person.

TABLE 2.1.3

EXAMPLE CHECKLIST OF POTENTIAL IMPROVEMENTS

SECTIONS

- Eliminate parking
- Install delineators
- Add guardrail-embankments
- Add guardrail-fixed objects
- Remove fixed objects
- Flatten fill slopes
- Add painted or raised median
- Deslicking
- Resurfacing
- Widen traveled way
- Reconstruction
- Install or improve edge marking
- Install or improve warning and/or directional signs
- Install median barrier
- Breakaway sign and light standards
- Install lighting
- Shoulder stabilization
- Widen shoulders
- Eliminate median crossovers
- Add climbing lanes

CURVES

- Install delineators
- Add guardrail
- Resurfacing
- Install warning signs
- Reconstruct curve

BRIDGE/UNDERPASS

- Install delineators
- Install lighting
- Energy absorption devices
- Add guardrails
- Bridge widening

INTERSECTIONS

- Install or improve warning and/or directional signs
- Install minor leg stop control
- Install lighting
- Install pedestrian signals
- Improve signals
- Install new signals
- Install warning signals
- Install stop ahead signs
- Install yield sign
- Install all-way stop signs
- Install warning signals
- Curtail left-turn movements
- Provide for left-turn movements
- Deslicking
- Install rumble strips

There are several sources of information that will assist in the selection of improvement alternatives. NCHRP Report No. 162, "Methods for Evaluating Highway Safety Improvements," provides accident reduction forecasts for various types of improvements. Table 2.1.3 presents an example checklist of potential improvements. Table 2.1.4 lists a number of general countermeasures associated with given accident patterns.

Several important items should be kept in mind during the process of selecting appropriate improvements:

1. Identify all practical improvements--everything from a do-nothing alternative to an ultimate alternative such as complete reconstruction. We are not making a final decision. The principal objective is to make certain we do not overlook an alternative that may be the most practical and economically-advisable solution.

2. Identify all practical combinations of improvements.

3. For each alternative, identify the potential effect of the improvement--the number of accidents, the types of accidents, and the severity of the accidents.

Documentation. There needs to be complete documentation of data and logic leading to prescription of applicable improvements. When the time comes to evaluate the results of implemented improvements, the analyst will need to know the background and considerations that led to the recommendations--questions related to:

- Problem Identification. What method was used to identify the problem at the hazardous location, and how was the problem defined?

- Accident Characteristics. What accident data were available and how were they utilized?

- Selection of Applicable Improvements. Which improvements or combinations of improvements were considered? Which were not considered applicable and why?

- Location of Peculiarities. Are there any peculiarities about the hazardous location that may cause the improvements to produce non-typical results?

2.1.5 EVALUATING ALTERNATIVE IMPROVEMENTS

The objective in this step is to determine which of the several alternatives will provide the greatest return for the resources expended. Generally, an evaluation pertains to the consideration of several alternatives at a single location. Care should be exercised, however, in evaluating the total effects of alternatives, particularly where an alternative will result in the re-routing of traffic or some other major change in the operational pattern. Where such major changes may be experienced, the evaluation should consider possible increase in accidents at other locations due to these changes.

Evaluations are based principally on economic analyses and will involve the following six steps:

1. Estimating accident reduction
2. Assigning values to accident reduction
3. Estimating secondary benefits
4. Estimating improvement costs
5. Analyzing improvement at each location
6. Assigning program priorities

It is generally agreed that an analysis identifying annual benefits and annual costs is acceptable for safety improvement evaluations.

TABLE 2.1.4

GENERAL COUNTERMEASURES FOR ACCIDENT PATTERNS AND THEIR PROBABLE CAUSES

ACCIDENT PATTERN	PROBABLE CAUSE	GENERAL COUNTERMEASURE
Right-angle collisions at unsignalized intersections	Restricted sight distance	Remove sight obstructions Restrict parking near corners Install stop signs (see MUTCD) Install warning signs (see MUTCD) Install/improve street lighting Reduce speed limit on approaches* Install signals (see MUTCD) Install yield signs (see MUTCD) Channelize intersection
	Large total intersection volume	Install signals (see MUTCD) Reroute through traffic
	High approach speed	Reduce speed limit on approaches* Install rumble strips
Right-angle collisions at signalized intersections	Poor visibility of signals	Install advanced warning devices (see MUTCD) Install 12-in. signal lenses (see MUTCD) Install overhead signals Install visors Install back plates
		Improve location of signal heads Add additional signal heads Reduce speed limit on approaches*
	Inadequate signal timing	Adjust amber phase Provide all-red clearance phases Add multi-dial controller Install signal actuation Retime signals Provide progression through a set of signalized intersections
Rear-end collisions at unsignalized intersections	Pedestrian crossing	Install/improve signing or marking of pedestrian crosswalks Relocate crosswalk
	Driver not aware of intersection	Install/improve warning signs
	Slippery surface	Overlay pavement Provide adequate drainage Groove pavement Reduce speed limit on approaches* Provide "SLIPPERY WHEN WET" signs
	Large numbers of turning vehicles	Create left- or right-turn lanes Prohibit turns Increase curb radii

* Spot speed study should be conducted to justify speed limit reduction.

TABLE 2.1.4 (Cont'd)

ACCIDENT PATTERN	PROBABLE CAUSE	GENERAL COUNTERMEASURE
Rear-end collisions at signalized intersections	Poor visibility of signals	Install/improve advance warning devices Install overhead signals Install 12 in. signal lenses (see MUTCD) Install visors Install back plates Relocate signals Add additional signal heads Remove obstacles Reduce speed limits on approaches*
	Inadequate signal timing	Adjust amber phase Provide progression through a set of signalized intersections
	Pedestrian crossings	Install/improve signing or marking of pedestrian crosswalks Provide pedestrian "WALK" phase
	Slippery surface	Overlay pavement Provide adequate drainage Groove pavement Reduce speed limit on approaches* Provide "SLIPPERY WHEN WET" signs
	Unwarranted signals	Remove signals (see MUTCD)
Pedestrian accidents at intersections	Large turning volumes	Create left- or right-turn lanes Prohibit turns Increase curb radii
	Restricted sight distance	Remove sight obstructions Install pedestrian crossings Improve/install pedestrian crossing signs Reroute pedestrian paths
	Inadequate protection for pedestrians	Add pedestrian refuge islands
	Inadequate signals	Install pedestrian signals (see MUTCD)
	Inadequate signal phasing	Add pedestrian "WALK" phase Change timing of pedestrian phase
	School crossing area	Use school crossing guards

* Spot speed study should be conducted to justify speed limit reduction.

TABLE 2.1.4 (Cont'd)

ACCIDENT PATTERN	PROBABLE CAUSE	GENERAL COUNTERMEASURE
Pedestrian accidents between intersections	Driver has inadequate warning of frequent mid-block crossings	Prohibit parking Install warning signs Lower speed limit* Install pedestrian barriers
	Pedestrians walking on roadway	Install sidewalks
	Long distance to nearest crosswalk	Install pedestrian crosswalk Install pedestrian actuated signals (see MUTCD)
Pedestrian accidents at driveway crossings	Sidewalk too close to traveled way	Move sidewalk laterally away from highway
Left-turn collisions at intersections	Large volume of left turns	Provide left-turn signal phases Prohibit left turns Reroute left-turn traffic Channelize intersection Install STOP signs (see MUTCD) Create one-way streets Provide turning guidelines (if there is a dual left-turn lane)
	Restricted sight distance	Remove obstacles Install warning signs Reduce speed limit on approaches*
Right-turn collisions at intersections	Short turning radii	Increase curb radii
Fixed-object collisions	Objects near traveled way	Remove obstacles near roadway Install barrier curbing Install breakaway feature to light poles, signposts, etc. Protect objects with guardrail
Fixed-object collisions and/or vehicles running off roadway	Slippery pavement	Overlay existing pavement Provide adequate drainage Groove existing pavement Reduce speed limit* Provide "SLIPPERY WHEN WET" signs
	Roadway design inadequate for traffic conditions	Widen lanes Relocate islands Close curb lane
	Poor delineation	Improve/install pavement markings Install roadside delineators Install advance warning signs (e.g., curves)

* Spot speed study should be conducted to justify speed limit reductions.

TABLE 2.1.4 (Cont'd)

ACCIDENT PATTERN	PROBABLE CAUSE	GENERAL COUNTERMEASURE
Sideswipe collisions between vehicles traveling in opposite directions or head-on collisions	Roadway design inadequate for traffic conditions	Install/improve pavement markings Channelize intersections Create one-way streets Remove constrictions such as parked vehicles Install median divider Widen lanes
Collisions between vehicles traveling in same direction such as sideswipe, turning or lane changing	Roadway design inadequate for traffic conditions	Widen lanes Channelize intersections Provide turning bays Install advance route or street signs Install/improve pavement lane lines Remove parking Reduce speed limit*
Collisions at driveways	Left-turning vehicles	Install median divider Install two-way left-turn lanes
	Improperly located driveway	Regulate minimum spacing of driveways Regulate mimumum corner clearance Move driveway to side street Install curbing to define driveway location Consolidate adjacent driveways
	Right-turning vehicles	Provide right-turn lanes Restrict parking near driveways Increase the width of the driveway Widen through lanes Increase curb radii
	Large volume of through traffic	Move driveway to side street Construct a local service road Reroute through traffic
	Large volume of driveway traffic	Signalize driveway Provide acceleration and deceleration lanes Channelize driveway
	Restricted sight distance	Remove sight obstructions Restrict parking near driveway Install/improve street lighting Reduce speed limit*
Night accidents	Poor visibility	Install/improve street lighting Install/improve delineation markings Install/improve warning signs.
Wet pavement accidents	Slippery pavement	Overlay existing pavement Provide adequate drainage Groove existing pavement Reduce speed limit* Provide "SLIPPERY WHEN WET" signs

* Spot speed study should be conducted to justify speed limit reduction.

The assumptions behind this approach are:

- The relative merit of an improvement is measured by its net annual benefit or benefit/cost ratio.
- All costs can be reduced to an equivalent uniform annual cost.
- All benefits can be reduced to an equivalent uniform annual benefit
- An improvement will be needed for its entire service life.

Information needed for analyses includes:

- Initial costs
- Annual costs
- Terminal values
- Service life
- Benefits
- Interest rate

Estimating Accident Reduction. The premise for proposing an improvement at a hazardous location is that there will be benefits resulting from accident reduction. The justification for any improvement, and its priority, is based on the ratio of the benefits and the costs of implementing the improvement. Therefore, the prediction of accident reduction becomes very critical in the process of evaluating improvements.

Benefits eventually will be identified in dollar amounts related to reductions in fatalities, personal injuries and property damage--but initially the yardstick will be reduction in accident rate with consideration of types of accidents and their severities. Required input data for each location will include:

- Historical accident experience--accident rate, types of accidents and severity.
- Estimated future ADT--the growth or decline of traffic volumes.
- Expected reduction in accident rate--by type of accident or by severity.

Accident reduction forecast tables in NCHRP 162 provide reasonable indicators (based on limited experience data) of potential accident reduction following implementaton of particular improvements. The organization of tables identifies several possible combinations of conditions:

- Type of location (sections, curves, intersections, etc.)
- Type of improvement
- Urban and rural
- Two lanes or more than two lanes

Expected accident reduction for any future year is calculated as:

Accidents Saved =

$$N \cdot P \frac{\text{ADT - future year}}{\text{ADT - record period}}$$

Where: N = the number of accidents in the period before the improvement project
P = the percent reduction selected from the table (expressed as a decimal).

Assigning Values to Accident Reduction. Two methods commonly are used for assigning economic values to accident reductions:

1. Costing by severity class, or
2. Costing by type of accident

Motorists pay for accidents in one or more of the following ways:

- Direct settlement paid to other persons
- Payment as a result of judicial proceedings
- Medical and property damage repair costs
- Automobile insurance premiums

Basic Accident Cost Data. The most reliable data on accident costs would be those which have been collected locally. Information from the Motor Vehicle Administration, local insurance companies, fleet operators and the Public Health Service, will be more suitable than nationwide statistics. Two of the more commonly used nationwide studies are summarized below:

Severity Class	Cost Per Accident	
	NHTSA	NSC
Fatal	$287,000	$113,500
Non-Fatal Injury (Avg)	$ 3,200	$ 6,200
Property Damage Only	$ 520	$ 570

While NHTSA did not attempt to place a value on human life, it did include calculable costs associated with the loss of human life--

wages lost, medical expenses, legal fees, insurance payments, home and family care and property damage. About eight percent of total costs were assigned to "pain and suffering."

Accident cost data from the source above reflects one philosophy as to what cost elements are included. Basic cost data adopted by any agency must reflect concepts and judgments acceptable to that agency. Top management should be involved in these decisions.

Benefit Computations--Based on Reduced Accident Severity. When calculating accident reduction benefits on the basis of severity of accidents, the following steps are followed:

1. Select or develop average cost data for each of several classes of severity--i.e., fatalities, one or more classes of injuries, and property-damage-only accidents.

2. Compute the expected accident reduction (numbers of accidents) by each severity class, for each year of the service life of the improvement.

3. Multiply the average costs for each severity class by accident reduction numbers for each year.

4. Compute the total of all classes for each year and calculate the total annual benefits.

Benefit Computations--By Types of Accidents. When benefits are computed on the basis of the types of accidents, the procedures are:

1. Select or develop average cost data by accident severity classes.

2. Establish categories of types of accidents (head-on, side swipe, left turn, etc.) and determine the frequency of each severity class for each accident type.

3. Compute the average cost for each accident type:

Average accident cost =

$$\frac{(Ff)(Cf) + (Fi)(Ci) + (Fp)(Cp)}{Ff + Fi + Fp}$$

Where: Ff = Number of fatal accidents for this accident type
Fi = Number of injury accidents for this accident type
Fp = Number of property damage accidents for this accident type
Cf = Average cost per fatal accident
Ci = Average cost per injury accident
Cp = Average cost per property damage accident

4. Multiply the average accident cost for each accident type by the reduction of each accident type and sum all types to obtain a total dollar value for each year.

Either of the above techniques is acceptable. Costing by type of accident reduces the influence of the rare event, a fatal accident, yet reflects its importance through the types of collision. If it is difficult to obtain data relating accident severity to types of accidents, costing by severity class may be more practical.

Estimating Secondary Benefits. The primary benefit to be expected from the implementation of an accident reduction improvement is a decrease in accident rate or severity, and the benefit analysis should focus on these factors. However, the possibility should not be overlooked that a safety improvement also may affect other road user and non-road user benefits. For example, a signal installation may reduce certain types of accidents while simultaneously increasing motorist delay; signal progression may reduce rear end collisions and lower auto emission levels; and street lighting has been shown to have a beneficial effect on both nighttime accidents and street crime.

Examples of secondary benefits might include:

- Reduced traffic congestion--which will not only decrease idling time and cost for vehicles but also reduce motorist delay.
- Improved roadway and roadside geometrics--which can minimize wear to vehicle components and also reduce fuel consumption.
- Higher speed of operation from realignment of a series of sharp horizontal curves.
- Smoother operation from implementation of a one-way street system with signal progression.
- Reduction in the need for vehicular "slow-downs" by improving the sight distance on the approach to a YIELD-controlled intersection.
- Reduction of the time and mileage for lost motorists by improving guide signing at an interchange.
- Elimination of motorist delay by prohibiting left-turns at selected locations.
- Reduction in street crime brought about by improved roadway lighting.
- More effective use of enforcement and other protective service personnel brought about by fewer accident-related duties.

Often these benefits will be negligible compared to the accident reduction benefits. But under some circumstances the secondary benefits will be significant and should be included in the analyses.

Estimating Improvement Costs -- The expected service life of each improvement and the cost of implementation are basic input for analysis.

Estimated Service Life -- The service life is the period of time that the improvement can reasonably be expected to affect accident rates. Twenty years usually is the maximum time for major geometric changes of roadways or bridges. Examples of estimated service life used in California are:

Improvement	Service Life
Signals	15 years
Safety Lighting	15 years
Median Barriers	15 years
Flashing Beacons	10 years
Guardrail	10 years
Pavement Grooving	10 years
Signing (major)	10 years
Signing (minor)	5 years
Raised Pavement Markers	5 years
Guide Markers	5 years
Painted Stripes	2 years

Both the costs and benefits of improvements should be calculated for the period of time designated as expected service life. The analysis period should not extend beyond the period of reliable forecast. Thus, the estimated service life should reflect the length of time that estimated accident reduction reasonably can be expected instead of the physical life of the improvement. For example, given a strong possibility of an intervening solution, such as improved vehicle design, traffic diversion or highway reconstruction, the service lives of the alternatives should be adjusted to reflect the shorter planning horizon.

Improvement Costs. There are three basic parts of improvement costs:

- Initial costs--the investment prior to and during construction.
- Annual costs--the annual expense required to keep the improvements operating.
- Terminal value--the amount recoverable at the end of the service life.

The manner of estimating initial costs will vary with the complexity of the improvement. Routine installation of signals, signs and similar standard installations can be based on average costs from experience. More extensive improvements will require preliminary design and estimating of quantities as bases for cost estimates.

Many improvements require an annual expenditure for maintenance and operation. For example, a traffic signal will have annual costs for electrical power and equipment maintenance. Annual cost figures can be obtained by analyzing operating cost data. For some improvements, the annual cost will be zero or so small that it can be ignored in the economic analysis.

The terminal value is the difference between the monetary value at the end of the period of service and the future cost of removal, repair, transfer and/or sale. For a safety improvement, it may include signing that is useable at another location or salvageable guardrail. If a proposed improvement will have terminal value, it should be included in the analysis. However, most improvements have very little terminal value.

Equivalent Uniform Annual Benefits and Costs. In order to conduct meaningful economic analyses, there is need to convert the computed benefits and costs to equivalent uniform annual values--with appropriate consideration of interest rates and the cost of capital investment.

Equivalent Uniform Annual Benefits (EUAB). When estimated benefits from accident reductions are expected to be reasonably uniform through each year of the lifespan of the improvement, the calculated annual benefit value may be used directly as the "equivalent uniform annual benefit."

Accident reduction usually will be related to projected traffic volumes--and if a significant increase in traffic is expected, the benefits will increase proportionally during the period. Simple averaging of the annual benefits will not give a proper basis for economic evaluation. It is necessary to establish an equivalent uniform annual benefit (EUAB) with consideration of the interest rate. The following formula should be used:

EUAB =

$CR_n^i \Sigma$ (each year's benefit)(each year's PW_n^i)

Where: CR_n^i = Capital Recovery Factor for n years (service life of improvement) at interest rate i

PW_n^i = Present Worth Factor for each year at interest rate i

Σ = Summation of all years of service life

The factors for capital recovery and present worth may be found in conventional interest tables.

Equivalent Uniform Annual Costs (EUAC). Because the cost of an improvement consists of an initial investment, recurring annual costs and consideration of salvage value, it is necessary to establish an equivalent uniform annual cost (EUAC) during the life of the improvement. If the annual costs are uniform through the period, use the formula:

EUAC =

$$CR_n^i(1) - T\ (n^{th}\text{year}\ SF_n^i) + K$$

If the annual costs vary from year to year, use the formula:

EUAC =

$$CR_n^i \left[I + \Sigma\ K_i PW_n^i \right] - T(SF_n^i)$$

Where: CR_n^i = Capital Recovery Factor for n years at an interest rate of i
PW_n^i = Present Worth Factor for each year at an interest rate of i
SF_n^i = Sinking Fund Factor for n years at an interest rate of i
1 = Initial cost of improvement
K = Constant annual cost
T = Terminal value of improvement
n = Service life of improvement
Σ = Summation of all years of service life

The values for capital recovery factor, present worth factor and series present worth factor can be found in conventional interest tables or calculated using the formulas in Table F-2 in Appendix F of the Research Report.

Criteria For Selection and Priorities. The values for the equivalent uniform annual benefits and costs computed for each improvement are used to establish indices for selecting improvements and establishing priorities.

The two indices are:

- NET ANNUAL BENEFITS--the difference between equivalent uniform annual benefits and costs (EUAB - EUAC)

- BENEFIT/COST RATIO--the ratio of the equivalent uniform annual benefits to costs (EUAB/EUAC)

The significance of these indices is quite evident. As long as the net annual benefit is a positive value, the improvement is a feasible investment. But if we have a choice between two or more improvements, the one with the largest net annual benefit should be selected. Similarly, any benefit/cost ratio value of more than one indicates a feasible improvement.

Ranking For Program Consideration. Once we have identified the improvement, or combination of improvements which will assure the maximum net annual benefits at each location, the next step is to establish priorities for implementation of the improvements.

The list may be quite long--and it will be evident that accomplishment of the total program will require several years. It is convenient to establish a series of annual programs which may be related to specific annual budgetary allocations.

Several factors may influence decisions on selecting specific improvements for a particular year. Some of these may be non-economic considerations. But, to the extent practical, priorities should be based on relative economic merit of individual improvements. And, one of the generally accepted indices for this purpose is the computed benefit/cost ratio.

The benefit/cost ratio (EUAB/EUAC) should be computed for the selected improvement at each location--and the total list should be structured in descending order of benefit/cost ratios. The current annual program should include projects selected from the top of the list. The amount of available funds would determine the number of projects that can be selected for that year.

Documentation. There needs to be complete documentation of data and logic leading to the selection of eligible improvements. When the time comes to evaluate the results of implemented improvements, the analyst will need to know the background and considerations that led to the recommendations--questions related to:

- Improvement parameters--What are the estimated service lives of the applicable improvements? Estimated costs?

- Improvement performance--What is the estimated accident reduction of each applicable improvement?

- Evaluation data--What is the equivalent uniform annual cost of each improvement? Equivalent uniform annual benefit?

- Evaluation results--Which improvements were selected? What were their benefit/cost ratios? Which improvements were rejected? Why?

2.1.57 Sensitivity of the Economic Analysis

It should be recognized that certain factors of the economic analysis may be highly sensitive and significantly affect the outcome of the analysis. Particular attention should be given to interest rates, terminal values and service lives. The key to realistic results is in the selection of realistic values and consistency in their selection.

In the selection of interest rates, it appears most logical to select the rate at which the agency can borrow for capital expenditures. Long-term improvements are affected substantially by a radical departure either higher or lower than the prevailing rate.

The sensitivity of terminal values and service life were previously discussed, but should be recognized as important considerations in the economic analysis.

2.1.6 PROGRAMMING AND IMPLEMENTING IMPROVEMENTS

There is need to place the highway safety improvement program in proper perspective--and to establish policies and procedures to implement the program effectively. Normally, this will involve top-level management decisions.

Basic Objectives and Policies. Most highway agencies operate on the basis of some established objectives and policies, written or unwritten, which define their purpose, goals and general framework of management. Quite often, safety is included as one of the criteria governing the design, construction and maintenance of highway facilities. But seldom has there been a singular effort directed solely at assuring safety in highway facilities. Only since the Federal government has stepped into the picture and established safety standards and provided financial incentives for safety programs have highway agencies been required to direct specific attention to safety.

In some ways this turn of events has been unfortunate--because it has placed highway safety considerations in a separate category of operations rather than as an integral part of the basic objectives and policies of a highway agency. But at least our attention has been directed to assuring safe highway facilities--and administrative officials of each agency now should acknowledge formally the extent of commitment to highway safety within their overall programs.

Some states have statutes requiring that available highway funds be used first for maintenance--so as to assure protection of the investment in existing facilities--and only the remaining funds can be used for new construction and improvements. The same type of priorities might be applicable to assure protection of the motorists.

The point is that safety programs need not be financed only with specified, earmarked funds. A well-conceived, totally comprehensive safety program might well be integrated with overall highway program objectives without finanacial limitations. But if highway agencies are not ready to make this commitment, the practice of earmarking specific funds will at least assure a minimum safety program.

Formal policy statements should be developed by each agency to clarify the status of the safety program and to define its scope and responsibilities.

Program Coordination. It is evident that safety improvements cannot be programmed and implemented entirely independent of other highway programs--even when separate funding is provided. Careful planning and coordination is essential.

After proposed safety improvements have been identified, the regular highway construction program should be reviewed to ascertain if any major improvements are planned at the locations of the proposed safety improvements.

If major work is not planned or if the proposed safety improvement will not be affected by reconstruction--the safety improvement should be implemented as soon as possible.

If the safety improvement will be replaced or negated by reconstruction in a few years, the following options are open:

- Recompute the net annual benefits or the benefit/cost ratio for the improvement with a reduced service life. If the results are still significantly positive, implementation should proceed.

- If the countermeasure cannot be economically justified with reduced service life, it should be determined whether the reconstruction can be accelerated for earlier completion.

- If reconstruction cannot be accelerated sufficiently, one or more lesser improvements should be re-evaluated to determine whether a smaller investment may be justified as a stopgap measure.

In some agencies, it will be necessary to coordinate safety improvement programs with other agency divisions--such as traffic engineering and maintenance--when these divisions have responsibilities for performing minor improvements and betterment work not included in the formal construction program; and certainly there is need for coordination with the design division to make certain the findings and conclusions relative to existing hazardous locations are

considered when establishing design standards and procedures for new construction projects.

Implementation Responsibilities. Various techniques and procedures can be used for implementing different types of improvements. When preparing programs, several questions need to be answered:

- How will the work be performed?
- Who will design the improvement?
- Who will be responsible for work performance?

Safety improvements may be accomplished by contract or with direct labor by agency work forces. Large complex improvements usually should be performed by contract, while agency work forces often can execute effectively the smaller-type improvements. Each agency must analyze its own capabilities and the economics of the alternative approaches.

Some improvements may be so simple that little design effort is required--removing obstacles, striping pavement, installing routine signs and signals, etc. Other improvements may require considerable study and design effort--intersection design, grade separations, channelization, large signs, sophisticated signal systems, etc. Work by contract will require more extensive design and the specification documentation. There is need to identify the extent of design work and the level of expertise required--and assign specific design responsibilities to appropriate units within the agency or to outside consultants.

Depending on the type of work involved, different organizational units and levels within an agency may be called upon to supervise work performance--units are as follows:

- Construction Division
- Maintenance Division
- Traffic Engineering Division
- District Engineers

Specific responsibilities should be identified for directing and/or supervising the work of both contractors and agency employees for each proposed improvement.

2.1.7 EVALUATING IMPLEMENTED IMPROVEMENTS

One of the principal weaknesses of past experience with highway safety programs has been lack of adequate follow-up and evaluation of the actual results of implemented improvements.

There are three possible results that may be expected from a safety improvement: there may be an increase in accidents, there may be a decrease, or there may be no change. If there is no change or a negative improvement, another alternative should be tried; if there is a significant improvement, the confidence in this alternative is needed for possible use at other locations.

2.1.71 Basis for Comparison

The purpose for implementing an improvement is to effect a significant accident reduction. Several techniques have been employed for evaluating results.

Before and After Analysis. This analysis compares accident experience at a particular location before and after improvement implementation. To obtain statistically-reliable data for evaluating a type of improvement, the before and after accident experiences at several locations may be grouped together.

Parallel or Control Group Analysis. This analysis compares accident experience at the improved location with accident experience at similar locations not receiving improvements. Comparisons are made with the experience at these "control group" locations during the "after" period or with the trend in experience from "before" to "after."

Performance Standard Analysis. This analysis compares improvement performance with standard performance for that improvement--and is applicable only when performance standards have been established.

Before and after analyses have been used more extensively than the other techniques, probably because of the difficulty in finding truly similar control locations.

2.1.72 Evaluation Procedures

Before and after studies may be conducted at a single location or at several locations with similar characteristics where the same improvement has been implemented. A regular program should be established for reviewing all implemented improvements every three months during the first year, and annually thereafter.

The basic data needed are:

- The improvement documentation--location, time, etc.
- Accident data--how many, what types, how severe
- Traffic volumes
- Any significant changes in the physical environment (other than the improvement)

which may influence accident records--illumination, skid resistance, etc.

Before and after comparisons normally will be made in terms of accident rates-accidents per million vehicles or per million vehicle miles. The basis for measurement will be percent reduction of accident rate. Comparisons also may be made in terms of numbers of accidents, but adjustments must be made for both time periods and changes in traffic volumes for meaningful results.

Before and after data should reflect comparable time periods, preferably at least twelve months. When less than twelve month data are available following implementation, the before data should be selected from the same months as the after data. For example, if after data are based on a period from October to March, the before data should be based on experience for the same months of the preceding year--or for the average of those months for several preceding years. For each location, or for each group of locations with similar characteristics and improvements, the change in accident experience is calculated and identified as:

Percent Accident Reduction =

$$100 \frac{\text{(Accident rate before)} - \text{(Accident rate after)}}{\text{(Accident rate before)}}$$

This procedure should then be repeated to identify changes in accident experience by types of accidents and severity of accidents. This will permit evaluation of the overall effect of the improvement. For example, the total accident rate may not have been materially reduced, but a significant decrease in severity of accidents will result in measurable overall benefits. On the other hand, a reduction in accident rate may produce little benefit if, for some unforeseen circumstance, the severity of accients shows a marked increase.

The original premise was that each improvement was economically justified. Using the actual findings on reduction of accidents by types and severity along with updated data on accident costs and the costs of implementing improvements, it can now be determined whether a wise decision was made. And, more importantly, the findings will help to make better decisions next year.

2.1.73 Significance of Results

Before jumping to a conclusion about the merits of a particular improvement and its effectiveness in reducing accidents, it is necessary to take a second look at the data to determine how much confidence to place in the findings.

There is a certain degree of chance in all happenings. Just because a coin comes up heads 7 times out of 10 flips, we would not have much confidence in predicting 70 heads out of 100 flips. We are reasonably sure it is going to even out about 50-50 in the long run. But if it happened that heads came up 70 out of the next 100 times, the results would start to be significant--we would begin to believe the coin was unbalanced, or that something other than mere chance was controlling the happening.

The same thing applies to accident data. We would have little confidence in predicting great changes on the basis of one week's experience, or a month--or probably even three months. The more experience we observe, the greater will be our confidence.

Suppose two locations had the accident experience shown below for periods of one year before and one year after implementation of an improvement.

Location	Before Accidents	After Accidents	% Reduction
A	50	30	40
B	5	3	40

Even though both locations experienced the same percent reduction during the same period, we would have a great deal more confidence in the findings at location A than at location B.

A simple test can be employed to determine whether the results at a particular location (or group of locations) are truly statistically significant. The test assumes that the distribution of accidents at a location has the general characteristics of a Poisson distribution. This distribution is illustrated graphically by the curves in Figure 2.1.3, and relates the total number of accidents in the data period preceding the improvement to the percent reduction of accidents following implementation of the improvement.

The curves in Figure 2.1.3 are designed to assure a 95% level of confidence that the indicated accident reduction was significant. This means there is only a 5% probability that the reduction occurred merely by chance. A 95% level of confidence is considered generally acceptable.

The lower of the two curves reflects a liberal test of significance--the upper curve a more conservative test. Testing of results at a particular location involves the following steps:

1. Identify the number of accidents in the data time period before implementation (the time should be comparable to the after-implementation data time period).

2. Compute the percent accident reduction at the location (See instructions in previous section).

3. Adjust, if necessary, the number of "before" accidents: If the before time period is longer than the after time period (this is the usual case), adjust the number of before accidents, B', to reflect differences in traffic volumes and time periods.

$$B' = B \frac{(\text{After ADT})(\text{Days in after period})}{(\text{Before ADT})(\text{Days in before period})}$$

For intersections, use the sum of the ADT on each of the legs and divide by 2 to obtain average ADT. If the traffic volume changes during the before or after period, multiply each ADT by the number of days it was applicable. If the after time period is longer than the before time period, the number of before accidents, b, need not be adjusted.

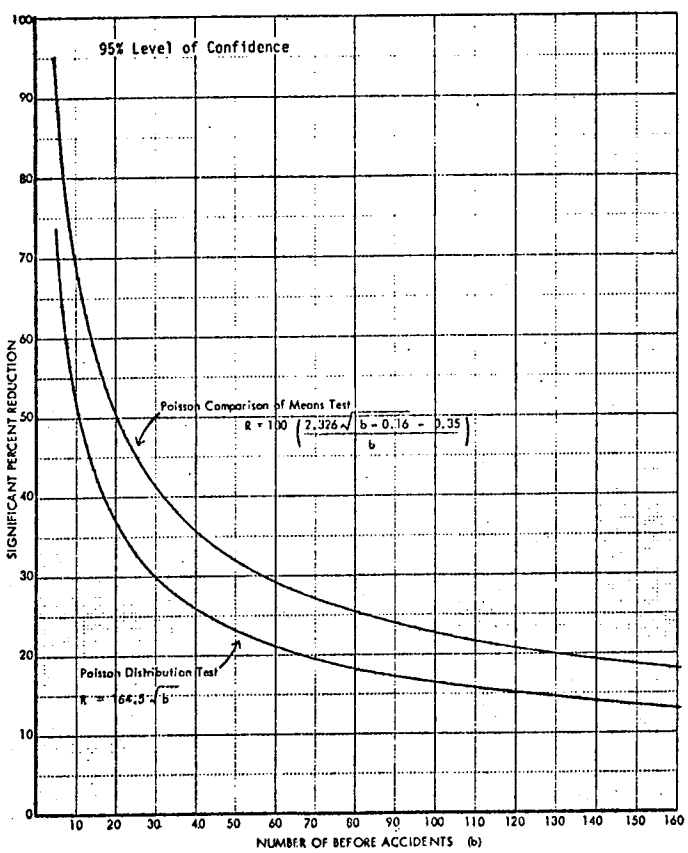

Figure 2.1.3 Poisson Tests For Significance

4. Refer to the curves in Figure 2.1.3 and locate the point of intersection of the number of accidents and the percent reduction.

5. If the intersection point is below the bottom curve, we are not sufficiently confident that the improvement actually caused that amount of accident reduction. The data are not considered significant as bases for future judgments.

6. If the intersection point is above the top curve, we are 95% certain that the accident reduction was attributable to the improvement. Data from these locations should be reliable for updating our standards, guides and criteria for future planning.

7. If the intersection point falls between the two curves, the significance of the results is uncertain. Continue to collect data from the location for an additional period of time and then re-evaluate the improvement.

2.1.8 EVALUATING THE SAFETY PROGRAM

Previously we discussed the need for a highway agency clearly to establish basic objectives and policies for its highway safety program. Subsequently, a management system framework was defined to facilitate accomplishment of objectives.

Periodically there is need to evaluate the total safety program in terms of:

Effectiveness. Are we getting results consistent with the defined objectives? Is there evidence of positive benefits? Are the expenditures economically justified?

Appropriateness of Objectives. Are current objectives appropriate? Have conditions changed or is there any indication of trends which would point to redirection of effort?

Criteria and Procedures. Based on experience, should changes be made in criteria and procedures for planning and implementing programs? Are the values used for forecasting still valid? Are the basic data on costs and benefits realistic? Is accident reporting of adequate quality and precision?

Legislators and top officials will have a principal interest in the first evaluation above--effectiveness. They have made a commitment to the program, established objectives and allocated resources for safety improvements. They should expect an accounting of the results--both costs and benefits. And they should know what to expect from future investments.

Those persons charged with responsibility for executing the safety program should have a keen interest in the third evaluation above--criteria and procedures. The effectiveness of future programs is dependent on reliable data, realistic criteria and sound decisions. There is a need continually to improve and refine the program management.

All levels of officials and management should have an awareness of changing conditions and of circumstances which might suggest changing the direction and scope of safety efforts.

2.1.81 Effectiveness

Resources have been invested in highway safety improvements. What did we get for our money?

Effectiveness is best measured in terms of:

- Increase or decrease in accident rates and accident severity at improved sites.
- Total reduction in cost of accident damage.
- Return on investment of improvement expenditures.

This type of information should be summarized from actual experience data and presented in graphical or tabular form.

- Are the cost estimates reliable for implementing improvements?
- Are the estimates of improved service life realistic?
- Is the interest rate appropriate for economic analysis?
- Are the analyses providing valid comparisons of costs and benefits?
- Are the priority indicators realistic?

2.1.82 Programming and Implementing Improvements

- Are all available earmarked safety funds being utilized?
- Is the safety program adequately coordinated with other agency improvement programs?
- Is there any overlap of responsibility for planning and implementing safety improvements?
- Are various improvements being implemented in the most economical manner? By contractors? By agency work forces?
- Is the scheduling process effective? Are there problems with manpower utilization and meeting target dates?
- Should there be changes in the approach to funding safety improvements?

2.1.83 Evaluating Implemented Improvements

- Is all necessary information being documented accurately?
- Are the data on post-implementation accidents reported adequately?
- Are the tests adequate to assure that the results are significant?
- Do we have a reasonable level of confidence that the results were primarily attributable to the improvements?
- Are the actual benefits and costs at each location reasonably close to our predictions? If not, why?
- Is there reasonable consistency in the results from one location to another?

2.1.84 Evaluating the Highway Safety Program

- Are the benefits identified with highway safety improvements sufficient to justify the investments in the program?
- Are there particular types of investments that are more productive than others?
- What is the total scope of the problem? Are we making progress?
- Is there need to adjust the levels of funding or to predict emphasis?
- Is top management getting the information they need for policy decisions?
- Based on experience, is there need to update standards, guides and criteria for planning safety improvements?
- Can improvements be made in the safety management system procedures?

2.1.85 Summary - Spot Safety Improvement Program

A spot safety improvement program involves six principal steps:

- Identifying Problem Locations
- Selecting Alternative Improvements
- Evaluating Improvement Alternatives

Those persons charged with responsibility for executing the safety program should have a keen interest in the third evaluation above--criteria and procedures. The effectiveness of future programs is dependent on reliable data, realistic criteria and sound decisions. There is a need continually to improve and refine the program management.

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- Is the interest rate appropriate for economic analysis?
- Are the analyses providing valid comparisons of costs and benefits?
- Are the priority indicators realistic?

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- Are the data on post-implementation accidents reported adequately?
- Are the tests adequate to assure that the results are significant?
- Do we have a reasonable level of confidence that the results were primarily attributable to the improvements?
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- Is there reasonable consistency in the results from one location to another?

2.1.84 Evaluating the Highway Safety Program

- Are the benefits identified with highway safety improvements sufficient to justify the investments in the program?
- Are there particular types of investments that are more productive than others?
- What is the total scope of the problem? Are we making progress?
- Is there need to adjust the levels of funding or to predict emphasis?
- Is top management getting the information they need for policy decisions?
- Based on experience, is there need to update standards, guides and criteria for planning safety improvements?
- Can improvements be made in the safety management system procedures?

2.1.85 Summary - Spot Safety Improvement Program

A spot safety improvement program involves six principal steps:

- Identifying Problem Locations
- Selecting Alternative Improvements
- Evaluating Improvement Alternatives

- Programming and Implementing Improvements
- Evaluating Effectiveness of Improvements
- Evaluating the Program

All of these steps are essential for a viable program, and it is evident that there are alternative methods for the accomplishment of these steps. Methods employed by a particular agency should be selected on the basis of the needs, availability of data, and the technical capability within the agency. Perhaps the most important final point is to stress the need for documentation. The entire process, throughout the six steps should be documented for the benefit of legal protection and information value.

The procedure described below is an alternative for the first three steps in the Spot Safety Improvement Program.

2.1.9 COST-EFFECTIVENESS SAFETY PROGRAM FOR ROADSIDE IMPROVEMENTS

Some types of hazards, particularly roadside hazards, do not have high accident rates on any single hazard, but, when struck, can produce a very severe accident. The improvement of these hazards must, therefore, be based on a probabilistic type of model of the type developed by the Texas Transportation Institute (3) and described below. Cost-effectiveness analysis depends upon the probability of the hazard being struck, the severity of the resulting collision, and the total cost of the improvement.

The Spot Safety Improvement Program is predicated on the assumption that the frequency or rate of accidents will indicate a need for safety improvements, and that the value of these improvements may be measured in dollar terms.

The procedure described below is an alternative for the first three steps in the Spot Safety Improvement Program:

- Identifying Problem Locations
- Selecting Improvement Alternatives
- Evaluating Alternative Improvements

It should be emphasized that this approach applies only to roadside hazards, whereas the Spot Improvement Program is applicable to all hazardous situations.

Cost-Effectiveness Model. The general cost-effectiveness model is expressed in the following equation:

Cost-Effectiveness Value =

$$\frac{\text{Annualized Cost of the Improvement}}{\text{Hazard Reduction Achieved}}$$

$$C/E = \frac{C_I + C_{MA} - C_{MB} + P_{HA}C_{HA} - C_{HB}P_{HB}}{H_B - H_A}$$

where:
- C/E = Cost in dollars to eliminate one serious injury or fatal accident
- C_I = Annualized first cost of the improvement
- C_{MA} = Annualized maintenance cost after improvement
- C_{MB} = Annualized maintenance cost before improvement
- C_{HA} = Annual cost to repair after being hit--after condition
- C_{HB} = Annual cost to repair after being hit--before condition
- P_{HA} = Probability of object being struck--after condition
- P_{HB} = Probability of object being struck--before condition
- HB = Hazard index, before improvement
- HA = Hazard index, after improvement (Hazard Index is product of probability of a vehicle leaving the roadway, probability that vehicle will travel laterally to the hazard, and the severity of the collision)

The cost elements in the numerator are all computed directly from the cost estimates. The probability of the object being hit is determined by the product of the probability of an errant vehicle reaching the object and the probability of a vehicle encroaching on the roadside.

The denominator elements are computed as the product of the probability of the vehicle leaving the roadway, the probability of an encroachment to the hazard, and the severity of a resulting collision.

Interpreting the C/E Value. The cost-effectiveness value is the cost of eliminating one serious or fatal accident. The nature of the model is such that the C/E value computed can be negative, either by the cost factors in the numerator being negative or by the difference of the hazard values in the denominator being negative. Assuming that a negative change in the relative hazard is not acceptable (i.e., a negative denominator will be disregarded), the negative C/E value will be meaningful and would indeed indicate that the agency would save money if the improvement is undertaken. The magnitude of the C/E value is also important.

The smaller the C/E number, the more benefit is derived from the work. This means a

smaller cost to accomplish the reduction of a possible fatality or injury. The end product of any program is to accomplish the greatest good with the least cost.

Program Application. The application procedure to evaluate safety improvements for roadside hazards comprises three related functions as summarized below:

1. Conducting a detailed physical inventory of the highway system to identify and locate each roadside hazard,

2. Recommending feasible safety improvement alternatives for each hazard or for groups of hazards, and

3. Evaluating the recommended safety improvement alternatives using a computerized cost-effectiveness analysis model. Data are recorded on the appropriate forms (See Figures 2.1.4, 2.1.5, and 2.1.6). These data sources provide the basic input information for analysis by the cost-effectiveness model.

The inventory scope includes all applicable roadside hazards located in the median or within a 30-ft lateral distance adjacent to the outer edge of the traveled lanes. Since the recommendations for alternative safety improvements will govern to a great extent the cost-effectiveness results, the inventory team must include personnel having considerable experience in traffic operations, geometric design, maintenance, and cost-estimating. Field trials of the inventory procedure indicated that a four-person team represents an efficient working force, to include as a minimum a driver, a data recorder, and two decision-makers to recommend safety improvements.

A one-week course which presents the cost-effectiveness application methodology was prepared recently by TTI and the Texas Department of Highways and Public Transportation for the Federal Highway Administration.

REFERENCES

1. Laughland, John C., et. al. Methods for Evaluating Highway Safety Improvements. NCHRP Report 162, Transportation Research Board, 1975.

2. Taylor, J.I. and Thompson, H.T. Identification of Hazardous Locations. Report No. FHWA-RD-76-44, Federal Highway Administration, 1976.

3. Weaver, G., et. al. Cost-Effectiveness Analysis of Roadside Safety Improvements. TRB Report 543, Transportation Research Board, 1975.

4. Colson, Cecil, CORRECT: Section 209, Phase 2, Alabama Highway Department, August 1975.

5. Manual on Identification, Analysis and Correction of High Accident Locations, Missouri Highway Department.

DISTRICT.....
CO.....
HWY.....

SHEET.....OF.....
PREPARED BY.....
DATE.....

ROADSIDE HAZARD INVENTORY FORM

HAZARD LOCATION AND CLASSIFICATION

BOX 1

HAZARD IDENTIFICATION						HIGHWAY DESCRIPTION						HAZARD CLASSIFICATION				MILEPOST AT HAZARD	
COUNTY CODE	CONTROL-SECTION: CONTROL NUMBER	CONTROL-SECTION: SECTION NUMBER	GROUPING NUMBER	HAZARD NUMBER		HIGHWAY TYPE	HIGHWAY NUMBER	CLASSIFICATION	TOTAL WIDTH CENTER-LINE TO SHLDR. ON INVENTORY SIDE*	ADT (TOTAL BOTH DIRECTIONS 1000'S)	RECORDING DIRECTION	CLASSIFICATION: IDENTIFICATION CODE	CLASSIFICATION: DESCRIPTOR CODE	OFFSET CODE	MEDIAN WIDTH (FT) LEAVE BLANK IF MED INV NEAR SIDE ONLY	BEGINNING	END (EXCEPT) FOR POINT HAZARD
1 2 3	4 5 6 7	8 9	10 11 12 13	14 15 16 17	18	19 20	21 22 23 24	25	26 27	28 29 30	31	32 33	34 35	36	37 38 39	40 41 42 43 44 45	46 47 48 49 50 51

* UNDIVIDED HIGHWAY ONLY

HAZARD DESC.....

POINT HAZARDS

BOX 2

HAZARD TYPE	HAZARD OFFSET, D(FT)		WIDTH W (FT)	LENGTH L (FT)	DROP INLETS ONLY (ENTER HEIGHT OR DEPTH): HEIGHT (FT)	DROP INLETS ONLY (ENTER HEIGHT OR DEPTH): DEPTH (FT)	UPDATE CODE	CARD TYPE
52	53 54	55 56 57 58 59 60 61 62 63 64 65 66	67 68 69	70 71 72	73 74 75	76 77 78	79	80
1								1

HIGHWAY TYPE CODE
- 00 US Spur
- 01 US
- 02 SH
- 03 SH Spur or Loop
- 05 FM-RM
- 06 US Alternate
- 08 IH

RECORDING DIRECTION CODES
- 1 With M.P.
- 2 Against M.P.

OFFSET CODES
- 1 Right
- 2 Median or left side

HIGHWAY CLASS. CODES
Full Control Access
- 1 Interstate
- 2 Non-Interstate

Non-Control Access
- 3 Two-Lane
- 4 Multi-Lane Divided
- 5 Multi-Lane Undivided

Other
- 6 One-Way Frontage Road
- 7 Two-Way Frontage Road

LONGITUDINAL HAZARDS

BOX 3

HAZARD TYPE	HAZARD OFFSET, D (FT): BEGINNING	HAZARD OFFSET, D (FT): END	GUARDRAIL END TREATMENT: BEGINNING	GUARDRAIL END TREATMENT: ENDING	GUARDRAIL END TREATMENT	HEIGHT OR DEPTH (FT)	WIDTH W(FT)		UPDATE CODE	CARD TYPE
52	53 54	55 56	57	58	59 60 61 62 63 64 65 66	67 68 69	70 71	72 73 74 75 76 77 78	79	80
2										1

GUARDRAIL END TREATMENT CODES
- 1 Not beginning or ending at structure - Safety treated
- 2 Not beginning or ending at structure - Not safety treated
- 3 Beginning or ending at structure - Full-beam connection
- 4 Beginning or ending at structure - Not full-beam connection

SLOPES

BOX 4

HAZARD TYPE	FRONT SLOPE: HINGE POINT OFFSET D_0(FT): BEGINNING	FRONT SLOPE: HINGE POINT OFFSET D_0(FT): END		FRONT SLOPE: STEEPNESS (HORIZ.:1FT): BEGINNING	FRONT SLOPE: STEEPNESS (HORIZ.:1FT): END	FRONT SLOPE: SLOPE DIRECTION	FRONT SLOPE: DISTANCE D_1(FT): BEGINNING	FRONT SLOPE: DISTANCE D_1(FT): END	FRONT SLOPE: SLOPE FACE EROSION CODE	SECOND OR BACK SLOPE: STEEPNESS (HORIZ.:1FT): BEGINNING	SECOND OR BACK SLOPE: STEEPNESS (HORIZ.:1FT): END	SECOND OR BACK SLOPE: SLOPE DIRECTION	SECOND OR BACK SLOPE: DISTANCE D_2(FT): BEGINNING	SECOND OR BACK SLOPE: DISTANCE D_2(FT): END	SECOND OR BACK SLOPE: SLOPE FACE EROSION CODE	UPDATE CODE	CARD TYPE
52	53 54	55 56	57 58	59 60	61 62	63	64 65	66 67	68	69 70	71 72	73	74 75	76 77	78	79	80
3																	1

SLOPE DIRECTION CODES
- 1 Positive
- 2 Negative

SLOPE FACE EROSION CODE
- 1 Slight or none
- 2 Severe (Ruts>1ft)

UPDATE CODES
- 1 Delete
- 2 Add
- 3 Change

ENTER DATA ON ONE LINE ONLY

RECOMMENDATIONS _____

Figure 2.1.4 Roadside Hazard Inventory Form

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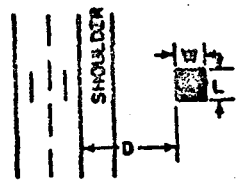
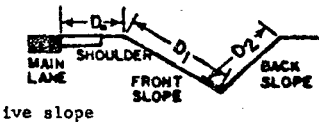
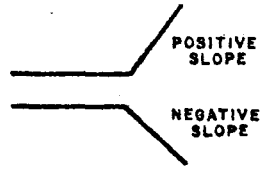
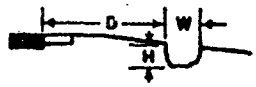
Identification Code	Descriptor Codes
(01) Utility Poles	(00)
(02) Trees	(00)
(03) Rigid Signpost	(01) single-pole-mounted (02) double-pole-mounted (03) triple-pole-mounted (04) cantilever support (05) overhead sign bridge
(04) Rigid Base Luminaire Support	(00)
05. Curbs	(01) mountable design (02) non-mountable design less than 10 inches high (03) barrier design greater than 10 inches high
06. Guardrail or Median Barrier	(01) w-section with standard post spacing(6 ft - 3 in.) (including departing guardrail at bridge) (02) w-section with other than standard post spacing (including departing guardrail at bridge) (03) approach guardrail to bridge--decreased post spacing (3 ft - 1½ in.) adjacent to bridge (04) approach guardrail to bridge--post spacing not decreased adjacent to bridge (05) post and cable (06) Metal Beam Guard Fence (Barrier) (in median) (07) median barrier (CMB design or equivalent)
07. Roadside Slope	(01) sod positive slope (02) sod negative slope (03) concrete-faced positive slope (04) concrete-faced negative slope (05) rubble rip-rap positive slope (06) rubble rip-rap negative slope
08. Ditch (includes erosion, riprap runoff ditches, etc.--does not include ditch formed by intersection of front and back slopes)	(00)
(09) Culverts	(01) headwall (or exposed end of pipe culvert) (02) gap between culverts on parallel roadways (03) sloped culvert with grate (04) sloped culvert without grate
(10) Inlets	(01) raised drop inlet (tabletop) (02) depressed drop inlet (03) sloped inlet
(11) Roadway Under Bridge Structure	(01) bridge piers (02) bridge abutment--vertical face (03) bridge abutment--sloped face
12. Roadway Over Bridge Structure	(01) open gap between parallel bridges (02) closed gap between parallel bridges (03) rigid bridgerail--smooth and continuous construction (04) semi-rigid bridgerail--smooth and continuous construction (05) other bridgerail--probable penetration, snagging, pocketing or vaulting (06) elevated gore abutment
13. Retaining Wall	(01) face (02) exposed end









(01) Denotes Point Hazard

Figure 2.1.5 Hazard Classification Codes

DISTRICT.....
CO.....
HWY.....

SHEET.....OF.....
PREPARED BY.....
DATE.....

ROADSIDE HAZARD IMPROVEMENT FORM

IMPROVEMENT IDENTIFICATION AND COSTS

BOX 1

IMPROVEMENT IDENTIFICATION						HAZARD/IMPROVEMENT COSTS			
COUNTY CODE	CONTROL-SECTION: CONTROL NUMBER	CONTROL-SECTION: SECTION NUMBER	GROUPING NUMBER	HAZARD NUMBER	IMPROVEMENT ALT	FIRST COST OF IMPROVEMENT ($)	REPAIR COST/COLLISION ($): HAZARD	REPAIR COST/COLLISION ($): IMPROVEMENT	NORMAL MAINTENANCE ($/YR): HAZARD	NORMAL MAINTENANCE ($/YR): IMPROVEMENT
1 2 3	4 5 6 7	8 9	10 11 12 13	14 15 16 17	18	19 20 21 22 23 24	25 26 27 28	29 30 31 32	33 34 35 36	37 38 39 40
☑										

HAZARD AND IMPROVEMENT DESCRIPTION

UPDATE CODES
2 Add
3 Change

POINT HAZARD IMPROVEMENTS

BOX 2

DO NOT ENTER DATA ON MORE THAN ONE LINE

IMPROVEMENT DESCRIPTION (NOTE: NUMBERS CORRESPOND TO DESC. CODES; FIRST IS DESCRIPTOR SECOND IS DETAIL AND THIRD IS BRIDGERAIL. TYPE REFERS TO POINT, ETC.)	DESC. CODES: TYPE	DESCRIPTOR	DETAIL		LATERAL OFFSET (FT) RIGHT, LEFT OR MEDIAN NEAR SIDE		LATERAL OFFSET (FT) MEDIAN FAR SIDE		LENGTH (FT)	WIDTH (FT)		UPDATE CODE	CARD TYPE
	41	42	43	44	45 46	47 48 49 50 51 52 53	54 55	56 57 58 59 60 61	62 63 64	65 66	67 68 69 70 71 72 73 74 75 76 77 78	79	80
1 Alleviate Hazard 1 Remove	1	1	1										2
1 Alleviate Hazard 2 Make Breakaway And/Or Relocate	1	1	2										2
1 Alleviate Hazard 3 Reconstruct Inlet To Safe Design	1	1	3										2
1 Alleviate Hazard; 4 Reconstruct Cross-Drainage System (Remove Headwalls, Extend Culv., Grade, Etc.)	1	1	4										2
2 Protect Hazard With Guardrail (Hazard Not On Critical Slope)	1	2											2
3 Protect Hazard With Concrete Median Barrier (CMB)	1	3											2
4 Protect Hazard With Energy Attenuation System	1	4											2

LONGITUDINAL HAZARD IMPROVEMENTS

BOX 3

COMPLETE LINE

IMPROVEMENT DESCRIPTION	DESC. CODES: TYPE	DESCRIPTOR	DETAIL	BRIDGERAIL	INSTALL GUARDRAIL: LATERAL OFFSET (FT) RIGHT, LEFT OR MEDIAN NEAR SIDE: BEGIN	END		INSTALL GUARDRAIL: LATERAL OFFSET (FT) MEDIAN FAR SIDE: BEGIN	END		CHANGES TO EXISTING GUARDRAIL: LENGTHEN (FT): BEGIN	END	SHORTEN (FT): BEGIN	END		UPDATE CODE	CARD TYPE
	41	42	43	44	45 46	47 48	49 50 51 52 53	54 55	56 57	58 59 60 61	62 63 64 65	66 67 68 69	70 71 72 73	74 75 76 77	78	79	80
1 Curb; 1 Remove And Regrade	2	1	1														2
1 Curb; 2 Install Wedge Modification	2	1	2														2
2 Bridgerail; 1 Rigid; 1 Upgrade To Full Safety Standards	2	2	1	1													2
2 Bridgerail; 1 Rigid; 2 Move Laterally	2	2	1	2													2
2 Bridgerail; 1 Rigid; 3 Install Guardrail Along Bridgerail Face	2	2	1	3													2
2 Bridgerail; 1 Rigid; 4 Deck Gap Over Between Parallel Bridges And Install Single Bridgerail	2	2	1	4													2
2 Bridgerail; 2 Semi-Rigid; 1 Upgrade To Full Safety Standards	2	2	2	1													2
2 Bridgerail; 2 Semi-Rigid; 2 Move Laterally	2	2	2	2													2
2 Bridgerail; 2 Semi-Rigid; 3 Install Guardrail Along Bridgerail Face	2	2	2	3													2
2 Bridgerail; 2 Semi-Rigid; 4 Deck Gap Over Between Parallel Bridges And Install Single Bridgerail	2	2	2	4													2
3 Guardrail; 1 Remove Existing Guardrail	2	3	1														2
3 Guardrail; 2 Upgrade To Full Safety Standards (Note: Supply Amount Of Change If Applicable)	2	3	2														2
3 Guardrail; 3 Upgrade To Full Safety Standards And Close-Up Gap	2	3	3														2
3 Guardrail; 4 Close-Up Gap Between Existing Guardrail	2	3	4														2
3 Guardrail; 5 Anchor Existing Guardrail To Bridgerail	2	3	5														2
3 Guardrail; 6 Safety Treat Guardrail Free-End Only	2	3	6														2
4 Ditch; 1 Reshape To Safe Cross Section	2	4	1														2
4 Ditch; 2 Replace With Storm Drain	2	4	2														2
4 Ditch; 3 Protect With Guardrail	2	4	3														2

SLOPE IMPROVEMENTS-GUARDRAIL

BOX 4

CHECK CORRESPONDING BOX

IMPROVEMENT DESCRIPTION	DESC. CODES: TYPE	DESCRIPTOR	DETAIL		INSTALL GUARDRAIL: LATERAL OFFSET (FT) RIGHT, LEFT OR MEDIAN NEAR SIDE OR APPROACH GUARDRAIL: BEGIN	END		INSTALL GUARDRAIL: LATERAL OFFSET (FT) MEDIAN FAR SIDE OR DEPARTING GUARDRAIL: BEGIN	END			UPDATE CODE	CARD TYPE
	41	42	43	44	45 46	47 48	49 50 51 52 53	54 55	56 57	58 59 60 61 62 63 64 65 66 67 68 69 70 71 72 73 74 75 76 77 78		79	80
1 Install Guardrail To Protect Slope Not At Bridge-May Include Point Hazards On Slope	3	1											2
2 Install Approach Or Departing Guardrail At Bridge-May Include Point Hazards On Slope	3	2											2
3 Install Continuous Guardrail Between Successive Bridges	3	3											2

SLOPE IMPROVEMENTS-FLATTEN SLOPE

BOX 5

SELECT PROPOSED IMPROVEMENT

IMPROVEMENT DESCRIPTION	DESC. CODES: TYPE	DESCRIPTOR	DETAIL		FRONT SLOPE: HINGE POINT OFFSET D_0 (FT): BEGIN	END	FRONT SLOPE: STEEPNESS (HORIZ.:1 FT): BEGIN	END	SLOPE DIRECT.*	FRONT SLOPE: DISTANCE D_1 (FT): BEGIN	END	SECOND OR BACK SLOPE: STEEPNESS (HORIZ.:1 FT): BEGIN	END	SLOPE DIRECT.*	SECOND OR BACK SLOPE: DISTANCE D_2 (FT): BEGIN	END	MILEPOST OF IMPROVED SLOPE (COMPLETE IF DIFFERENT FROM INVENTORY): BEGINNING	END	UPDATE CODE	CARD TYPE
	41	42	43	44	45 46	47 48	49 50	51 52	53	54 55	56 57	58 59	60 61	62	63 64	65 66	67 68 69 70 71 72	73 74 75 76 77 78	79	80
4 Flatten Slope	3	4					•	•				•	•				•	•		2

NO IMPROVEMENT RECOMMENDED

BOX 6

	41	42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71 72 73 74 75 76 77 78	79	80
4 No Improvement Recommended	4			2

• SLOPE DIRECT. CODES: 1 POSITIVE; 2 NEGATIVE

Figure 2.1.6 Roadside Hazard Improvement Form

Rev: 1/78

TOPIC 3 SESSION 1
DRIVING EXPECTANCY IN SAFETY DESIGN

Objectives:

1. The participant should become aware of the importance of driving expectancy in obtaining safe operation of the highway system and be able to evaluate a segment of roadway for driving expectancy problems, and

2. Be able to utilize the primacy concept in the design of the driver information system.

3.1.1 THE DESIGN PROCESS

The design of a roadway is a complex interaction of the design policy of the agency, the preferences of the design engineer, and the environmental, political, social and economic constraints imposed by society. The combination frequently does not result in a design that easily will meet the expectations of all the motoring public. These areas of expectancy conflicts (or design inconsistencies) result in driver uncertainty, an increase in response time, an increase in the probability of inappropriate driver response and, in concept at least, an increase in accident potential.

For these reasons a review of the design process and its relationship to safety is appropriate.

Design requirements usually are expressed in terms of observable conditions (speed, volume, delay, etc). The driver decision-making portion of the driver-roadway-vehicle complex system cannot be quantified easily. They are difficult to translate into design criteria and, therefore, difficult to incorporate into the design process. This can lead to compromises in the design process, particularly in the use of minimum standards when there is little or no justification for the use of such standards. AASHTO recognized this problem when the Policy on the Geometric Design of Rural Highways (1) adopted a desirable as well as a minimum standard for most design features.

If driving expectancy is to be integrated into the design process, a rather comprehensive knowledge of the concept is required, as well as a sincere desire on the part of the designer to utilize the concept in the design process.

3.1.2 EXPECTANCY AS A BEHAVIORAL CONCEPT

The general concept of expectancy has been a part of psychology for many years. Indeed, the concept of "set" was explored before the turn of the century. It can be defined as the process in which an individual with an established set of concepts and ideas is presented with a stimulus and responds to that stimulus in some manner. If the stimulus and situation match the individual's expectancy, there is no conflict, and the response generally can be expected to be the proper one. Conversely, if the situation is not as the individual expects (i.e., as he perceives), then uncertainty exists, and the response (or lack of response) may be unrelated to the situation except that the stimulus triggered the response cycle.

The basic expectancy model can be described as a complex servo-mechanism with a feedback loop. The driver learns from previous experience and adapts his response pattern more nearly to the demands of the situation. Figure 3.1.1 is a graphical representation of the model.

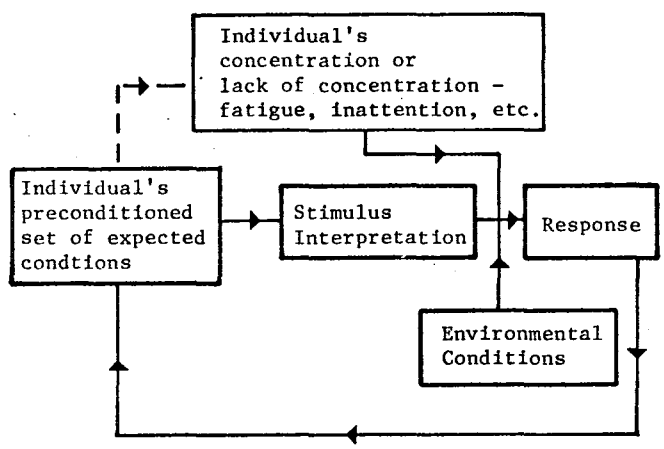


Figure 3.1.1 Simplified Expectancy-Response System.

This model presumes the existance of a preconditioned set of expectations followed by the presentation of the stimulus within the environment created by the environmental

and psychological modifiers. The absence of a predetermined set (i.e., expectancy) dictates random behavior or utter confusion on the part of the operator. Thus, the model, even though highly simplistic, is reasonable and necessary to the successful operation of a motor vehicle.

3.1.3 EXAMPLES OF EXPECTANCY IN DRIVING SITUATIONS

The highway engineer long has recognized that certain situations were very difficult for the motoring public. Possibly the most graphic example is the replacement of stop signs with signals. For the first few days of operation, it is common for drivers to complete a stop and proceed on through the intersection, regardless of the signal display that is illuminated. The use of tangential exit ramps when the main roadway makes a curve to the left results in many exiting maneuvers that were not planned. Many other cases can be cited. For example, midblock traffic signals, a sharp curve at the end of a long tangent section, the problem of the first signal encountered upon entering a large metropolitan area, etc. These examples suggest that driving expectancy has been a part of the highway engineering process for a long time. However, driving expectancy was regarded as an individual design criterion rather than as a comprehensive design philosophy.

3.1.4 DEFINITION OF DRIVING EXPECTANCY

Driving expectancy can be defined as "an inclination, based on previous experience, to respond in a set manner to a roadway or traffic situation." It must be stressed that the initial response is to the expected situation rather than to the actual situation. In most instances the two are similar enough so as to create minimal problems. Where design inconsistencies occur due to changes in standards with time, designer preference or any of a variety of reasons, the initial response will be delayed at best and wrong at the worst.

3.1.5 DRIVING INFORMATION SOURCES

The sources of information used by the motorist are extensive. Studies by Rockwell (2) and others have demonstrated that drivers do not concentrate on the road surface but rather spend much of their driving time viewing roadside objects, other traffic, the horizon, etc. While a complete listing of all the sources is impractical, two broad groups can be defined: 1) Formal Information Sources - those that are designed specifically to convey information; and 2) Informal Information Sources - those that are intended primarily for some other purpose and incidentally are used as an information source by the driver. Formal sources include all traffic control devices (i.e., signs, signals, markings, channelization, etc.), whereas informal sources include buildings, rivers, geometric situations, and other similar items.

The unique part of the information source concept is that the informal, not the formal, is the dominant source for the motorist.

3.1.6 TYPES OF DRIVERS

To further complicate the information processing problem, drivers have a wide variety of experience and training. With the exception of the first year of driving, most drivers have sufficient experience behind the wheel to know the fundamentals of operating a motor vehicle safely. Experience with a particular roadway section, however, depends entirely on the frequency of use of that segment of roadway by the driver. In this context, three driver types can be identified: 1) The familiar driver - one who drives the roadway on a regular basis; 2) Infrequent driver - one who uses the roadway once or twice each year; and 3) The unfamiliar driver - one who is using the roadway segment for the first time. While there is not a specific "design driver," most designs tend toward the familiar driver since this is the source of a majority of the pressure. A system which meets the needs of the unfamiliar motorist also would satisfy most of the other drivers' needs and, therefore, should be the basic design driver. This point will be discussed in greater detail later in this session.

3.1.7 DRIVER INFORMATION PROCESSING

The driver has available a wide range of information sources at one point in time. The process by which the information is selectively accepted and used by the driver is referred to as information processing. Alexander and Lunenfeld (3) have described three basic levels:

- Control Level - Activities and information which relate to manipulation of the vehicle.
- Guidance Level - Speed and path selection.
- Navigation Level - The process of planning and executing a trip from its origin to its destination.

Rowan and Woods (4) also define three levels of driver information processing. These definitions parallel very closely with those of King and Lunenfeld above, differing only in the scope of the definition.

- Positional Information - Information utilized in maintaining lane position and

desired speed. This information is predominantly near the vehicle, generally less than 300 ft (90 m) away.

- Situational Information - Information used to avoid undesirable interaction with pedestrians, other vehicles or roadway geometric situations. These sources are usually 300 to 1000 ft (90 to 300 m) from the vehicle.

- Navigational Information - That information used in selecting the path from the origin to the destination of the trip. Navigational information is variable but is commonly distant from the point where the action must be taken.

3.1.71 Information Primacy

The three general levels of information are not of equal importance to the motorist. Alexander and Lunenfeld (3) suggest that a distinct processing order exists. At the top of the primacy hierarchy is information utilized to maintain position on the roadway (control level or positional level). Second in priority is situational or guidance information. At the bottom of the hierarchy is navigational information. The primacy concept thus suggests that positional or control information must be processed completely before processing can begin at the situational or guidance level. Further, this level must be satisfied before navigational information will be processed.

The importance of this concept in safety design is the recognition that maintaining a high level of lane identification and demarcation reduces the demand for time to process positional information, and thereby increases the time available to process situational and navigational information. This, in turn, reduces the potential for erratic maneuvers and the accident potential.

3.1.8 THE DESIGN DRIVER

The concept of a design driver has been discussed at great length in the literature in recent years. The fact remains that while most design standards implicitly assume a design driver, they never actually define the characteristics of such a driver. As previously noted, drivers have vastly differing experience and training backgrounds. This, combined with physical and mental characteristics of the driving population, suggests the need for a design driver. The information relative to design driver characteristics presented in Table 3.1.1 is presented with the intent that it be used as supplemental information for existing design and traffic control standards. With some experience in the application of such information we may at some future date be better prepared to develop explicit design driver criteria.

3.1.9 SUMMARY

Several significant points have been included in this section. Briefly, they are:

- Driving expectancy has been a part of the design process for many years, but as an independent criterion rather than as a comprehensive design philosophy.

- The initial response to any situation is to the expected condition rather than to the actual condition.

- The higher the level of maintenance of the roadway delineation system, the lower the need for time to process positional information and the more time that can be devoted to situational and navigational vigilance.

- A design driver should be established to guide design decisions.

A definitive procedure has not yet been developed to formally incorporate driving expectancy into the design process (5). The "Driver Expectancy Checklist" (6), the "Positive Guidance" paper (3) and the "Driving Postulate Concept" (7) are attempts to provide some guidance to the designer in this regard. Since the design practices of the states differ in many details, the driver expectancies will be somewhat different in each area. A set of driving postulates must be established which conform to the practices of the individual state and

TABLE 3.1.1

SUGGESTED DESIGN DRIVER CHARACTERISTICS

Characteristics	Level of Value
Visual Acuity	20/40
Perception-Reaction Time	3.5 Seconds
Eye Height	3.0 Ft (0.9 m)
Color Blindness	Red & Green
Visual Angle	
Horizontal	
Detection	160°
Identification	10°
Vertical	
Up	60°
Down	10°
Intelligence Quotient	70

the design prepared accordingly. Simply stated, the designer's attitude must reflect a concern for not "surprising" the motorist. Stated more concisely, "Every design must be engineered rather than handbooked."

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4. Rowan, N.J. and Woods, D.L. Final Report - Diagnostic Studies of Highway Visual Communication Systems. Texas Transportation Institute Research Report 606-9F, March 1971.

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7. Woods, D.L. The Driver and the Road. Transportation Engineering Journal, American Society of Civil Engineers, TE4, November 1971.

TOPIC 3 SESSION 2
DESIGN CONSISTENCY

Objectives:

The objective of this session is to familiarize the participant with design consistency such that he will be able to:

1. Identify basic inconsistencies in geometric design,

2. Identify the operational and safety requirements for achieving design consistency, and

3. Utilize an analysis of traffic operations as an integral part of alignment design to achieve design consistency.

3.2.1 INTRODUCTION

Streets and highways are intended to provide quality transportation, but how do we measure the quality of transportation? To describe the attributes of a good highway, a user may propose the following list:

1. High speed -- a speed consistent with development, terrain and type of highway.

2. Uniform speed -- few stops, and no critical, slow-speed sections.

3. No congestion -- sufficient capacity to handle demand without excessive delay and reduced speed.

4. Wide road -- a roadway with adequate lane width and shoulders for driving comfort at the selected speed.

There are perhaps many descriptors of quality in a roadway, but these illustrate two specific points: operating speed appears in practically every descriptor of quality, and consistency in operation (and in design) appears to be a major factor. It is by designing a highway according to uniform operating speed, a design speed, that we achieve consistency in design.

3.2.11 Definition

Design consistency is a relatively new term although designers have been concerned about it for years. Even in the 1930's when the design speed concept was established, the intent was to achieve uniformity in operation--to design a highway so that the driver may know what to expect in geometric characteristics as he drives along a roadway. Thus, we may define design consistency as the "condition which exists when the geometric features of the roadway are consistent with the operational characteristics as determined by what the driver expects of the roadway ahead and what he is willing to accept in terms of operational quality." Leisch describes design consistency as the achievement of an appearance--a message conveyed by the visual appearance of the facility which is consistent with the desired operational characteristics of the highway. It is human factors at work.

3.2.12 Types of Consistency Problems

Design consistency describes the positive approach of achieving operational quality in the design of new facilities or in the redesign of existing facilities. More important, perhaps from the safety viewpoint on existing facilities, is the identification and correction of inconsistencies. Inconsistencies normally relate to:

1. Changes in design speed

2. Changes in cross section, and

3. The incompatibility in geometry and operational requirements.

Changes in Design Speed. Changes in design speed may occur on a given highway due to the highway being built in separate projects over an extended time period. This is due to a number of factors, including changes in standards, agency policy, re-classification of the facility, and availability of funding. Other inconsistencies may be only apparent from policy decision, such as the selection of a design speed which is not consistent with the terrain, the development, and the character of the roadway. For example, a design speed of 50 mph (80 km/h) may be chosen for a local road that has all other appearances of a high-speed rural highway. If the design consists of long tangents with occasional curves at critical design curvature, then the roadway presents an "apparent" design inconsistency to the driver. The long tangents convey an almost unlimited operating speed, whereas the curves impose severe restrictions. It is important that design speed be selected on the basis of anticipated operational characteristics rather than available funds. Historically, economic conditions are far more temporary than highway alignment.

Changes in Cross Section. There are two major types of design inconsistencies relative to cross section: "point" or discrete

inconsistencies, and a general incompatibility between cross section and alignment.

Significant changes in cross section at a specific point generally are characterized by changes from 2-lane to multi-lane, other lane drops, the elimination of shoulders, narrowing lane widths, narrow bridges, and others. These are generally the result of changes in standards, policies and funding between successive projects. These must be dealt with through a combination of design and traffic control measures.

The second form of cross section inconsistency is the result of upgrading the highway cross section without upgrading alignment. Sometimes pavements are widened and shoulders added on an older 2-lane highway to accomplish an "economical" improvement. The wider cross section on an old alignment conveys a conflicting message to the driver: He develops an expectancy based on the visual aspects of the cross section because it is more apparent than the alignment.

Incompatibility of Geometric Features and Operating Requirements. There are several elements of the design process which appear to have been done for the purpose of fitting the geometric pieces together conveniently and economically rather than for the purpose of satisfying operational requirements. A classic illustration of this inconsistency is the direct entry ramp or turning roadway. The unquestionable objective of this design concept was to permit vehicles to enter or merge with a moving traffic stream without coming to a complete stop. However, the direct entry ramp presents only the "go," "no-go" choice and forces the driver to come to a complete stop if a gap in the traffic stream is not immediately available. Once the vehicle has stopped, then the function of the ramp or turning roadway has been defeated. There are numerous other such inconsistencies that the designer must deal with through operational analysis.

It is incumbent upon the design engineer and the traffic operations engineer to cooperatively and in a coordinated effort identify existing inconsistencies and correct them, and to establish design practices which will minimize to every extent possible inconsistencies, or, from a positive approach, maximize the consistency between geometrics and operations.

3.2.2 METHODS OF IDENTIFYING INCONSISTENT DESIGN

For the most part, the methods that are available for the identification of inconsistent designs are the same as those presented earlier in the model safety improvement program. Inconsistent design relates to safety problems, and accident records are used extensively to identify the location, the nature, and the magnitude of the problem. Also, inconsistencies may result in operational problems which can be identified through measures of operational efficiency such as travel time, speed, delay, etc. The reader should refer to the section on safety improvement programs for a detailed treatment of most of the methods of identifying inconsistencies. The discussion here will be limited to those methods that are unique to this section.

In a recent project dealing with Highway Geometric Design Consistency Related to Driver Expectancy (1), Messer suggested several traffic operations measures that can be used to detect inconsistencies. They are:

1. Speed
2. Speed changes
3. Lateral placement
4. Erratic maneuvers
5. Driver behavior studies

One of the most practical methods of identifying inconsistencies is through an operations analysis. Through an operations analysis, it is determined what level or quality of operation is expected or should be expected by the driver, and field observations are made to determine whether the roadway geometry permits the expected operational quality.

Skewed Speed Distributions. Traffic speeds at a point on the highway are normally distributed. That is, a sample of speeds will display the characteristic bell shaped curve when conditions are normal. When they are not normal, skewed distributions occur (2). When speeds are skewed to the left, as shown in Figure 3.2.1, then this may mean that a geometric condition is as bad or worse than it appears to be. Also, it could mean that the speed restrictions of the geometric condition are below what is desired by the drivers. The flatness of the distribution curve on the right indicates a high frequency of speeds at the physical limit. It also indicates that many drivers would drive faster if permitted.

A skew to the right indicates that the geometric condition appears worse than it actually is. A disproportionate number of drivers slow down to a given speed, whereas others who are familiar with the location do not. This results in a large number of speeds at the "apparent" physical limit.

3.2.3 METHODS OF ACHIEVING DESIGN CONSISTENCY

There are several methods of achieving design consistency, including the conventional graphical methods, perspective plotting techniques, and modeling.

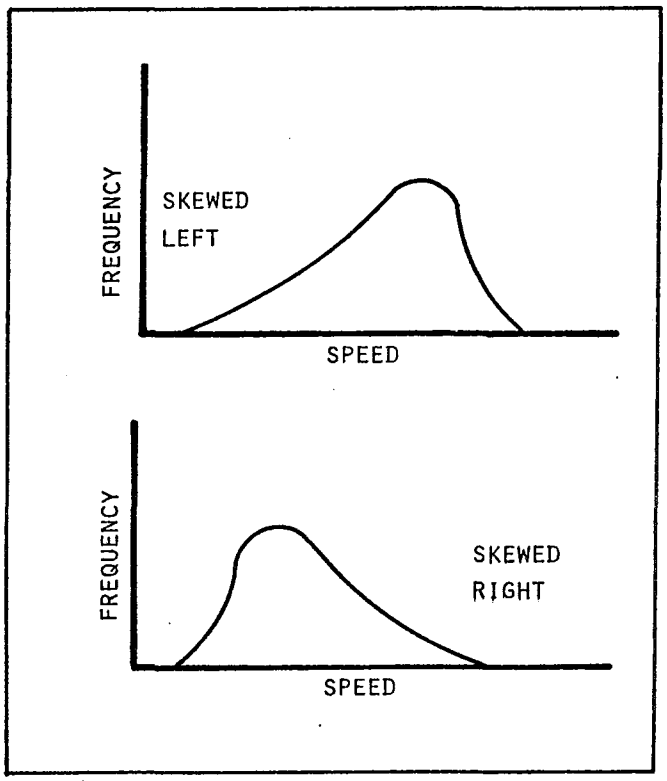

Figure 3.2.1 Skewed Speed Distribution

3.2.31 Conventional Methods

Typically, the designer uses a combination of plan view and profile of the roadway plus extensive experience in design to help him to visualize the finished product. Much helpful information is included in the Blue Book and the Red Book in the forms of guidelines for the coordination of horizontal and vertical alignment. Still, these are only guides to help the designer in gaining experience. Even the experienced designer needs more than simply a plan and profile. Leisch (3) utilizes an operational profile in which he plots the operating speed of automobiles and trucks in conjunction with normal plan and profile data. This approach, which he calls his design speed application, will be covered subsequently.

3.2.32 Driving Expectancy Checklist

A driving expectancy checklist prepared by Rowan and Woods (4) and published by AASHTO is one approach available to the designer so that he may check to make sure that all operational requirements are included for consideration in the design process. It is not a replacement for experience and design; it serves to assist the designer in considering all operational requirements.

3.2.33 Perspective Plotting

There are numerous perspective plotting techniques available to the designer to assist him in visualizing the finished product. One such system, the RDS, Roadway Design System (5), generates, among many other things, a series of perspective plots that are built from alignment and cross section data. Again, perspective plotting does not replace experience and technique in road design; it simply serves as a tool to help the designer do a better job in applying his experience and technical expertise.

3.2.34 Modeling

Physical modeling is used on many complex interchanges and other complex designs. It is used more to fit proposed designs to an existing land area than to assure operational responsive geometrics. Modeling is used frequently as a public relations tool to sell projects to the public and/or political factions.

Modeling is difficult to use in analyzing the operational effects of geometry because of the difficulty of viewing the model from the drivers' viewpoint. Some have developed optical viewing devices similar to periscopes so that the designer may, through a series of prisms, view along the model roadway and achieve some semblance of driver perspective. Generally, however, models are too expensive to construct for this single purpose. Perspective plotting will accomplish basically the same thing at a much lower cost.

3.2.4 ILLUSTRATIVE EXAMPLES OF DESIGN CONSISTENCY

3.2.41 Design Speed Application

Design speed has been used for 40 years as a means of achieving consistency in geometrics. It has served well as a general guide, but its major effect has been dependent upon how it was applied by the designer. In some instances, there is a lack of assuring operational consistency dealing with a route as a whole, including the relationship between cars and trucks. Jack Leisch, in his paper, "New Concepts in Design Speed Application as a Control in Achieving Consistent Highway Design," presented at the 56th Annual Meeting of TRB, January 1977, has suggested new concepts in design speed applications. These new concepts consist of the following:

1. 10 mph (15 km/h) rule -- Reduction of design speed should be avoided where possible, and limited to 10 mph (15 km/h) where absolutely necessary. Potential passenger car speeds should not be permitted to vary more than 10 mph (15 km/h), and potential truck speeds should not be permitted to drop more than 10 mph (15 km/h) in the main lanes (see Figure 3.2.2).

2. Speed profile -- Potential passenger car and truck speeds are charted along with the plan and profile of the proposed roadway (See Figure 3.2.3).

DESIGN PRINCIPLE
THE 10 MPH *(15 KPH)* RULE

- WITHIN A GIVEN DESIGN SPEED, POTENTIAL AVERAGE PASSENGER CAR SPEEDS GENERALLY SHOULD NOT VARY MORE THAN 10 MPH *(15 KPH)*
- A REDUCTION IN DESIGN SPEED WHERE CALLED FOR NORMALLY SHOULD NOT BE MORE THAN 10 MPH *(15 KPH)*
- POTENTIAL AVERAGE TRUCK SPEEDS GENERALLY SHOULD BE NOT MORE THAN 10 MPH *(15 KPH)* BELOW AVERAGE PASSENGER CAR SPEEDS AT ANY TIME ON COMMON LANES

Figure 3.2.2 The 10 mph Design Rule *(Source: Leisch (3))*

Problems occur when low design speeds are used and the controlling features are dispersed along the highway such that drivers tend to speed up on flat tangent sections and then slow down to negotiate the controlling curvatures. When it is necessary to utilize the lower design speed, then all features of the highway should be designed in accordance with that speed.

Design speeds are sometimes too low for the driver's expectation and his judgment of the surrounding conditions. There are some basic values that should be adhered to wherever practicable. These are illustrated in Figure 3.2.4. Essentially, we should be designing for 70 mph (112 km/h) in flat, open terrain, 60 mph (96 km/h) in rolling terrain, and 50 mph (80 km/h) in urban areas. Of course, rugged mountainous terrain and highly urbanized areas may necessitate deviation from these values, but the designer should be required to justify without question any variations from these recommended values. Figure 3.2.5 presents Leisch's design speed designation concept.

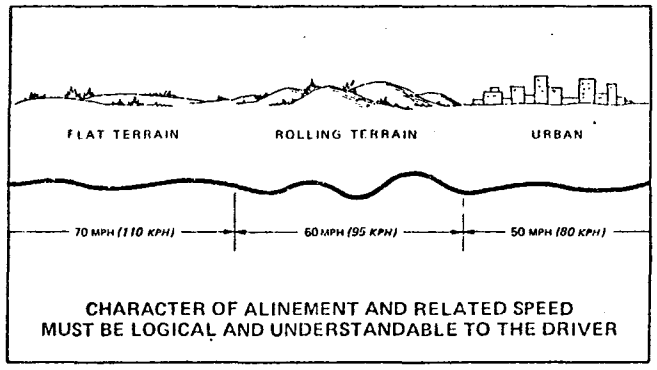

Figure 3.2.4 Design Speed As Related to Terrain [*Source: Leisch (3)*]

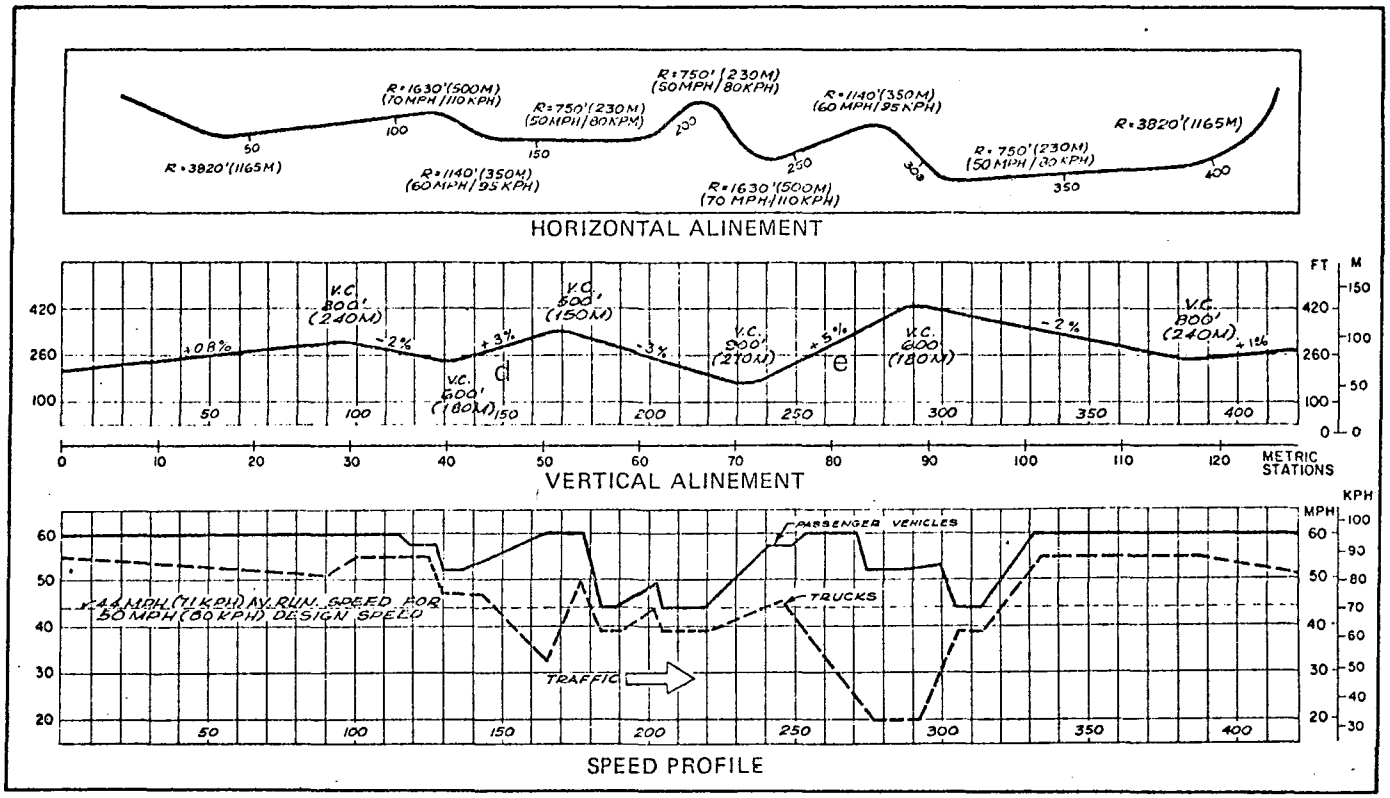

Figure 3.2.3 Example of Speed Profile *(Source: Leisch (3))*

DESIGN SPEED DESIGNATION CONCEPT

DESIGNATED DESIGN SPEED MPH	IMPERIAL UNITS	POTENTIAL INCREASE PERMISSIBLE WITHIN DESIGN SPEED MPH
30		30 TO 40 MAX.*
40		40 TO 50 MAX.*
50		50 TO 60 MAX.*
60		60 TO 70 MAX.*
70		70**
80		80**

KPH	METRIC UNITS	KPH
50		50 TO 65 MAX.*
65		65 TO 80 MAX.*
80		80 TO 95 MAX.*
95		95 TO 110 MAX.*
110		110**
125		125**

* ALINEMENT CONFIGURATION TO BE ADJUSTED, AS REQUIRED, TO LIMIT POTENTIAL SPEED TO NOT MORE THAN 10 MPH (15 KPH) ABOVE THE DESIGN SPEED DESIGNATION. APPROPRIATE DESIGN MEASURES SHOULD BE APPLIED TO SECTIONS OF "MAX." SPEED.

** FOR DESIGN PURPOSES, NO POTENTIAL SPEED INCREASE IS CONSIDERED FOR DESIGNATIONS OF 70 AND 80 MPH (110 AND 125 KPH); HOWEVER SPECIAL ATTENTION SHOULD BE ACCORDED TO SECTIONS INVOLVING LONG TRIPS AT HIGH, SUSTAINED SPEEDS.

Figure 3.2.5 [*Source: Leisch (3)*]

The key to applying Leisch's new application concept is the determination of operating speed at successive points along the route to facilitate plotting of the speed profile. As outlined in Leisch's paper, the basic characteristics of the speed profile are predicated on the following:

1. Low volumes (free-flow conditions)

2. Average (running) speeds of traffic

3. Favorable roadway conditions--daylight, good weather, etc.

4. Average (running) speeds on horizontal curves, in accordance with the "low volume" relations of average running speed to design speed

5. "Top average speeds" representative of free moving vehicles (passenger cars and trucks) on relatively straight open sections of roads, outside the influence of any other geometric constraints

6. Separate average (running) speeds for passenger cars and for trucks plotted in juxtaposition; on near level grades, average truck speeds are assumed to be 5 mph (8 km/h) below average passenger car speeds

7. Truck speeds selected to be representative for a particular highway; weight-power ratio of 200 assumed as an average.

8. Acceleration and deceleration for passenger cars predicated primarily on and extrapolated from 1965 AASHO Geometric Design Policy

9. Acceleration and deceleration for trucks compiled from 1965 Highway Capacity Manual, 1965 AASHO Geometric Design Policy, and 1972 FHWA Dynamic Design for Safety

As can be noted, the speed profile consists basically of three speed elements: speed on curves; top average speed on the straight portions of the highway; and truck speeds (3). The process of developing the speed profile utilizes the predetermined values and elements as identified below:

a. Top average speeds of passenger cars--guidelines, Tables 3.2.1 and 3.2.2

b. Top average speeds of trucks--5 mph (or 8 km/h) below top average speeds of passenger cars

c. Average speeds of passenger cars on horizontal curves--Figure 3.2.6

d. Average speeds of trucks on horizontal curves--Figure 3.2.6, less 5 mph (or 8 km/h)

e. Deceleration of passenger cars approaching a horizontal curve--Figure 3.2.8

f. Acceleration of passenger cars departing a horizontal curve--Figures 3.2.7 and 3.2.9

g. Deceleration of trucks approaching a horizontal curve--Figure 3.2.10

h. Acceleration of trucks departing a horizontal curve--Figure 3.2.11

i. Deceleration of trucks operating on grades--Figure 3.2.12

To provide a better understanding of how the various charts and related material are used in constructing the speed profile, the following illustration extracted from Reference 3 is presented for this purpose.

An application of the speed profile to an actual problem involving approximately an 8-mile section of highway is demonstrated in Figure 3.2.13. The horizontal and vertical alignment of the existing highway (except for the dash lines at points a, b and c) are shown in the two upper blocks of the figure. The variable horizontal curvature (which originally was constructed on the basis of a

TABLE 3.2.1

AVERAGE SPEED OF FREE-MOVING VEHICLES
Compiled from Highway Statistics-1972

Type of Highway	Average Speed - mph Pass. Car	Truck	Bus	All
Rural Interstate	66.6	59.8	64.0	64.9
Rural Primary	58.6	54.0	57.5	57.5
Rural Secondary	53.3	49.8	51.2	52.6
Urban Interstate	57.5	53.0	56.1	56.4
Urban Primary	43.4	40.7	39.0	42.8
Urban Secondary	40.0	38.6	37.4	39.9

TABLE 3.2.2

GUIDELINES FOR TOP AVERAGE SPEEDS OF PASSENGER CARS
FOR VARIOUS TYPES OF HIGHWAYS*

Type of Facility	Highway Quality and Condition Favorable		Moderate	
	mph	(kph)	mph	(kph)
Rural Highways				
Interstate	65	(100)	60	(95)
Primary - Main	60	(95)	55	(90)
Primary - Intermediate	55	(90)	50	(80)
Secondary	50	(80)	45	(70)
Urban Highways				
Interstate	60	(95)	55	(90)
Arterial - Main	50	(80)	45	(70)
Arterial - Intermediate	45	(70)	40	(65)
Secondary - Feeder	40	(65)	35	(55)

* Representative of low-volume, free-flowing conditions on open, near level and straight highways.

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SPEED-CURVATURE RELATIONSHIPS

PERMISSIBLE AVERAGE RUNNING SPEEDS ON CURVES OF GIVEN RADII AT LOW-VOLUME, FREE-FLOW CONDITIONS – APPLICABLE TO PASSENGER CARS

IMPERIAL UNITS

V – DESIGN SPEED, MPH	30				40				50				60			70			80
CORRESPONDING AVERAGE RUNNING SPEED, MPH																58	60	62	64
													52	54	56	58	60	62	64
									44	46	48	50	52	54	56	58 • →			
					36	38	40	42	44	46	48	50	52	54 • →					
	28	30	32	34	36	38	40	42	44	46	48 • →								
D_c – MAX. DEGREE OF CURVE*	23.0	19.1	16.4	14.1	12.5	10.8	9.5	8.5	7.5	6.8	6.2	5.6	5.0	4.5	4.0	3.5	3.1	2.8	2.5
R_c – MIN. RADIUS, FEET	250	300	350	400	460	530	600	680	760	840	930	1030	1140	1280	1440	1630	1820	2030	2240

METRIC UNITS

V – DESIGN SPEED, KPH	50			65			80			95			110		125
CORRESPONDING AVERAGE RUNNING SPEED, KPH													92	96	100
										82	86	90	92	96	100
							70	74	78	82	86	90	92 • →		
				58	62	66	70	74	78	82	86 • →				
	46	50	54	58	62	66	70	74	78 • →						
D_m – MAX. DEGREE OF CURVE**	21.0	17.8	14.4	11.9	10.4	8.9	7.5	6.8	5.9	5.1	4.6	4.0	3.7	3.2	2.7
R_m – MIN. RADIUS, METERS	80	95	120	145	165	195	230	255	290	320	375	425	465	540	650

* $D_c = 5729.5780 \div R_c$ (BASED ON CENTRAL ANGLE SUBTENDING 100-FOOT ARC)

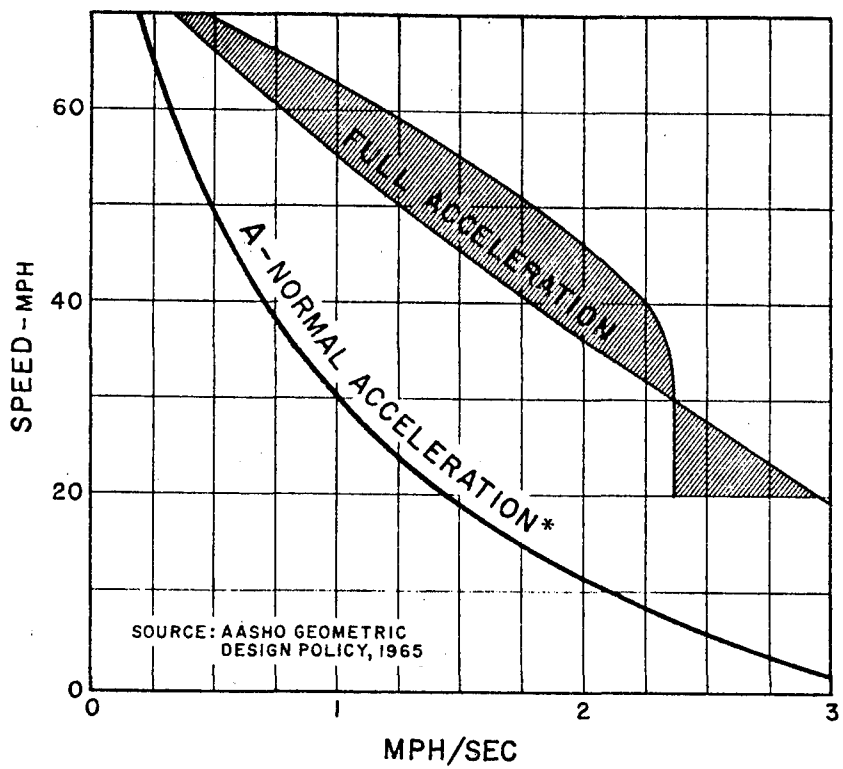
** $D_m = 1718.8734 \div R_m$ (BASED ON CENTRAL ANGLE SUBTENDING 30-METER ARC)

TABULAR VALUES ARE BASED ON AN AVERAGE MAXIMUM SUPERELEVATION RATE OF .08.

• → FOR A DESIGNATED OR ESTIMATED DESIGN SPEED, ANY LARGER RADII BEYOND THE ARROW ARE ASSUMED TO HAVE THE SAME AVERAGE RUNNING SPEED AS AT THE ARROW.

ADAPTED FROM: AASHO GEOMETRIC DESIGN POLICY, 1965

Figure 3.2.6 [*Source: Leisch (3)*]



ACCELERATION RATES AT VARIOUS SPEEDS OF PASSENGER CARS – MPH/SEC

*Curve "A" relates to acceleration departure of vehicles from intersections or acceleration of vehicles entering a highway

ACCELERATION RATES OF PASSENGER CARS FROM A POINT ALONG A HIGHWAY WHEN ALINEMENT AHEAD BECOMES CONDUCIVE TO HIGHER SPEED

PERCENTAGE OF ACCELERATION RATES FROM CURVE "A" ASSUMED APPLICABLE FOR CONDITION WHEN			
S_R–REQUIRED SPEED REDUCTION PRIOR TO LIMITING CURVE POINT 1 TO 2	VIEW PERCEIVED BY DRIVER DEPARTING LIMITING CURVE AT POINT 3		
	UNRESTRICTED	SEMI-RESTRICTED	RESTRICTED
<10 MPH	50%	40%	20%
10-20 MPH	75%	60%	30%
>20 MPH	100%	80%	40%

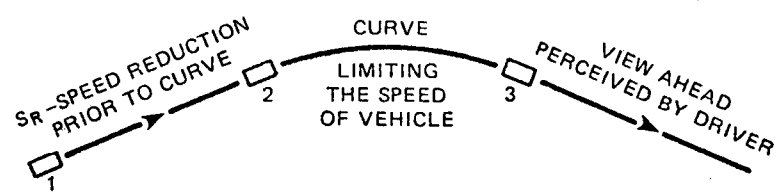
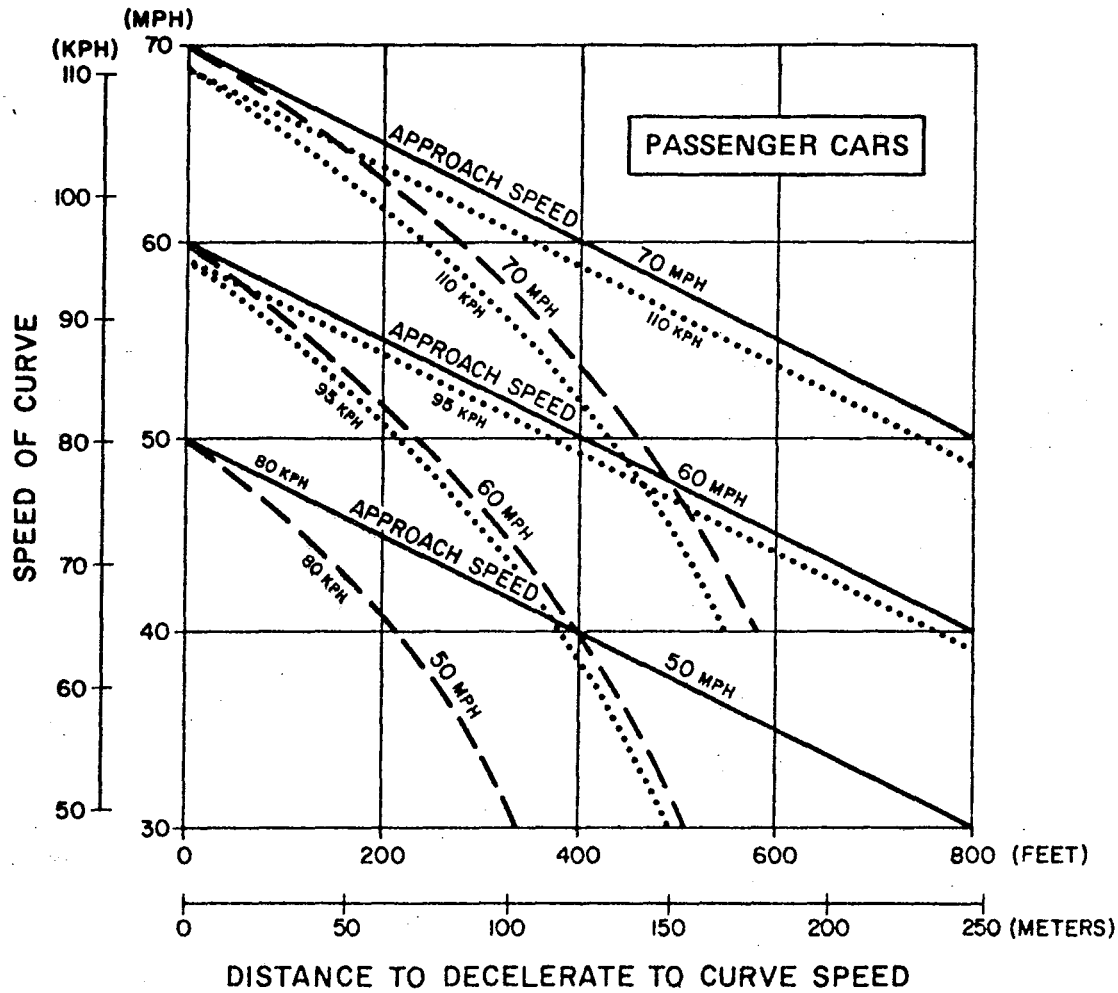


Figure 3.2.7 Basis for Passenger Car Accelerations for Continuous (Uninterrupted-Flow) Operation on Highways
[*Source: Leisch (3)*]



LEGEND

Deceleration for required speed reduction of 15 MPH *(25 KPH)* or less (based on deceleration in gear)

Deceleration for required speed reduction of 20 MPH *(30 KPH)* or more (based on "light" braking)

SOURCE:
AASHO Geometric Design Policy, 1965

Figure 3.2.8 Deceleration of Passenger Cars Approaching a Curve which Limits the Speed [*Source: Leisch (3)*]

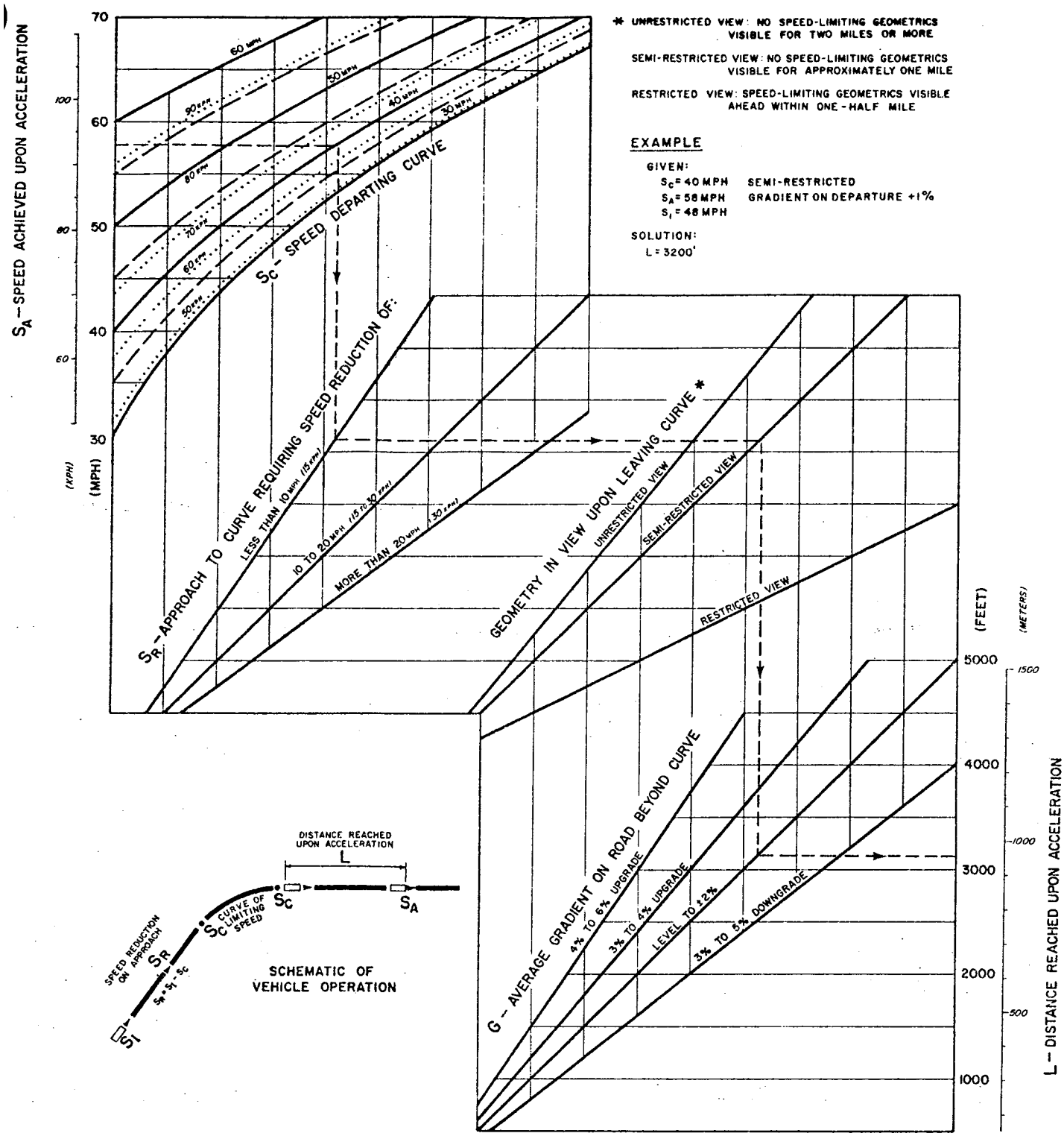
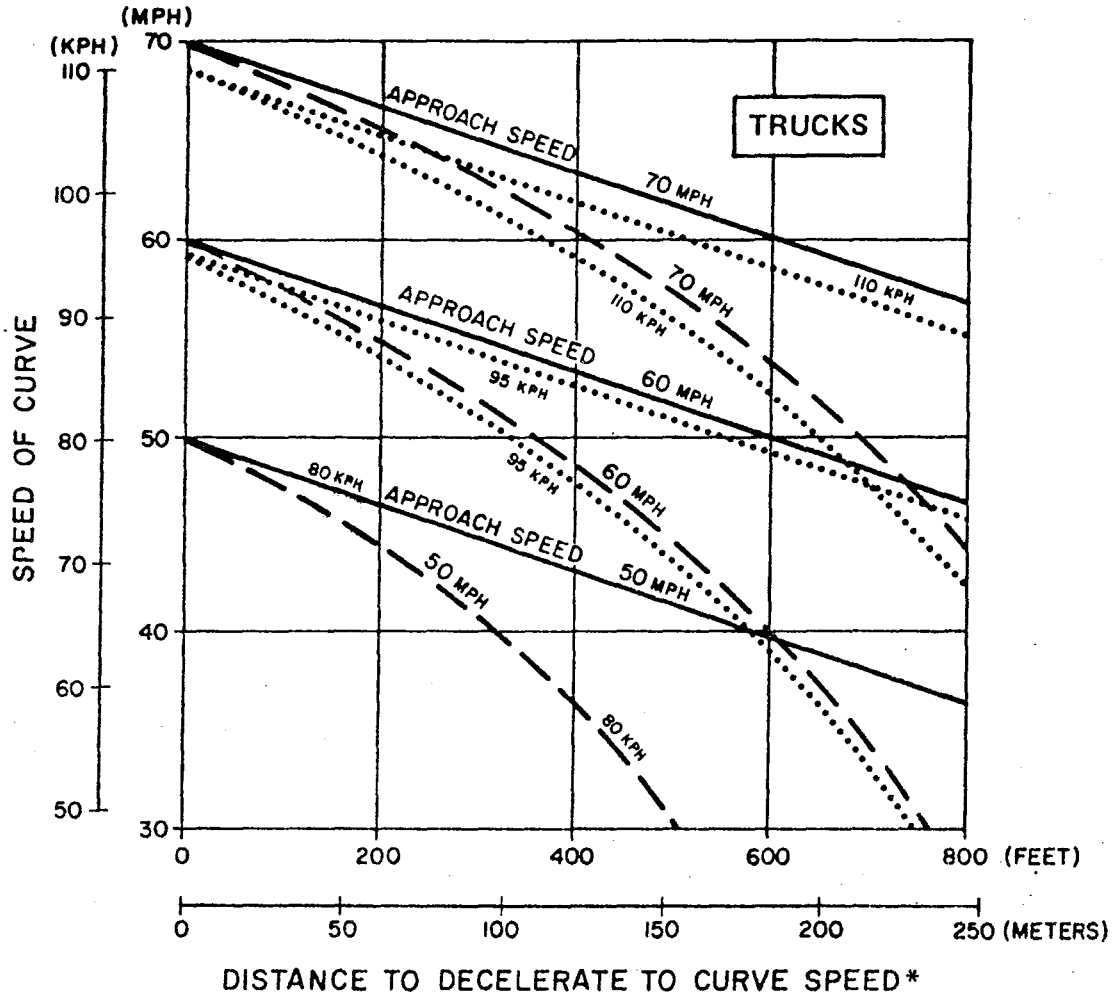


Figure 3.2.9 Acceleration of Passenger Cars Departing a Curve which Limits the Speed [*Source: Leisch (3)*]



LEGEND

Deceleration for required speed reduction of 15 MPH *(25 KPH)* or less (based on deceleration in gear)

Deceleration for required speed reduction of 20 MPH *(30 KPH)* or more (based on "light" braking)

SOURCES:

AASHO Geometric Design Policy, 1965

Dynamic Design for Safety, FHWA, 1972; Section 3A — Three-Dimensional and Dynamic Consideration of Highway Alinement and Cross Section (Sub-heading — Sight Distance)

* Taken as 1.5 times Values used for Passenger Cars.

Figure 3.2.10 Deceleration of Trucks Approaching a Curve which Limits the Speed on or Near Level Conditions [*Source: Leisch (3)*]

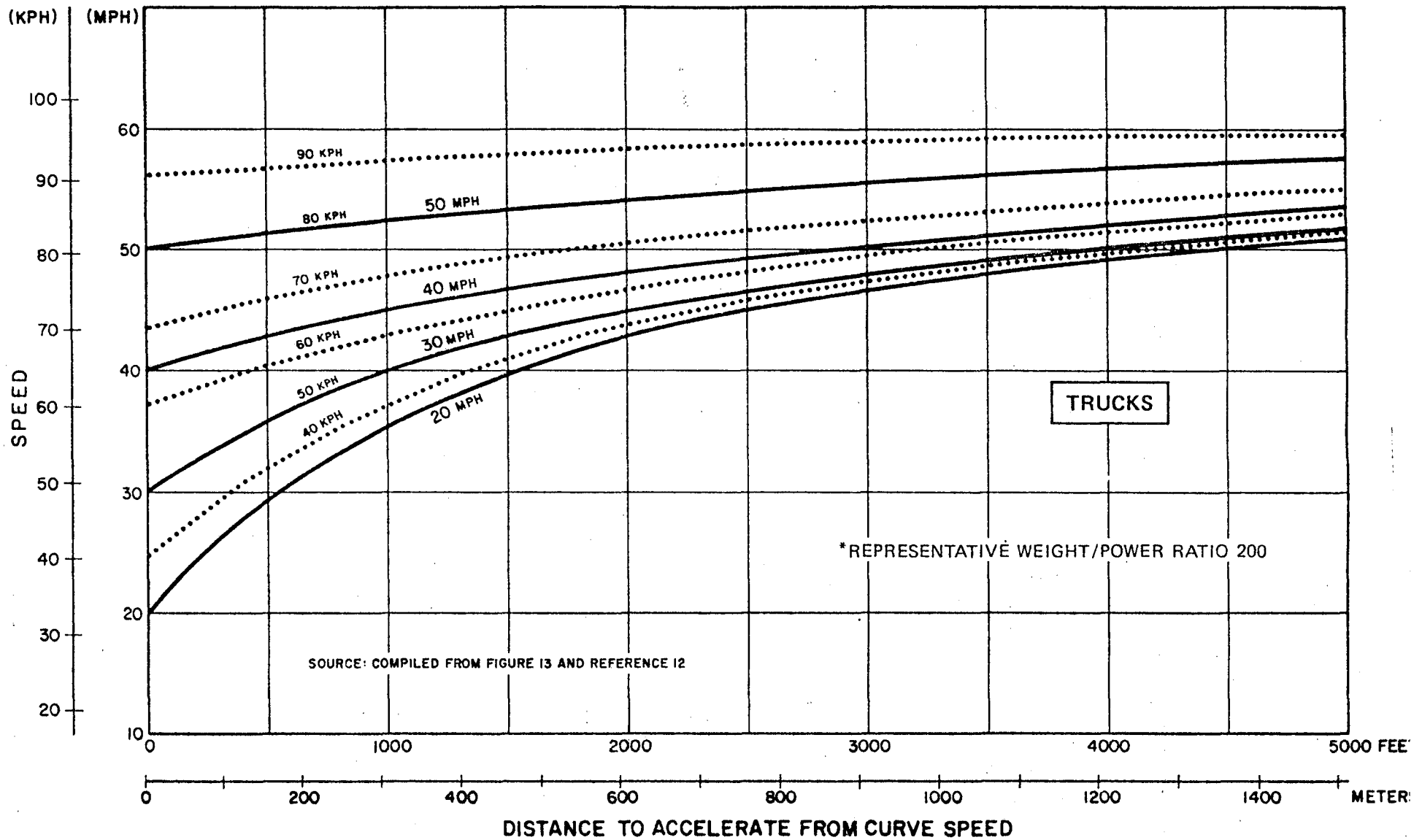


Figure 3.2.11 Acceleration of Trucks Departing a Curve Which Limits the Speed on or Near Level Conditions [*Source: Leisch (3)*]

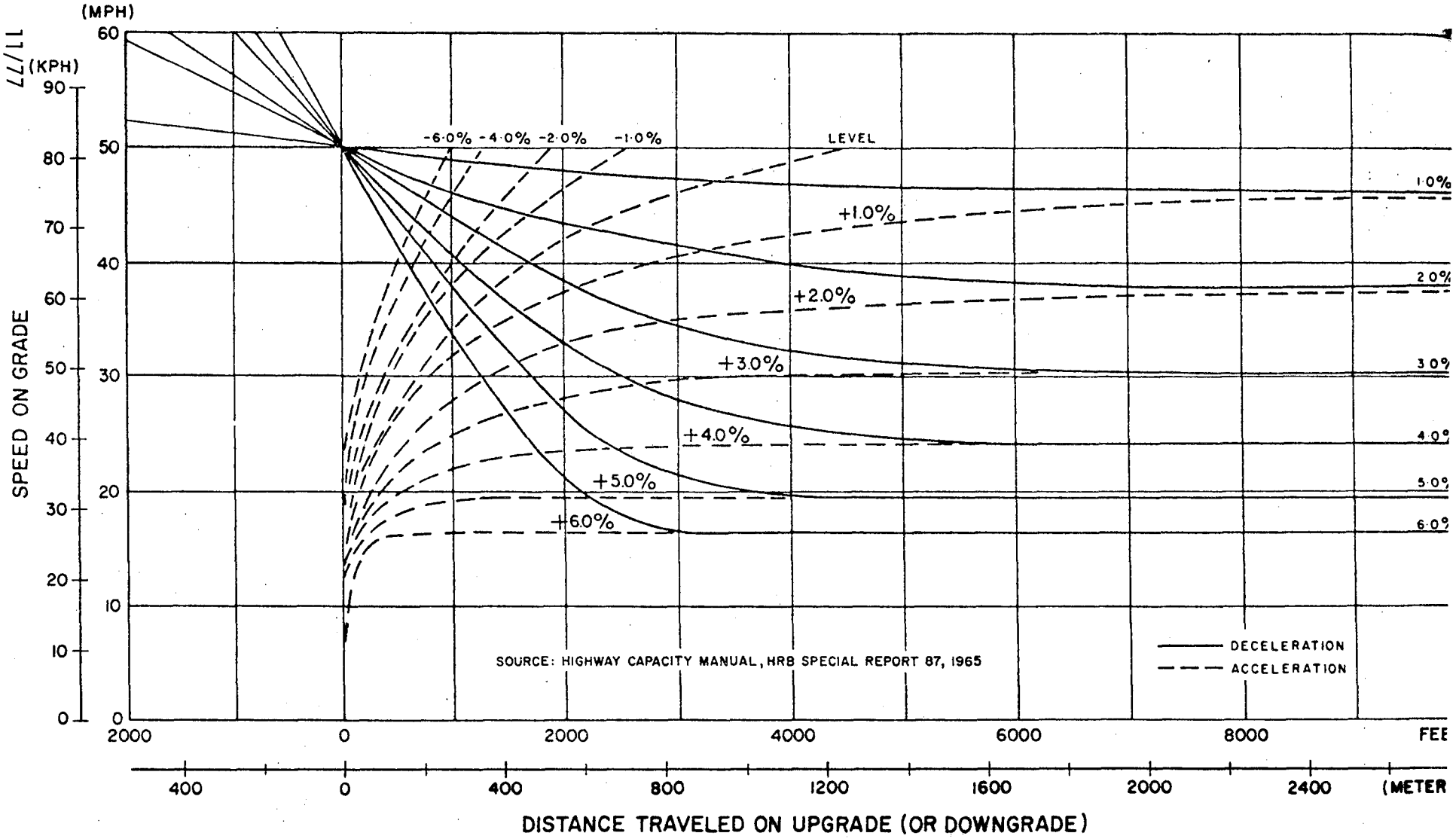


Figure 3.2.12 Speed-Distance Relations for Operating Trucks on Grades (Weight-Power Ratio: 200) [*Source: Leisch (3)*]

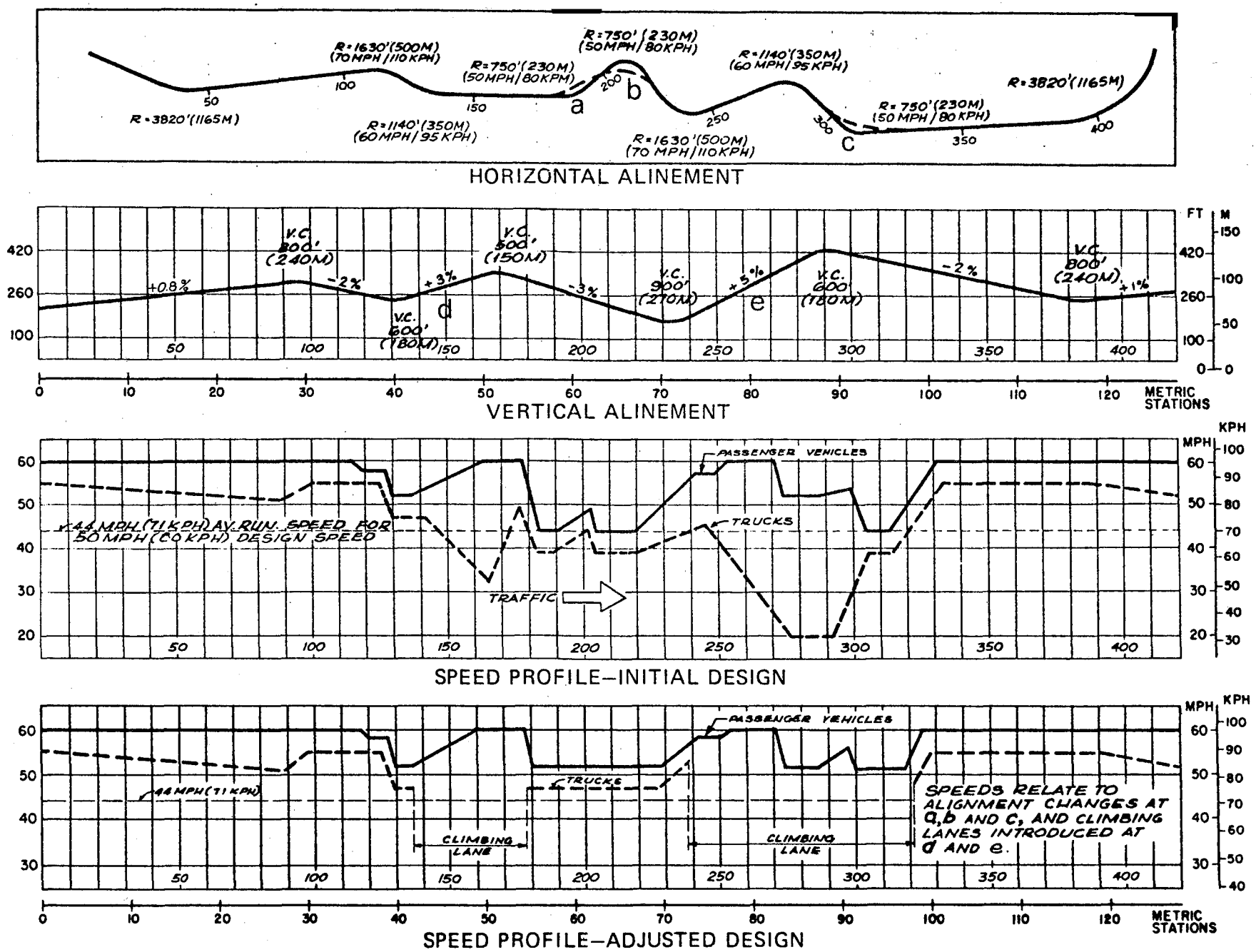


Figure 3.2.13 Demonstration of Speed Profile Application
[*Source: Leisch (3)*]

design speed of 50 mph (80 km/h), in combination with the undulating profile, produces highly variable speeds as demonstrated in the third block of the figure. The speed profile was constructed in accordance with speeds and acceleration data obtained from Figures 3.2.6 to 3.2.12.

The speed profile for the existing condition (shown for travel in one direction) reveals the high inconsistency of the design and its resulting operations. The speed trend for passenger cars indicates considerable variations requiring a series of decelerations and accelerations, with changes in average speed as much as 15 mph (24 km/h). Even more erratic is the truck speed profile which shows critical variations not only in its speed but in the relative speeds between passenger cars and trucks.

The "trouble-spots" can be spotted readily by examining the speed profile. The problems become apparent with respect to the character of the alignment to achieve a more appropriate design speed and a more consistent operation for passenger vehicles, as well as the combination of the horizontal and vertical alignment to improve truck operations. The lack of conformity to the 10 mph (15 km/h) rule is quite obvious on this existing facility. The situation at any one point or at a given short section of the facility is quite obvious. The location of the relatively rapid changes in speed can be noted readily, along with the more dangerous speed changes of more than 10 mph (15 km/h). The most obvious are the locations where the speed between passenger cars and trucks exceeds 10 mph (15 km/h). At several locations, the differences are of the order of 25 and 30 mph (40 and 48 km/h, respectively).

Some situations are of particular concern. For example, between Stations 140 and 155, trucks are rapidly decelerating while passenger cars are accelerating. Such a condition is most hazardous, and in this case particularly so, since the average speed differential between the two vehicle types over this section of road is nearly 20 mph (32 km/h). Another undesirable situation may be noted between Stations 300 and 305 where passenger cars are decelerating to negotiate a horizontal curve, while trucks are continuing to accelerate on a downgrade.

These are just some of the points of concern which are quite obvious. A methodical study of the speed profile provides a thorough insight to the operational characteristics of the highway and in spotting the inconsistencies of design and potentially hazardous locations. Although not mentioned in this writing, a plot of a sight distance profile in consonance with the speed profile provides further and added perception to the problem and a more thorough means of evaluation and diagnosis for improvement.

A demonstration of this analysis by the use of speed profile on the existing highway immediately reveals the means for remedial measures. Any number of improvements come to mind, depending largely upon socioeconomic considerations. An example of a rather minimal improvement, but complying with the design principle of the 10 mph (15 km/h) rule is demonstrated in the same figure as points a, b and c on the horizontal alignment, and at points d and e on the vertical alignment (profile). The improvement indicates the realignment of three horizontal curves (at a, b and c) from a design speed of 50 mph (80 km/h) to a design speed of 60 mph (96 km/h)(a change in radius of from 750 feet (230 m) to 1140 feet (350 m)), and in profile the introduction of a truck climbing lane at two locations (d and e), at approximate Stations 137 to 178 and 238 to 322.

The suggested improvements were selected and coordinated with the development of an adjusted and revised speed profile, as shown in the bottom block of Figure 3.2.13. The new speed profile indicates the compliance with the 10 mph (15 km/h) rule. By the indicated adjustments, the design speed designation has been changed from 50 mph to 60 mph (80 to 96 km/h), providing a more consistent design and uniform operation. As shown, the speed variance of passenger cars is within 10 mph (16 km/h), and truck speeds on common lanes are not more than 10 mph (16 km/h) below the speed of passenger cars. A similar speed profile was developed for travel in the other direction (not shown) which required further profile adjustments. This illustrates a simple example of what can be accomplished with the application of the speed profile. For more complex problems where extensive improvements or road relocations are encountered, or where new highways are proposed, including a series of alternative locations and highway type variations, the speed profile becomes even more significant in optimizing design and operational conditions.

3.2.42 Consistency in Cross Section

As mentioned previously, inconsistencies in cross section may fall into two major categories: those that occur at a point, and those in which there is a general inconsistency between cross section and alignment. Examples of point inconsistencies are reduction in the number of lanes, transitions to and from divided highways, narrowing of lanes and/or shoulders, and many more. There are several primary considerations in the treatment of point or discrete inconsistencies.

1. Satisfactory sight distance should be provided. The driver must be able to see the geometry and respond to its requirements. Some of the special considerations such as Decision Sight Distance, to be discussed in Session 3.4, should be utilized.

2. The inconsistency should be transitioned within the realm of acceptable driver performance. For example, taper lengths should be adequate when transitioning from a divided to an undivided section.

3. Decisions and responses should be distributed such that the driver workload is not exceeded. For example, the transition to a 2-lane undivided from a divided roadway should not occur precisely at the point where the number of lanes is reduced. The reduction in lanes should occur prior to the elimination of the median.

In the other case, the general incompatibility between cross section and alignment is correctable through administrative and programming decisions rather than specific design processes. In these times of austerity programs, this inconsistency is more likely to occur. As highway agencies attempt to achieve maximum results from the resources available, the comparatively lower cost of upgrading the cross section without changing the alignment will become increasingly attractive. To prevent the occurrence of this inconsistency, design supervisors must stress the importance of design balance in the formulation and review of safety improvement programs.

3.2.43 Coordination of Horizontal and Vertical Alignment

The Blue Book presents general design controls relative to the combination of horizontal and vertical alignment. These are presented on pages 212 and 213 of the Blue Book. Leisch, in an earlier paper, expands these general controls in a human factors treatment of the subject (6).

1. The horizontal and vertical alignments should be in balance. A generous, flowing alignment in one plane is not compatible with small and frequent breaks in the other.

2. Horizontal and vertical elements should coincide with profile elements--both with respect to position and length.

3. An important feature in achieving balanced alignment design is the proper proportioning of the tangent and curve elements of the roadway. In composition of the alignment, long tangent-short curve arrangements should be avoided in lieu of shorter tangents and longer curves, and, where feasible, approach gradual curvilinear form.

4. Compound curves should not be used where a sharp curve follows a long flat curve.

5. Sight loss, where the road disappears and reappears, should be avoided.

6. A sag curve on a long straight road appears to be a sharp break in the profile and should be avoided. The greater the distance to the sag when it first appears in view, the longer should be the sag curve. A general rule which may be applied is shown in Figure 3.2.14.

7. Generally, long sight distances are desirable, but extremely long views may be misleading to the driver.

8. The designer should impose a generalized limit to the number of alignment breaks that are visible to the driver at any one time. Normally, these should be limited to two breaks horizontally and three breaks vertically.

3.2.5 SUMMARY

One of the principal considerations in providing consistency in design is providing sufficient sight distance--the ability to see

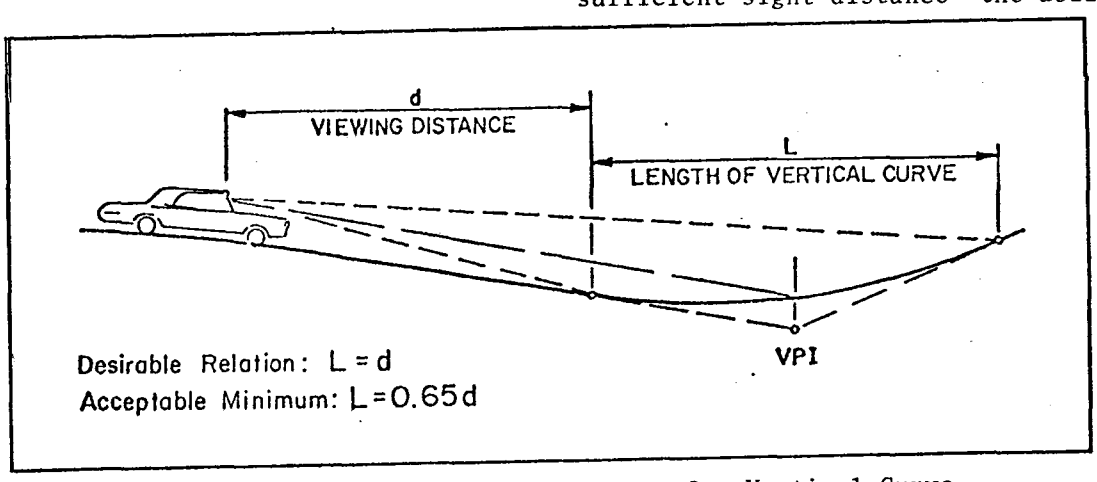


Figure 3.2.14 Viewing Distance to Sag Vertical Curve
[*Source: Leisch (6)*]

the road ahead. This is probably the most important single design element from the safety standpoint. The complexity of operating on modern highways, with the need to frequently process information while driving, requires even longer sight distances at certain locations. This, referred to as "Anticipatory Sight Distance," is particularly important at areas of potential hazard and at points requiring complex driver decisions. Areas or points of concern may involve intersections, interchanges, lane drops, railroad grade crossings, draw bridges, toll collection booths, design speed reduction zones, and others. Another term which may be used for this type of sight distance is "Decision Sight Distance." These terms will be discussed in greater detail in subsequent sessions.

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TOPIC 3 SESSION 3
HORIZONTAL ALIGNMENT AND SKID RESISTANCE

Objectives:

As a result of this session, the participant should be able to:

1. Identify the design and operational variables influencing dynamic vehicle behavior on horizontal curves.

2. Identify several alternatives available for the improvement of dynamic vehicle performance on horizontal curves,

3. Evaluate the relative effectiveness of these various alternatives.

3.3.1 INTRODUCTION

Horizontal alignment design is a very broad subject area that is treated quite thoroughly in the Blue Book, the Red Book, and numerous highway engineering textbooks. We will not attempt to cover the broad range of material that may be found in these references. Instead, we will concentrate on factors related quite specifically to the safety aspects of horizontal alignment and to more recent developments in technology.

Part of this presentation centers around the four variables included in the fundamental horizontal alignment relationship: curvature, speed, superelevation and friction. For a moment, let's explore some history relative to these four variables to see how and perhaps why they are related as they are today.

Early highways were designed for animal-drawn vehicles that rarely exceeded 8 mph (13 km/h) and, therefore, speed was not a design factor (1). Length and maneuverability of vehicles were prime considerations. The typical freight wagon pulled by four horses was approximately 50 ft (15 m) in length and required a turning radius of 105 ft (32 m) to remain within a 12 ft (3.6 m) roadway. If a shorter radius was required, curve widening was necessary.

Because of the low speed conditions, roadways were crowned rather than banked in the curved section. The objective was to remove the water from the roadway as quickly as possible, and there was no apparent need for banking the curves for wagon transportation.

As automobiles were developed and began using the highways at speeds up to 20 or 25 mph (32 or 40 km/h), the crowned sections on horizontal curves became objectionable (1). Drivers would cross over and take the inside of the curve in order to avoid the negative cross slope. Needless to say, this created a certain element of danger, particularly on highly-travelled roadways and those with blind corners.

One of the earliest responses to speed as a design criterion was the introduction of superelevation in 1912 by the state of New York. Superelevation was applied to curves sharper than R=500 ft. (150 m). The rates were one inch per foot (8.33 cm/m) for macadam surfaces and 5/8 inch per foot (5.2 cm/m) for concrete surfaces. These rates were fixed regardless of speed and regardless of curvature. The statement was made that variations in these rates were considered useless.

Concurrent with the introduction of superelevation, some states introduced curve widening. The objective was to widen the curve so that side tracking of the vehicles could be accommodated. Widening ranged from 3 ft (1 m) on flat curves up to 8 ft (2.4 m) for very sharp curves.

The first mathematical treatment of superelevation was provided in 1920 when Ludeke and Harrison developed a mathematical model (1):

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

It was noted that this model required up to 5 times the superelevation used in practice at that time. Thus, pavement skid resistance or friction took care of the difference.

The introduction to superelevation by the use of spirals came in 1926 when the General Motors Proving Ground built the first fully-spiraled roadway in this country. In 1929 the Bureau of Public Roads built the first public roadway with spiral curves in Virginia, the Mount Vernon Memorial Highway.

In the 1930's Ralph Moyer conducted studies to determine acceptable friction factors for horizontal alignment design. He blindfolded passengers in automobiles traveling on curves and attempted to determine the relationship between passenger comfort and coefficient of friction. He found that when the coefficient was less than 0.10, passengers could not sense clearly whether they were on a curve or tangent. At speeds that would provide a coefficient of 0.10 or greater, the passengers clearly could sense that they were

on a curve; and when the coefficient was increased to 0.30, the passengers felt distinctly uncomfortable. Thus, we find the development of friction factors was based on "seat of the pants" friction. The question now is, how did all of this affect our current approach to horizontal alignment design?

Over the years, we have established a strong relationship between speed and curvature whereby we choose to control curvature to accommodate speed. Further, we utilize superelevation to enhance driver comfort as well as to compensate for friction demand. Early in the development of design criteria, we selected friction factors on driver comfort instead of friction capability. Current technology has contributed much more, as we will see.

3.3.2 BASIC HORIZONTAL CURVATURE RELATIONSHIP

By assuming that the vehicle is in an instantaneously static position relative to the cross section of a curve, we may use engineering mechanics to derive the standard AASHTO centripetal force equation,

$$e + f = \frac{V^2}{15R} \qquad (1)$$

In metric units, the equation becomes

$$e + f = \frac{v^2}{9.8R} = \frac{V^2}{127R} \qquad (2)$$

Where: e = Superelevation
f = Coefficient of friction
V = speed in mph (km/h)
v = speed in (meters/sec)
R = feet (meters)

This equation is used principally to determine the minimum radius or maximum speed for critical design conditions. On the basis of optimum design as discussed earlier, we should avoid using critical levels for design wherever possible. The importance of this recommendation can be illustrated by considering some assumptions made in the application of the AASHTO Formula. Specifically, the application of the formula assumes that the design superelevation exists at the begining of the curve, and that the vehicle tracks precisely the design radius of curvature. Neither of these is assured under most conditions.

The full design superelevation is achieved at the beginning of the curve only when spiral transition curves are employed. When the 2/3 - 1/3 practice is applied, only two-thirds of the design superelevation is achieved at the beginning of the curve. What does this mean to the vehicle-roadway interaction? An additional demand - up to about .03 - is placed on the friction supply. This would be critical only when the friction supply is low, or a critical radius of curvature is selected. If we avoid these two situations through design decisions, a safer condition is achieved.

Another critical assumption in applying the AASHTO formula is that vehicles will track precisely the vehicle curve. A comprehensive research study (2) showed that a substantial percentage of drivers experience brief curve segments that are appreciably sharper than the design radius. This means that some of the margin of safety - that difference between available friction and accountable friction demand - is used to maintain traction through the sharper vehicle paths. What can the designer do about this? Principally, there are two available actions; avoid critical horizontal curves and maintain friction supply as high as practicable.

Perhaps we should point out typical friction supply-demand conditions. Figure 3.3.1 shows the design friction coefficients for the stopping maneuver. Presumably, these can be used to estimate available cornering friction supply. Superimposed on this same figure is the design friction coefficients for horizontal curve design. The difference in these two curves is the "margin of safety" that must accommodate the friction demands that are not predictable such as differences in vehicle paths, braking, acceleration, wind gusts, surface irregularities and others.

3.3.3 FRICTION: SUPPLY VS. DEMAND

The cardinal rule in horizontal alignment design is that friction supply should always be greater than friction demand. This is simply stated, but the question follows: To accomplish this rule, do we lower the demand or increase the supply? Generally, efforts in recent years have been to increase the supply rather than to modify demand. Perhaps there is a need for both as will be discussed herein. First, we should explore factors which influence supply and demand.

3.3.31 Factors Influencing Friction Supply

Factors which influence the available friction are listed and discussed briefly in this section.

- Weather Conditions
- Surface Texture (Macrotexture, Microtexture)
- Surface Drainage
- Cross Slope
- Tire Characteristics
- Vehicle Speed

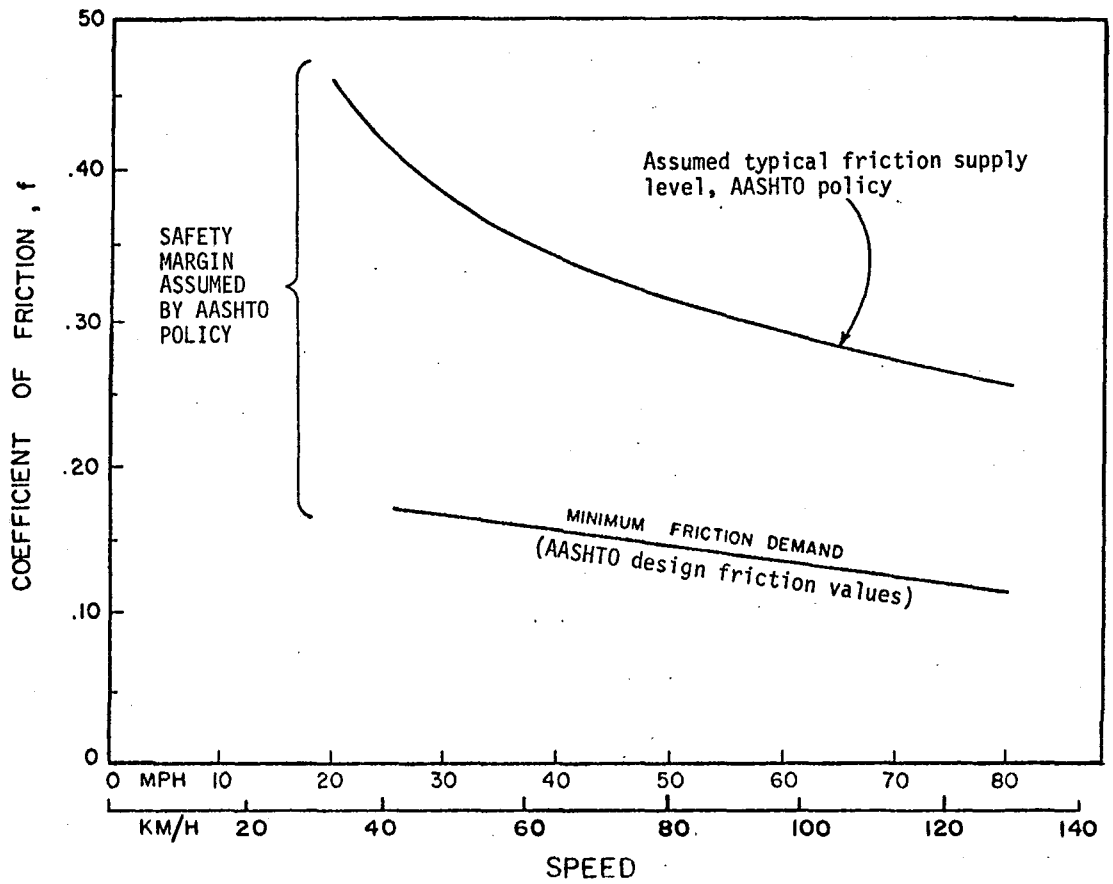

Figure 3.3.1 Typical Friction Supply - Demand Relationship

Weather Conditions. Weather has the greatest influence on pavement friction. For example, a dry pavement may have a coefficient of about 0.60. When it is wet the coefficient may range from .10 to .60 and when the wet surface freezes, the coefficient is reduced to about 0.10. It appears most practical to design for wet pavements, but ice should be given some consideration in areas where it is most prevalent.

Surface Texture. Macrotexture and microtexture are terms used to describe surface texture. Microtexture refers to the gritty aggregate surface that makes intimate contact with the tire and provides skid resistance. Macrotexture is the coarseness of the pavement surface which is provided by the size, shape and gradation of aggregate particles making up the pavement surface. Both are necessary for good, consistent friction characteristics. Each is achieved in different ways, and illustrations are included in a recent AASHTO "Guidelines for Skid Resistant Pavement Design" (3). The methods of achieving a desirable microtexture include:

- Using good non-polishing aggregates in paving mixtures, both bituminous and portland cement.
- Using the sprinkle treatment to place non-polishing aggregates in the upper layer of bituminous concrete pavements. This is accomplished by sprinkling the surface with 3-7 psy or 1.1 - 2.7 kg/m^2 of aggregate after placement of the hot mix and rolling in using conventional rolling methods.
- Using a silica sand mix on low speed roads
- Using synthetic aggregates that degrade slowly with use, always providing a new high skid resistant surface.
- Using a bituminous surface treatment (seal coat) with non-polishing aggregate.
- Controlling cement factor and water-cement ratio in portland cement concrete.
- Using air entrainment in portland cement concrete.
- Using finish methods that will result in a gritty surface.

Macrotexture also is achieved in several ways (3), including the following:

- Using proper proportions of angular non-polishing coarse aggregate

- Using bituminous surface treatment (seal coat) of suitable gradation.

- Using open-graded asphalt friction course to facilitate surface drainage.

- Texturing portland cement concrete pavements by brooming or dragging with tines or other apparatus. Sawing is used on in-place pavements.

Surface Drainage. Surface drainage is essential to avoid hydroplaning. Surface drainage is accomplished using a combination of macrotexture and cross slope. Also, open graded asphaltic paving mixtures are being used to facilitate subsurface drainage. The open-graded mix forms a porous pavement that water may penetrate, and an impervious layer under the pavement carries the water to the edge of the roadway.

Recent research indicates that the drainage potential for open graded mixes is relatively low (actually about 15 percent of the cross sectional area). This means that only low intensity rainfalls could be expected to drain through the open-graded course. In areas subjected to long periods of relatively low intensity rainfall, the open-graded surface course can be most effective. Where higher intensity rainfalls are involved, the open-graded surface course cannot substitute for adequate cross slope in providing pavement surface drainage.

The use of pavement grooving to provide a path for the water to escape from beneath the tire has been proven effective. The grooves can be provided initially by brooms or tines. Grooving of pavements with low skid resistance can substantially reduce the problem. Longitudinal grooves should be used to increase lateral stability and transverse grooves should be used when increased stopping frictions is desired.

Tire Characteristics. Tread depth, tread pattern and tire pressure are highly influential on the occurrence of hydroplaning. However, these factors are not within the realm of control by the designer.

Speed Effects. The speed of the vehicle has two major effects on friction supply. First, friction decreases as speed increases. This phenomenon is defined as "speed gradient." Second, speed is one of the primary factors in the cause of hydroplaning.

The "speed gradient" of friction, the relationship of skid number to vehicle speed, is very important to the design process, but perhaps even more important in the friction measurement process. Figure 3.3.2 illustrates the variation in frictional characteristics on roadways within a state. The percentile distribution could be very important information for decisions relative to providing highly skid-resistant surfaces.

3.3.32 Factors Influencing Friction Demand

According to the centripetal force equation,

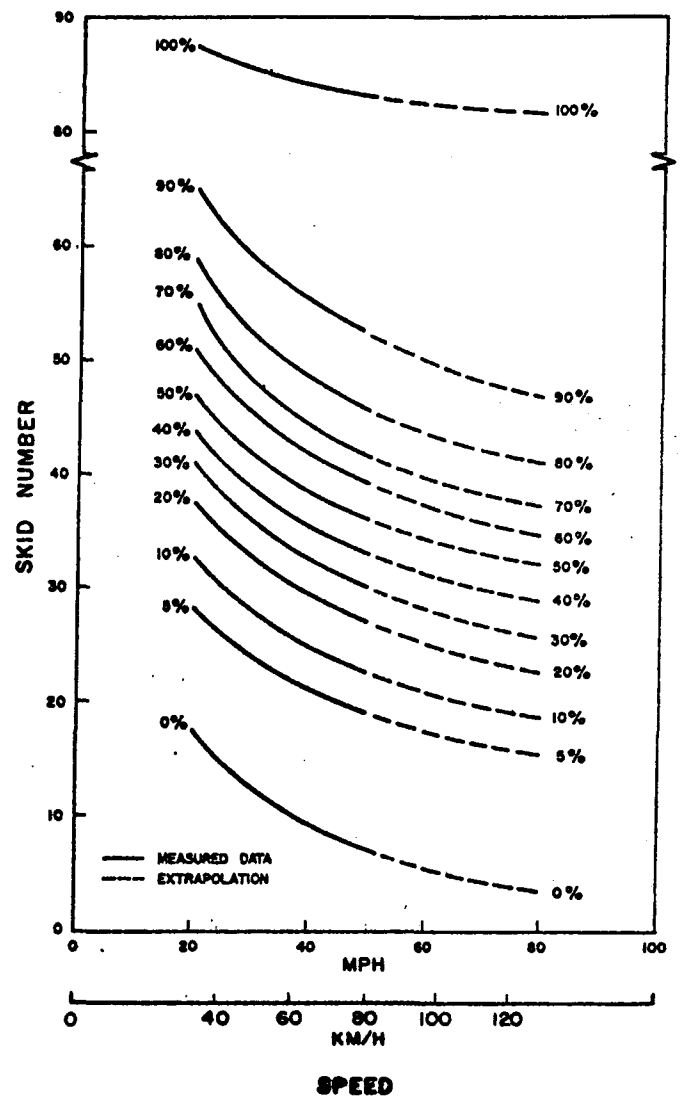

Figure 3.3.2 Percentile Distribution of Relation between Friction Capability and Vehicle Speed for 500 Pavements in One State

friction demand is a function of speed, curvature and superelevation but, in reality, there are several other factors that influence friction demand. These are:

- Tracking or cornering
- Vertical acceleration
- Longitudinal acceleration
- Braking
- Wind velocity

Tracking or Cornering. Application of the AASHTO centripetal force equation generally

assumes that the vehicle will conform precisely to the design radius of the curve when, in fact, a driver makes frequent steering adjustments in rounding a curve. Each steering adjustment constitutes a decreased radius of curvature, and these irregularities result in the driver traversing several successive curve segments, many of them much sharper than the design radius. This effect has been verified through research (2). A photographic study was conducted using motion pictures made from a vehicle following other vehicles through a horizontal curve. The motion picture photography permitted the measurement of the lateral position of a vehicle from the centerline as it moved through the horizontal curve. Then, by geometric computations, the incremental curvature of the vehicle path could be defined.

By using regression techniques, a relationship has been developed for relating the radius of vehicle path to the design radius of curvature for a given speed. Figure 3.3.3 provides the radius of vehicle path curve that may be expected with a given highway curve. This relationship is based on the estimated percentage of drivers that can be expected to exceed the indicated vehicle path.

What is the application of this added information? The designer may use it to justify the selection of horizontal curves with greater radius than is determined for minimum requirements.

Vertical Accelerations. Vertical acceleration effects on friction demand are the result of two types of behavior. Vertical curves reduce the net effect of the normal force (weight of the vehicle) and thus increase friction demand. This is not very critical on well-designed highways because of long vertical curves used for visual controls. Vertical accelerations also result

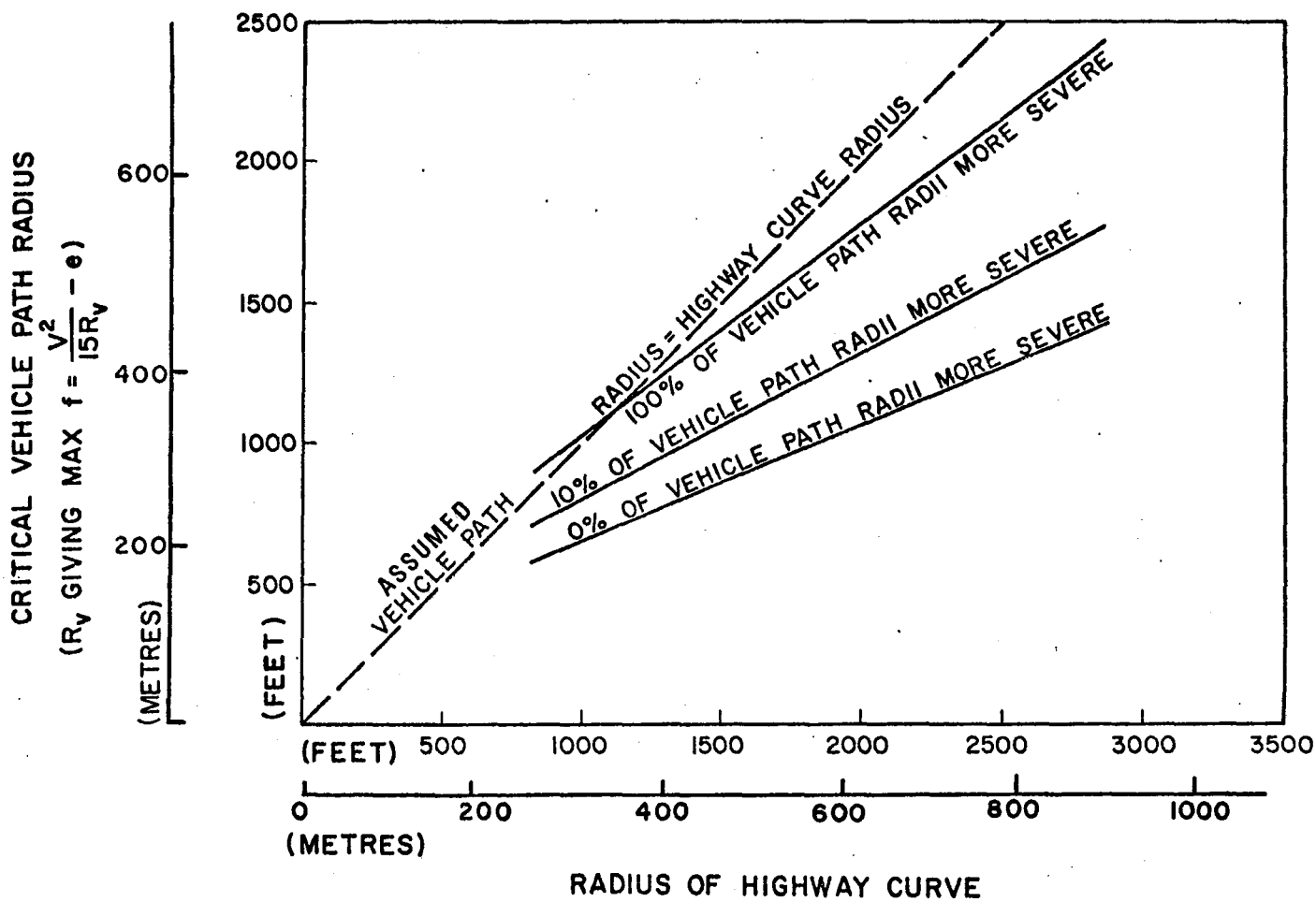

Figure 3.3.3 Percentile Distribution of Vehicle Path vs. Highway Radius

from surface irregularities. If such irregularities occur on a horizontal curve, their effects can be very significant. Correction of the pavement durability problem is the obvious answer.

3.3.4 MEASURING PAVEMENT FRICTION - THE SKID NUMBER AND SURFACE TEXTURE

The relative skid resistance of various pavements can best be determined through measurements. ASTM method E274 is recommended, with due attention given to standardized procedures and the calibration of skid testers. The FHWA has developed calibration procedures and has established skid calibration centers. Several portable pavement friction testers are available, but these are not sensitive to the effect of speed, and should, therefore, not be considered as substitutes for skid testers.

From measurements taken at two or more different speeds, a skid resistance-speed gradient can be established for estimating the loss in skid resistance at higher speeds and for evaluating the effects of coarse texture. The skid resistance-speed gradient is determined by dividing the change in skid resistance by the corresponding change in speed. A pavement with coarse macrotexture provides better relief of water pressure and hence a smaller skid resistance-speed gradient than a pavement with a finer macrotexture. Thus speed gradients can, in principle, be determined from measurements of macrotexture, but so far correlations have not been consistently high. Methods which have been used by various agencies include stereophotographic analysis (ASTM E559), the sand patch test, the grease smear method, and the outflow meter.

A simple depth gage such as that used to measure the tread depth of automobile tires may be used to measure the average depth of grooves on tine finishes and grooved pavement surfaces. A typical speed vs. skid number relationship is presented in Figure 3.3.4 (5).

3.3.5 CRITICAL MANEUVERS AND FRICTIONAL REQUIREMENTS

The safe performance of desired maneuvers is dependent upon the existence of sufficient pavement friction. It is well known that the friction required (demand) increases with speed. The friction available tends to decrease with increasing speed. Loss of control is the usual result when friction demand exceeds friction available. The friction at the point where available friction and demand friction are equal is defined as the "critical friction," and the associated speed is defined as the "critical speed."

Stopping Maneuvers. The design of a highway assumes as a minimum that the driver will be able to stop before hitting an obstacle in his path. The minimum stopping distance is expressed as the sum of the distance traveled during perception-reaction time and the distance required to stop after the brake is applied. This results in the basic equation for the required skid number,

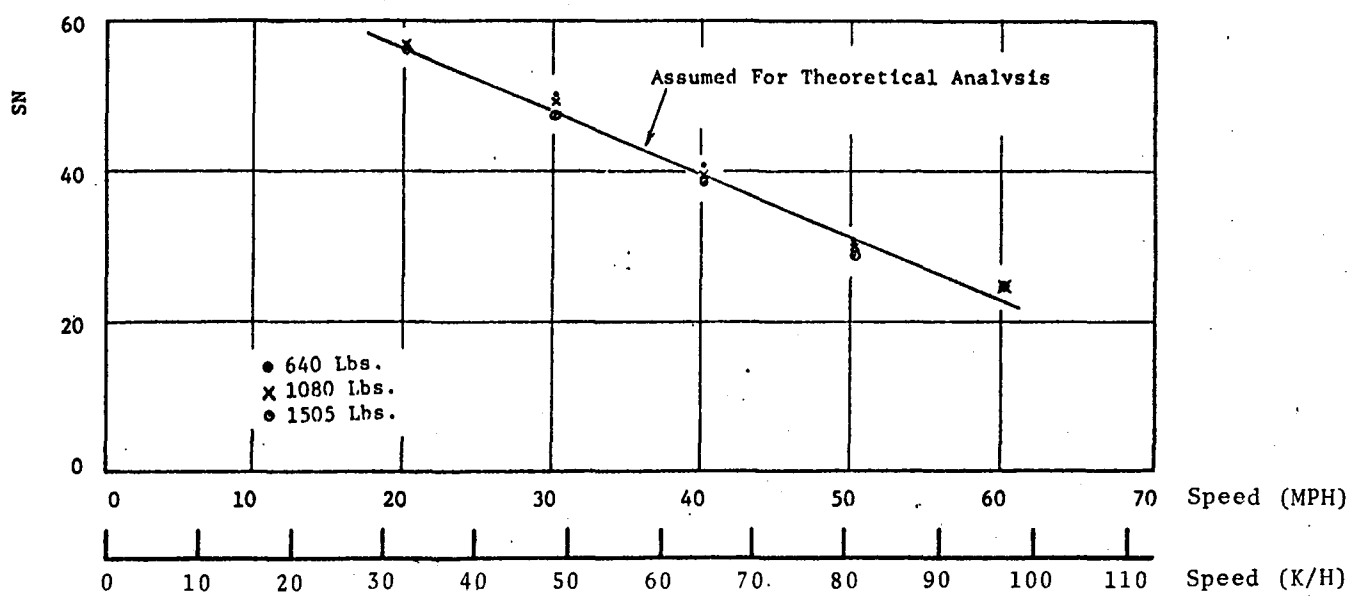

Figure 3.3-4 Effect of Speed on SN for One Tire at 3 Load on Unpainted Concrete

Source: Reference 5

$$FN_s = \frac{V^2}{0.3\ (d_{min} - 1.47V\ T_{PR})}$$

where:

FN_s = Skid number demand for stopping

d_{min} = Available stopping sight distance in feet

V = Initial velocity in miles per hour

T_{PR} = Perception-reaction time in seconds

Figure 3.3.5 illustrates the relationship of available friction and demand friction for the stopping maneuver for sight distances ranging from 200 to 1000 ft (61 to 305 m) assuming a 2.5 second perception-reaction time.

Cornering Maneuvers. Lateral friction during cornering must be sufficient to generate the lateral forces necessary to traverse a given curve. From the centripetal forces equation, the demand skid number in cornering can be determined. The equation is as follows:

$$Fn_c = (\frac{V^2}{15R} - e)\ 100$$

Observations of vehicles traversing horizontal curvature indicate that a degree of curvature somewhat greater than the design value consistently is used. The empirical equation to approximate the 90th percentile curvature is given by the following equation:

$$D_v = 1.014 + 1.128\ D$$

where:

D_v = 90th percentile curvatures

D = Design degree of curvature

Passing Maneuvers. Passing maneuvers are probably the most critical non-emergency maneuvers performed on a two-lane highway. Several characteristics combine during the passing maneuver to influence the demand for friction: the maneuver is performed at relatively high speeds, the passing vehicle executes both the pull-out and return maneuvers against negative superelevation due to the normal crown and, finally, the maneuver involves combinations of forward and lateral acceleration.

Ninety percent of the passing maneuvers radii were 1470 ft (450 m) or greater and, therefore, this value can be and has been substituted into the passing maneuver demand friction number to yield the following relationship:

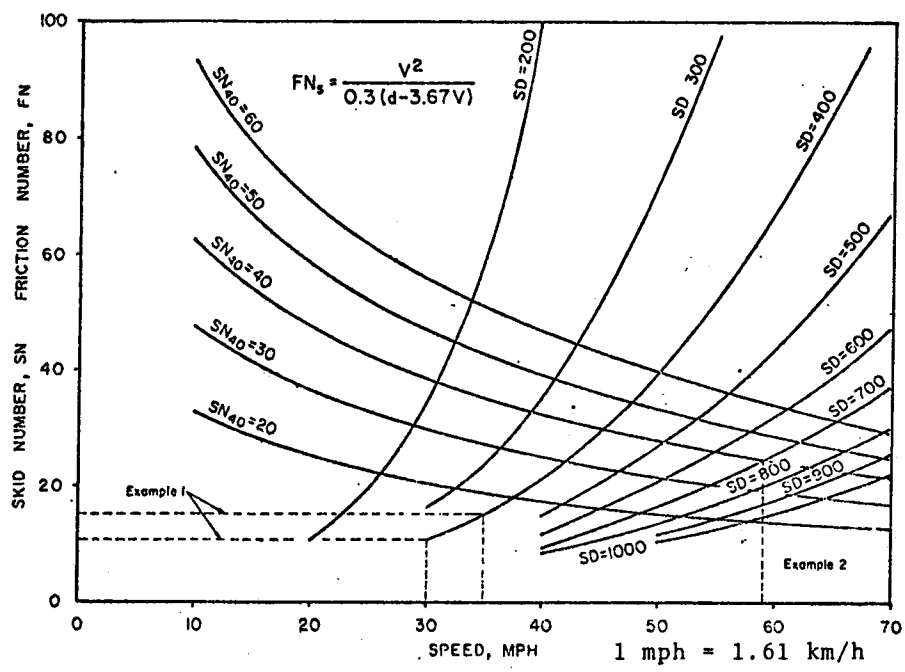

Figure 3.3.5 Critical Speed For Emergency Stop Imposed By Sight Distance and Available Friction

$$FN_p = 100\left(\frac{V^2}{15(1470)} - e\right)$$

$$= \frac{V^2}{220} - 100e$$

if a normal crown of 0.02 feet per foot is assumed, this reduces to

$$FN_p = \frac{V^2}{220} - 2$$

Hydroplaning. When operating on wet pavements, a vehicle's steering and braking capabilities are impaired. As the depth of water increases, the wheel becomes physically separated from the pavement, and hydroplaning exists. Hydroplaning is an exceedingly complex phenomenon that commonly is associated with an appreciable depth of water on the pavement surface. This condition is encountered during exceptionally high intensity rain or on pavements with poor drainage characteristics. Wheel path ruts produce puddling and, therefore, provide the potential for hydroplaning.

The general equation for estimating the critical speed for the onset of hydroplaning is:

$$V_c = K\sqrt{P}$$

where:

V_c = Critical Speed (mph)

P = Tire Pressure (psi)

K = Constant

for: Low texture pavements or smooth tires K = 8

High texture pavements or fully treaded tires K = 10

Table 3.3.1 illustrates typical critical speeds for varying tire pressures:

TABLE 3.3.1 CRITICAL HYDROPLANING SPEEDS

Tire Pressure psi (kPa)	Critical Speed, mph (km/h) K=8	K=10
20 (138)	36 (58)	45 (72)
24 (165)	39 (63)	49 (79)
26 (179)	41 (66)	51 (82)
28 (193)	42 (68)	53 (85)
30 (207)	44 (71)	55 (86)
35 (241)	47 (76)	59 (95)
50 (345)	52 (84)	71 (114)

3.3.6 SUMMARY OF HORIZONTAL ALIGNMENT DESIGN PRACTICES TO IMPROVE SAFETY

There are a number of practices in horizontal alignment that may be exercised to assure improved safety. Basic to these practices is the question posed earlier: Is it more practical to reduce friction demand through a modification of design standards, or to increase friction supply? Actually both approaches are appropriate, but there is greater potential in increasing friction supply than in modifying existing formulas to reduce demand. To summarize there are several points to be considered in both approaches.

- The designer should exercise practices of achieving the maximum skid resistance through the selection of materials and design methods of pavement surface design recommended in the AASHTO Guidelines for Skid Resistant Pavement Design should be considered.

- Designer should consider optimum design rather than minimum design philosophy. A proper balance between initial cost, operational costs, and service life should be sought.

- Flat cross sections are unavoidable in transitions to horizontal curves. This problem is compounded when transitions are placed where the longitudinal slope is also flat or essentially flat. The designer should have a system for detecting these situations and avoiding them. Plotting contours, flow paths, etc., will reduce the likelihood of catastrophic circumstances.

- Horizontal sight distance problems have been virtually eliminated through the high standards normally used on modern, high-speed highways. These problems may have been re-introduced by the placement of rigid barriers in narrow medians and by the installation of roadside barriers in advance of ramps. Very serious problems have arisen at diamond interchanges where bridge rails and traffic barriers block the view of drivers negotiating the intersection of a ramp or frontage road with the overpassing roadway. The designer should consider stopping sight distance as a minimum condition. For increased safety, he should consider anticipatory or decision sight distance. Providing stopping sight distance may present a false sense of security when the driver's needs are not to stop but to make an important decision relative to a maneuver required of him by a geometric constraint that he cannot see in time to make the proper decision.

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TOPIC 3 SESSION 4
VERTICAL ALIGNMENT

Objectives:

The participant should be able to:

1. Select maximum and minimum grades based on safety and operational controls,

2. Design satisfactory truck climbing and auxiliary passing lanes, and

3. Select and apply appropriate sight distance controls relative to the safety and operational requirements of streets and highway.

3.4.1 INTRODUCTION

To identify a general objective of vertical alignment design, it seems entirely appropriate to quote the new Red Book, A Policy On Design of Urban Highways and Arterial Streets (1). It is stated very succinctly that "highways should be designed to encourage (and permit) uniform operation throughout." This could be stated as a general objective for the entire design process, but it seems most appropriate for vertical alignment design.

3.4.11 Scope

There are three specific areas of vertical alignment that we will explore in greater detail. These are:

- Control of Grades - The steepness and length of grades
- Vertical Curves - Joining grades
- Sight Distance Alternatives

It is not the intent in this session to delve into vertical alignment in its most minute detail. Since time is limited, we will concentrate on some historical background of design controls so we may better understand the standards available to us today. Further, we will examine some new controls for vertical alignment design. For example, viewing time and anticipatory sight distance or decision sight distance may be more meaningful in certain instances than stopping sight distance.

3.4.12 Background and Evolution of Design Controls

The general objective stated above is entirely appropriate for the present day mode of operation on our streets and highways, and it was a good general objective for the engineers of highways and roadways during the pre-automobile era. In fact, our predecessors in highway engineering were far more concerned with vertical alignment than we are today.

Grades. During the era of horse-drawn wagons there was a very critical concern for grades because the steeper the grade, the more horses were required for pulling a given load and, consequently, higher operating costs were experienced. Figure 3.4.1 illustrates the grade effects on number of horses and the relative costs taken from Wiley's Highway Text published in 1928 (2). It is easy to realize even today that grades were most important particularly when the maximum speed was something on the order of eight mph (13 km/h). At such speeds, horizontal alignment was of little concern and, therefore, the road could be built as indirect as necessary to achieve relatively flat grades.

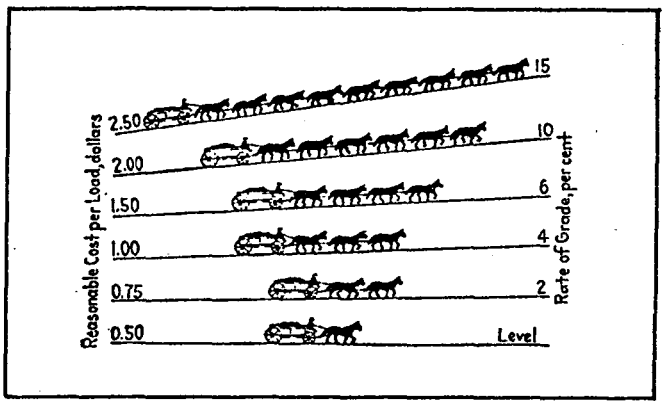

Figure 3.4.1 Effect of Grades on Hauling Power *(Source: Wiley)*

The average grade resistance (2) was computed as

$$R_R = 20g$$

where: R_R = rolling resistance per ton of load

g = the grade in percent

For example, a 4-ton (3600 kg) freight wagon would offer a resistance of 400 pounds (182 kg) when being pulled up a grade of 5%. Early design guides pointed out that one horse typically could pull one-tenth his weight under sustained loading. In other words, it would require four, 1000-pound

(454 kg) horses to pull a 4-ton (3600 kg) freight wagon up a 5% grade without stopping for rest. Under extreme loading conditions a horse was able to pull one-half of his weight but only for brief loading periods. Our predecessors had done their homework well, and they had things under control until the automobile came along.

Economics of construction always has been a principal issue in the planning and design of highways. With the advent of automobiles, economics was a primary consideration for a major change in layout and construction methods. The automobiles and trucks had much greater power than horse-drawn wagons, and, as a result, were not restricted to the flatter grades that were previously necessary. The highway industry turned to much steeper "grass roots" grades, and, in fact, conducted a great deal of research in the gradeability of trucks. With reduced restrictions on grades, economic factions pressed for straight alignment, on the basis that highway costs were equated directly with road mileage.

In more recent years, we have observed some tempering of the strict economic philosophies in the interest of operational, environmental and safety issues. As highway technology has advanced, we realized that grass roots grade lines may not be the most desirable from the operational standpoint. As capacity considerations came into being and certainly the maintenance of uniform operating speeds, we came to realize that the flat grades required by early horse-drawn vehicles provided definite operational advantages. Further, flatter grades and smoother vertical curves provided greater sight distance and more passing opportunities, and thus improved the safety of the facilities.

With the modern construction equipment, we found that we could slice through the hills and fill the valleys to achieve a smooth flat grade line at moderate costs. However, the massive cuts and fills exposed the insides of the mountains and brought about strong objections by the environmentalists. At this point in time, we are seeking a reasonable balance between environmental, operational, and economic factors.

Vertical Curves. Until the advent of the motor vehicle, there was very little concern relative to vertical curves in alignment design. For example, two 5% connecting grades resulted in an approximate 6-degree intersection angle which would be of no real concern to wagon and buggy traffic operating at about 8 mph (13 km) maximum. If there were any problems with the intersection of the relatively steep grades, only a small amount of rounding would be sufficient.

However, as motor vehicles became more common, speeds increased, and two new requirements emerged in the design of the vertical profile--these were comfort and sight distance. C. C. Wiley in his 1928 Highway Engineering text points out the need for some vertical curvature to relieve the discomfort of vertical accelerations, but mainly he expressed concern for drivers being able to see each other in the meeting situation. He stated that drivers should be able to see each other and avoid a head-on collision, and that this would require approximately 400 ft (122 m). There was no mention of speed in connection with this sight distance.

Also, in the early 1920's another authority stated that vertical curves should be of a reasonably large radius to avoid the discomfort aspects--not less than 50 ft (15 m). About the same time, another authority suggested that parabolic vertical curves be used, and that the lengths should be 100 ft (30 m) for algebraic differences in grade of 1 to 3 percent, 200 ft (61 m) for differences of 3 to 6 percent, and 300 ft (91 m) for algebraic differences greater than 6 percent.

In 1929 AASHO established a minimum sight distance of 500 ft (152 m) for vertical curve design (3). As measurement criteria, they set the height of eye at 4.5 ft (1.4 m) and the height of object at 4.5 ft (1.4 m).

In the 1930's the German engineers who designed the autobahnen used 370 meters (1215 feet) as sight distance criteria for vertical curves. This was also the first indication of design speed considerations, for the autobahnen was designed for 180 kilometers per hour, which is roughly 112 miles per hour. The Germans used 3.9 ft (1.2 m) for height of eye and 8 inches (20 cm) for height of object.

In 1939 AASHO reviewed the work of the Germans and collectively adopted new standards that related to providing visibility of small objects that may be in the path of the vehicle. Heretofore, the only concern had been that drivers see each other in the meeting situation and be able to avoid head-on collisions. Here was the first provision in this country for drivers to see small objects. It was readily labeled the "dead cat" theory. As usual, there was considerable controversy in the committee that composed these new standards, some favoring zero object height whereas others favored larger values. The 4-inch (10 cm) object height was a compromise. Thus, the standards called for height of eye of 4.5 ft (1.4 m) and height of object of 4 inches (10 cm). Design speeds which were adopted in 1938 were incorporated into sight distance standards.

Current AASHO standards provide for a height

of eye of 3.75 ft (1.1 m) and height of object of 6 inches (15 cm).

Many feel that current stopping and passing sight distance standards are inadequate for certain elements of vertical alignment design, and new measures such as Minnesota's viewing time of 11 seconds and Jack Leisch's suggested anticipatory sight distance are under consideration.

3.4.2 DESIGN CONSIDERATIONS IN CONTROL OF THE GRADE LINE

3.4.21 Design Controls

The two AASHTO design manuals, A Policy on Design of Urban Highways and Arterial Streets and A Policy on Geometric Design of Rural Highways, provide detailed, although not very specific, information relative to control of grades. These relate primarily to minimums, maximums, and relative lengths of grade.

Minimum Grades. For rural highway cross sections where surface water is drained by ditches, the travel way may be built to a 0% or flat grade. It is presumed that the ditch section will have sufficient slope for satisfactory drainage. Where curbs are used for drainage, the minimum grade is 0.35% for high-type facilities and 0.50% for other facilities. In concept, these minimum grades are excellent; however experience in many of the midwestern states, where topography is virtually flat, shows that these minimum grades cannot be attained. Fortunately, in a number of these cities, rainfall frequency is such that street flooding is an acceptable trade-off.

Maximum Grades. Maximum grades are controlled on the basis of the type of facility and the type of terrain. Standards for maximum grades are primarily arbitrary policy because their effects on operation are not scientifically well-defined. The various states, through AASHO, have had extreme difficulty in arriving at acceptable maximum grades. In 1956, AASHO established standards for the Interstate system as follows:

> "For design speeds of 70, 60, and 50 miles per hour, gradients generally shall not be steeper than 3, 4, and 5 percent, respectively. Gradients 2% steeper may be provided in rugged terrain (3)."

However, it was not until 1961 that the states reached a consensus concerning maximum grades to be used for highways other than freeways. Standards adopted by AASHO in 1961 are presented in Table 3.4.1.

Length of Grade. Maximum grade in itself is not a complete design control. It is also necessary to consider the length of a particular grade in relation to the desirable vehicle operation. The term "critical length of grade" is used to indicate the maximum length of a designated up-grade upon which a loaded truck can operate with a specified minimum speed reduction. AASHTO currently recommends that a minimum speed reduction of 15 mph (24 km/h) be used for design (4). It is anticipated that the recommended minimum speed reduction will be 10 mph (16 km/h) when the new AASHTO standards are published. Figure 3.4.2 gives the critical length of grade related to percent upgrade and speed reduction.

TABLE 3.4.1 AASHO STANDARDS FOR MAXIMUM GRADES

Topography	Design Speed, mph (km/h) 30 (48)	40 (64)	50 (80)	60 (97)	70 (113)
Flat	6	5	4	3	3
Rolling	7	6	5	4	4
Mountainous	9	8	7	6	-

3.4.22 Design Alternatives

The controls for designing the grade line are rather specific. Satisfying these controls is necessary to achieve operational efficiency. There are several methods, however, in which these controls may be satisfied. For example, the designer may flatten the grade to accommodate loaded trucks within a specified speed range. Also, the designer may alter the length of grade to insure the desired level of operational quality. There are times, however, when it is not economically feasible to flatten the grade or alter the length. Under such conditions there is yet a third alternative, and that is to provide an auxiliary lane, a truck climbing lane, for the slower moving vehicle. Truck climbing lanes are also beneficial to slower moving passenger cars as well.

3.4.23 Climbing Lane Design

There are three principal elements in climbing lane design: analyzing the need for a climbing lane, determining the beginning point, and determining the point at which the climbing point may be terminated. These three elements are treated in this section.

Analyzing Need. Perhaps the most important element of climbing lane design is determining whether or not a climbing lane is needed. Most agencies utilize a capacity and level-of-service analysis procedure to determine need. Such a procedure is outlined in "A Policy on Geometric Design of

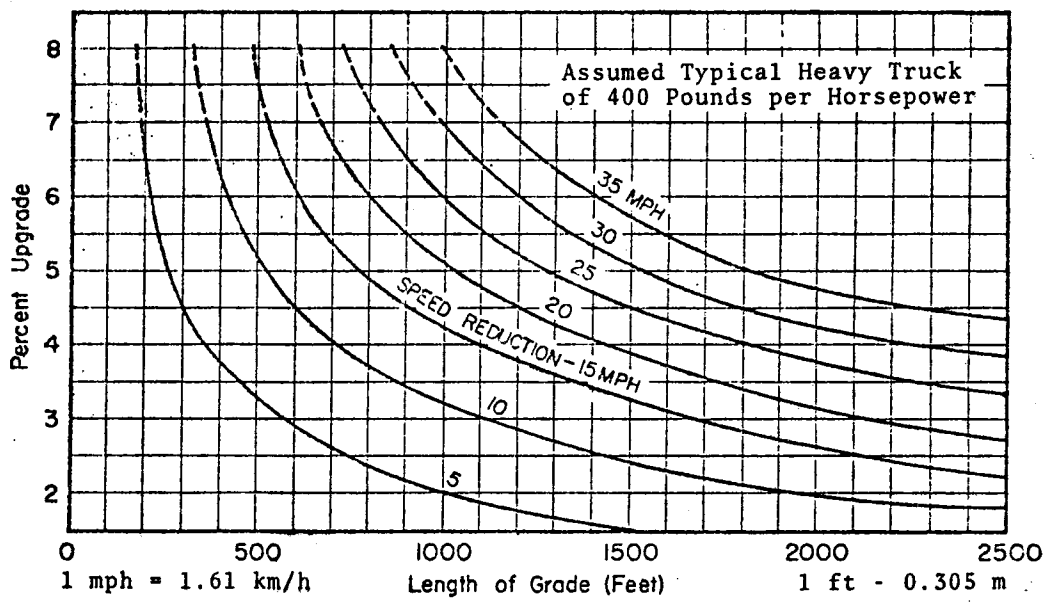

Figure 3.4.2 Critical Lengths of Grade for Design

Rural Highways." In applying such a procedure, care should be exercised in assuming that a high level-of-service be maintained. The level-of-service to be achieved in the section where a climbing lane is being considered should not be lower than in the adjacent sections of highway.

It is believed that analysis of climbing lane needs have been too conservative, resulting in reduced safety and operational efficiency. In a more liberal approach, some agencies have based the need for climbing lanes more on the occurrence of reduced truck speed rather than capacity reduction. Generally, the class of facility is taken into consideration in applying this more liberal practice. For example, climbing lanes may be provided on primary two-lane highways based on speed reduction only. For secondary class of highways, the speed differential and capacity analysis may be used. The important point is, consider the climbing lane as a useful and efficient tool to provide uniformity of operation rather than as a necessity to avoid extreme congestion and disruption.

Climbing lanes are applicable on multi-lane highways where extreme grade conditions reduce the level of service below that provided in adjacent sections.

Determining the Beginning of Climbing Lane. The best method of illustrating climbing lane design is by example. The proposed grade line shown in Figure 3.4.3 is to receive a climbing lane. The major problem at this point is to determine where the lane starts and where the lane ends. The starting point is determined from Figure 3.4.3 as the point where the truck speed decreases 15 mph (24 km/h) below entry speed. Of course, other speed differentials may be selected as designer choice, but this differential and the data included in the figure are current AASHTO values (4). From the truck entry speed of 47 mph (76 km/h), the loaded truck is assumed to decelerate along the 2% grade line curve throughout the length of the grade, 2500 ft (762 m). It will be noted that at the end of the 2500 ft (762 m), 2% grade, the truck speed should be about 33 mph (52 km/h). Then we note that the dotted line extends horizontally to the left to intercept the next proposed grade, 4% grade, 1100 ft (335 m) in length. Now, the deceleration would occur along the line of the 4% grade until it reaches 32 mph (52 km/h). Thus, the distance from the beginning of the grade at which the truck speed is reduced to 32 mph (52 km/h) is 2500 ft (762 m) on a 2% grade, plus 100 ft (30 m) on the 4% grade, for a total of 2600 ft (792 m). This marks the beginning of the climbing lane, and the lane should be full width at this point. A 50 to 1 entrance taper should be provided in advance of this point. Also, this is the maximum distance along the grade at which the climbing lane should be started. It certainly would be acceptable to begin the climbing lane sooner.

Determining the Terminal Point for the Climbing Lane. The climbing lane should be continued until the truck regains an appropriate speed for re-entering the traffic stream. As a minimum, this should

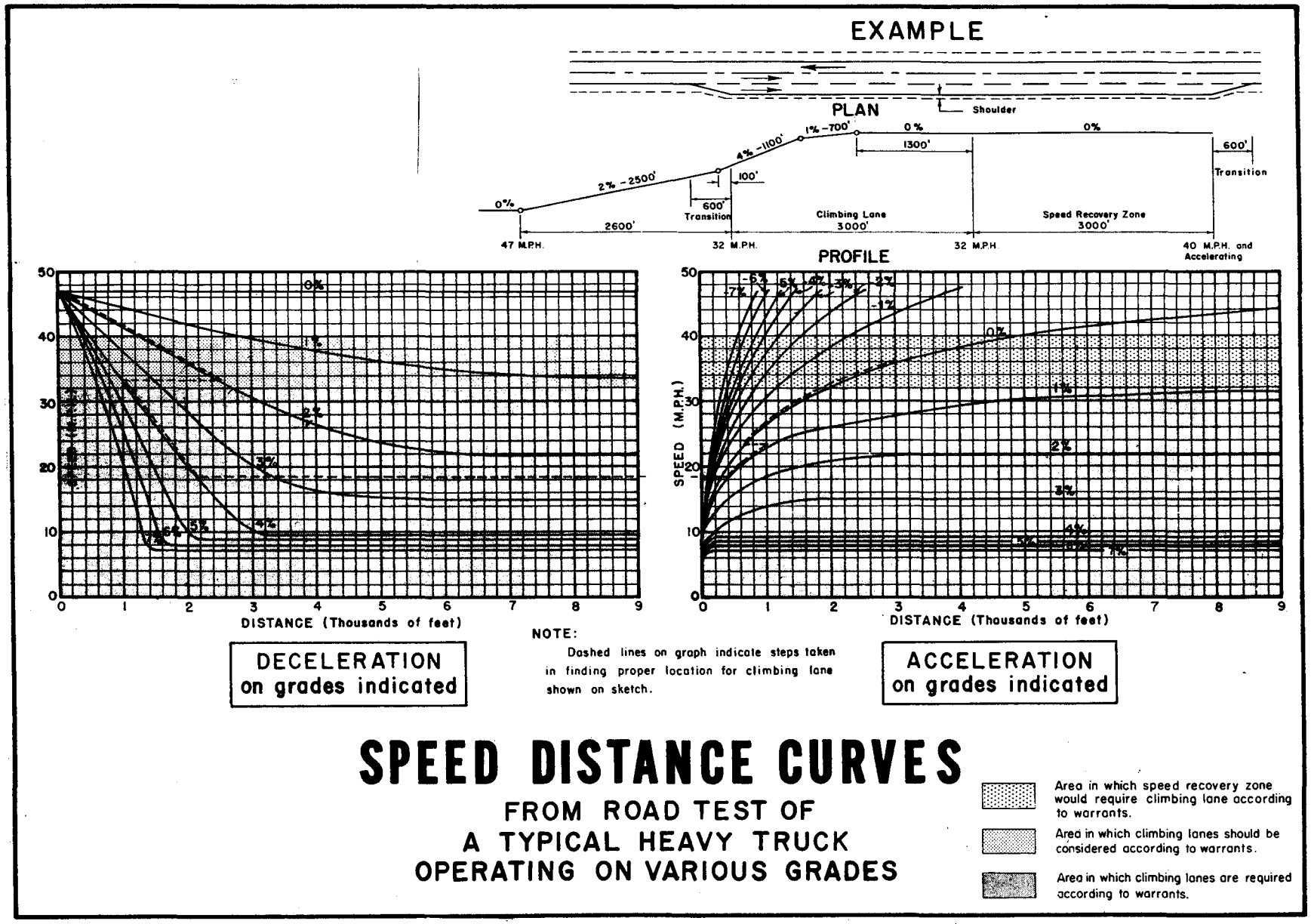

Figure 3.4.3 Truck Climbing Lane Design

be 32 mph (52 km/h). It is preferable that higher speeds be achieved before reentry. One major factor is whether or not the climbing lane terminates in a passing or no passing zone. If termination is in a passing zone, then the minimum acceptable speed can be tolerated because higher speed vehicles will normally be able to pass. If, however, the climbing lane terminates in a no passing zone then it should be continued until the truck is able to regain speed to at least 40 mph (64 km/h). To determine this point, refer again to Figure 3.4.3 where the dotted lines show deceleration of the truck to approximately 18 mph (28.8 m).

At that point the dotted line shows transfer of truck operating characteristics to a plus 1% grade, and it may be noted that the truck begins to accelerate and continues to accelerate along the 1% curve throughout the 700 ft (210 m) grade. The grade changes to 0%, and the truck characteristics change also as noted by the horizontal line from the plus 1% to 0% grade line. This curve shows the speed-distance relationship of the truck as it returns to 32 mph (51 km/h) at 1300 ft (400 m) along the flat grade. To achieve a speed of 40 mph (64 km/h), an additional 3000 ft (910 m) of the 0% grade line is required. Beyond that point a transition is provided at a 50 to 1 taper. Thus, the length of a truck climbing lane is established.

3.4.24 Designing for Passing Opportunities

One of the major issues with safety on two-lane highways is the head-on collision. Many times, head-on collisions occur because drivers attempt to pass when there is insufficient clearance from opposing vehicles. Much of this may be due to frustrations incurred while drivers follow slow-moving vehicles over an extended section with no passing opportunities. In rough terrain where passing restrictions are common, it may be desirable to design for passing opportunities by adding an auxiliary lane even though it may not be justified totally on the basis of truck speeds.

Design Controls

It is difficult to identify specifically the controls that should be utilized in determining whether or not auxiliary lanes should be provided. Two controls have been selected and are presented here. They are: (1) following time and (2) traffic queue length. For the purpose of illustration, following time has been arbitrarily set at two minutes or one mile, whichever is shorter. This is an assumption based on the supposition that drivers will begin to take undue chances in passing after they have followed a slow-moving vehicle for this specified time.

In applying the queue length criterion to establishing the distance into a no-passing zone where it would be desirable to provide an auxiliary passing lane, there are four variables that must be considered by the designer. These are:

1) Maximum queue length

2) Average time headways of vehicles forming the queue

3) The speed of the slow vehicle

4) Normal operating speed (the operating speed of the queue before it is impeded by the slow vehicle)

Maximum Queue Length. The maximum queue length is variable dependent upon the service expected of the facility and the volume of traffic. For secondary roadways and for higher volume facilities, the queue length would be greater than for low volume primary roads. A range of 2 to 4 vehicles in the queue appears to be reasonable, but this is certainly subject to more deliberate consideration.

Average Time Headways. Average time headways is computed using the traffic volume during the peak period. For example, a volume of 400 vehicles per hour will result in 3600 seconds/400 vehicles = 9 second average headway.

Speed of Slow-moving Vehicle. The speed of the slow-moving vehicle is a function of the assumed operational characteristics. The slow speed may be the result of a truck negotiating grades in which case the speed may be determined from an operational profile discussed in Session 3.2. Also, the slow speed may be determined as the 15-percentile speed of the traffic stream. Each situation should be analyzed to determine the most realistic slow-speed vehicle to be considered.

Normal Operating Speed. The operating speed of the queue before it is impeded by the slow-moving vehicle can be determined in several ways, including the use of the 85-percentile speed, the average running speed or the speed limit for the facility.

The objective of this approach is to determine the distance into a no-passing zone where it would be desirable to provide an auxiliary passing lane. This is accomplished by first determining the time which will be required to form a specified queue length and then converting this to distance on the basis of the speed of the slow-moving vehicle. To illustrate, let us assume a slow-moving vehicle at the 15-percentile speed of 35 mph and a normal operating speed as the 85-percentile speed of 60 mph. Further, let us assume a peak hour volume of 300 vehicles per hour and a maximum queue

length of three vehicles. Thus, the average time headway is 3600/300 = 12 seconds. The distance required for the queue to be formed is computed as follows:

$$D = V_1 t = \frac{V_1 V_2 H}{V_2 - V_1}$$

where

V_1 = Speed of slow vehicle (15 percentile speed)

V_2 = Normal operating speed (85 percentile speed)

H = Total headway time of queue (max. no. of vehicles in queue times average headway)

t = Total elapsed time from the point where the slow-moving vehicle enters the no-passing zone

For our example

$$D = \frac{35(1.47)(60)(1.47)(36)}{60(1.47) - 35(1.47)}$$

In this illustration, an auxiliary passing lane should be provided at 4460 feet within the no-passing zone.

When the auxiliary lane is added, it should be continued until all vehicles that have joined the queue have an opportunity to pass. Here, the designer should use a queue length based on 95% probability of occurrence rather than an average value. For an average arrival of three, one may expect as many as six or more arrivals in one out of twenty situations (95% probability level). If we assume five seconds per vehicle for the passing situation, then the required passing time will be 5 x 6 = 30 seconds. During the passing operation, as many as three additional vehicles could have arrived. Thus, the minimum passing distance would be:

$$D_p = 9 \times 5 \times 35(1.47) = 2315'$$

Certainly, it should be noted that this is a conceptual treatment of providing passing opportunities on two-lane highways. The principal objective in presenting this concept is to stimulate designers into positive thinking regarding passing opportunities rather than accepting the negative approach of prohibiting passing where sight distances are inadequate. It is hoped that designers will use the concepts presented herein to develop a rational design method for providing satisfactory passing opportunities.

3.4.3 DESIGN CONSIDERATIONS IN THE APPLICATION OF VERTICAL CURVES

The principal objectives in designing vertical curves are to insure driving comfort, to enhance the aesthetic quality of the highway and provide sufficient sight distance for the driver to operate on the highway with reasonable safety and efficiency. AASHTO, in its Policy on Geometric Design of Rural Highways and Policy on Design of Urban Highways and Arterial Streets, provides four different controls for vertical curves. These are: (1) stopping sight distance, (2) passing sight distance, (3) comfort, and (4) headlight sight distance (for sag type vertical curves). All of these controls are applied through formulas for the computation of the length of curve required to provide sight distance or comfort, given the algebraic difference in grades and the design speed. It is not the intent here to delve into AASHTO standards in detail as they are readily accessible in the Blue Book or Red Book. It should suffice to point out that the latest changes in AASHTO were adopted in 1971 when AASHTO established desirable versus minimum values for stopping sight distance. In essence, AASHTO retained its previous stopping sight distance standards as minimum values. These were based on the assumption that drivers voluntarily would driver slower during wet weather conditions and, thus, speeds assumed for actual operating conditions were less than the design speeds. Table 3.4.3 presents these minimum values as well as the assumed speed for conditions used to compute the minimum values.

The desirable stopping sight distance values, to be used in preference to minimum values, are determined on the basis of design speed. Experience has shown that a substantial percentage of drivers travel at the design speed under wet pavement conditions. Desirable values also are shown in Table 3.4.3.

The length of vertical curve required for specified conditions can be computed using formulas in the Blue Book. However, cookbook solutions such as the graphical solution shown in Figure 3.4.5 are also available in the Blue Book as well as practically every state design manual.

Two points are to be made relative to the application of AASHTO's standards. First, desirable stopping sight distance values should be used on all new construction and should be applied to every extent possible when upgrading existing facilities. Minimum values should be applied only where achieving desirable values would be economically prohibitive and unreasonably disruptive to the abutting property and general area served by the roadway. The second point, lengths of vertical curve determined by the formulas or by the charts, are minimum values

TABLE 3.4.3

STOPPING SIGHT DISTANCE

Design Speed	Assumed Speed For Condition	Perception and Brake Reaction Time	Perception and Brake Reaction Distance	Coefficient of Friction	Braking Distance on Level	Stopping Sight Distance Computed	Stopping Sight Distance Rounded for Design
mph	mph	sec.	feet	f	feet	feet	feet
-			Minimum Design Criteria				
30	28	2.5	103	.36	73	176	200
40	36	2.5	132	.33	131	263	275
50	44	2.5	161	.31	208	369	350
60	52	2.5	191	.30	300	491	475
65	55	2.5	202	.30	336	538	550
70	58	2.5	213	.29	387	600	600
75	61	2.5	224	.28	443	667	675
80	64	2.5	235	.27	506	741	750
			Desirable Design Criteria				
30	Not applicable	2.5	110	.35	86	196	200
40		2.5	147	.32	167	314	300
50		2.5	183	.30	278	461	450
60		2.5	220	.29	414	634	650
65		2.5	238	.29	485	723	750
70		2.5	257	.28	584	841	850
75		2.5	275	.28	670	945	950
80		2.5	293	.27	790	1083	1050

1 mph = 1.61 km/h 1 ft = 0.305 m

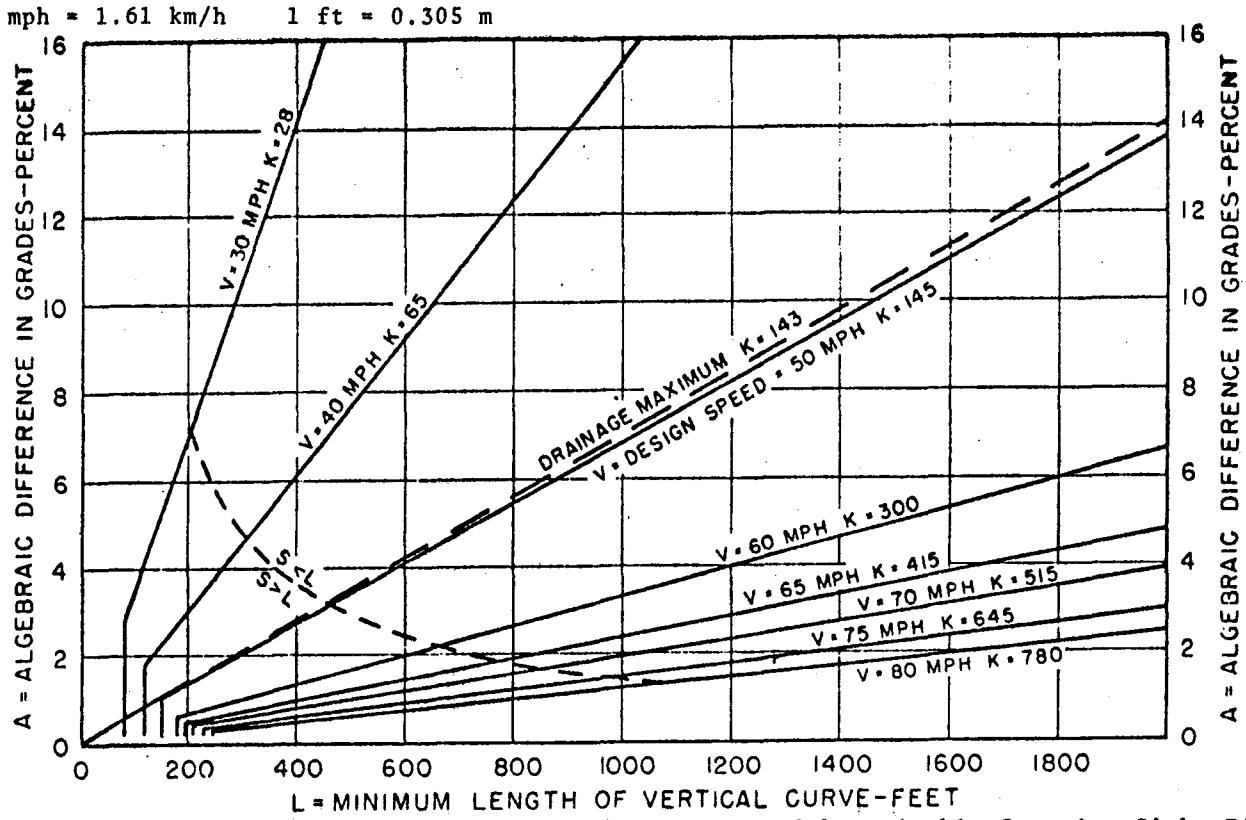

Figure 3.4.5 Design Controls For Crest Vertical Curves With Desirable Stopping Sight Distance

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for the conditions described. This means that shorter lengths of curves should not be employed; however, it is perfectly acceptable, and most frequently desirable, to use lengths that are longer than the minimum values.

3.4.31 The Effects of Viewing Time/Distance

A fundamental rule in geometric design is that stopping sight distance shall be provided throughout the entire length of highway. A driver should be able to drive at a reasonable and prudent speed with confidence that he will be able to stop his vehicle before colliding with a foreign body intruding into the vehicle path. Also, it is common practice to provide passing sight distance where it is economically feasible. These are basic rules that were appropriate and sufficient for low volume 2-lane rural highways which were prevalent when these controls were established. With the complex roadways of today and high-speed, high-density traffic operations, additional controls are necessary.

Drivers are faced with complex decisions and operational tasks at intersections, interchanges, lane drops, etc., that take considerably more time than the simple decision and reaction to stop. New design controls which take into consideration operational perception and reaction times should be adopted. Four alternatives are proposed herein.

Zero Object Height

One alternative is to provide the normal stopping sight distance, but base the measurement of sight distance on a zero object height to permit the driver to see the pavement surface (See Table 3.4.4). If the driver is able to see the pavement surface he should be able to interpret the geometry and perceive the need for a particular maneuver. Using zero object height while maintaining the same sight distance will require flatter vertical curves.

Viewing Time

Another alternative is to provide a fixed amount of viewing time so that the driver can view the pavement surface in the area where the decision is to be made and reach a proper decision and response. The Minnesota DOT attempts to provide an 11-second viewing time for decision points such as entrance and exit ramps at freeway interchanges. The 11 seconds is an allowance for detection, decision and response to the conditons at the ramp terminal. Sight distances and K values for vertical curves for 11-second viewing times are presented in Table 3.4.4.

Anticipatory Sight Distance

A third alternative is to base design on anticipatory sight distance. This sight distance is based on a time allowance for the detection, decision and response process to allow drivers to make logical and proper responses to complex decision points on the highway. Anticipatory sight distances are quite long as illustrated in Table 3.4.4 and result in extremely long vertical curves as illustrated in the same table. A typical example of anticipatory sight distance is a design practice in California a few years ago whereby a driver must be able to see the pavement surface for a distance of 2000 ft (610 m) in advance of a lane drop.

Decision Sight Distance

The fourth alternative is to base design on decision sight distance. Decision sight distance, as developed in conjunction with the positive guidance concept, is a sight distance based on time requirements for various elements of the driver decision process. The components of decision sight distance are presented in Table 3.4.5.

Pre-maneuver time is the time required for a driver to process information relative to a situation. It consists of the time to: (1) detect and recognize the situation, (2) decide on the proper maneuver and (3) initiate the required action.

- Detection and Recognition Time--These two elements of the information-handling process include time periods for latency (delay between time when stimulus is presented and time eyes begin to move), eye movement to situation, eye fixation, and finally recognition or perception of the situation. Time for these elements increases with the complexity and number of stimuli and with increasing vehicle speed.

- Decision and Response Initiation Time--Having perceived the situation, the next step in the process is to identify the alternative maneuvers, select one and then initiate the required action. These steps are often referred to as the reaction to the stimuli. Since the required maneuver is likely to be a lane change, the time required to decide on this maneuver, search for possible gaps and to initiate the action, can range from 2 to 7.1 seconds (or even longer under heavy traffic volume conditions).

The final component is the time required to accomplish a vehicle maneuver. Since it is the intent to allow the driver time to take an evasive action other than a quick stop, the assumption is made that a lane change maneuver will be required. Based on data provided by AASHTO's GDRH, this is assumed to be between 4.5 - 3.5 seconds, decreasing with increasing speed.

It is interesting to note that the design values for decision sight distance (Table 3.4.4) are comparable to those sight distances

TABLE 3.4.4

CREST VERTICAL CURVE DESIGN DATA
Based on Various Sight Distance Criteria

$L = KA$ $\quad K = S^2 \; 100 \left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2$ $\quad h_1 = 3.75'$ $\quad h_2$ as indicated

Design Speed	Normal Stopping Sight Distance		Stopping Sight Distance 0" Object Height		11-Second Viewing Time (Minn.)		Anticipatory Sight Distance		Decision Sight Distance	
	Sight Distance	K-Value $h_2 = 0.5'$	Sight Distance	K-Value $h_2 = 0'$	Sight Distance	K-Value $h_2 = 0'$	Sight Distance	K-Value $h_2 = 0'$	Sight Distance	K-Value $h_2 = 0'$
30	200	28	200	53	485	310	600	480	500	330
40	300	65	300	120	645	550	800	850	675	610
50	450	145	450	270	800	850	1100	1610	800	940
60	650	300	650	560	970	1250	1500	3000	1060	1500
70	850	515	850	960	1130	1700	2000	5330	1175	1840
80	1050	780	1050	1470	1290	2210	3000	12000	1350	2430

TABLE 3.4.5 DECISION SIGHT DISTANCE (5)

Design Speed mph (kph)	Times				Decision Sight Distance	
	Pre-Maneuver		Maneuver (Lane Change) sec.	Summation sec.	Computed ft. (m)	Rounded for Design ft. (m)
	Detection and Recognition sec.	Decision and Response Initiation sec.				
30 (48)	1.5	4.2 - 6.6	4.5	10.2 - 12.6	449 - 556 (137 - 170)	450 - 550 (137 - 168)
40 (64)	1.5	4.2 - 6.6	4.5	10.2 - 12.6	559 - 741 (183 - 226)	600 - 750 (183 - 229)
50 (80)	1.5	4.2 - 6.6	4.0	9.7 - 12.1	713 - 889 (217 - 271)	725 - 900 (221 - 274)
60 (97)	2.0	4.7 - 7.1	4.0	10.7 - 13.1	944 - 1155 (288 - 352)	950 - 1175 (290 - 358)
70 (113)	2.0	4.7 - 7.1	3.5	10.2 - 12.6	1050 - 1296 (320 - 395)	1050 - 1300 (320 - 396)
80 (129)	2.0	4.7 - 7.1	3.5	10.2 - 12.6	1199 - 1482 (366 - 452)	1200 - 1500 (366 - 457)

change manuever will be required. Based on data provided by AASHTO's GDRH, this is assumed to be between 4.5 - 3.5 seconds, decreasing with increasing speed.

It is interesting to note that the design values for decision sight distance (Table 3.4.4) are comparable to those sight distances provided in the 11-second viewing time as used by Minnesota. These values are considerably less than anticipatory sight distances.

Of these four methods introduced, the most logical is the decision sight distance. It provides a rational basis for the selection of these values whereas others are, for the most part, arbitrary decisions. It is much easier to improve upon rationally-based design standards than it is those which are established arbitrarily.

3.4.32 Effects of Drainage

Vertical curves controlled by high speeds and long sight distances may present drainage problems in the general vicinity of the crest of the vertical curve. These problems result from the combination of curbed sections, relatively flat cross sections, and flat longitudinal profiles. Critical drainage areas are fairly easily defined using the criteria outlined in AASHTO policy (4). In effect, any curve with a K-value greater than 143 may present drainage problems when curbs are used for drainage control. Alternatives for improving this situation include the use of shoulders to carry the drainage run-off, frequent drainage inlets to remove surface water and reduce ponding, and perhaps most important, to increase the cross slope to remove surface water from the bridge more quickly. The solutions are complex, but it is very important to recognize the factors that create the problem.

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TOPIC 3 SESSION 5
CROSS SECTION: ROADWAY ELEMENTS

Objectives:

The participant should be able to select roadway cross section elements and establish appropriate dimensions in such a manner as to enhance the operational and safety aspects of the roadway.

1. To identify various roadway cross section elements having a significant influence on safety.

2. To review the capacity and safety effects of various lane widths.

3. To explore some "good practices" relative to traffic safety and the roadway cross section.

3.5.1 INTRODUCTION

Sessions 5 and 6 of this topic area deal with safety aspects of street and highway cross sections. Because of the broad range of operational characteristics and requirements, the general cross section subject has been divided further into roadway elements and roadside elements. This session deals with elements of the roadway, whereas Session 6 deals with roadside elements.

Subjects to be covered relative to roadway elements include lane widths, cross slopes, shoulders, median treatments and auxiliary lanes.

3.5.2 LANE WIDTHS

There is considerable controversy over acceptable minimum lane widths for different types of streets and highways, particularly for urban streets built and operated by local jurisdictions.

For a considerable period of time, however, the lane width question for freeways and primary rural highways was resolved. Both AASHTO Policy Manuals, (1,2), the Blue Book and the Red Book, specifically recommend 12-ft (3.6-m) lanes. In this period of austerity, even these standards are coming under close scrutiny. There are several research/demonstration projects around the country involving experimentation with narrower lanes on freeways. One notable study is a section of U.S. 59 (Southwest Freeway) in Houston (3), where a 2.5-mile (4-km) section of freeway was converted to 10.5-ft (3.2-m) lane widths in order to increase the number of lanes.

Cross section comparisons for "before" and "after" conditions were as follows:

Three-lane Section	Before	After
Left Shoulder	10 ft (3 m)	10 ft (3 m)
Main Lanes	36 ft (11 m)	42 ft (13 m)
Right Shoulder	10 ft (3 m)	4 ft (1.2 m)

Four-lane Section	Before	After
Left Shoulders	10 ft (3 m)	10 ft (3 m)
Main Lanes	48 ft (15 m)	52.5 ft (16 m)
Right Shoulder	10 ft (3 m)	5.5 ft (1.7 m)

This section of freeway was converted in May 1976 at a cost of approximately $38,800. The results to date have been very favorable, as shown in the following summary:

- Increase in ADT: 85,000 to 90,000
- Increase in 2-hour PM Peak flow: 13,500 to 15,000 vehicles (1500 vehicles)
- Increase in Peak Hour flow: 6400 → 7700 vehicles
- Reduction in Delay: 1000 vehicle-hours/day (PM Peak)
- Accident Experience: Accident rate in the test section decreased, whereas rate in adjacent sections continued to increase.

It is significant to note that these demonstration projects have involved high-density urban freeways, and efforts have been made to relieve serious congestion. Compromises are reasonable in such instances, but perhaps should not imply the relaxation of current standards under other conditions. In other words, a successful project in Houston under 90,000 vpd should not imply that 10.5-ft (3.2-m) lanes are acceptable on rural Interstate highways with considerably lower volumes.

Lane widths on two-lane primary highways are believed to be more critical than lane widths for freeways. Narrowing the lanes on two-lane highways will force opposing traffic even closer together. On two-lane primary highways we should be striving for more space between traffic streams. For several years Texas has used a modified cross-section known as the "divided two-lane highway," as illustrated in Figure 3.5.1. It is noted that 12-ft (3.6 m) lanes are standard, and reductions are made in paved shoulder widths. It is conceivable that lane widths could be reduced to 11 ft (3.4 m) to insure sufficient space between opposing flows if this concept is applied to narrower crown widths.

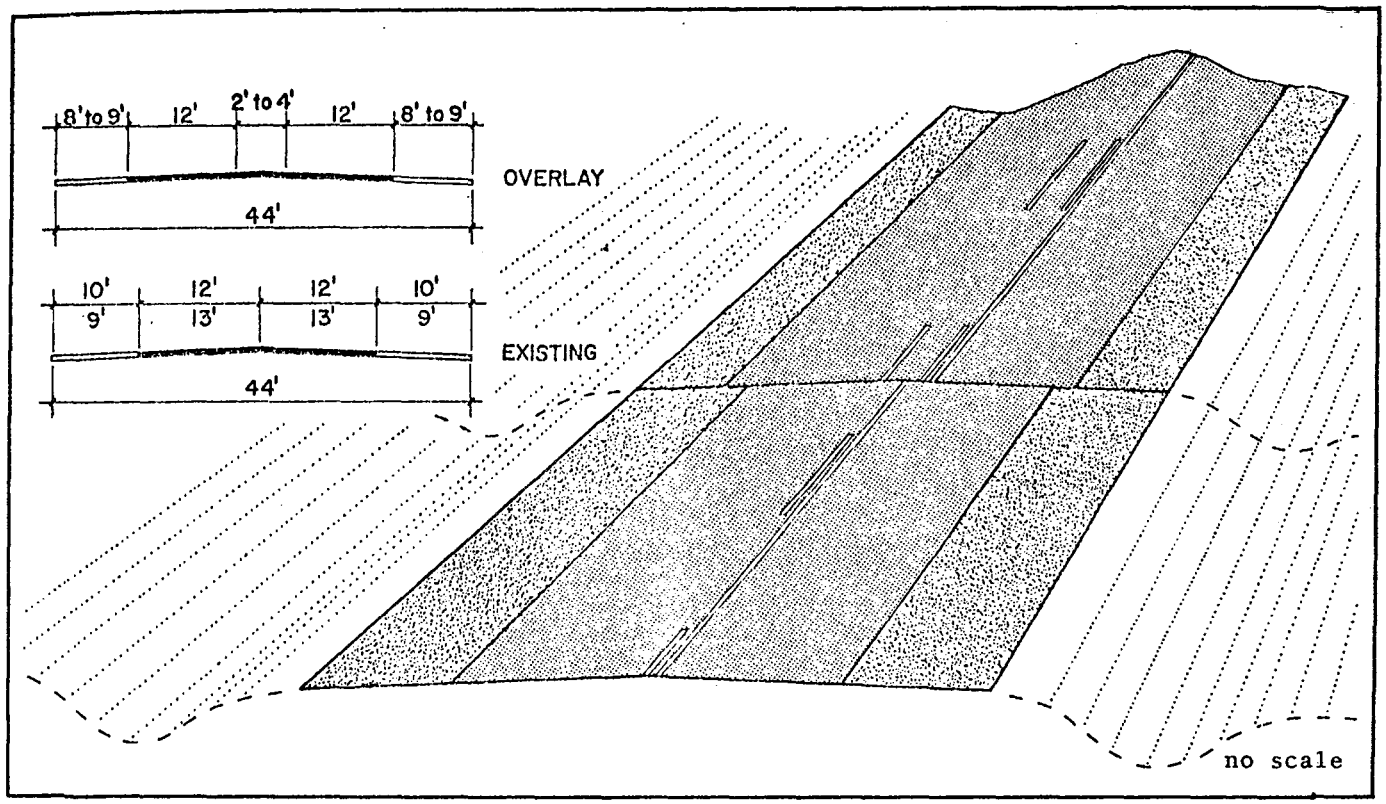

Figure 3.5.1 Two-Lane Double Striped Highway Overlay of Existing Pavement

Lane widths in urban areas are highly variable, and there is considerable controversy as to the widths that lanes should be. APWA reported in 1969 on a survey of urban arterial design standards (4) which is indicative of practices, standards, and philosophies concerning arterial design. In the following paragraphs, some direct quotes and others paraphrased, the practices relative to lane widths in 24 urban areas are reviewed.

The discussion of lane widths centers upon striping policies in the study areas. Widths, in general, are measured from the center of a lane stripe and, for the curb lane, from the face of the curb. The curb lane, inside lane, and turn lane are discussed both as separate entities and as they affect one another in the operations of the whole street.

The curb lane has the most variable width because it may include the gutter width and may include or be a parking lane, or have a specialized use such as an exclusive bus lane. Side friction caused by cars entering and leaving the roadway has the greatest effect on this lane. All of these factors must be considered in the design of the curb lane.

When the curb lane is not used exclusively as a parking lane it should be wider than the other travel lanes. One reason is that the gutter width is not always considered as part of the usable lane width. It has been stated that the curb lane should be wide enough to keep cars from riding in the gutter where the pavement may fail, especially near drainage inlets. A wider curb lane helps turning vehicles to stay in the lane while turning into and out of driveways.

Column 1 of Table 3.5.1, Lane Width Standards by Type, contains a summary of the preferred or most common curb lane widths used as through travel lanes. In practice, the range of lane widths is much wider, varying from 10 to 14 ft (3 to 4.3 m). The most common curb lane width to serve moving traffic is 12 ft (3.6 m) (4).

Where the curb lane is used as a parking lane during all but peak hours, a 10-ft (3-m) lane width is often required. While narrow, it is still of sufficient width to handle peak hour traffic. Cities which provide permanent parking lanes on arterial streets often use only a 7- or 8-ft (2.1- or 2.4-m) lane width. In some of these areas it is felt that when the parking lane is converted to a travel lane all the lane widths on the street will be adjusted; and so, until then, the travel lanes are made as wide as possible (4).

The inside lane includes any lane between the

TABLE 3.5.1 LANE WIDTH STANDARDS BY TYPE.

Jurisdiction	Curb (ft.)	Inside (ft.)	Turn (ft.)
Atlanta, Ga.	11	10	9
Baltimore, Md.	12	12	10
Chicago, Il.	12	12	11
Cincinnati, Oh.	11	11	10
Dade County, Fl.	12½	11	10
Denver, Co.	12	11	10
Detroit, Mi.	11(17)	11	10
Eugene, Or.	12	12	10
Ft. Worth, Tx.	11(18)	12	12
Glendale, Ca.	11	11	10
Kansas City, Mo.	12	12	10
Lansing, Mi.	12	11	11
Los Angeles, Ca.	12	11	10
Louisville, Ky.	11	11	10
Middletown, Oh.	12	12	12
Omaha, Nb.	11½(19½)	10	10
Pasadena, Ca.	12(19)	11	10
Phoenix, Az.	14(20)	12	12
Portland, Or.	11½	10½	11½
Salem, Or.	12	11½	11
Skokie, Il.	12	12	—
Toronto, On.	13	12	10
Tulsa, Ok.	11	11	10
Washington, D.C.	11	11	11

	Range	Median	Mode	Average
curb lane:	10-14'	12'	12'	11.8'
inside lane:	9-12'	11'	11'	11.3'
turn lane:	9-12'	10'	10'	10.4'

curb lane and the street centerline or median. Traffic on an inside lane should move relatively smoothly because most of the points of conflict along a street are located at the sides. A major exception is the left-turning vehicle, which may disrupt traffic at points where exclusive turn lanes are not provided. Because of these relatively free conditions, inside lanes are generally not as wide as the curb lane. A summary of the preferred or most used inside lane widths is in column 2 of Table 3.5.1, Lane Width Standards by Type. The range of inside lane widths varies between 9 and 12 ft (2.7 and 3.6 m) with an 11-foot inside lane most commonly designated. Officials generally agree that a 10-ft (3.0-m) lane width may be an acceptable minimum on arterials carrying few commercial vehicles. As it is usually very difficult to control the movement of commercial traffic, officials generally stated that the 11-ft (3.4-m) lane would better accommodate most traffic conditions. Twelve-foot (3.6-m) inside lane widths are mostly in the "preferred" category and are most often used in new roadway construction through relatively undeveloped areas.

A Los Angeles report discusses the acceptability of an 11-ft (3.4-m) lane width as opposed to the 12-ft (3.6-m) width found in most state and federal requirements. The object of the report was to demonstrate to the state that a 12-ft (3.6-m) lane width requirement on projects using state gas-tax money was both unrealistic and unnecessary in the Los Angeles area (4).

Studies and observations made in various locations differ as to whether traffic volumes decrease with decreasing lane width. One city observed a 6:5 volume ratio when comparing 12-ft (3.6-m) lanes to 11-ft (3.4-m) lanes. At least three jurisdictions reported that there has been no observable difference in capacity on lane widths varying between 10 and 12 ft. (3.0 and 3.6 m).

Acceptable standards for turn lane widths are generally less than for those carrying through traffic because vehicles entering and leaving the lane generally do so at a low speed. Table 3.5.1, column 3, lists the preferred or most used turn lane widths. Turn lane widths were found to vary from 9 to 12 ft (2.1-and 3.6-m), with the 10-ft (3.0-m) turn lane by far the most common. In many areas the geometrics of the transition into the turn bay and the signalization associated with the turn lane appear to be more important than the width of the lane.

In some jurisdictions a turn lane is felt to be a sufficiently valuable addition to the roadway that one will be added even at the expense of the other lane widths at an intersection. For example, a 50-ft (15-m) four-lane arterial with 13-ft (4.0-m) curb lanes and 12-ft (3.6-m) inside lanes may be striped as five 10-ft (3.0-m) lanes at an intersection to include a turn lane (4).

The APWA survey included case studies relative to narrow lanes and accident experience. A case study in Detroit, Michigan, compared Fort West and Grand River Avenues. Fort West with four 10-ft (3.0-m) moving lanes, a 9-ft (2.7-m) two-way left turn lane and two, 11.5-ft (3.5-m) parking lanes (with peak hour restrictions) had an accident rate of 1600 accidents/100 million vehicle miles (1000 accidents/100 million vehicle kilometres). Grand River, with four, 11-ft (3.4-m) moving lanes, two 11-ft (3.4-m) parking lanes (with peak hour restrictions), and a 10-ft (3.0-m) two-way left-turn lane had an accident rate of 700 accidents/100 million vehicle miles (435 accidents/100 million vehicle kilometres). The difference was attributed primarily to the narrower lanes along Fort West (4).

A case study in Portland, Oregon, involved a "before" and "after" study on a widening project of a one-mile (1.6-km) section of West Burnside. Before widening, the street was 36 ft (11 m) wide with two, 9-ft (2.7-m) moving lanes and two, 9-ft (2.7-m) parking lanes with peak hour restrictions. The street was widened to 44 ft (13.4 m) with two, 10-ft (3.0-m) inside lanes and two, 11.5-ft (3.5-m) curb lanes with parking prohibited. Volumes increased from 15,600 to 19,500 ADT, and the accident rate decreased from 7400 to 3900 accidents/ 100 million vehicle miles (4600 to 2400 accidents/ 100 million vehicle kilometres).

3.5.3 CROSS SLOPES

The cross slope of the highway or street is a design compromise between the requirements for drainage and the requirements for satisfactory and comfortable vehicle operation. On the one hand, it would be preferable to have a very steep cross slope that would remove the water from the pavement surface very quickly and reduce the probability of hydroplaning. On the other hand, steering control in steep cross slopes requires a very concerted effort on the part of the driver, and is not good from the standpoint of driver comfort and safety. Thus, design values seek an optimum balance between drainage requirements and steerage control. It appears, however, that steerage control has dominated in recent years, because many of our high-speed highways have cross slopes as low as 1/8 inch per foot (1%). It is the opinion of some that designers should strive toward the maximum recommended cross slope of high-speed facilities of 1/4 inch per foot (2%) as a minimum condition.

Urban cross sections, particularly those on arterial streets pose entirely different problems. Not only must the pavement drain the surface water, but the pavement along the curb line is expected to serve as a drainage channel as well. Many city streets have a parabolic cross section which permits steeper slopes in the lane next to the curb. Steep slopes in the curb lanes pose some problems for tall trucks and buses, particularly where utility poles and other fixed objects are placed immediately adjacent to the curb.

Flatter cross slopes in the outside lane may improve traffic operation, but these also increase the width of street inundated by water during heavy rainfall. There are general rules which should be followed in determining the portion of the street that may be inundated by water during a design rainfall. These are as follows:

- Freeways: none
- Major Arterials: outside lanes only
- Collector Streets: all except for one lane in each direction
- Local Streets: all except for one lane in the center of the street

3.5.4 SHOULDERS

Perhaps one of the greatest assets of rural highways is the shoulder. The safety effects of shoulders, however, can not be fully documented by research. Certainly, a shoulder is an element of driver comfort and has a direct bearing on the relative safety of a roadway. Roadway shoulders appear in various forms and, based on the type of construction, laws and local driving practices, serve different functions. For example, grass shoulders are definitely emergency stopping areas, and areas where a vehicle may avoid an immediate collision with an approaching vehicle. Gravel shoulders provide some greater stability and may be utilized for stopping, whether it be emergency or routine. Paved shoulders provide the greatest comfort and the greatest operational efficiency, while serving many functions. Not only do paved shoulders serve for emergency escape and emergency stops, but they may also serve as slow-moving vehicle lanes where permitted by law. Also, paved shoulders facilitate the development of acceleration and deceleration lanes at rural intersections. In states that permit driving on shoulders, paved shoulders are naturally used as acceleration-deceleration lanes.

The Highway Capacity Manual states that the effect of paved shoulders on capacity is equivalent to an added one foot of lane width. For example, a highway with 11-ft (3.4-m) lanes and paved shoulders would have the same capacity as a highway with 12-ft (3.6-m) lanes but no paved shoulders. The major safety advantage to paved shoulders on narrow highways is due to the greater tolerances in vehicle steerage permitted by the paved shoulder. It is the opinion of many that head-on collisions frequently are caused by persons running off the pavement to the right and then losing control and going into the opposing traffic lanes as they attempt to regain the pavement. This loss of control generally is due to the rutting and wind erosion by trucks on roadways without paved shoulders. Any width of paved shoulder, even 2 to 3 ft (0.6 to 0.9 m), will serve to nullify this effect.

In states where permitted, the paved shoulder on two-lane highways permits through vehicles to pull out and go around vehicles waiting to make a left turn. In other states, where it is unlawful to pass on the right unless there is a specified lane, states have designated the shoulder as a driving lane so the traffic can legally pass to the right. An example of a specific design providing this operational feature is illustrated in Figure 3.5.2.

The desirable and minimum standards for shoulder widths are dependent upon the function of the shoulder and the character of the highway. The AASHTO Red Book (2) indicates that 10-ft (3.0-m) surfaced right shoulders should be provided on freeways and other high-type highways. Where there are high proportions of heavy trucks, 12-ft (3.6-m) surfaced shoulders are justified. This width is selected apparently on the basis of having a 2-ft (0.6-m) clearance between the edge of the vehicle and the through traffic lane for emergency stops. For emergency stopping conditions on other types of facilities, particularly arterial streets and long-span, high-cost structures, a desirable minimum width is 8-ft (2.4-m)

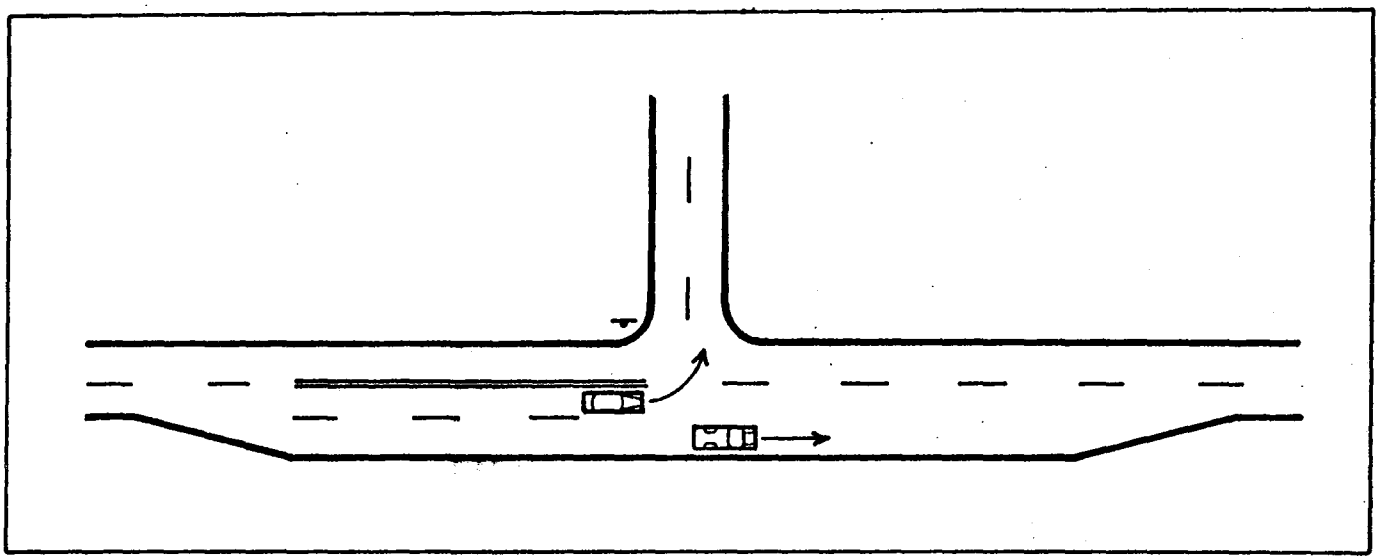

Figure 3.5.2 Left-Turn Bypass Lane

while an absolute minimim is 6 ft (1.8 m). These widths serve the purpose of emergency stopping space.

In states that allow slow vehicles to move onto the shoulder to permit others to pass, desirable shoulder width is 10 ft (3.0 m). Minimum shoulder width for this type of operation is 8 ft (2.4 m). It should be noted that reduced shoulder widths create a hesitancy on the part of drivers to pull over onto the shoulder and, further, a hesitancy on the part of other drivers to pass under restricted conditions.

Left shoulders are desirable on all divided urban highways. On urban freeways with 4 lanes, paved shoulders of a minimum of 4 ft (1.2 m), and preferably 6 ft (1.8 m), should be provided on the median side. On freeways with 6 or more lanes, this paved shoulder width should be increased to 10 ft (3.0 m) because the left shoulder becomes a frequently-used emergency stopping area for disabled vehicles.

Design standards do not discuss the matter of utilizing narrow paved shoulders under conditions of temporary improvements to existing two-lane highways. Mention is made, however, of utilizing shoulders as narrow as 2 ft (0.6 m) to provide edge protection to pavements. From the safety standpoint, the designers should recognize a very significant value to paving a narrow strip along two-lane highways which may be used temporarily as high-volume bypass facilities. As discussed previously, this measure could reduce the occurrence of head-on collisions significantly.

A case in point is the final segment of U.S. 75 between Dallas and Houston in the early 1970's when Interstate 45 was being completed. The old highway which was to be downgraded to a local road when I-45 was complete, was a 22-ft (6.7-m) wide concrete pavement with 6-ft (1.8-m) gravel shoulders. The heavy truck traffic caused an erosion of shoulder material away from the pavement edge. In meeting trucks on the narrow pavement, passenger cars frequently would run off the pavement, particularly at night; in an attempt to regain the lane on the pavement, drivers could be thrown out of control and across the centerline into the opposing traffic lane. During a period of approximately two years, this section of roadway had a very high fatality rate, most of the fatalities resulting from head-on collisions. In retrospect, even a narrow paved shoulder would have been highly cost-effective in reducing the severity and frequency of accidents.

3.5.5 MEDIANS

Medians serve a wide range of purposes dependent upon the type and location of the facility. Medians on rural highways are intended primarily to physically separate opposing traffic streams, reducing the probability of encroachment and reducing the effects of headlight glare at night.

In urban areas, and particularly on urban arterials, medians prevent illegal or unwanted crossovers, prohibit or channelize left turns, provide pedestrian refuge, shield cars stopped in the middle of the street, and serve as an area for street beautification. A well-designed median should increase the safety and efficiency of a street and enhance the appearance.

In urban design, the major question is, barrier median or painted median? The answer depends on the intended function: Access

or access control. The painted median accommodates access, whereas the barrier median prevents access. Barrier medians should be used on new arterials where access is limited in the interest of efficient traffic operation. Where barrier medians are used to prohibit left turns, right turns should be restricted or limited to major drives or entrances to major developments. If right-turn access is permitted, then U-turns will be needed at crossovers to facilitate an alternative to direct left turns.

Where it is desirable to permit access, then two-way left-turn lanes or painted medians should be provided. Two-way left-turn lanes are preferred over painted medians because they leave little doubt relative to the legality of driving in the area, whereas some drivers are hesitant to enter a painted median for fear of legal action.

Two-way left-turn lanes offer a significant safety advantage, particularly when compared to an arterial with no median. They facilitate the removal of traffic slowing or stopping to make a left turn from the through lanes. The literature indicates that installation of a median left-turn lane, regardless of type, reduces accidents involving left-turning vehicles. Installation of the two-way left-turn lane to replace a conventional left-turn lane was found in one study to reduce total accidents by about 33% with especially favorable reductions of 45% and 62%, respectively, for head-on and rear-end type accidents (6). According to the literature, the head-on collision, which has been a primary concern among those considering the installation of the two-way left-turn lane design, has proven in every study to be an uncommon occurrence and of negligible concern.

The width of barrier-type medians has considerable influence on their safety. Narrow medians on arterial streets have been found to have negative safety effects (5). They prevent mid-block left-turns but, in doing so, they create the need for U-turns at intersections and median openings. The U-turns are more hazardous than mid-block turns from a two-way left-turn lane. The width of median required to accommodate U-turns will be covered later in Session 3.7.

In addition to providing space for left-turn lanes, the median serves other purposes, including the "shadowing" of vehicles, i.e., vehicles stopping in a median opening while crossing one traffic stream at a time. A median width of 20 ft (6.1 m) and preferably 24 ft (7.3 m) is needed where traffic is expected to operate in this manner.

3.5.6 AUXILIARY LANES

Most safety aspects of auxiliary lanes have already been discussed relative to other elements of this session. To reiterate, however, acceleration and deceleration lanes are significant safety features for the rural highway and the urban arterial as well as the freeway. Under rural conditions, however, the need for such lanes may be alleviated to some degree if paved shoulders are provided and it is permissible for them to be used as acceleration and deceleration lanes.

Also, climbing lanes and other auxiliary lanes for permitting more convenient passing maneuvers have been discussed in other sections of this course. There is no doubt that climbing lanes add to the safety and the operational efficiency of a highway.

In urban areas where parking may be permitted on arterial streets, there are a number of recent innovations that improve safety and efficiency. One is the gap arrangement of parallel parking. In Figure 3.5.3 it will be noted that very little additional space is required to leave 10-ft (3.0-m) spaces between every other vehicle in the parking lane. This configuration permits drivers to pull directly into the parking space

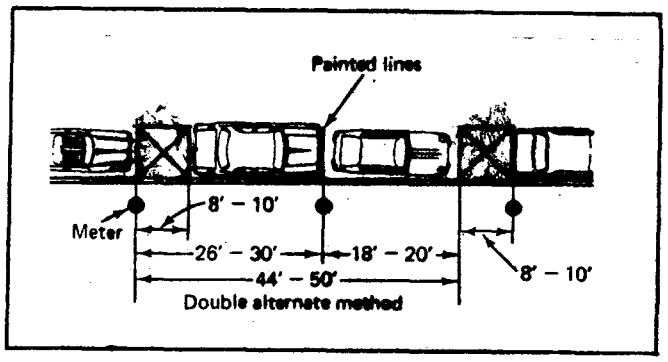

Figure 3.5.3 Paired Parking Layout

and maneuver into the parking position outside of the influence of through traffic. Where traffic must utilize the outside through lane in order to maneuver into a parking space, there is a significant loss in street capacity as well as the hazard of a vehicle backing up in a through traffic lane.

Angle parking is taboo on most arterial streets. A recent application of angle parking at a 22.5-degree angle improves operating conditions. The main safety problem is associated with the fact that a driver backing out of a normal angle parking space cannot see until he is well into the traffic stream. This results in frequent collisions. At 22.5 degrees, however, the driver is able to see back up the traffic stream and avoid conflicts with through vehicles as he backs out of the parking position.

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TOPIC 3 SESSION 6
CROSS SECTION: ROADSIDE ELEMENTS

Objectives:

1. The participant should become aware of the need for a clear zone of greater width than the basic 30-foot requirement and of the additional width required for safety under various roadside conditions, and

2. Be able to select roadside design elements to insure safe traversal of the roadside including:

a) Ditches
b) Curbs
c) Side slopes
d) Drainage structures

3.6.1 INTRODUCTION

In the early 1960's, as substantial mileage of the Interstate system was built, significant changes were observed in traffic operating characteristics. Speeds increased significantly and the characteristics of the typical rural highway accident changed appreciably. Instead of running head-on into opposing traffic or striking trees and other fixed objects immediately adjacent to the roadway, many drivers were simply running off the new freeways, colliding with the ditch, the backslope, bridge piers, sign supports, and any other objects that may be in the way. Since these were higher speed accidents, and consequently more spectacular, and because they were happening on a facility that had been justified on the basis of safety, there was considerable dilemma. From all of this emerged the clear roadside concept. In its report, the Special AASHO Traffic Safety Committee (1) reported in 1967 the desirability to improve the design of the roadside and the appurtenances along highway facilities. This committee published 19 recommendations relative to roadside design in their 1967 report, fondly referred to as the Yellow Book. From these 19 recommendations the clear roadside concept was formulated by the Federal Highway Administration and made a requirement on Federal Aid Highway Projects, with major emphasis, of course, on the Interstate Highway System.

The clear roadside concept is, in fact, a design practice which requires that, to every extent possible, the area adjacent to the driving lanes be maintained clear of fixed objects and with a gentle topography which can be negotiated by an errant vehicle with a good chance for recovery of control. The clear roadside concept suggested that insofar as practicable, the entire roadside should be kept clear of hazards, but particular attention should be given to a lateral clearance of 30 (9.15 m) ft. The Yellow Book states specifically the basis for the 30 (9.15 m) ft clear distance as follows:

"For adequate safety, it is desirable to provide an unencumbered recovery area up to 30 ft (9.15 m) from the edge of the traveled way; studies have indicated that about 80% of the vehicles in the 'run-off the road' accidents did not travel beyond this limit." (1)

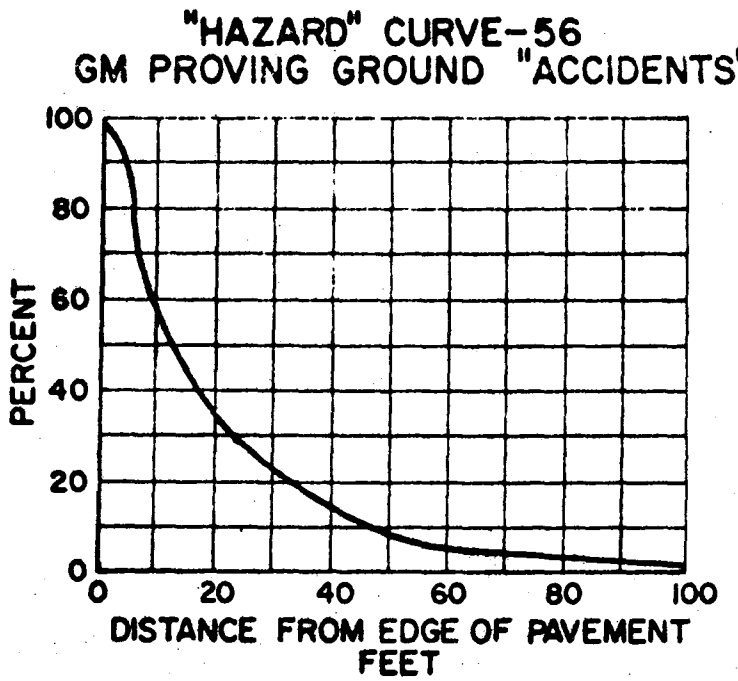
In other words, the committee says, clear the roadside as far as you can but make sure it is at least 30 ft (9.15 m).

The committee further recognized that it would be practically impossible to totally clear the roadside, even for the 30-ft (9.15-m) width. Therefore, they specified an alternative condition of making fixed objects breakaway or "install an appropriate vehicular guardrail or barrier that will afford protection to motorists."

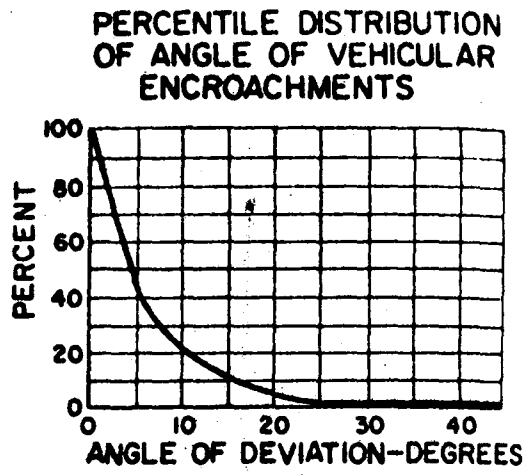
Since a rather stormy beginning amid the turmoil of the 60's, the clear roadside concept has become a common practice in the design and redesign of highway facilities. Most important, it has created a different attitude or a different philosophy on the part of designers. More definite criteria have now been developed to aid the designer in creating clear roadsides. These are discussed in more detail in Topic 5.

3.6.2 BACKGROUND INFORMATION

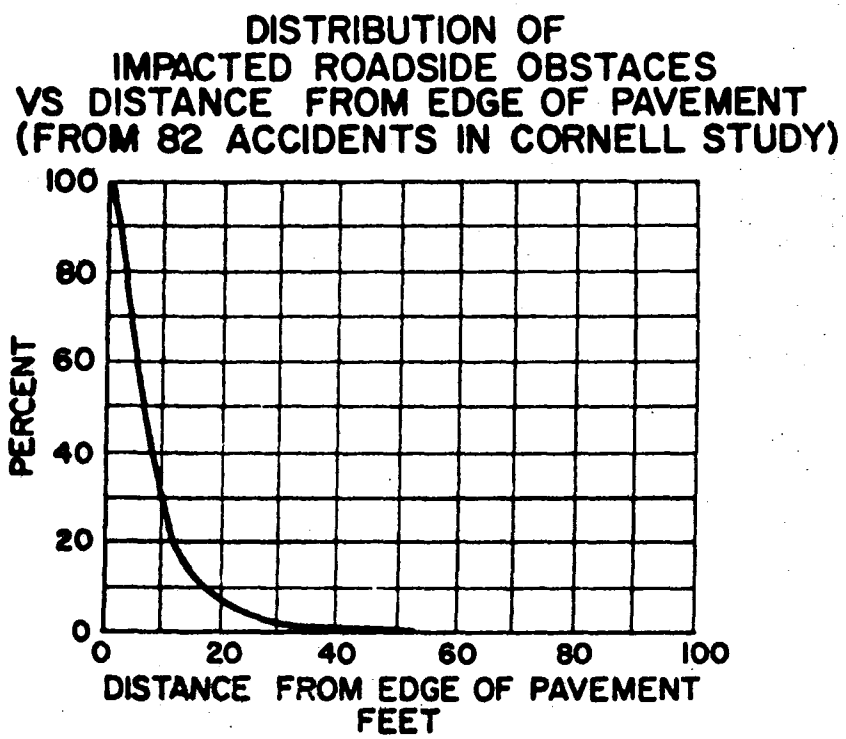
While the concern for roadside safety was mounting, there were a number of pioneering research efforts around the country that contributed data that would be extremely valuable to the development of technology for the improvement of roadside safety. At the General Motors Proving Ground, researchers were amassing data relative to run-off-the-road accidents in conjunction with their normal automobile proving ground activities. Due to the abundance of driving activity on the roadways of the proving ground, numerous single vehicle accidents occurred -- so many in fact that the researchers were able to compile data on the distribution of lateral encroachments and angular encroachments. These data are presented in Figure 3.6.1. Also, in the interest of improving roadside safety on



2(a) *



2(b)*



* Based on relatively flat side slopes.

Figure 3.6.1 Relation of Cross-Section Design and Highway Safety

the proving grounds, GM engineers conducted extensive tests in full-scale vehicle traversals of various slope and ditch configurations (2). This pioneering research is reported in Volume 39 of Highway Research Board Proceedings.

Hutchinson and Kennedy (3), while at the Illinois Engineering Experiment Station, conducted an extensive study of vehicle encroachments onto highway medians. The results of this study were subsequently published in Illinois Engineering Experiment Station Bulletin 482 and subsequently in Transportation Research Board publications. The curves developed by Hutchinson and Kennedy on the distribution of lateral encroachments and encroachment angles are illustrated in Figure 3.6.2.

In a much later study a comprehensive project of vehicle behavior on various side slope, back slope, and ditch combinations was conducted under the auspices of the National Cooperative Highway Research Program (4). The results of this project will be discussed later. It should be remembered that the encroachment data presented above are for very flat roadsides.

3.6.3 PRIORITY FOR ROADSIDE TREATMENT

Through the several years of research and study of the roadside safety problem, there has emerged a basic set of priorities that should be followed in improving roadside safety. These priorities should be followed regardless of whether it is a new project, an improvement to an existing project, and whether it is rural or urban, and whether it is local, state, and/or federally funded. These priorities are:

- Eliminate the hazard
- Relocate the hazard
- Make the hazard breakaway
- Redirect and/or attenuate the vehicle in providing protection from the hazard.

The application of these priorities is illustrated briefly in the following paragraphs.

3.6.31 Eliminate the Hazard

To accomplish this step, it is necessary to review all objects along the roadway and eliminate those which are unnecessary to the operation of the facility. This could mean eliminating signs that are installed on the basis of practice rather than need, and it could mean combining signs on one support to eliminate excessive sign supports. Removal or elimination of hazards also involves the flattening of critical slopes to facilitate the driver's maintaining control of an encroaching vehicle. Eliminating the hazard could also include the modification of drainage facilities to eliminate the culvert headwalls, table top drainage inlets and other features that constitute unnecessary fixed objects near the roadway.

3.6.32 Relocation of Appurtenances

Appurtenances which constitute fixed object hazards may be relocated longitudinally or laterally to reduce their likelihood of producing a fixed object collision. Some of the most obvious problems are the placement of signs in gore areas and on the outside edges of curves where they have a high probability of being hit. These signs may be moved longitudinally to a point where there is less probability of a collision. Overhead signs may be moved to existing bridge structures eliminating the need for supports altogether. Also, overhead signs may be moved to a point where the sign bridge supports may be located behind a protective traffic barrier. This may be done so long as relocation of the sign does not reduce the operational effectiveness of the sign. Fixed objects such as signs and luminaire supports may be relocated laterally to reduce the probability of a collision. Lateral relocations should not be used, however, as a tradeoff for nonbreakaway bases. Even when these objects are moved laterally, breakaway devices should be employed because they are relatively economical. The only exception would be where objects are located behind an existing traffic barrier.

3.6.33 Make Fixed Objects Breakaway

All sign supports, luminaire supports, and mile-post markers should be made to yield or breakaway if struck by an out-of-control vehicle. This does not apply to devices which are placed behind an existing traffic barrier, but certainly it is impractical to consider the installation of a traffic barrier to protect devices which can be made breakaway.

3.6.34 Redirect or Attenuate Vehicles

There are many objects along the roadway that cannot be eliminated, relocated, or made breakaway. These may be in the form of continuous hazards such as steep side slopes, natural streams, rock face cuts, and the opposing traffic stream. These hazards may also be point hazards such as bridge piers, elevated gores, and overhead sign bridges. Generally continuous hazards are treated using traffic barriers which intercept and redirect out-of-control vehicles before they intrude into the hazard area. Point hazards, on the other hand, utilize crash cushions to attenuate the vehicle. That is, the crash cushion absorbs the kinetic energy of the vehicle, slowing it within the deceleration limits of the passengers within.

A great deal of this course is built around the technology by which these roadside treatment priorities are accomplished. In this session, we will deal with the first two, elimination and relocation, while separate sessions later on will deal with priorities 3 and 4.

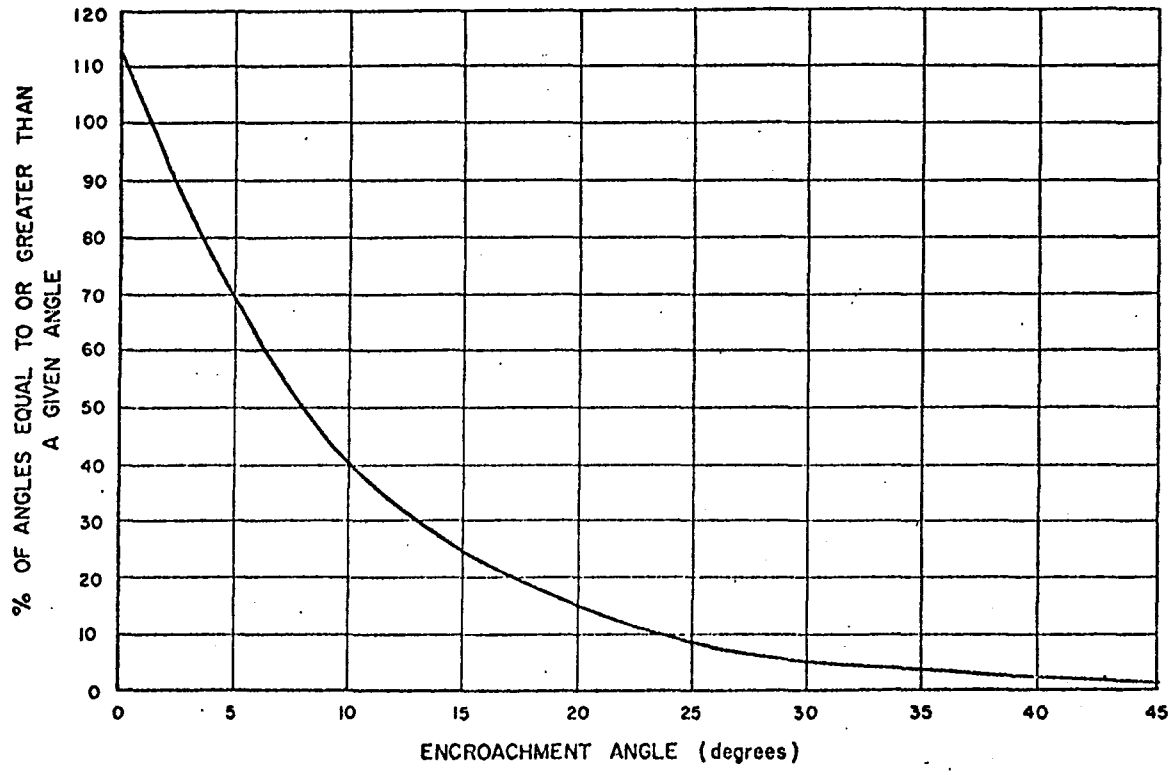


Figure 3.6.2a Distribution of Encroachment Angles

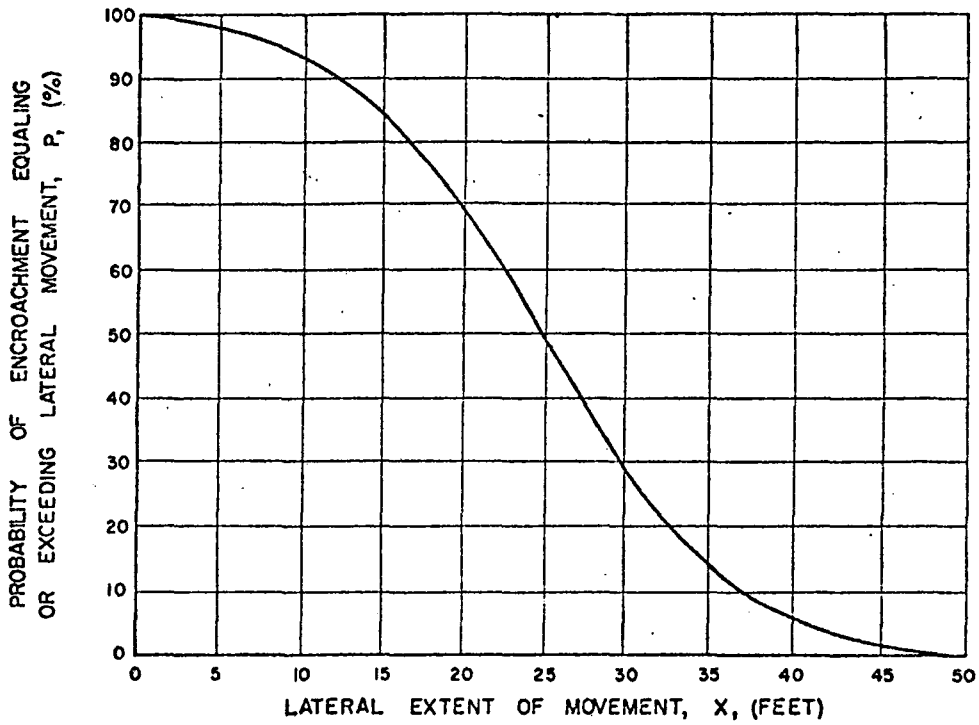


Figure 3.6.2b Distribution of Lateral Displacements of Encroaching Vehicles

Figure 3.6.2 Encroachment Data

3.6.4 ELIMINATION OF HAZARDS

We have already pointed out that elimination of hazards involves the removal of unnecessary fixed objects along the roadside. This can be achieved through a comprehensive field survey and study of traffic operations requirements. Our main concern in this session dealing with the roadside cross section is the control of side slopes, back slopes, and ditches and in the treatment of drainage structures.

Only recently have efforts been directed toward quantifying how the incremental variation of front slopes, back slopes and the ditch region between the slopes relates to accident severity. It is generally accepted that wide flat side slopes, free of fixed objects, contribute greatly to the safety of the roadway, but for a majority of highways, the cost of providing this standard would be prohibitive. The design engineer has been handicapped by the lack of objective criteria in the area of selecting safe combinations of slopes for roadway design. To enable him to evaluate alternatives and thus achieve optimum roadside safety in his design, objective criteria must be made available to him.

3.6.41 Roadside Critical Areas

The sequence of events that can occur when a vehicle leaves the roadway is greatly influenced by the roadside geometry. Three regions of the roadside are particularly important when evaluating the safety aspects: the top of the slope (hinge-point); the front slope, and the toe-of-the-slope (intersection of the front slope forming a ditch). The hinge-point and front slope regions are particularly important with regard to the design of long slopes where a driver could attempt a recovery maneuver or reduce speed before impacting the ditch area. In addition, the hinge-point adds to the loss of steering control because the vehicle tends to become airborne. A driver's normal instinct is to attempt to return to the roadway, and obviously there is a front slope steepness at which the vehicle will roll during a recovery maneuver. This maximum negotiable steepness must be considered in design. Also, there are situations where the toe-of-slope is close to the roadway so that the probability of reaching the ditch is high, in which case safe transition regions between front and back slope must be provided.

Each region affects vehicle response in a different way and for a different set of operating conditions. When individual criteria are determined for each, the pieces may be put together to produce safety guidelines for total roadside slope design.

An NCHRP project (5) has addressed the toe-of-slope region with particular emphasis on safe combinations of slopes forming various ditch shapes. This work involved a combination of mathematical simulation (HVOSM) and full-scale vehicle testing. Other research (6) has investigated the hinge point and front slope region and the driver return maneuver.

The tentative design curves and discussion of vehicle behavior during ditch traversal presented here are based on the NCHRP study. This study included investigation of simulated traversals at 60 mph (37.3 km/h) and 25-degree encroachment angle for all combinations of front and back slopes from 3:1 to 6:1 and 24 full-scale tests on 7:1 front slopes and 3:1, 4:1, and 6:1 back slopes on both vee and round ditches. Four ditch configurations were evaluated: vee ditch, round ditch, trapezoidal ditch, and rounded trapezoidal ditch. Ditch widths were varied from zero (vee ditch) to 16 ft (4.9 m). All traversals including full-scale tests, were conducted in a free-wheeling mode (no driver steering control).

3.6.42 Criteria

The tentative design curves are developed on the basis of a severity index of 1.0 and 1.6 from which safe combinations of front and back slope forming each of the ditch configurations may be selected. The severity indices of 1.0 and 1.6 represent degrees of expected occupant restraint of "no restraint" and "seat belt restraint," respectively, as discussed below.

To fully evaluate the response expected during traversal of a particular ditch configuration, the resultant acceleration of the three axes must be considered. Although vertical G's usually represent the predominant component, lateral and longitudinal G's can be appreciable in certain combinations of slope and ditch shape. Since tolerable (limit) accelerations are not equal in the three axes, a method of evaluating the resultant effect is as follows:

$$SI = \sqrt{\left(\frac{ALON}{G_{XL}}\right)^2 + \left(\frac{ALAT}{G_{YL}}\right)^2 + \left(\frac{AVER}{G_{ZL}}\right)^2}$$

where SI = Severity Index
ALON = Acceleration experienced in longitudinal axis, G's
ALAT = Acceleration experienced in lateral axis, G's
AVER = Acceleration experienced in vertical axis, G's
G_{XL} = Tolerable acceleration in longitudinal (X-axis) direction, G's
G_{YL} = Tolerable acceleration in lateral (Y-axis) direction, G's
G_{ZL} = Tolerable acceleration in vertical (Z-axis) direction, G's

This form follows the elipsoidal theory of

failure indicated by Hyde (7). The normalizing values used in the development of design curves are based on unrestrained occupant values:

G_{XL} = 7 G's

G_{YL} = 5 G's

G_{ZL} = 6 G's

Therefore, the severity index equation becomes:

$$SI = \sqrt{\left(\frac{ALON}{7}\right)^2 + \left(\frac{ALAT}{5}\right)^2 + \left(\frac{AVER}{6}\right)^2}$$

A severity index of 1.0 represents a resultant acceleration which may be safely tolerated by an unrestrained occupant. A severity index of 1.6 represents the upper limit of acceleration considered safe for seat belt restraint.

3.6.43 Effects of Ditch Type

Vee Ditches. Although it might appear contrary to intuitive reasoning for a 60 mph (97 km/h) 23 degree traversal, crossing the vee ditch generally produced g-forces which were less severe than those caused by traversing rounded or trapezoidal ditches having widths of 8 ft (2.4 m) or less, or the rounded trapezoidal ditches having widths of 4 to 8 ft (1.2 to 2.4 m). The effect of ditch width on vehicle acceleration is discussed separately under each of the other three ditches.

The similarity of g-forces produced by crossing the vee ditch and the other ditch configurations was substantiated using several comparison bases: 50-msec average g-forces, event time average g-forces, or the 50-msec severity index particularly for conditions producing severity indices in the range of 1.0 to 1.6. G-forces were greater for pairs of slopes when the steeper slope was located on the front slope.

It was also found that vehicle bumper penetration into the back slope during vee ditch traversal was approximately equal to that occurring in the other three configurations when the width was less than 8 ft (2.4 m) and comparable slope combinations were used.

Rounded Ditches. A phenomenon observed in the basic study using an 8 ft (2.4 m) round ditch was evident throughout the entire width spectrum: round ditches generally produced g-forces that were more severe than the other three configurations for comparable slope combinations. This was particularly evident for steep slope combinations and narrow ditch widths. Crossing a 3:1-4:1 round ditch in the range of 4 to 8 ft (1.2 to 2.4 m) wide is more severe than crossing a vee ditch of comparable slope combination. On the other hand, the round ditch is less than the vee ditch for the flatter slope combinations and widths greater than 4 ft (1.2 m). It was also noted that there was very little difference in severity among all shaped ditches having widths in the 16-ft (4.8-m) range.

Trapezoidal Ditches. The trapezoidal shape is a construction compromise between a round and a vee ditch. From a safety standpoint, the trapezoidal cross-section appears to offer a ditch section that is safer to cross at high speeds particularly for the steeper slope combinations. The g-forces produced by the trapezoidal ditch were, in general, lower than the vee or the rounded ditch.

Rounded Trapezoidal Ditches. Very little safety benefit was realized by rounding the basic trapezoidal cross-section to produce the rounded trapezoidal configuration. The vehicle accelerations for the two trapezoidal cross-sections were very similar for comparable widths and slope combinations; however, the rounded trapezoidal produced slightly lower g forces in several cases.

The rounded corners provide a more gradual transition to and from the flat bottom. The fact that bumper penetration for this ditch section was considerably less than for all other shapes may be attributed to the more gradual transition. Therefore, less severe vehicle damage could be expected during a rounded trapezoidal ditch traversal.

No significant reduction in severity was realized after the ditch width was increased beyond 12 ft (3.6 m). This width apparently is ample to allow the vehicle to stabilize after reaching the flat bottom before it impacts the back slope.

3.6.4 SUMMARY OF DTICH TRAVERSAL STUDIES

The 16 tests on the 4:1 and 5:1 back slopes with a front slope of approximately 7:1 revealed that these combinations could be safely negotiated at speeds up to 60 mph with no rollover hazard and only moderate discomfort if the driver was adequately restrained.

The test driver experienced considerable difficulty in achieving the 20-degree exit angle at speeds of 50 to 60 mph (80 to 97 km/h) due to rear wheel drift, yet he had a 42-ft (12.8-m) wide pavement in which to negotiate the turn. A 25-degree encroachment angle at these speeds can be executed by a professional driver under certain conditions, but probably is too severe for design purposes.

Response is influenced appreciably by the speed and angle at which the vehicle enters the ditch region. At the 25-degree encroachment angle, bumper contact and rear overhang

drag were observed in all tests above 40 mph (64 km/h).

Vertical accelerations comprise the predominant accelerations in ditch traversal and consequently contribute most significantly to the resultant acceleration. Lateral and longitudinal accelerations can be considered to be virtually negligible in round ditch traversals and in vee ditch traversals at speeds less than 50 mph (80 km/h). Although relatively high acceleration "spikes" may be experienced, their time duration is so short that they do not appreciably affect the average acceleration.

When no steering or braking control was applied, the path followed by the test vehicle throughout the maneuver agreed, on a qualitative basis, with the predicted path. In general, very little redirection was observed as the vehicle passed through the ditch.

Although the resultant accelerations observed at each site were higher for the vee ditch than the corresponding round ditch, the vee ditch did not appear to be significantly more severe. The driver considered the vee ditch to be less severe than the round at the higher speeds, whereas the reverse was observed for the low-speed tests.

The vee ditch generally produced g-forces that were less severe than those caused by traversing the round or trapezoidal ditches having widths of 8 ft (2.4 m) or less, or the rounded trapezoidal ditch in the 4 to 8 ft (1.2 to 2.4 m) range.

Round ditches generally produced g-forces that were more severe than the other three configurations for comparable slope combinations, particularly for steep slope combinations and narrow ditch widths.

Very little difference in severity can be expected between the shaped ditches when the width exceeds 16 ft (4.8 m).

The trapezoidal ditch configuration offers a cross-section that is safer than the others to cross at high speeds, particularly for the steeper slope combinations. The g-forces in general, were lower than the vee or the rounded ditch.

Very little safety benefit was realized by rounding the basic trapezoidal cross-section to produce the rounded trapezoidal ditch.

No significant reduction in severity was realized after the trapezoidal ditch was widened beyond 12 ft (3.6 m).

Desirably, slope combinations would be selected such that unrestrained occupants could be expected to sustain no injury and the vehicle would not incur major damage during traversal. However, site conditions such as restricted right-of-way or other factors beyond the designer's control may dictate the use of slope combinations steeper than desirable. Therefore, design curves are presented for both conditions in Figures 3.6.3 and 3.6.4 for various ditch configurations. Only the "desirable" design curve is shown in Figure 3.6.5 because the "limiting" curve lies above 2:1. Slopes steeper than 2:1 are difficult to construct and maintain and are therefore considered not desirable. In Figures 3.6.3 through 3.6.5 the "desirable" design curve is based on a severity index of 1.0 and bumper penetration of 4 to 4.5 in. (10.16 to 11.43 cm) whereas the "maximum" design curve is based on a severity index of 1.6 and bumper penetration of 6.0 in. (15.24 cm).

These curves provide the design engineer with objective criteria for selection of traversable slope combinations and ditch shape under 60 mph/25-degree encroachment conditions such as might be encountered on high-speed facilities. The design curves are applicable for ditch location up to 60 ft (18.3 m) from the edge of the roadway.

3.6.5 USE OF CURBS

Curbs are installed on highways in urban areas on and near bridges, at intersections for lane dividers, near underpasses, and in other selected locations. The diverse functions of curbs include: drainage, delineation, access control, aesthetics, and safety. Also, some configurations are intended to serve as barriers, and some enhance maintenance operations.

Examination of standard designs employed in more than 30 states indicated that these states follow the guidelines set out in the current Blue Book. A study of earlier guidelines (9, 10) suggests that the use of curbs dates to the time when highways were routed through cities. On such street routes, protective islands for pedestrians were necessary. Curbs also were used by passengers when stepping down from running boards of automobiles, and they served to redirect slow-moving automobiles away from sidewalks. Photographs of early divided highways on which speeds were limited clearly show that curbs provide an attractive method of delineating the edges of the roadway. The evolution of curbs has been an orderly process of applying existing practices to new locations.

In such urban areas, provision must be made for pedestrians on bridges and along the roadway. These pedestrian areas usually are separated from the roadway by a curb. Frequently highways are designed for a specific speed, and later the speed limit is increased, yet the geometrics of the highway and appurtenances such as curbs remain the same. Increased speeds, greater traffic volumes, and constantly changing vehicle capabilities

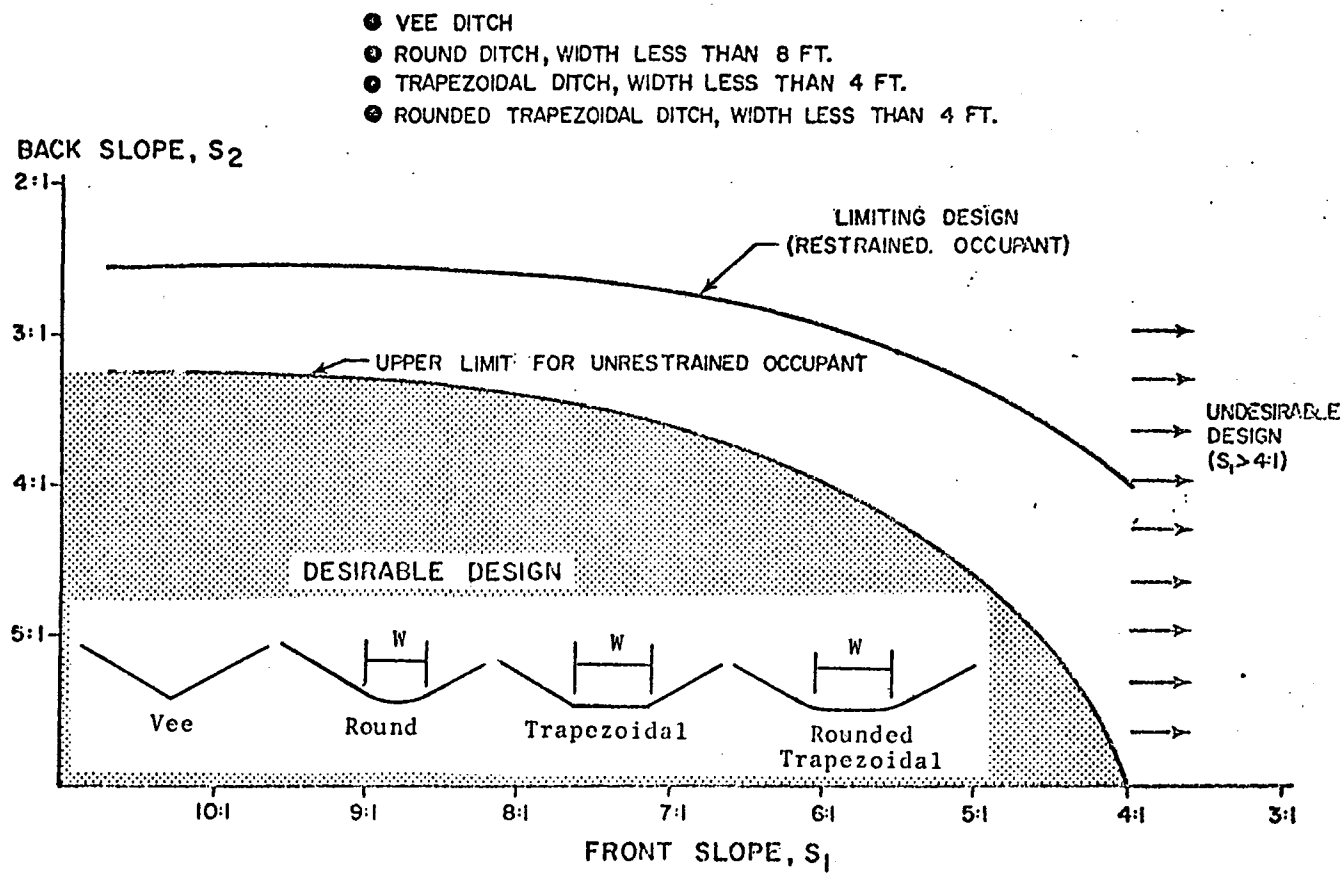


Figure 3.6.3 Tentative Design Recommendations for Roadside Slope Combinations - Curve A

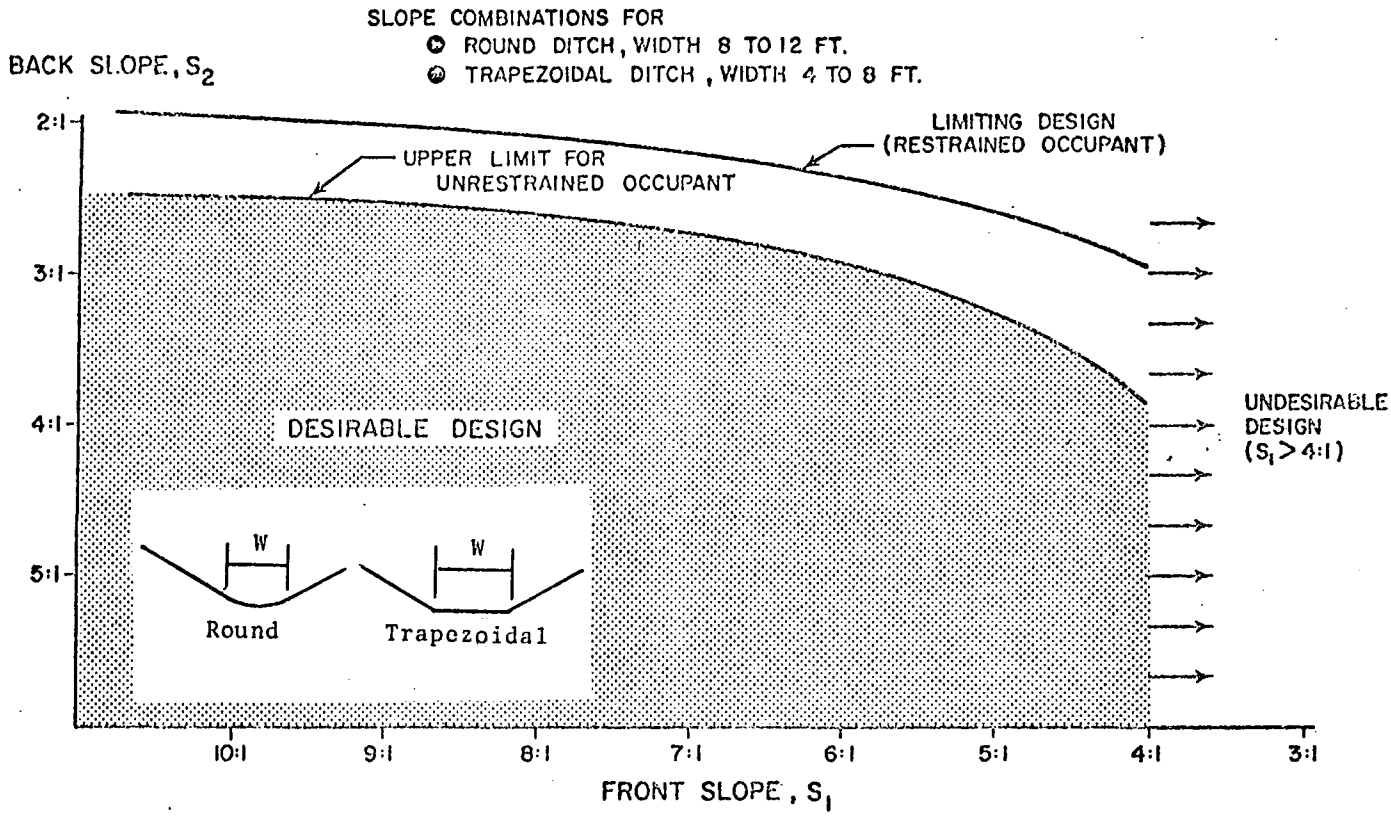


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Figure 3.6.4 Tentative Design Recommendations for Roadside Slope Combinations - Curve B

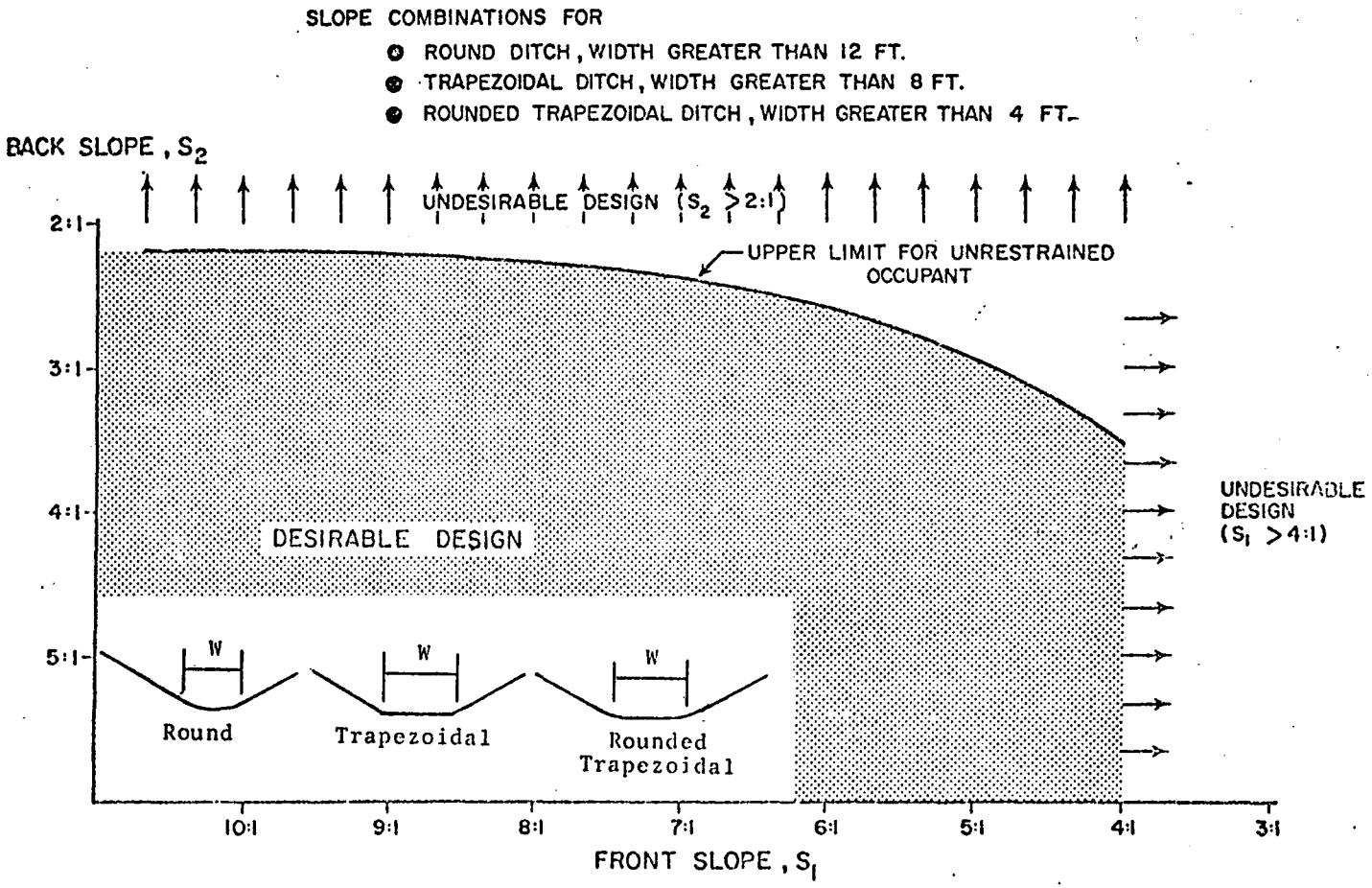


Figure 3.6.5 Tentative Design Recommendations for Roadside Slope Combinations - Curve C

can result in collisions, the severity of which can be aggravated by curbs. Beaton and and Peterson (11) conducted full-scale crash tests in 1953 to ascertain the "...ability of various types of curbing to serve as a physical barrier to cars striking the curb, and also to determine the potential damage to both car and curb." Subsequently, Beaton and Field (12) reported findings of tests on bridge curbs and rails. These studies clearly demonstrated the behavior of an automobile following a collision with a curb. The "jump curves" presented in these earlier studies were examined and led to those presented in the present study.

Employment of mountable curbs in medians and along the edges of roadways and the use of traffic barriers in conjunction with curbs continues in many states. A series of live-driver tests in Washington (13) clearly indicated that a mountable curb in the median did not produce redirection of a speeding automobile. Earlier, full-scale tests on raised medians were conducted by the California Division of Highways in conjunction with development of cable median barriers. Standard-sized automobiles and smaller sports cars easily mounted raised medians having 6 in (15.2 cm) high curbs (14).

In recent years, a sloped face concrete median barrier has been adapted for use on bridges and as a barrier between the edge of the traveled way and fixed hazards such as bridge columns, or steep-cut sections. The use of this configuration seems to be replacing the two-step barrier curb shown in the Blue Book (AASHO Curb Type B) (15). Full-scale tests (16, 17, 18) on "safety shape" median barriers, and on an adaptation of this shape to bridge barriers (19) have led to the current trend for employing such barriers.

Often roadside barriers or bridge barriers are located behind curbs, and the behavior of colliding vehicles has been discussed by others (20, 12, 14). Such installations aggravate a secondary collision incident.

Knowledge of the action on impact should be a major tool in design decisions as to the use (or omission) of curbs and their specific location in relation to the edge of the traveled lane.

An NCHRP study (8) was recently completed, having as its objective investigation of the safety aspects of curbs from the standpoint of vehicle impact.

The approach taken to investigate the effects of curbs on vehicle behavior included a combination of full-scale testing and simulated impacts using the Highway-Vehicle-Object-Simulation-Model (HVOSM). Three curbs (AASHO Types C, E, and H) were selected for detailed study because they represented the most commonly used curb configurations throughout the country. A fourth configuration designated Type X was selected as an experimental "barrier" curb. The dimensions of the 13-in (33-cm) high Type X curb were those of the lower portion of the "New Jersey" concrete median barrier. The study conditions are summarized in Tables 3.6.1 and 3.6.2. Vehicle behavior was evaluated during impact with four curb systems. The results are discussed individually below.

3.6.51 Redirection

None of the AASHO curb designs investigated are satisfactory for installation on high-speed facilities where redirection is the primary design intent. Examination of Figures 3.6.6 and 3.6.7 leads to the conclusion that redirection may be expected when encroachment angles are five degrees or less at speeds in excess of 60 mph (96 km/h).

Conventional curbs do not function as barriers. At present, the most promising highway barrier concepts are the New Jersey "safety shape," and the California Type 20 bridge barrier. It is realized that none of these fit into the curb classification, but it is clear that a height of 2.67 ft (0.8 m) is required to achieve consistent vehicle redirection.

3.6.52 Vehicle Attitude

Curbs similar to AASHO Types C, E, and H can produce vehicle ramping under various combinations of speed and angle impact conditions such that there is a strong possibility that the vehicle will vault a 2.25-ft (0.68-m) guardrail located behind the curb. The guardrail offset distance to restrain a ramped vehicle differs for various angles, speeds, and curb geometry. A secondary collision with guardrail located behind a curb can be compounded if the offset is such that the initial vehicle front-end dipping causes the bumper to snag beneath the rail face.

Obviously, it is uneconomical to remove all curb in front of guardrail; however, use of rubbing rails is recommended to alleviate bumper snagging. The deep face rail (Thrie-Beam) can also be used effectively.

Maximum bumper rise occurs in the range of 8-10 ft (2.4 - 3 m) behind the 6-in (15.2-cm) curbs. Therefore, existing curb/guardrail combinations in which the rail offset is in this range should be considered most critical.

3.6.53 Vehicle Accelerations

Curbs 6 in (15.2 cm) high or less produce slight vehicle accelerations. However, although decelerations are slight, a curb will aggravate any collision resulting off the traveled lane because it represents a discontinuity in the vehicle path with which the driver must contend. Additionally, curb impact at high speeds (35 mph or greater) is capable of damaging the vehicle steering mechanism which diminishes control of a car by its operator.

Curbs offer no enhancement to safety on high-speed highways from the viewpoint of vehicle behavior following impact. For this reason, it is recommended that the use of curbs on high-speed highways be discontinued.

Figures 3.6.6 through 3.6.7 indicate that curbs may have potential redirection capabilities on low-speed facilities; however, the decision to construct them should be based on considerations other than redirection alone. Typical reasons for curb installation include access control, delineation and drainage. Delineation and drainage may be achieved by other means which do not produce a discontinuity in the roadway.

Curbs located in front of traffic barriers can aggravate a secondary collision with the guardrail by producing vehicle ramping. Curbs should not be used in front of guardrails, and consideration should be given to removing existing curbs in front of traffic barriers on high-speed highways. The AASHTO Barrier Design Guide (22) should be followed when barriers and curbs are used together.

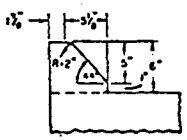
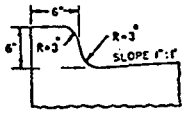
3.6.6 SAFETY DESIGN OF ROADSIDE DRAINAGE STRUCTURES

As discussed in an NCHRP publication (1), some highway drainage structures are potentially hazardous and, if located in the path of an errant vehicle, can substantially increase the probability and/or severity of an accident. Sound engineering can minimize a number of these hazards and modify the form of remaining hazards to reduce the number and severity of accidents. NCHRP 3 outlines four principal objectives for providing safer roadsides. In order of priority they are:

- Unnecessary drainage structures should be eliminated.
- Necessary drainage structures should be located so that they create the least possible hazard.
- Structures that cannot be eliminated should be designed to inflict minimum damage.
- Where the first three objectives cannot be feasibly accomplished, a traffic barrier should be installed.

TABLE 3.6.1

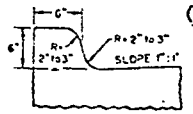
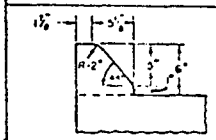
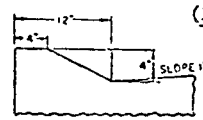
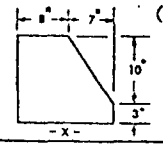
FULL-SCALE TEST SEQUENCE

	Test Reference Number	Curb Type	Speed (mph)	Angle (degrees)
 TYPE E CURB	N-1	E	30	5
	N-2 (rerun)	E	30	5
	N-3	E	45	5
	N-4	E	60	5
	N-5	E	30	12.5
	N-6	E	45	12.5
	N-7	E	60	12.5
	N-8	E	30	20
	N-9	E	45	20
	N-10	E	60	20
 TYPE C CURB	N-11	C	30	5
	N-12	C	45	5
	N-13	C	30	12.5
	N-14	C	45	12.5
	N-15	C	30	20
	N-16	C	45	20
	N-17	C	60	5
	N-18	C	60	12.5
	N-19	C	60	20

Note: All tests conducted in a "hands-off" free-wheeling mode.

TABLE 3.6.2

CURB COLLISIONS SIMULATED BY HVOSM
(48 SIMULATED TESTS)

Curb	Encroachment Angle (degrees)	Speed (mph) 30	45	60	75
 (1) -C-	5	X	X	X	X
	10				X
	12.5	X	X	X	
	15				X
	20	X	X	X	
 (1) -E-	5	X	X	X	X
	10				X
	12.5	X	X	X	
	15				X
	20	X	X	X	
 (1) -H-	5	X	X	X	X
	10				X
	12.5	X	X	X	
	15				X
	20	X	X	X	
 (2) -X-	5	X	X	X	X
	10				X
	12.5	X	X	X	
	15				X
	20	X	X	X	

Notes

(1) Ref. 1965 AASHO Blue Book, pg. 228.

(2) Modified form of New Jersey concrete median barrier.

(3) All simulation conducted in "hands-off" free-wheeling mode.

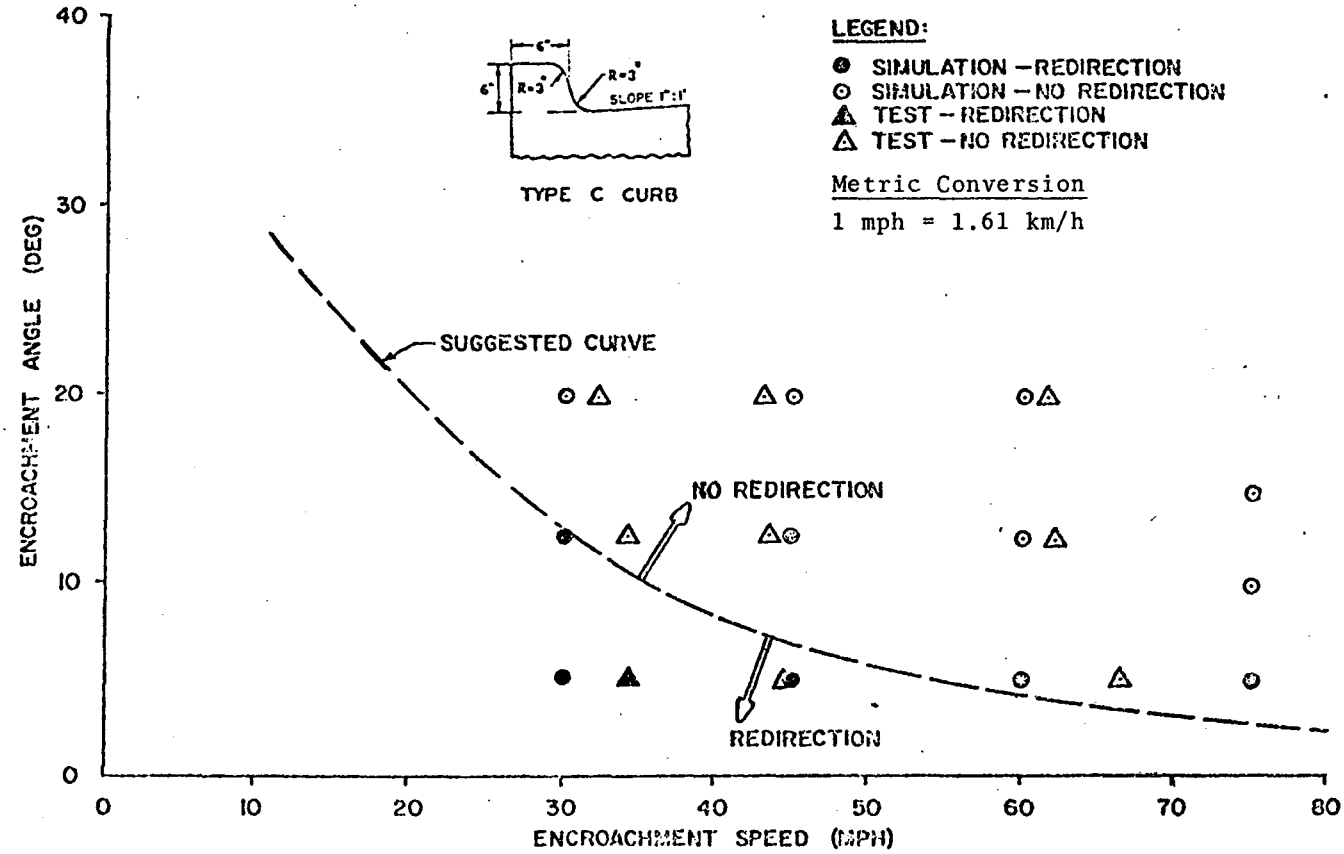


Figure 3.6.6 Vehicle Redirection Capabilities of Type C Curb

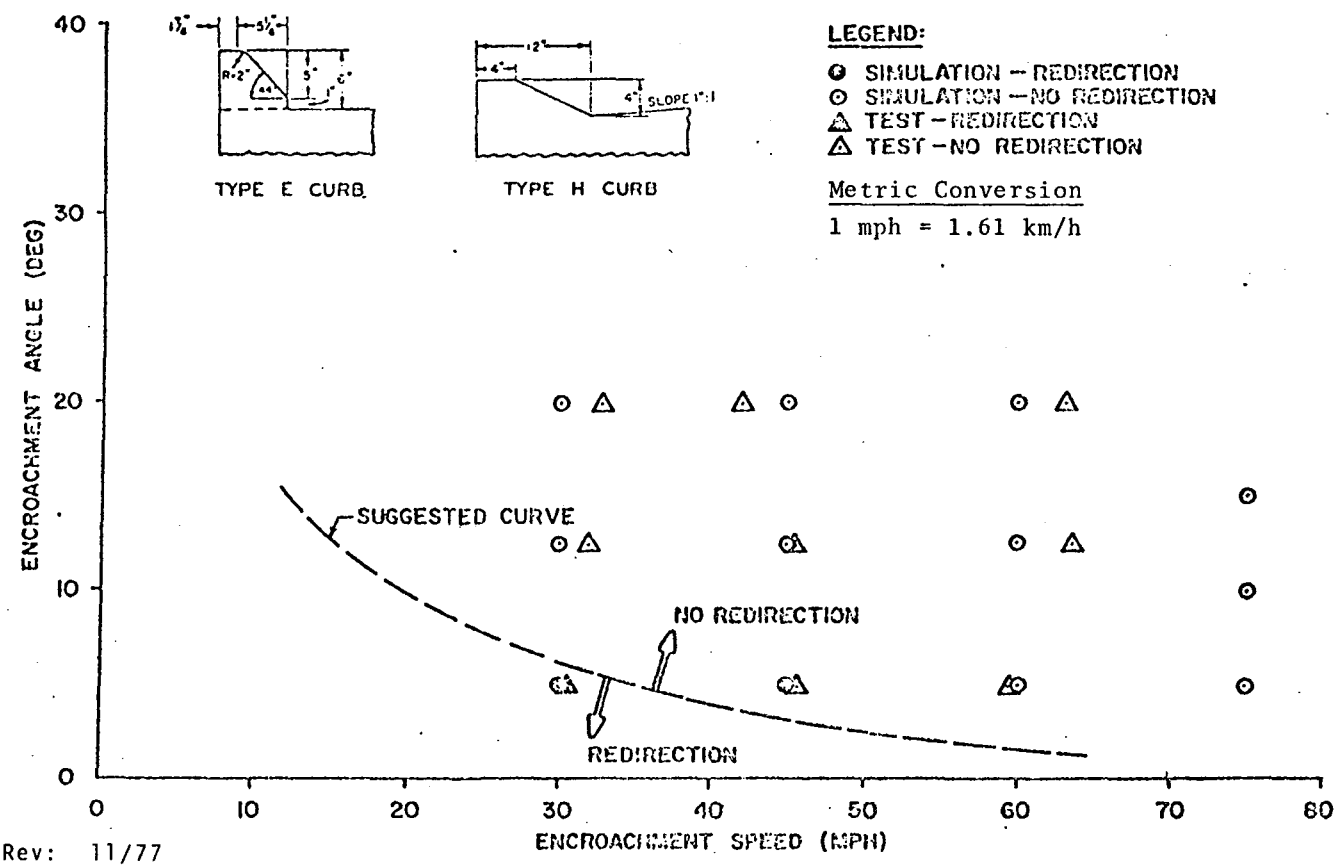


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Figure 3.6.7 Vehicle Redirection Capabilities of Types E and H Curbs

The principal function of the drainage structure should not be overlooked when considering the above objectives; therefore, traffic-safe design should not be allowed to seriously impair the hydraulic function of the structure. Consideration of hydraulic requirements may result in increasing the size, number, or length of drainage structures to satisfy safety objectives. In the final analysis, judgments may need to be based on economic considerations of a number of alternative designs. Traffic-safe drainage structures should not have steep faces protruding above the ground or steep-sided depressions below the surface.

3.6.61 Drop Inlets

Drop inlets, usually located in medians, fall within two categories: Flush-type and projecting.

Flush type usually consist of metal grates mounted atop concrete catch basins and are traffic-safe in that a vehicle can pass over them without striking an obstruction. The grate must have sufficient strength to support the design wheel with bars spaced and arranged such as to prevent penetration by the narrowest vehicle tire using the facility. Other desirable characteristics of flush median drop inlets are:

- Hydraulic efficiency of a flush-type grated median is satisfactory in as much as debris usually is not a problem.

- A wider or longer inlet will reduce bypass flow during peak runoff.

- Orientation of the bars should be parallel to the flow for greater hydraulic efficiency.

- The concrete apron around the inlet discourages overgrowth, improves inlet efficiency, assists in maintenance, and reduces erosion.

Projecting drop inlets present serious safety problems for vehicles striking the projecting surfaces, particularly since they usually are located quite close to the roadway. Projecting inlets include concrete slab-topped inlets (table-top), metal gratetopped concrete catch basins, and inlets flush with the ground on three sides but having an exposed vertical face where the flow enters the structure.

Vertical projections should not be permitted except where medians are very wide and the structure is beyond the recovery area. Projecting inlets should be replaced by flush inlets where possible, or as a last resort, protected by guardrail or other barriers.

3.6.62 Culvert Headwalls

In selecting a satisfactory design, it is necessary to consider the physical condition of the site, the hydraulic requirements and the cost of alternative treatments. Three general solutions have been used:

- Lengthening the culvert to place the ends beyond the specified recovery area.

- Modifying the culvert end structure to accept a grate that is designed to carry a vehicle.

- Installing a roadside barrier to protect traffic.

In comparing these alternatives, the location and form of any longitudinal open channels should be taken into consideration. Steep-sided longitudinal channels are hazards in themselves; relocation of the channel together with extending the culvert may prove to be the best overall solution. At many locations, the hydraulic efficiency of grated inlets may be impaired by the collection of debris. In such cases, lengthening of the culvert may result in improved reliability and lower maintenance costs.

Listed below are desirable characteristics of culvert headwalls:

- Culvert end structure should be located outside the designated recovery area where there is sufficient right-of-way.

- Vehicle should be protected from falling into the structure by grates when the installation is within the recovery area.

- Any projection of the end structure above the ground surface should be minimal.

- When the above cannot be accomplished, vehicle occupants should be protected by sufficient length of properly installed guardrail.

- Guardrail should not be installed if it constitutes a greater hazard than the drainage structure itself.

- When possible, culvert end structures should be oriented away from the direction of traffic.

3.6.63 Access Road Culvert Headwalls

Most culverts are constructed to carry streamflow across the right-of-way. Access roads present special problems with culvert design because the culvert headwall is exposed directly to traffic flow either in a median crossover or on a side access road. The abrupt culvert headwall and associated steeper drainage slopes create a "pocket" in which an errant vehicle can become trapped.

Side access road culverts can often be moved laterally beyond the normal recovery area; however, median crossover culverts are usually close to the travel lanes and must be protected

by other means. Sloped grates offer one method of reducing the severity of impact. A computer simulation of a vehicle traveling over a grate slope of 10:1 resulted in a non-injury vehicle action. Slopes of 6:1 produced vehicle rollover (actually pitch over) even on ditches 2 to 3 feet (0.61 to 0.91 m) deep. Some designers advocate a maximum longitudinal slope no steeper than 20:1 in ditch areas. The dynamic vertical tire load on the sloping grate is about 5 times the static wheel load; therefore, the grate must be adequately designed to accommodate this loading.

3.6.7 RELOCATION OF APPURTENANCES

We have already discussed to some degree the relocation of appurtenances to improve roadside safety. One of the classic examples of relocation is the removal of the big T-mount gore sign. The T-mount gore sign is perhaps one of the greatest errors we have made in the application of signing technology. It was intended to present information relative to the adjacent exit plus information relative to the subsequent exits, all on one support. The decision to utilize the gore mount for information relative to the adjacent exit was a mistake. At the point the driver can see the sign and respond to it, it is too late for him to negotiate the exit safely. This information is needed in advance of the exit ramp and should be presented either in a series of roadside signs, or on an overhead sign when there are 3 or more lanes on the facility. The only information needed at the exit ramp is a confirmation and definition of the exit point. This is accomplished effectively by the common "EXIT" sign. For a detailed treatment of the guide signing procedure, one should refer to the Manual on Uniform Traffic Control Devices or the State Manual.

Wherever practicable, overhead signs should be attached to bridge structures. Bridge designers are likely to point out all of the negative effects, such as the bridge was not designed for that purpose; there are no mounting brackets; and wind loading effects were not included in the bridge design; also, the backs of the signs protruding up above the bridge rail will be detrimental to the visual aesthetics from the overcrossing roadway. From a positive viewpoint, the signs constitute virtually no additional wind load above and beyond that imposed by the bridge rails; and drivers rarely notice the backs of signs, and if they do they can be dressed up; one should be able to economize significantly by eliminating an expensive structure, and, at the same time, improve the roadside safety through the elimination of a major fixed object. If it is not practicable to locate the sign on a nearby bridge, then a sign bridge structure should be employed. There are several ways in which this structure may be treated to enhance the roadside safety. First, there is the possibility of making an overhead sign support breakaway. A breakaway overhead sign support was developed by the Texas Transportation Institute in a multistate cooperative research project (21).

The overhead sign bridge can be moved longitudinally for several hundred feet without major consequence in most cases. The designer should remember that drivers tend to key on the sign; that is, drivers will tend to take the action indicated at the sign. This means that the sign should be located at or in advance of the point where the designer wants the action to take place. Thus, an overhead sign pertaining to an exit ramp can be located several hundred feet in advance of the beginning of the exit maneuver but should not be located beyond the exit point.

In recent years, designers have reconsidered practices in bridge design relative to the use and location of bridge piers in the design of interchange structures. Often times designers used 3 and 4 sets of piers in order to keep spans short and more economical. Now they realize that this may be a false economy and have increased span lengths to improve the safety of the roadside.

A great safety advantage may be realized by the lateral relocation of many devices. These include primarily luminaire supports which can be moved to the median and protected by a median barrier, installed for other reasons. Also, sign supports may be integrated with the median barrier, eliminating their hazard as roadside obstacles.

Where there is no median barrier or where it is not practical to move objects to the median, they may be moved laterally from the roadway. This may be done with signs and with luminaire supports. Lateral relocation should be performed to reduce the probability of impact and/or to locate the objects behind a traffic barrier, on a back slope, or on or behind a retaining wall. In the case of luminaire supports, the designer may vary the mast arm length in order to achieve an optimum location for the support. One outstanding example of this practice is the design of the lighting system for I-405 in Portland, Oregon.

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TOPIC 3 SESSION 7
INTERSECTION DESIGN

Objectives:

1. The participant should be able to identify the driver's needs at intersections,

2. Be able to apply realistic sight distance criteria in intersection design, and

3. Select intersection design features to maximize the safety and efficiency of intersection operation.

3.7.1 INTRODUCTION

In the development of this session, it was assumed that the participant has available the latest copies of the Blue Book and the Red Book. The intricate detail of intersection design criteria and design elements are left to the participant to gain through study of these documents. Materials presented herein relate primarily to supplementing these documents. Extractions are utilized only in unique circumstances.

3.7.2 FUNCTIONAL CLASSIFICATIONS OF INTERSECTIONS

While the classification of streets is necessary for effective communication throughout the planning-design-operation process, a functional intersection classification system is greatly needed to properly coordinate design and traffic operations. A system of intersection classification has been devised to reflect the operational and design requirements based on the types of facilities comprising the intersection, as follows:

1. Minor to Minor
2. Major to Minor
3. Major to Major
4. Major to Expressway/Freeway

A description of each of the intersection classes is provided in the subsequent paragraphs.

3.7.21 Minor-Minor Intersection

This is an intersection of two low-volume roads, characterized by the intersection of two residential streets. The basic intersection is not greatly different from the two intersecting roadway cross sections.

Control--Basic right-of-way rule applies.

Operational requirements

- Drivers must be able to identify the intersection.
- Drivers must be able to see each other and judge speed and distance.

Design requirements

- Horizontal and vertical alignment must be compatible with operational requirements.
- Paths on approach and through intersection must be natural.
- Sight distance must be sufficient for conditions.
- Design must be compatible with environmental conditions.

3.7.22 Major-Minor Intersection

Because of intensified operating conditions, some changes in configuration are frequently required to accommodate changes in operation.

Control--Priority of use assigned by law and reinforced by stop signs or signals.

Operational requirements

- Drivers on the minor road must be able to detect the importance of the major road.
- Drivers on the minor road must have sight distance to judge speed, distance, and gaps in major traffic streams from stopped position.
- Drivers on the major road must be able to identify the minor road.
- Drivers on the major road must be able to see a vehicle stopped on the minor road.

Design requirements

- Horizontal and vertical alignment must be compatible with operational requirements.
- Paths on approach and through intersection must be natural.
- Sight distance must be sufficient for conditions.
- Design must be compatible with environmental conditions.

3.7.23 Major-Major Intersection

Not only are there drastic changes in operating conditions, but now capacity considerations are necessary due to the regulated sharing of common space. The configuration of the intersection is now the principal determinant of operational safety and efficiency.

Control--Priority is not specifically assigned by law except basic right-of-way rule; authority is given to establish control devices; type of control (present and future) should be established prior to design (yield, two-way stop, four-way stop, signals).

Operational requirements

- Drivers on each approach must be able to detect the importance of the intersecting roadway.
- Drivers must be able to determine the required lane or position in order to accomplish the intended maneuver.
- Drivers must be able to identify and respond to the control device.
- Drivers must be able to identify the requirements of the intersecting traffic stream and judge the characteristics of that stream as they relate to the maneuver.

Design requirements

- Horizontal and vertical alignment must be compatible with operational requirements.
- Paths on approach and through intersection must be natural.
- Traffic should be separated according to maneuvers.
- Turning maneuvers should be physically accommodated.
- Turning roadways should be designed for reasonable speeds.
- Good transitions to turning roadways and auxiliary lanes should be provided.
- Sight distance must be sufficient for conditions.
- Design must be compatible with environmental conditions.

3.7.24 Major To Expressway/Freeway

The grade separation or low-order interchange emerges when one or both of two operating conditions prevail: the at-grade intersection can no longer accommodate the demand volume, or it is desirable to have continuous flow on one or both facilities.

Control--Priority assigned by law (single lane entering multiple lanes); control devices are provided for any right angle crossings.

Operational requirements

- Requirements for intersections within the interchange are the same as for the major-minor or major-major intersection.
- Operational requirements of ramps

+ The driver must be able to define the maneuver points in adequate time to respond.

+ The driver must be able to transition naturally from the freeway environment to the highway environment.

+ The driver must be able to comprehend and respond to control measures and directional signing in the transition from freeway to highway environment.

Design requirements

- All of the requirements for major-major intersections apply.
- Additional requirements for ramps

+ Exit ramps should be single lane at the gore.

+ Exit ramps should leave at flat angle (4 to 5 degrees) and have an adequate taper.

+ Exit ramps should not have critical vertical alignment.

+ Exit ramps should be of sufficient length to permit normal deceleration.

+ Exit ramps should always leave main lanes on a tangent section.

+ The section from the exit gore to the intersection of the cross road or frontage road should be designed as a system to facilitate the environmental transition.

+ Entrance ramps should also be single lane.

+ Entrance ramps should approach at flat angles (aim them to the taper) and should be at essentially the same grade as main lanes.

+ Entrance ramps should always join main lanes on a tangent section.

+ The taper of the entrance ramp is most important. A long, gradual taper fits the natural manevuer.

+ Entrance and exit ramps should always be on the right. Major bifurcations are a very different problem. All must satisfy driving expectancy to every extent possible.

3.7.3 SAFETY DESIGN ELEMENTS

A detailed treatment of the design elements for urban intersections at grade is provided on pages 675 -725 of the Red Book. There are however, some elements that are specifically related to safety and should be treated in greater detail or on the basis of a different interpretation.

3.7.31 Sight Distance

Several different approaches for considering sight distance and the design of intersections are available to us. Most commonly, stopping sight distance is a basic rule - stopping sight distance should be provided at every point along any given facility. Stopping sight distance, however, is not sufficient for intersection design. Generally, the perception-reaction time used in computing stopping sight distance is insufficient for intersection requirements. In approaching the intersection, the driver must be able to interpret the geometry, reach a decision as to what will be required of him, and carry out whatever maneuvers may be required to negotiate the intersection. Early, we covered several approaches to sight distance at decision points. At that time they were related primarily to freeway interchange and lane drop conditions. Here, they should be related to the intersection. Table 3.7.1 provides decision sight distance, sight distances which should be appropriate for intersections. It can be noted that these decision-sight distances provide for perception-time and for a reaction time related to lane changing. This should be sufficient for most intersection maneuvers.

The measurement criteria for the sight distances are equally important. The recommended eye height of 3.75 feet (1.14 m) is representative of practically all drivers. The 6-inch (0.15 m) object height normally used for stopping sight distance seems inappropriate as does the 4.5 foot (1.37 m) object height normally associated with passing sight distance. In the intersection, it is necessary, most generally, that the driver be able to see the intersection surface; in other words, there is a zero object height. Only under these conditions can the driver observe and place into proper perspective the vertical relief features of the intersection.

In addition, provisions must be made for the driver to see other vehicles approaching the intersection. There are three conditions or cases of intersection sight distance. The determination of intersection sight distances in these three cases is made using one of the two schematics shown in Figure 3.7.1.

Case I--Enabling Vehicles to Adjust Speed. At uncontrolled intersections, it may be desirable to provide for drivers to simply adjust speed to avoid a collision. This case should be limited to low-volume, low-speed intersections. AASHTO recommends that 2 seconds be allowed for perception-reaction time, plus 1 additional second to activate the brake or accelerator. Distances traveled in three seconds are as follows:

Speed, mph	20	30	40	50	60	70
Speed, km/h	32	48	64	80	96	112
Distance, ft	90	130	180	220	260	310
Distance, m	27	40	53	67	80	93

Substituting these distances in the sight triangle schematic for d_a and d_b will facilitate the determination of a clear sight triangle. These distances are quite critical and should be used with extreme care.

TABLE 3.7.1 DECISION SIGHT DISTANCE

Design Speed mph (kph)	Times				Decision Sight Distance	
	Pre-Maneuver		Maneuver (Lane Change) sec.	Summation sec.	Computed ft. (m)	Rounded for Design ft. (m)
	Detection and Recognition sec.	Decision and Response Initiation sec.				
30 (48)	1.5	4.2 - 6.6	4.5	10.2 - 12.6	449 - 556 (137 - 170)	450 - 550 (137 - 168)
40 (64)	1.5	4.2 - 6.6	4.5	10.2 - 12.6	559 - 741 (183 - 226)	600 - 750 (183 - 229)
50 (80)	1.5	4.2 - 6.6	4.0	9.7 - 12.1	713 - 889 (217 - 271)	725 - 900 (221 - 274)
60 (97)	2.0	4.7 - 7.1	4.0	10.7 - 13.1	944 - 1155 (288 - 352)	950 - 1175 (290 - 358)
70 (113)	2.0	4.7 - 7.1	3.5	10.2 - 12.6	1050 - 1296 (320 - 395)	1050 - 1300 (320 - 396)
80 (129)	2.0	4.7 - 7.1	3.5	10.2 - 12.6	1199 - 1482 (366 - 452)	1200 - 1500 (366 - 457)

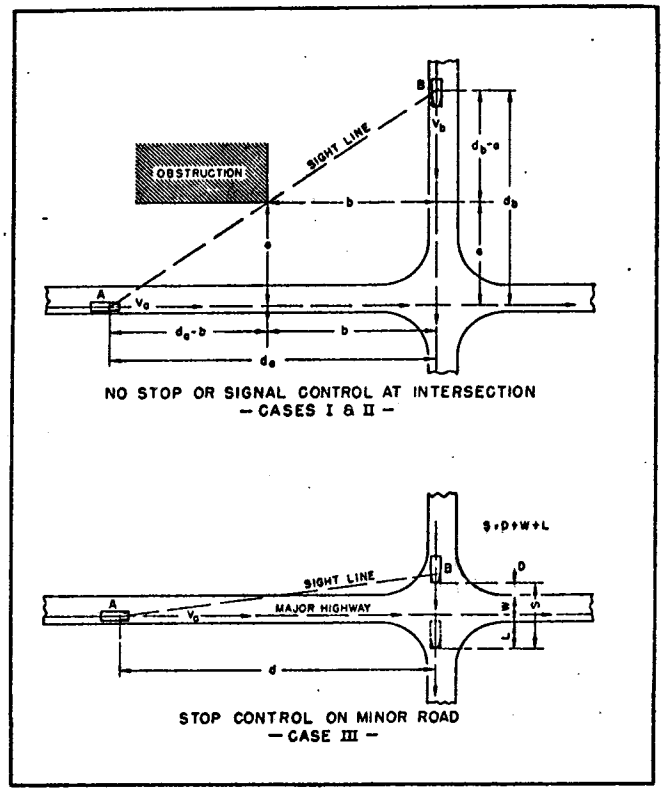


Figure 3.7.1 Sight Distance At Intersections --Minimum Sight Triangle

Case II--Enabling Vehicles to Stop. In comparison to Case I, it is more desirable to provide for both vehicles approaching an uncontrolled intersection to be able to stop before arriving at the intersection. The stopping distances recommended by AASHTO are as follows:

Design Speed, mph	30	40	50	60	70
Design Speed, km/h	48	64	80	96	112
Safe Stopping Distance, feet					
Minimum	200	275	350	475	600
Desirable	200	300	450	650	850
Safe Stopping Distance, metres					
Minimum	61	84	107	145	183
Desirable	61	92	137	198	259

These values are substituted in the sight triangle for distances d_a and d_b, or they may be used in the following formula developed by similar triangles from this sight-triangle:

$$d_b = \frac{a\ d_a}{d_a - b}$$

Case III--Enabling Stopped Vehicles to Cross a Major Highway. Where stop control is used on a minor road, it is necessary to provide for the driver of the stopped vehicle to see along the major highway a sufficient distance for him to cross the highway without interfering with oncoming vehicles. This is illustrated in the Case III schematic. The sight distance, d, is computed as follows:

$$d = 1.47\ V\ (J + t_a)$$

where, V = design speed on major highway, mph

J = perception and reaction time, sec

t_a = time required to traverse distance S = D + W + L

The time, t_a, varies with different drivers and vehicles, but AASHTO design information is provided on the graph (see Figure 3.8.2).

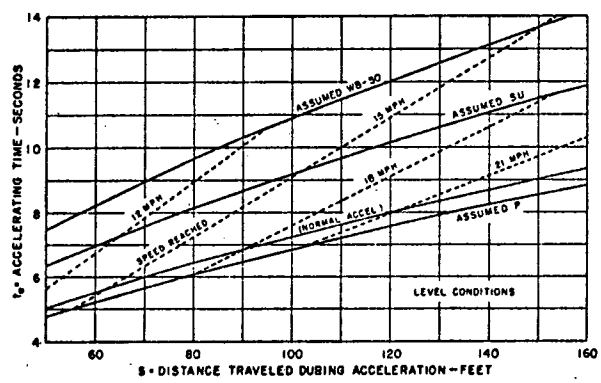


Figure 3.7.2 Sight Distance At Intersections --Case III Data On Acceleration From Stop

3.7.32 Turning Radii

The radius of curb returns seems to be one of the most controversial and variable elements of intersection design. AASHTO in its Blue Book and Red Book present very comprehensive and complicated treatments while the municipalities tend to practice a very simplified treatment. The attitude of AASHTO on urban design seems to be summed up in the following quotation from the Red Book.

"In conclusion, corner radii at intersections on arterial streets should satisfy the requirements of the vehicles using them to the extent practical in consideration of right-of-way available, angle of intersection, pedestrians, width and number of lanes on the intersecting streets, and speed reductions. The following summary is offered as a guide:

- Radii of 15 to 25 feet (4.5 to 7.6 m) are adequate for passenger vehicles. These may be provided at minor cross streets where there is little occasion for trucks to turn, or at major intersections where there are parking lanes. Where the street has sufficient capacity to retain the curb lane as a parking lane for the foreseeable future, parking should be restricted for appropriate distances from the crossing.

- Radii of 25 feet (7.6 m) or more at minor cross streets should be provided on new construction and on reconstruction where space permits.

- Radii of 30 feet (9.1 m) or more at minor cross streets should be provided where feasible so that an occasional truck can turn without too much encroachment.

- Radii of 40 feet (12.2 m) or more, and preferably three-centered compound curves or simple curves with tapers to fit the paths of appropriate design vehicles, should be provided where large truck combinations and buses turn frequently. Larger radii are also desirable where speed reductions would cause problems.

- Radii dimensions should be coordinated with crosswalk distances or special designs to make crosswalks safer for pedestrians.

Corner curb radii on two-way streets have little effect on left-turning movements. Where the width of the arterial street is equivalent to four or more lanes, generally there is no problem of encroachment by left-turning vehicles."

In a survey of 24 cities (3), APWA found a range of curb radius of 15 to 50 feet (4.5 to 15.2 m) with 15 to 30 feet (4.5 to 9.1 m) being most common. In their survey, they found a consensus of opinion that large turning radii are necessary at major intersections to handle the turning movements of large vehicles. Many municipal engineers feel, however, that radii greater than 30 feet (9.1 m) cause problems for pedestrians and, in some cases, for traffic signal placement. To minimize these problems, it is suggested that, when a radius larger than 30 feet (9.1 m) is necessary to provide adequate vehicle turning movement, a radius of about 75 feet (22.9 m) be used in conjunction with intersection channelization. The results of the APWA survey are presented in Table 3.7.2.

The ultimate objective in standards for turning radii is to satisfy the design vehicle requirements. The new Red Book presents scale drawings of vehicle paths for all design vehicles, the passenger car up through the WB-60. Included in this series is a new design vehicle, the BUS. It is similar in many respects to the SU vehicle which was previously utilized to depict buses, except the BUS has a 25-foot (7.6-m) wheel base comparable to newer large buses being used in urban transit systems. These design vehicle paths except for the passenger cars are reproduced in Figures 3.7.3 through 3.7.7 in scale dimensions so they may be utilized to make transparencies for design purposes.

TABLE 3.7.2 CURB, GUTTER, AND CORNER RADIUS STANDARDS

Jurisdiction	Curb Height (in.)	Gutter Width (ft.)	Corner Radius (ft.)
Atlanta, Ga.	6[1]	2	25
Baltimore, Md.	8	2	15
Chicago, Il.	6[2]	1	15
Cincinnati, Oh.	6	2[3]	30
Dade County, Fl.	6	1½	40
Denver, Co.	6	2	30
Detroit, Mi.	7	—	15[4]
Eugene, Or.	6	1½	16
Ft. Worth, Tx.	7[5]	...	20
Glendale, Ca.	8	...	15
Kansas City, Mo.	7	2	25
Lansing, Mi.	6	1½	15-30
Los Angeles, Ca.	8[6]	2	25[4]
Louisville, Ky.	6	1½[7]	15
Middletown, Oh.	6[8]	2	30[9]
Omaha, Nb.	6	1½[10]	50
Pasadena, Ca.	8	...	25
Phoenix, Az.	6	...	20 30
Portland, Or.	6	—	15, 25
Salem, Or.	6	—	20 30
Skokie, Il.	6	1	...
Toronto, On.	5	1	25[11]
Tulsa, Ok.	6	—	30
Washington, D.C.	6, 8	—	20

	Range	Median	Mode	Average
curb height:	5-8"	6"	6"	6.5"
gutterwidth:	1-2'	1.5'	2'	1.6'
corner radius:	15-50'	25'	15'	23.4'

Notes:
[1] 10" where heavy equipment
[2] 9" on median (state roads)
[3] on state constructed roads
[4] 20' at arterial intersections
[5] 6" on medians
[6] 35' at freeways
[7] seldom built
[8] 7" on medians
[9] 3-centered curve
[10] curb generally integral with pavement
[11] 50-55' where heavy bus traffic

1 ft = 0.303 m

3.7.33 Lane Widths

Lane widths for urban arterials are also controversial and variable from one agency to another. The Red Book points out that 12-foot (3.66 m) lanes are desirable and required on freeways and principal arterials. Where right-of-way is limited, the Red Book suggests a minimum of 11-foot (3.35 m) lanes.

The APWA survey generally supports the AASHTO position as illustrated in Table 3.7.3. In the approaches to intersections, however, cities sometimes relax their standards in order to achieve certain lane arrangements. It will be noted in Table 3.7.3 that turning lanes are most generally much narrower than the through lanes. Further, even the through lanes can be reduced in width with a reasonable degree of success if the lengths of reduced lane widths are maintained rather short. Certainly, it is not recommended that through lanes be reduced in width as a general practice. However, if a more workable arrangement can be achieved only through reducing lane widths, then such should be given consideration. As an extreme example, Figure 3.7.8 shows how the city of Houston gained left-turn lanes within a 44-foot (13.4 m) cross section (4). This was done on an experimental basis, and at last report the accident experience had reduced significantly, and the delay at the intersection was reduced substantially.

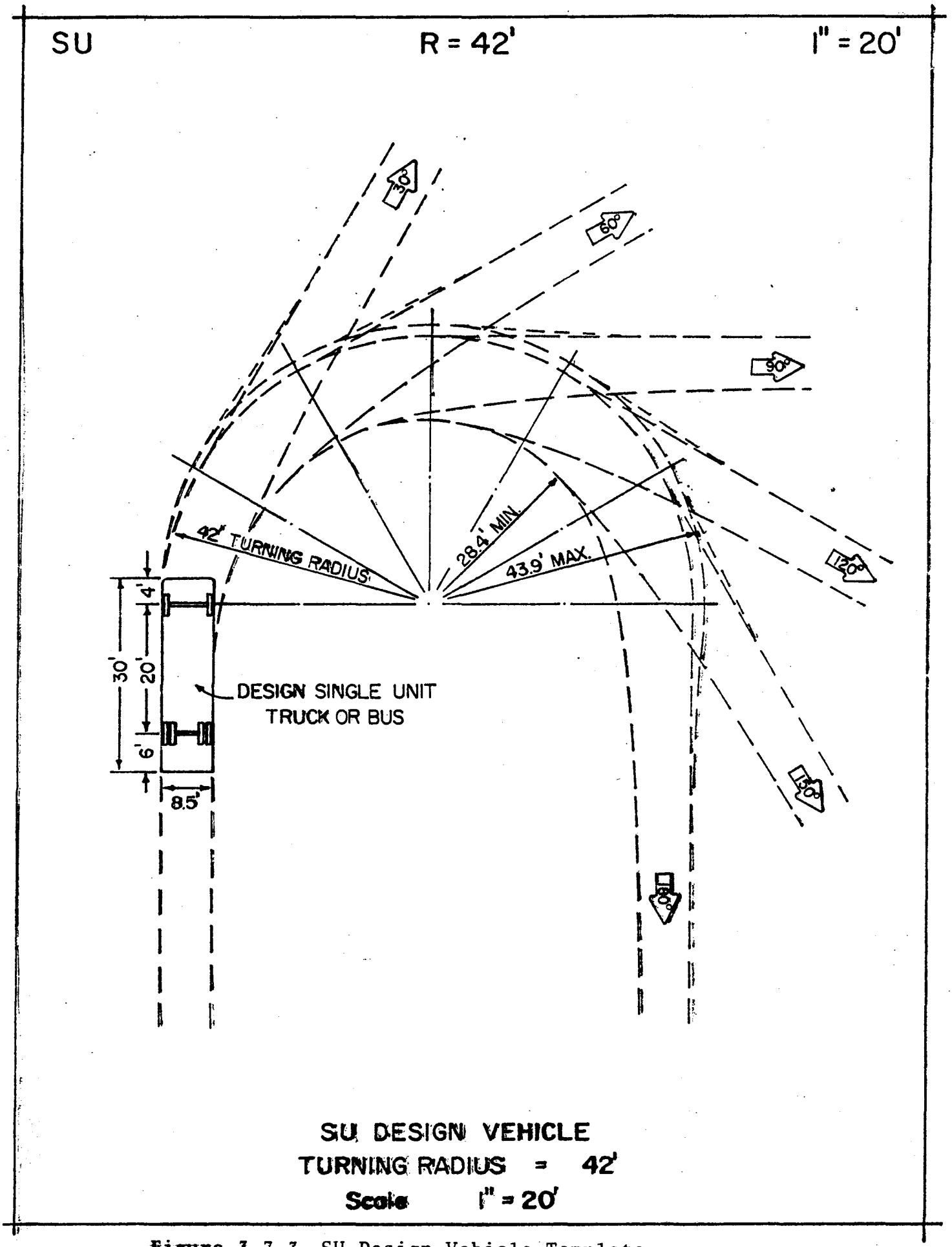


Figure 3.7.3 SU Design Vehicle Template

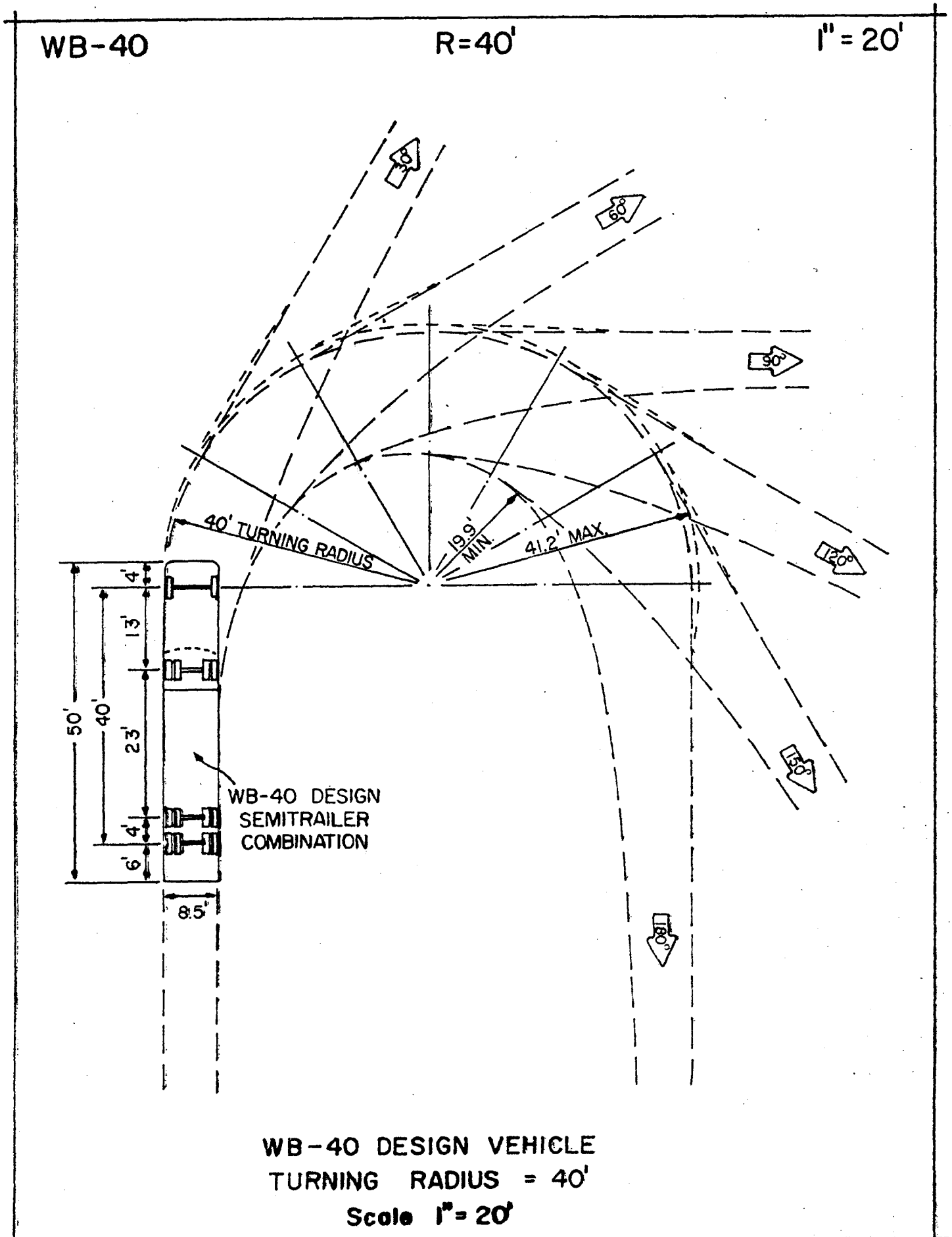


Figure 3.7.4 WB-40 Design Vehicle Template

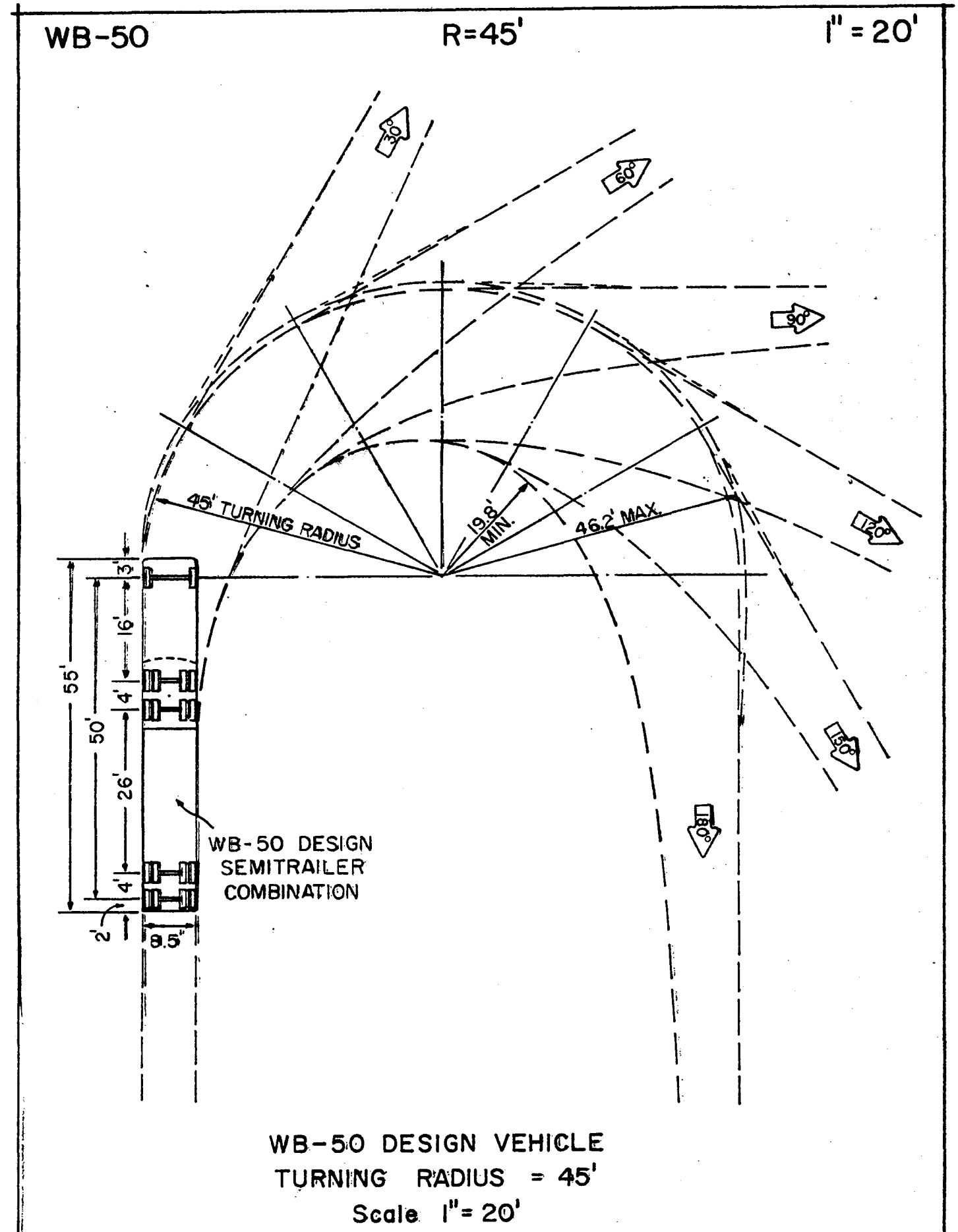


Figure 3.7.5 WB-50 Design Vehicle Template

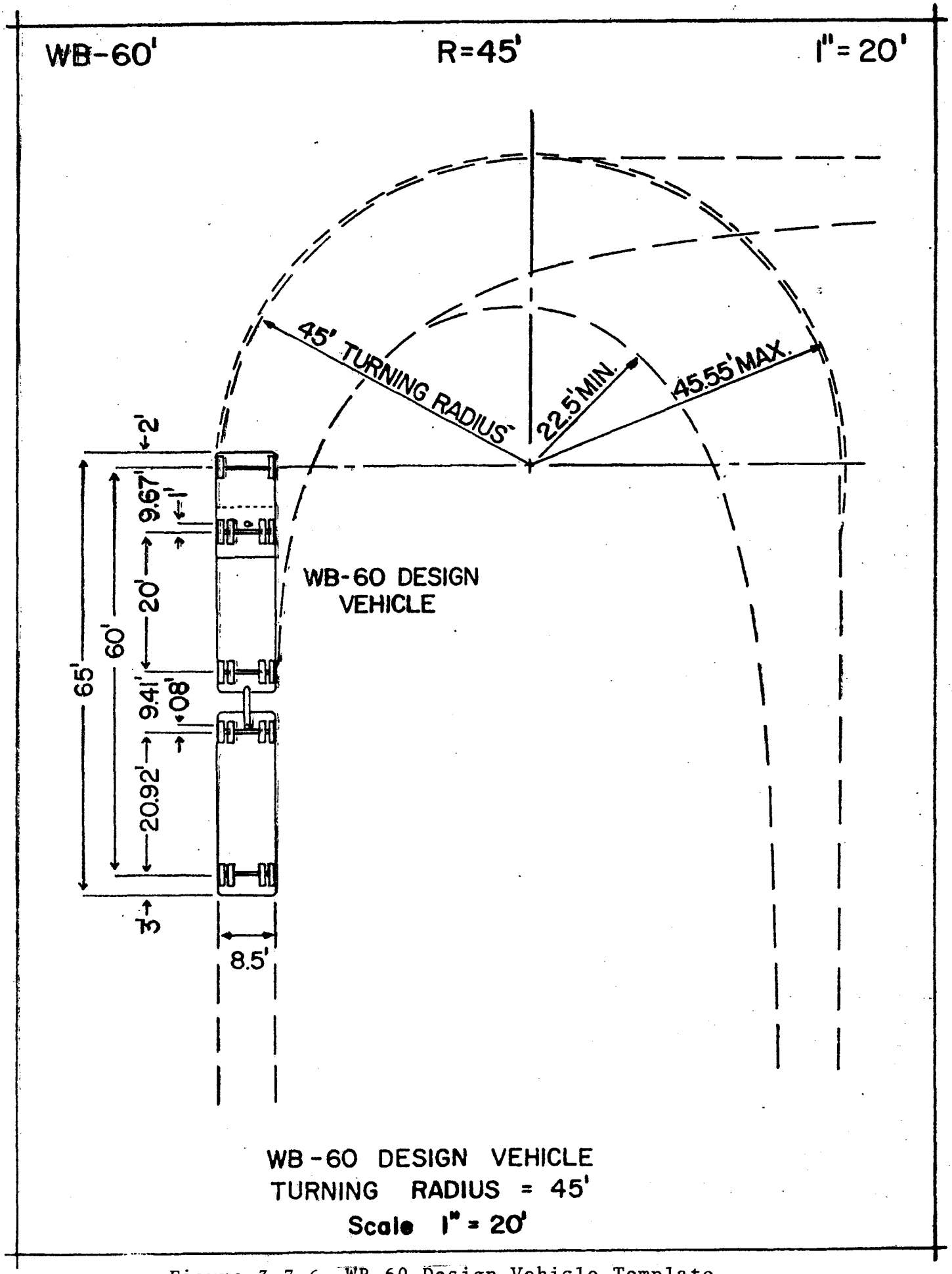


Figure 3.7.6 WB-60 Design Vehicle Template

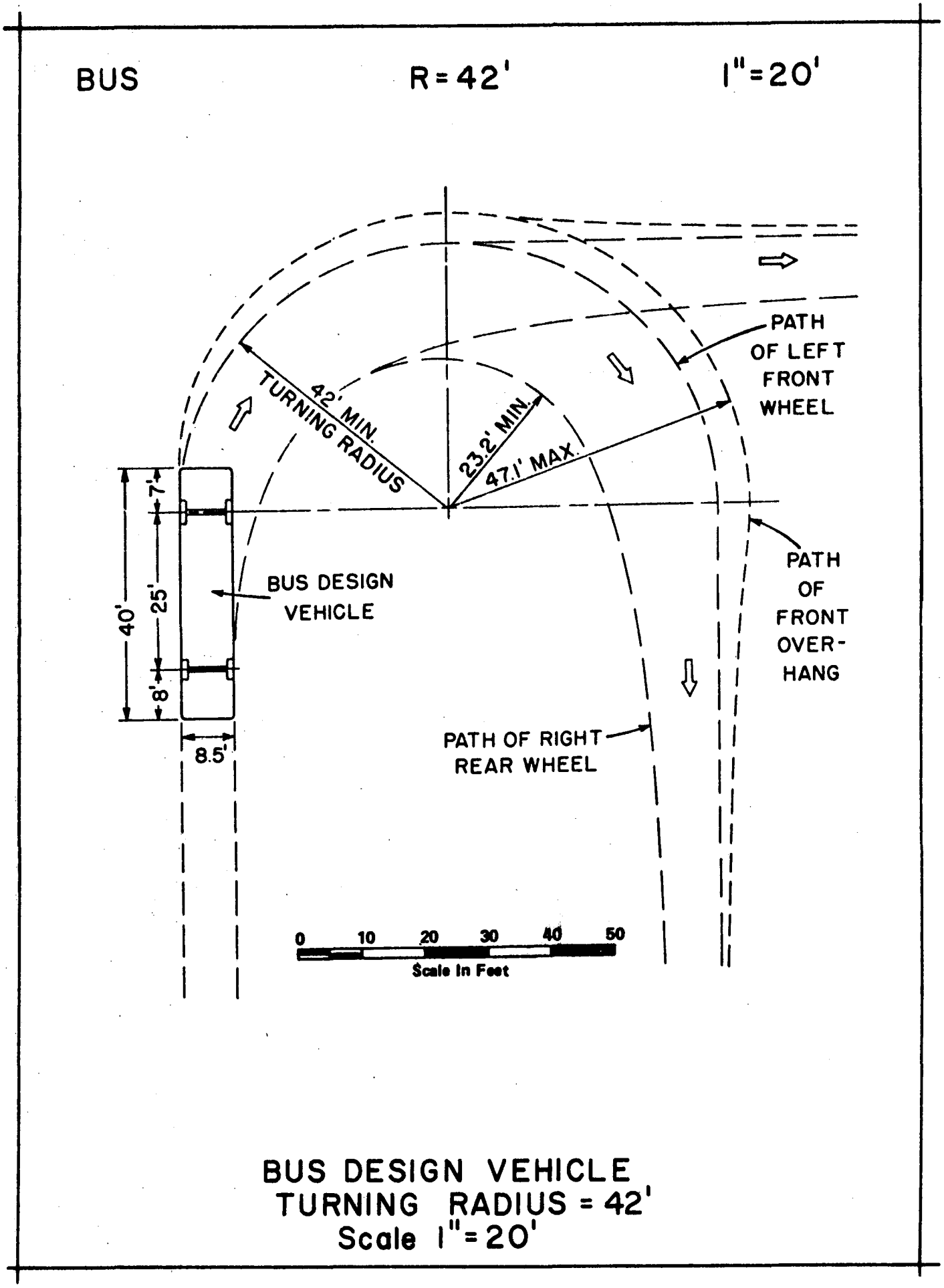


Figure 3.7.7 Bus Design Vehicle Template

TABLE 3.7.3 LANE WIDTH STANDARDS BY TYPE

Jurisdiction	Curb[1] (ft.)	Inside (ft.)	Turn[2] (ft.)
Atlanta, Ga.	11	10	9
Baltimore, Md.	12	12	10
Chicago, Il.	12	12	11
Cincinnati, Oh.	11	11	10
Dade County, Fl.	12½	11	10
Denver, Co.	12	11	10
Detroit, Mi.	11(17)[3]	11	10
Eugene, Or.	12	12	10[4]
Ft. Worth, Tx.	11(18)	12	12
Glendale, Ca.	11	11	10
Kansas City, Mo.	12	12	10[4]
Lansing, Mi.	12	11	11
Los Angeles, Ca.	12	11	10
Louisville, Ky.	11	11	10[4]
Middletown, Oh.	12	12	12
Omaha, Nb.	11½(19½)	10	10
Pasadena, Ca.	12(19)	11	10
Phoenix, Az.	14(20)	12	12
Portland, Or.	11½	10½	11½
Salem, Or.	12	11½	11
Skokie, Il.	12	12	—
Toronto, On.	13	12	10
Tulsa, Ok.	11	11	10
Washington, D.C.	11	11	11

	Range	Median	Mode	Average
curb lane:	10-14'	12'	12'	11.8'
inside lane:	9-12'	11'	11'	11.3'
turn lane:	9-12'	10'	10'	10.4'

Notes: the preferred or most used lane widths are given
[1] curb lane width includes gutter (when existing)
[2] left turn only lanes
[3] curb lane widths in parentheses include parking
[4] turn lanes seldom used

1 ft = 0.305 m

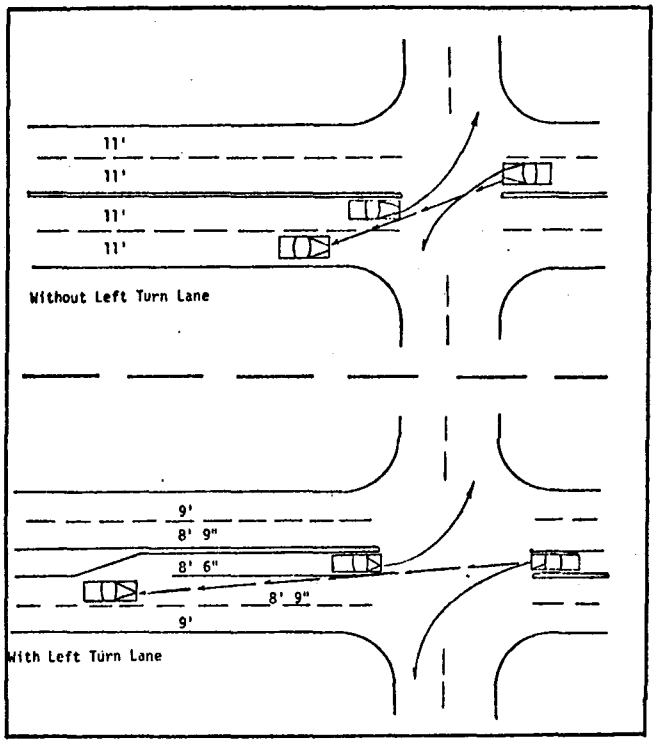


Figure 3.7.8 Left-Turn Lane Addition (4)

3.7.34 Deceleration Distances

From a safety standpoint, the urban designer very frequently does not provide for adequate deceleration into turning lanes and thus creates a delay and needless speed differential hazard in the through lanes. It is desirable that deceleration distances be provided in accordance with the values set forth in the Red Book. They are as follows:

Average Running Speed		Deceleration Length	
20 mph	32 km/h	160 ft	49 m
30 mph	48 km/h	250 ft	76 m
40 mph	64 km/h	370 ft	113 m
50 mph	80 km/h	500 ft	152 m

Equally important are the acceleration distances. The Red Book does not treat acceleration distances other than in association with ramps and interchanges. Greatly improved operation may be achieved by providing straight taper acceleration areas using basically the same dimensions that are set forth above for deceleration distance.

3.7.35 Storage Requirements for Turning Lanes

Too frequently, city street design standards call for a fixed length storage in the left and right turn lanes regardless of the number of vehicles to use that lane. The length of storage should be computed on the basis of the number of vehicles to be stored. A good approach for storage requirements is a length to satisfy 1 1/2 to 2 times the length required for the average number of vehicles to arrive during a signal cycle. For unsignalized intersections, a storage capacity of approximately two minutes of operation should be provided.

3.7.4 AUXILIARY LANES

Auxiliary lanes at intersections serve a wide range of purposes including the storage for turning vehicles both left and right. Other purposes include space for deceleration and acceleration, for bus stops and for access to abutting property.

3.7.41 Left Turn Lanes

The left turn lane is perhaps the most important of all auxiliary lanes. It has advantages that are directly related to safety. For example, the left turn lane permits the traffic with major operational differences to separate and thus avoid drastic differentials in speed and delay characteristics. By increasing the operational efficiency of the intersection, the capacity and safety are also increased.

One major improvement as a result of separate left turn lanes is the increased visibility afforded the turning vehicle. Although it is frequently overlooked, it is a major factor in improving the safety of intersection operations. The improved visibility is illustrated in Figure 3.7.8. In essence, the driver waiting to make a left turn can see

a greater distance down the opposite intersection approach and judge better the available gaps in the traffic stream. Of course, this is applicable only at intersections where separate left turn signal indications are not provided.

At extremely high volume intersections, where there are two or three through lanes on each approach, the single left turn lane is generally considered detrimental to the overall intersection capacity. This is true because of the movement of left turning vehicles on a single lane as opposed to the movement of through vehicles on 2 or 3 lanes. This situation can be improved greatly by providing dual left turn lanes where possible. Dual left turn lanes may not be applicable where there is insufficient width in the throat of the cross street, and where the median on the arterial is extremely wide. Dual left turn lanes cannot be expected to reduce left turn signal timing requirements by 50%, but certainly a reduction on the order of 30 to 35% is reasonable.

Designers have attempted several ways to handle left turn movements without reducing the through movement capacity at the intersection. One such method is to provide an advance left turn as shown in Figure 3.7.9. This would have to be a special circumstance where sufficient right of way is available. On the other hand, however, a minor street may be used in lieu of a diagonal cutoff.

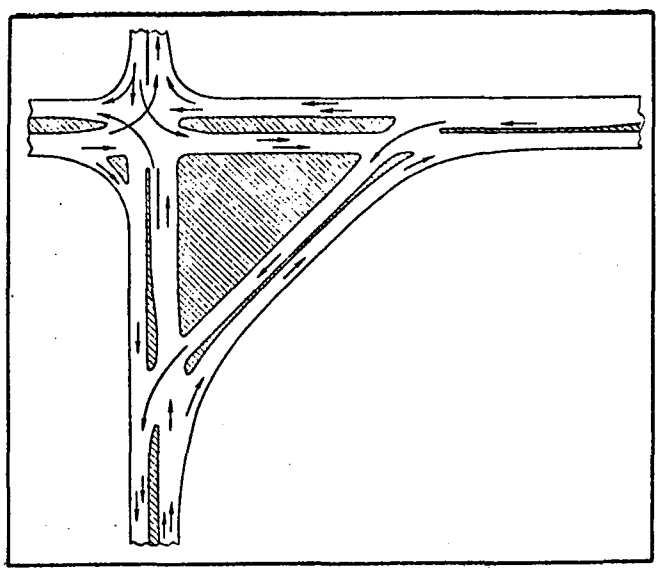


Figure 3.7.9 Special Design For Turning Movements

Other treatments that have been used are the indirect left turns as illustrated in Figure 3.7.10 and 3.7.11.

The integration of exclusive left turn lanes with 2-way left turn lanes has certain safety overtones, not necessarily from the aspect.

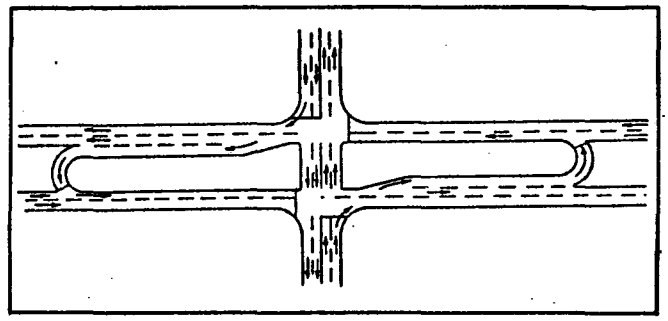


Figure 3.7.10 Indirect Left Turn Through A Crossover

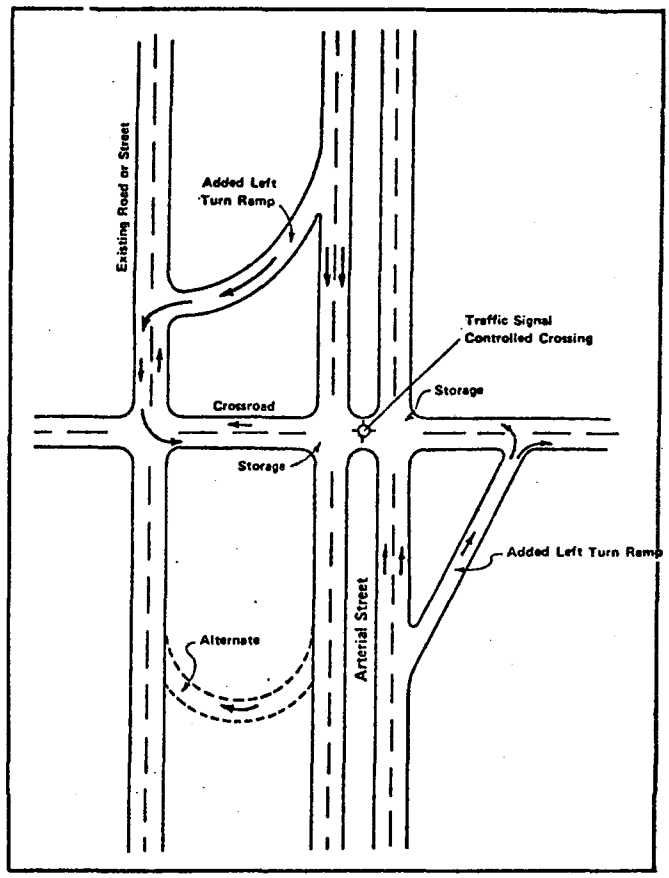


Figure 3.7.11 Special Left-Turn Designs For Traffic Leaving Highway With Narrow Median

of head-on collisions, but collisions with islands and channelization used to assure an exclusive left turn lane. Unfortunately, early documentation of the two-way left turn lane suggested the use of a barrier-curb divisional island along the exclusive left turn lane and complete shadowing of the left turn lane as shown in the upper half of Figure 3.7.12. Typically, accident reports on the early installation of two-way left turn lanes showed a reduction in vehicle-to-vehicle collisions but an increase in fixed object collisions as a result of the raised islands. As a result of extensive

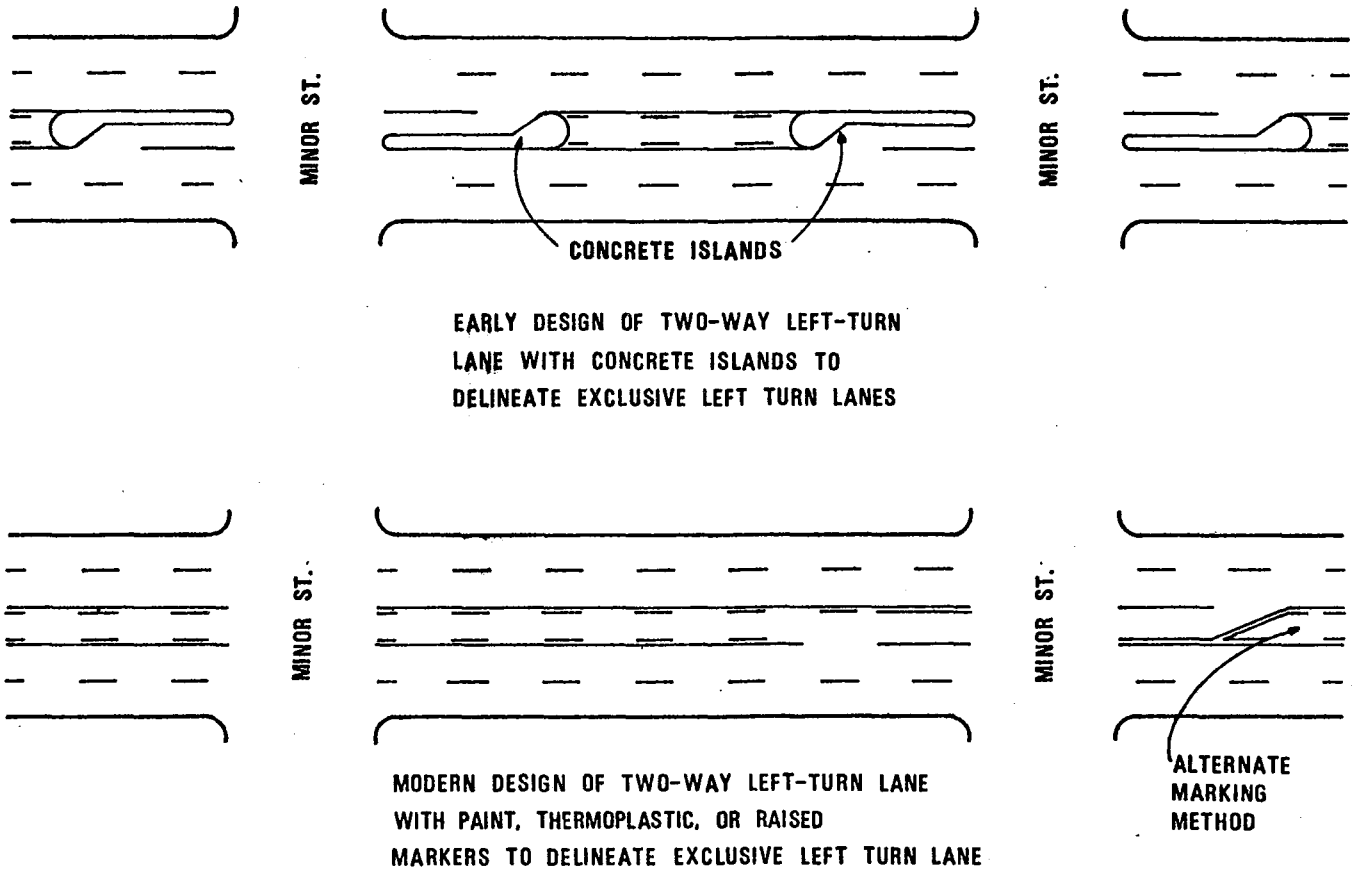


Figure 3.7.12 A Comparison of Early and Modern Two-Way Left-Turn Lane Designs to Illustrate Changes in Channelization Methods

experience with the two-way left turn lane, certain desirable practices have emerged.

- The two-way left turn lane is generally continued on through minor or unsignalized intersections (See Figure 3.7.12).

- For signalized intersections or those controlled by four-way stops, it is generally advisable to restrict entry into the lane for a reasonable distance from the intersection. This is accomplished much more effectively with paint lines, thermoplastic stripes, or raised pavement markers. A double yellow stripe is fairly effective in restricting opposing entry. An overload in the left-turn lane does not block the through lanes (See Figure 3.7.12).

- The shadowing effect does not appear necessary. Some use paint to delineate between the exclusive and the two-way left turn lane while others simply leave the area open. The only thing that they change is the double yellow stripe alongside the exclusive lane. Both systems seem to work quite well.

3.7.42 Right Turn Lanes

Right turn lanes are a great asset to the operational efficiency of urban arterial intersections. Right turns from the through lanes do not create the same safety hazard as do left turns from the through lanes. However, the right turn lane is significantly beneficial for efficient operation. It permits widening of the intersection to accommodate a greater capacity of traffic; but most important it separates out the right turn maneuvers so that they may be handled as a free right turn, right turn on red, or on multiple signal phases. The design of the right turn lane is very similar to the design of the left turn lane. There should be adequate storage and a smooth taper into the lane.

3.7.43 Bus Loading Facilities

With the increased emphasis on the use of public transit, specific attention should be given to the accommodation of buses and passengers at the intersection. Ideally, the bus stop should be removed from the intersection. This can be accomplished through mid-block bus stops as illustrated in Figure 3.7.13.

It is not always possible or desirable to remove the bus stop completely from the intersection. Then the question becomes, is a near side or far side bus stop better? In most instances, it appears that a far side stop is more appropriate particularly where right turn lanes are provided. The far side stop provides a minimum of interference with turning traffic and with through traffic where the bus stop is an auxiliary lane. When the bus is not at the stop, then the auxiliary lane-bus stop is an area that can be used for acceleration. Near side bus stops are preferable where there is parallel parking and a low percentage of buses. An illustration of near side and far side bus stops is presented in Figure 3.7.14.

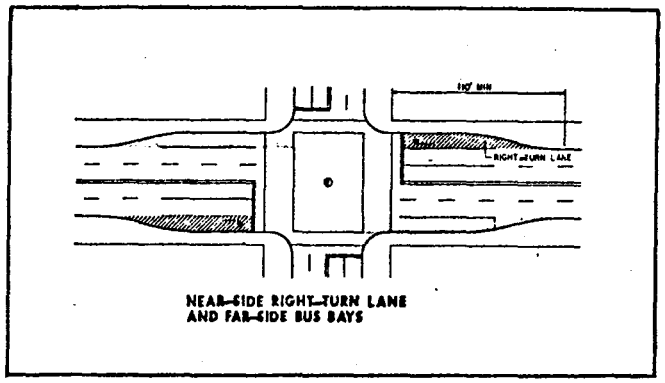


Figure 3.7.14 Near-Side Right-Turn Lane and Far-Side Bus Bays

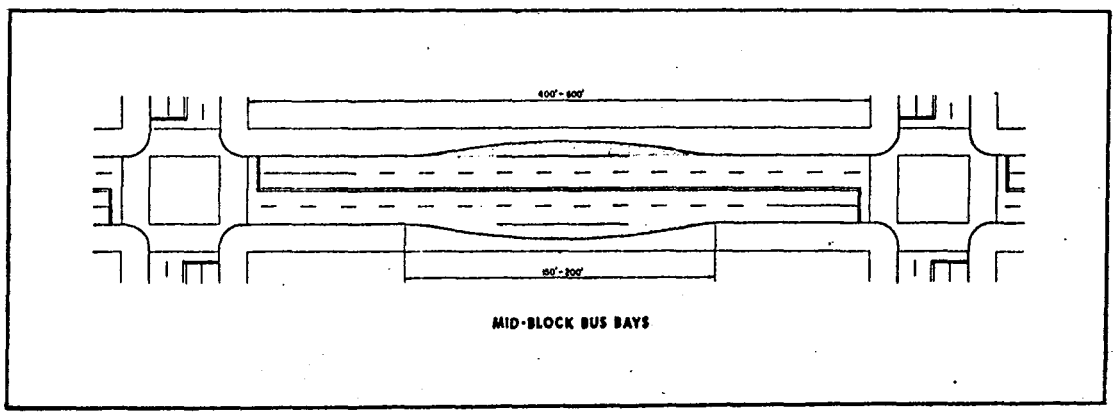


Figure 3.7.13 Mid-Block Bus Bays

3.7.5 MID-BLOCK ACCESS CONTROL

In general, operational problems at mid-block locations relate to lack of or improper control of access. Too many driveways, poor driveway design and the lack of acceleration-deceleration lanes invariably reduce the quality of traffic flow and safety on the facility. On such streets, these conditions generally exist in the vicinity of the intersections and result in reduced capacity.

The most critical problem with mid-block access is the left-turn maneuver into a driveway, when such a maneuver is made from the inside through lane. One obvious correction is the introduction of a barrier type median that prohibits the mid-block left-turn. However, this generally is not a practical solution where access has already been granted. To say the least, it is politically unpopular. On the other hand, barrier medians should be employed in the development of new arterials where land use has not yet developed. When used in this manner, the land use will be forced to develop according to another access form. When barrier medians are used, and direct access to land is permitted, consideration should be given to the circulatory effects of gaining access to the land use. U-turns at the intersection are most impractical. Access by "circling the block" can be provided by the proper street layout. The most desirable, however, is to encourage access from a side street after turning left at an intersection.

Where right-turns into driveways are permitted, one-way angle drives are preferred.

NCHRP Report 91 provides recommended design standards for driveways that will minimize the effects on traffic operation.

Where there is existing development without access control, many cities have found painted medians or two-way left-turn lanes to be very effective in improving the quality of traffic flow, safety and capacity. The success of painted medians is dependent upon the local or state laws pertaining to crossing painted medians, and the degree to which they are enforced. In either case the operational concept is the same - provide a space where turning vehicles may pull out of the through lanes and wait for an opportunity to turn.

Two Way Left Turn Lanes. The two-way left-turn lane is considered superior to the painted median channelization because it removes the confusion that may arise as to whether or not turns are permissible. The Manual on Uniform Traffic Control Devices now includes standard markings and signs for two-way left turn lanes, and they are used extensively throughout the country.

The safety benefits of the two-way left-turn lanes are documented, and the improvements in quality of flow are obvious. Some have advanced the idea that they increase the capacity of intersections by reducing the number of left-turns to be accommodated at the intersection. Obviously, if the turns are permitted at mid-block, those that would have otherwise been routed through the intersection to gain access have been eliminated.

An alternate to the two-way left turn lane design is the U-turn at a median opening. This permits the driver to change direction by making a U-turn and enter the abutting property through a right-hand driveway entrance. This design is impractical for most urban areas because of the extensive right-of-way required. Figure 3.7.15 shows that very wide medians are required even to accommodate a passenger car, and these widths increase more than twofold for the larger design vehicles.

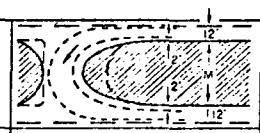
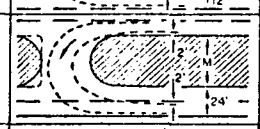
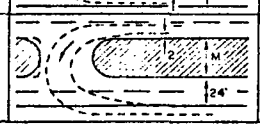
Type of Maneuver		M – Min. width of median – feet for design vehicle				
		P	WB-40	SU	BUS	WB-50
		Length of design vehicle				
		19'	50'	30'	40'	55'
Inner Lane to Inner Lane		32	60	64	68	70
Inner Lane to Outer Lane		20	48	52	56	58
Inner Lane to Shoulder		10	38	42	46	48

Figure 3.7.15 Minimum Designs for U-Turns

Frontage Road Connections. Frontage roads on principal arterial streets are not very common, but exist in numbers that perhaps deserve mention because of the serious problems developed at intersections. The new Red Book makes a very significant point in the design of intersections with frontage roads. A direct quotation makes the point very clearly!

"For satisfactory operation with moderate-to-heavy traffic volumes on the frontage roads, the outer separation should preferably be 150 feet (46 m) or more in width at the intersection. The 150 foot (46 m) dimension is derived on the basis of the following considerations:

(1) It is about the minimum acceptable length needed for placing signs and other

traffic control devices to give proper direction to traffic on the cross street.

(2) It usually affords acceptable storage space on the cross street in advance of the main intersection to avoid blocking the frontage road.

(3) It enables turning movements to be made from the main lanes onto the frontage roads without seriously disrupting the orderly movement of traffic.

(4) It facilitates U-turns between the main lanes and the two-way frontage road. (Such a maneuver is geometrically possible with a somewhat narrower separation but is extremely difficult with commercial vehicles.)

(5) It alleviates the problem of wrong-way entry onto the through lanes of the arterial."

3.7.6 CHANNELIZATION

Channelization in general is a rather basic subject that is treated in a number of references including both the Blue Book, the Red Book and the Manual on Uniform Traffic Control Devices. The fundamental values of channelization are summarized in 10 points presented and quoted directly from the Red Book.

"There are many advantages that can be obtained from a properly channelized intersection. These include the following:

(1) The paths of vehicles can be channelized so that not more than two paths cross at any one point.

(2) The angle and location at which vehicles merge, diverge, or cross can be controlled.

(3) The amount of paved area can be reduced, thereby decreasing vehicle wander and narrowing the area of conflict between vehicles.

(4) Clearer indications can be given of the proper manner in which movements are to be made.

(5) The predominant movements can be given preference.

(6) Areas can be reserved for pedestrian refuge.

(7) Separate storage lanes or areas can be provided that allow turning vehicles to wait clear of through traffic lanes.

(8) Space can be provided for traffic control devices so that they can be more readily perceived.

(9) Prohibited turns can be discouraged, if not prevented.

(10) The speeds of vehicles can be controlled to some extent."

It has long been recognized that the driver associates the width of the roadway with the intended operating speed. The wider the roadway and traffic lanes the higher the expected speed of operation. On high speed facilities (Design speeds over 45 mph) the use of any type of curb is discouraged. Where they must be used, it is recommended that the curbs be located outside the shoulder edge. On high speed roadways, all channelization must be accomplished with flush pavement markings.

On urban roadways the primary factor in selecting raised curb or flush channelization is the nature of the situation approaching the channelizing element. Where the driver must make an overt maneuver to be in a position to impact the curb, raised curb channelization is recommended. A lane drop or redesignation of the function of a down stream lane (i.e., the trap lane situation) a flush channelization system is appropriate. Rolled curbs or mountable curbs may be used in circumstances where the downstream situation is much more hazardous than impacting the curb or what is behind the curb (i.e., conversion from one-way to two-way operation). General use of such devices is however discouraged.

The major point to be made here relative to channelization is the need for detailed design to fit vehicle path. It is imperative that the designer utilize, as a minimum design control, the templates illustrated in Figure 3.7.3 to 3.7.8. These are minimum design paths for the indicated vehicles. It would be even more desirable to utilize the operating characteristics of the design vehicles for 10 to 15 mile per hour operation.

Problems with channelization generally result when operational limitations are overlooked. Convenience in design and layout generally results in straight lines and circular treatments such as illustrated in Figure 3.7.16 where a semi-circle is used to treat the end of an island. Considering the needs for traffic operation, it is desirable to give the island a bullet-nose shape as illustrated in Figure 3.7.16. The traces of vehicular paths illustrate the operational problems caused by the semi-circular nose.

3.7.7 SPECIAL RURAL INTERSECTION DESIGN FEATURES

Rural intersections pose significant safety problems in perhaps a slightly different way as compared to urban arterial intersect-

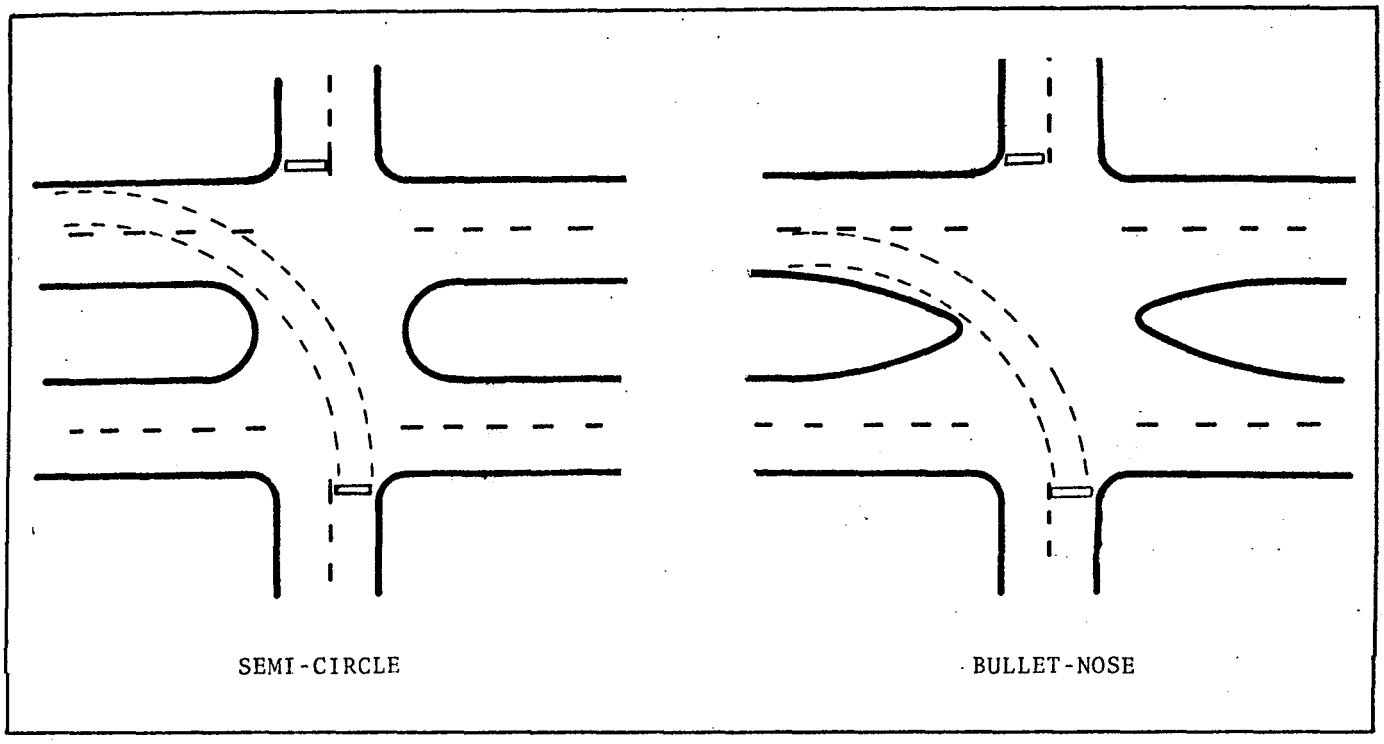


Figure 3.7.16 Vehicular Paths for Alternate Median Designs

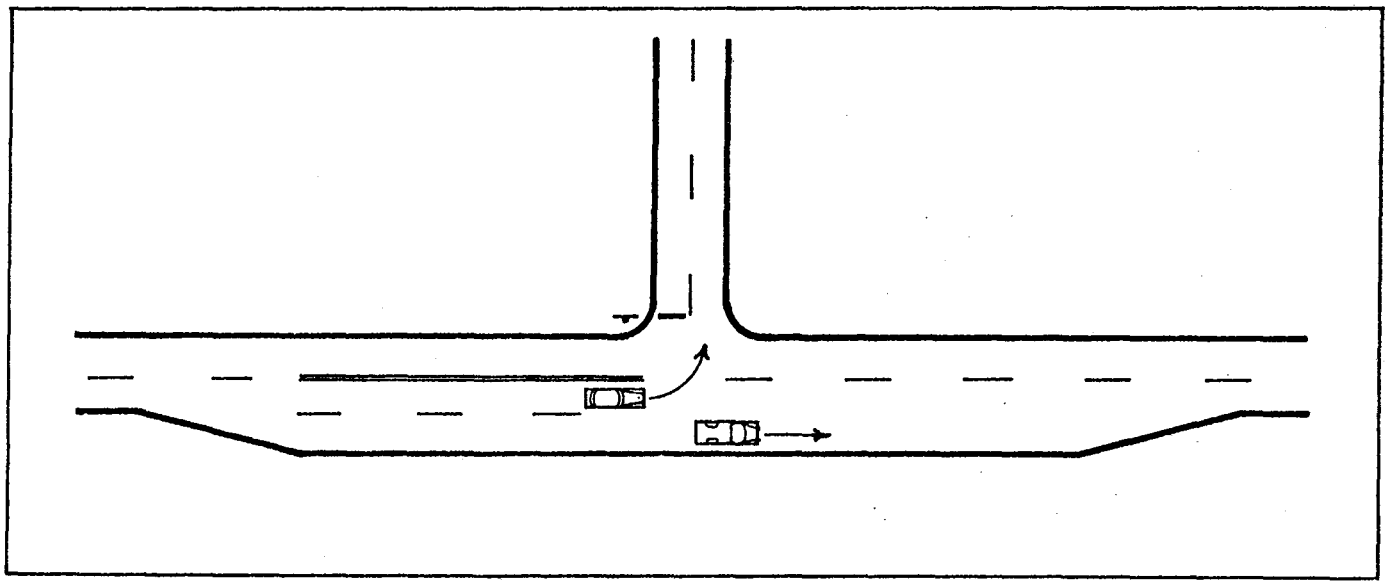


Figure 3.7.17 Left-Turn Bypass Lane

ions. Major safety problems in the rural area are the result of speed differentials, generally due to vehicles slowing or stopped to make a turning movement at the intersection. For minor rural intersections the effects of speed differential may be reduced greatly by flaring the intersection and permitting through traffic to by-pass to the to the right of a vehicle waiting to make a left turn. This is, in essence, the provision of a left turn lane, but when a left-turning vehicle is not present, the left turn lane becomes the through lane. This is illustrated in Figure 3.7.17.

Major rural intersections frequently employ turning roadways as a means of maintaining high operating efficiency for the right turning vehicles. The geometric design of turning roadways has generally been based on a balance between available right-of-way and the desired level of operation (design speed) of the turning roadway. Recent experiences have shown that the distance to the beginning of the turning roadway from the intersection is a critical dimension. There is a range of dimensions that should be avoided because of the response characteristics of drivers crossing at the intersection.

Accidents appear to occur when a driver waiting to cross the roadway notes a lateral movement of the vehicle approaching the intersection. He assumes that the vehicle is entering the turning roadway when, in fact, the vehicle may simply be responding to a widened pavement area. The driver, waiting to cross, checks traffic from the opposing direction and, finding none, pulls out into the intersection only to find that he is struck by the vehicle that he had misjudged previously. To avoid this conflict, the designer has two alternatives. First, he can design the turning roadways very conservatively, pulling the connection points in very close to the intersection. A second and more preferable alternative when right-of-way is available is to make the physical separation point between turning roadway and through lanes at least 800 feet (245 m) from the intersection. For normal highway speeds, this suggests that we should avoid any design which would result in the physical separation being between 200 feet (60 m) and 800 feet (245 m) from the intersection.

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3. A Survey of Urban Arterial Design Standards, American Public Works Association, 1313 East 60th Street, Chicago, Illinois 60637, 1969.

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TOPIC 3 SESSION 8

SAFETY AND OPERATIONAL REQUIREMENTS FOR INTERCHANGES

Objectives:

The participant should be able to:

1. Identify the features of interchanges that produce safety and operational problems,

2. Apply a generalized operational analysis to existing and proposed interchanges, and

3. Evaluate the adequacy of geometric elements of interchanges based on the desirable safety and operational characteristics of the elements.

3.8.1 INTRODUCTION

3.8.11 Definition and Purpose

Fundamentally, an interchange as it is applied in modern highway technology simply provides an opportunity for traffic to transfer from one roadway to another. Implicitly, an interchange pertains to those facilities which provide a vertical separation of two main traffic flows from which the transfers take place. In other words, a modern interchange is a grade separation with connecting ramps.

Interchanges are justified on the basis of policy (the Interstate system) and operational criteria: to provide greater capacity; to provide for the maintaining of higher operating speeds; and to reduce the probability of vehicular conflict and, as a result, improve traffic safety. In summary, the purpose of the interchange is to separate conflicting flows and to facilitate interchanging movements with a minimal amount of interference. The purpose here is not to delve deeply into the fundamentals of interchange warrants and policy but to review very briefly the characteristics of interchanges which appear to contribute positively and adversely to safety. The basic requirements of interchange design have been spelled out to a reasonable degree at this point, and we should add perhaps that the traffic movements in an interchange should be accomplished as simply as possible and with a minimum of undue surprises to the driver. When this is accomplished, a good design generally has been provided.

3.8.12 Interchange Types

There are many interchange types that have been employed throughout the country - many more types than are actually justified. One of the major problems with interchanges is the strong personal influence of the designer. Although innovation of the designer is commendable, uniformity in design is a very strong virtue. Uniformity permits the driver to anticipate requirements based on what he sees at the moment.

We will not attempt to identify and discuss all types of interchanges. For such a detailed treatment one should consult the Red Book (1) and the Blue Book (2). The major point is that interchange type should depend upon the type of facilities between which interchange is to take place. For example, the diamond is the most efficient of the interchanges used for the connection between freeways and surface streets. It requires a minimum travel distance and permits a uniform change from freeway operating conditions to surface street operating conditions.

The clover-leaf is reasonably efficient for low and intermediate volume interchanging movements between two controlled-access facilities. Inherent problems of the clover-leaf relate to extreme operational changes imposed on the interchanging drivers. Thus, the key issues in using such an interchange type are sufficient land area to design the loops properly, sufficient weaving space for ingress and egress, and the provision of collector-distributor roads to handle the higher volume weaving conditions.

Interchanging traffic between two high-type, controlled-access facilities, i.e., two Interstate highways, generally requires a directional interchange - one in which all possible movements are provided by direct turning roadways rather than loops and intersections. Directional interchanges take many forms, dependent upon the topography, the available land area and to a great degree the innovative thinking of the designer. Again, uniformity permits a driver to think ahead and to plan ahead. Lack of uniformity maintains a certain level of driver confusion. Confused drivers make more mistakes, have more accidents, and require more signing and traffic control measures.

3.8.13 Typical Interchange Problems

Thirty years' experience with various types of interchanges have revealed certain operational problems (3). We will identify some of these problems and discuss them briefly. Later, corrective alternatives will be discussed.

Poorly-defined Exit Points. Particularly in early designs, approaching drivers were unable to see the pavement surface in the exit area due to vertical alignment. Parallel deceleration lanes were frequently designed, and there is no assurance that a driver will use a parallel deceleration lane. If the driver does not see the ramp terminal early, and if he does not utilize existing deceleration space, then he may exceed the design conditions of the ramp after he leaves the main lanes.

Drastic Speed Changes. Critical exit ramp geometry often imposes more severe change than is expected or considered acceptable by the driver.

Inadequate Visibility. Many times, we do not provide sufficient visibility distance so that the driver can interpret properly the roadway geometrics and plan his maneuvers accordingly.

Confusing Geometrics. In many instances, we develop highly channelized intersections and ramp terminals that look good in plan view on paper but result in driver confusion when viewed from the vehicle.

Poorly-defined Entrance Points. There are two major problems: First, the entrance to the ramp from the cross street and, second, the entrance or merge point with the freeway may not be well defined. The failure to define either of these points very well may result in reduced efficiency due to increased driver confusion.

Exceeding Driver Work Load Capacity. Some entrance ramp designs require the driver to look over his shoulder to find a gap in the traffic stream, to look ahead to make sure the way is clear, and to steer a compound curve, superelevated and tapered into a horizontal curve on the freeway. This simply overloads the driver, and he is unable to accomplish all these tasks at one time.

Unnatural Merge Requirements. Left-hand entrances require unnatural maneuvers. The driver has less visibility and less experience and, therefore, does not merge right as well as he merges left. Further, it is unreal to expect a driver to merge with a high density traffic stream at 45 to 50 miles per hour through a direct entry-type ramp terminal. Thus, such a ramp is totally incompatible with the operational requirements.

Deceiving or Inadequate Sight Distance. Accomplishing a merge on a freeway facility is a very complex driving maneuver. To merge safely and efficiently, the driver must be able to see in proper perspective the other traffic with which he is merging.

3.8.14 Types of Accidents

Certain types of accidents are associated directly with and/or influenced to a great degree by the type and design of the interchange. These will be reviewed briefly.

Rear-end Collision. The rear-end collision is by far the most frequent accident type associated with interchanges. It occurs on the entrance ramp and is the result of the driver attempting to simultaneously look over his shoulder to judge the traffic, and to maintain a sufficient gap with the vehicle in front.

Fixed Object Collisions. Fixed object collisions relate to interchanges primarily because of the fixed objects that are inherent due to the configuration of the overpassing structure. The proximity of piers to the roadway and to the point where decisions must be made simply results in a higher frequency of occurrence in the interchange area.

Ran-off-the Road Collisions. Interchanges are obviously points of decision, and unfortunately, they are points of critical geometry. These two in combination result in a higher frequency of out-of-control type collisions.

Intersection Conflicts. Interchanges with surface-type facilities result in intersections at grade. These intersections are frequently located very near the structure and, consequently, are subject to limited sight distance due to bridge rails, traffic barriers and vertical alignment of the crossing roadway.

Wrong-Way Movements. In almost all cases, wrong-way movements begin at the interchange. Some simply cannot be avoided; others, however, generally result from driver confusion which is usually a direct result of the design.

3.8.2 INTERCHANGE ACCIDENT EXPERIENCE

There are several methods of identifying critical elements and operational problems in interchanges, but perhaps the most direct method of examining safety problems is to compare accident statistics relative to the various interchange elements. Several recent research projects have provided a substantial amount of data upon which we may make a study of interchange elements. Perhaps the most comprehensive of these studies is the Interstate System Accident Research Study II by the Federal Highway Administration (4). This study is used to identify the critical elements of the interchange and to give them a certain priority relative to accident rates. Because accident characteristics are so different under rural and urban conditions, we will look at the general distribution of accidents for each of the two conditions.

3.8.21 Rural Interchange Accident Rates

Figure 3.8.1 presents data on accident rates by interchange element. It should be noted that for rural conditions the highest rate is on the exit ramp (346 accidents per 100 million vehicle miles of travel). It is difficult to identify the exact cause in such a generalized study, but most of these accidents are due to drivers being exposed to critical ramp geometry in their transition from freeway operating speeds.

The second most critical element in rural interchanges is the entrance ramp. It should be noted in Figure 3.8.1 that the rate on entrance ramps is roughly half that of exit ramps. Again, these accidents are attributable mainly to critical geometry.

The third most critical element in rural interchanges is the deceleration area. All other elements have roughly the same rate.

3.8.22 Urban Interchange Accident Rates

In Figure 3.8.1 it is quickly noted that the highest accident rate on any element of the interchange is on the entrance ramp (718 accidents per 100 million vehicle miles). This high rate is obviously due to the situation where the driver must look over his shoulder to find a gap in the traffic stream and, when doing so, increases the likelihood of being involved in a rear-end collision with the car ahead.

The second highest rate is on the exit ramp (370 accidents per 100 million vehicle miles). This could be due, in part, to rear-end collisions with traffic that is stopped for a control device on the surface street system.

Also in Figure 3.8.1, it can be noted that the deceleration lane and the acceleration lane elements have comparable rates. In general, however, the rates for urban interchanges are roughly two times the rate for rural interchanges.

Table 3.8.1 presents the accident frequency and accident rates for various distances on each side of the interchange (4). On the exit side, the higher accident rates are observed at distances of less than one mile from the interchange area (the interchange area being defined as the limits of the ramp terminals). Further, it can be noted

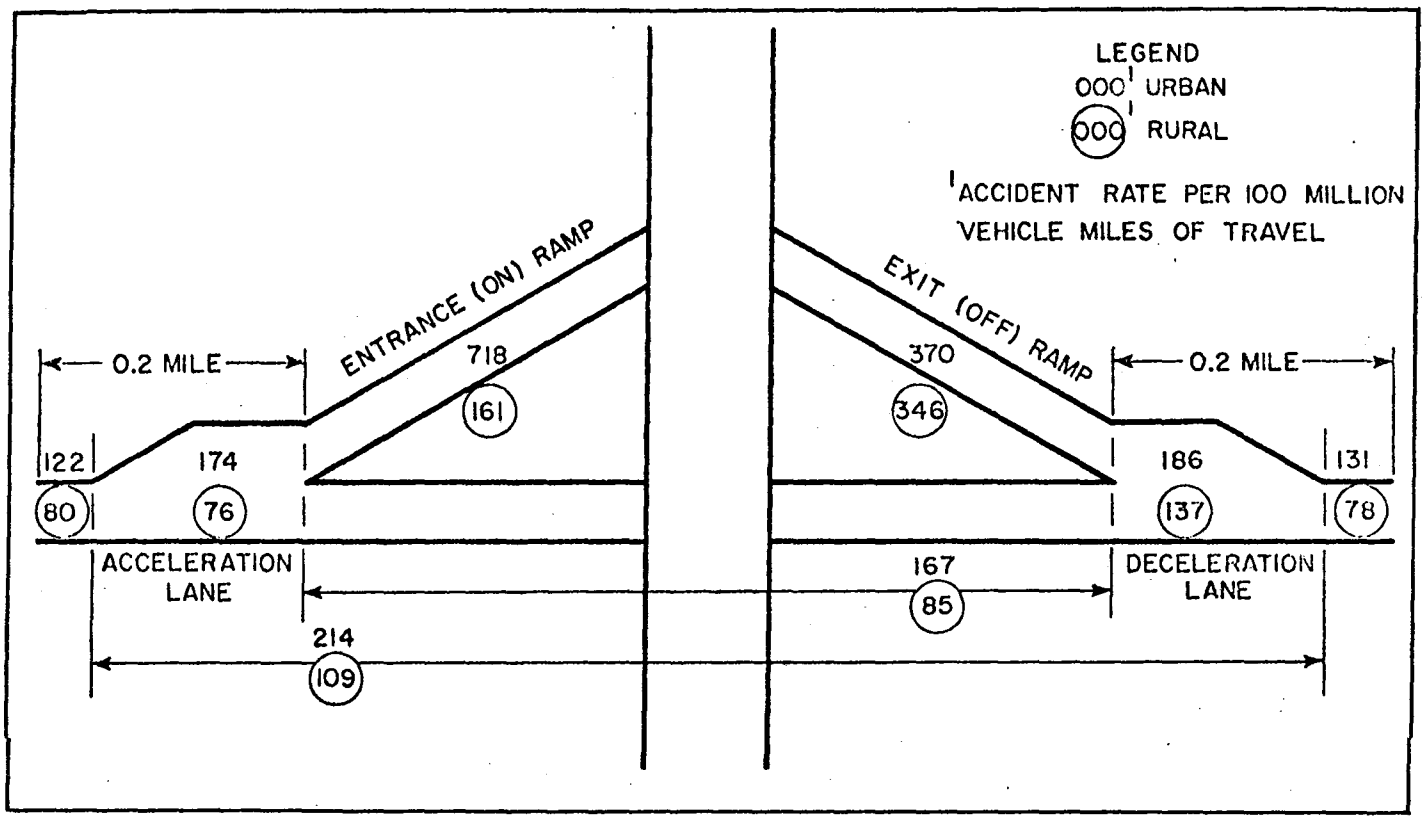

Figure 3.8.1 Accident Rate by Type of Interchange Unit (4).

TABLE 3.8.1 ACCIDENT RATE BY PROXIMITY TO INTERCHANGE

EXIT SIDE			ENTRANCE SIDE		
Distance to exit-ramp nose ahead	Accidents	Accident rate [1]	Distance to entrance-ramp nose behind	Accidents	Accident rate [1]
URBAN			**URBAN**		
	Number	Number		Number	Number
Less than .2 miles	722	131	Less than .2 miles	426	122
.2- .4 miles	1,209	127	.2- .4 miles	1,156	125
.5- .9 miles	786	110	.5- .9 miles	655	105
1.0-1.9 miles	280	75	1.0-1.9 miles	278	84
2.0-3.9 miles	166	63	2.0-3.9 miles	151	59
4.0-7.9 miles	19 [2]	69	4.0-7.9 miles	200	75
More than 8 miles [3]	—	—	More than 8 miles [3]	—	—
RURAL			**RURAL**		
Less than .2 miles	160	76	Less than .2 miles	117	80
.2- .4 miles	459	75	.2- .4 miles	482	82
.5- .9 miles	559	69	.5- .9 miles	560	72
1.0-1.9 miles	479	69	1.0-1.9 miles	435	64
2.0-3.9 miles	222	68	2.0-3.9 miles	169	51
4.0-7.9 miles	46	62	4.0-7.9 miles	52	40
More than 8 miles [3]	—	—	More than 8 miles [3]	—	—

[1] Number of accidents per 100 million vehicle miles.
[2] Small sample size.
[3] No data available.

that these rates are roughly 1.5 times the rates on other sections of the freeway. For rural conditions, however, there is virtually no increase in rates outside of the immediate interchange area.

It should be noted that the rate for rural entrance ramps is roughly twice the rate for normal highway conditions. This is perhaps due to critical geometric characteristics of entrance ramps. The lower rate is indictative of lower speeds and relatively fewer surprises to the driver. Entrance ramps on urban freeways are entirely a different matter. It is noted that the highest rate of all is observed on urban entrance ramps. This high rate is due largely to the rear-end collisions resulting from exceeding the driver work load. It is the situation in which the driver must look over his shoulder to select a gap in the traffic stream and move forward to intercept that gap. Frequently, however, there is a vehicle in his path that is detected too late to avoid a collision. Fortunately, these conditions are minor, but their effect on operation of the interchange is most devastating. They tend to block the facility during the times of greatest activity.

Rear-end type collisions have been greatly reduced through the implementation of ramp signal control (12). Ramp control reduces the congestion in the merge area by permitting only one vehicle at a time in the merge process. From a design standpoint these rear-end collisions may be reduced greatly by designing the ramp to parallel the main lanes such that the driver can select or maintain visual contact with his gap through the rearview mirror while keeping his eyes directed ahead of the vehicle. These design features will be discussed in greater detail later.

3.8.23 Accident Rates By Type of Ramp

A rather significant research study (5) has shown that some ramps are inherently safer than others, and these results are reflected in Table 3.8.2. Looking first at entrance-ramps, it is evident that the accident rates are relative to the confusion factors - the greater the confusion, the greater the accident rate. Evidence to this effect is reflected in the rates for left-hand, scissor, and trumpet ramps.

For entrance ramps presenting only a moderate level of confusion, we find moderate accident rates. Note that "button hook" ramps, loops without collector-distributor roads, and clover-leaf ramps without C-D roads have rates ranging from 64 to 78 accidents per 100 million vehicle miles. The simplest ramps - the diamond, the clover-leaf ramps with C-D roads and direct connections - have the lowest rates. From this we should conclude that simplicity is a prime virtue.

Noting again information in Table 3.8.2, we find essentially the same patterns for the various exit ramp types, but the rates are higher - in fact, they are disproportionately higher for the major confusion types. It may be noted further that button hook and direct connection exit ramps have relatively high rates. Again, this reflects the intensity of the combination of high speed and critical geometry.

TABLE 3.8.2 ACCIDENT RATES BY TYPE OF FREEWAY RAMP (5).

Ramp Type	Accident Rate [1] On	Off	On & Off
1. Diamond ramps	0.40	0.67	0.53
2. Cloverleaf ramps with collector-distributor roads [2]	0.45	0.62	0.61
3. Direct connections	0.50	0.91	0.67
4. Cloverleaf loops with collector-distributor roads [2]	0.38	0.40	0.69
5. Buttonhook ramps	0.64	0.96	0.80
6. Loops without collector-distributor roads	0.78	0.88	0.83
7. Cloverleaf ramps without collector-distributor roads	0.72	0.95	0.84
8. Trumpet ramps	0.84	0.85	0.85
9. Scissors ramps	0.88	1.48	1.28
10. Left side ramps	0.93	2.19	1.91
Average	0.59	0.95	0.79

[1] Accidents per million vehicles.
[2] Only the On & Off rate includes the accidents occurring on the collector-distributor roads.

3.8.3 OPERATIONAL ANALYSIS

In this section we will review briefly the operational requirements of the basic entrance and exit movements through an interchange. We will attempt to identify and present a non-technical approach to defining driver informational needs that must be satisfied in order to achieve the safest operating conditions in the interchange area.

3.8.31 The Exit Maneuver

In reviewing a preliminary design of an interchange, the designer must, through an imaginative process, put himself in the position of the approaching driver and consider all aspects of interpreting the freeway interchange geometry. In doing so, he will find that the first requirement is good effective directional signing, placed well in advance of the exit. Additional signing and markings should identify the specific exit point. The driver cannot depend totally on the signing system. He must have visibility of the deceleration lane, the taper, the ramp gore, and other elements of the ramp in proper order. It is necessary that the driver be able to view this area of departure well in advance so that he can plan his maneuver and reduce his speed to conform to the geometric requirements. If, however, he is unable to see the area of departure and the ramp beyond it, he likely will not adjust speed sufficiently.

Unless otherwise alerted and prompted, most drivers will utilize leisure, in-gear deceleration rather than braking. Braking seems to be a last resort. Thus, it is essential that sufficient deceleration distance be provided on the ramp beyond the ramp gore. One cannot assume that all drivers will utilize a parallel deceleration lane if there is an opportunity for them to enter the ramp without it.

It is important that all geometric features of the ramp and the connection to the surface street be visible as soon as possible. Further, the driver must comprehend and be ready to respond to the requirements of this geometry. There are several connection possibilities at the other end of the ramp:

- A diamond ramp that connects directly to the surface street
- A merge with another traffic stream
- A merge with a collector-distributor road
- A merge with a one-way frontage road
- An intersection with a two-way frontage road

The first two are generally expected by the driver but the latter three generally are not expected, and the information must be conveyed primarily by the visual aspects of the geometry. Roadway lighting can be used effectively to enhance the safety of these latter alternatives, particularly the collector-distributor road connection.

3.8.32 The Entrance Maneuver

Again following the driver through his maneuvers by an imaginative process, the designer would first find a possible breakdown in driver information in locating the connection from the cross street onto the ramp. This is accomplished most effectively by a combination of good directional signing and by good geometry which puts the connection in full view of the driver well in time for him to make whatever maneuver is necessary to gain access to the connection. Complications arise when there are multiple entry points, or driveways that appear to be entry points. Partial cloverleafs and other designs that bring the entrances and exits into a single point of convergence frequently introduce unncessary confusion. In these instances, the designer must use channelization and signing to prevent wrong-way entries. This is one point where the simple diamond interchange has a tremendous advantage.

Another very awkward situation arises when the ramps are not integrated into the frontage road system. In many cases frontage roads are forced outside the access limits and result in two successive intersections with the cross street - one for the frontage road and one for the ramp. The drivers fail to comprehend the jurisdictional problems that have prevailed, and the result is confusion.

Two-lane entrance ramps that are quite common with directional interchanges can be virtual nightmares from the standpoint of operations. In a situation such as illustrated in Figure 3.8.3, where a two-lane ramp joins a three-lane freeway to result in four lanes downstream, merging the left lane of the ramp with the outside lane of the freeway is a disaster. There is no relief valve. Both lanes are enclosed from the outside, and speeds are high. This situation is avoided by most drivers until, under high-volume conditions, they are simply forced to remain in their respective lanes. This situation can be solved as shown in Figure 3.8.4 where the two lanes are brought parallel, and the outside lane is dropped at a point downstream.

Once the driver has gained access to the ramp, there are certain characteristics that must be provided if the driver is to make a safe and efficient entry. At some point on the ramp prior to entry, he should look over his shoulder to get a general view of the traffic stream with which he is to merge. This first look should be fairly easy and should be achievable in a short period of time. To do this he needs to be at essentially the same grade and perhaps even a little above the main lanes so that he can get a good view. From there, he must quickly come into a position parallel with the main lanes so that he can pick up the traffic stream in his rearview mirror before reaching the gore of the ramp. From that point he must be able to operate with the information given in his rearview mirror. Ideally, he will have a long, tapered entry at the same grade as the through lanes, and the length of the taper will be 50 to 1 over a distance of something on the order of one thousand feet (330 m). Further, a paved shoulder should permit the driver to continue at a reduced speed and not be forced to stop at the end of the merge area if he has been unable to accomplish the merge. Above all, this merge area should not be on a horizontal curve. This simply adds an additional element to the driver workload and one that will induce stopping rather than merging.

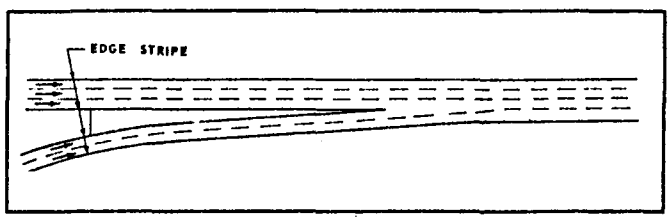

Figure 3.8.3 Two-Lane Entrance Terminal - Inside Merge

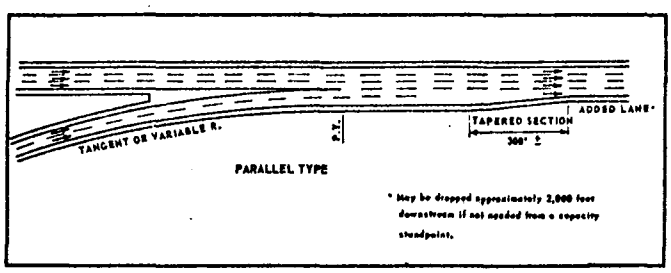

Figure 3.8.4 Two-Lane Entrance Terminal - Outside Merge

3.8.4 OPERATIONAL UNIFORMITY IN DESIGN

Aside from elements that have already been discussed, there are other elements for which uniformity is of the essence. These include interchange patterns, interchange spacing, lane drops, frontage roads, and cross-street characteristics.

3.8.41 Interchange Patterns

Although there is a certain amount of uniqueness to every interchange, it is detrimental to the operational quality for every interchange to be designed totally different from the next. Further, many highway agencies around the country have approached interchange design with the philosophy that every interchange should be different. On the other hand, some states have selected basic interchange types or patterns to be employed for the two basic operational conditions, that is freeway-to-surface street and freeway-to-freeway operations. The uniqueness of individual sites has been used to effect minor modifications that adapt the general freeway pattern to the specific location. This latter design approach has proven far more desirable.

Many states have adopted the diamond interchange as the basic interchange for freeway to surface street type operation. This easily conforms to a fundamental driver expectancy rule that the exit point be located on the advance side of the interchange, and the entrance point be located on the downstream side of the interchange.

All interchanges should provide for all legal maneuvers. To omit a movement because it is of a low volume in the estimated traffic demand is unrealistic. It is a point of extreme confusion for those few who are looking for the connection and, further, it puts the designer in a position of predicting future land use that may influence the demand for the omitted maneuver.

Some agencies build crossovers rather than interchanges. When drivers see a structure ahead, they associate this with an interchange. When they arrive on the scene, there may be some confusion in searching for the nonexistent ramp connections. Thus, any crossover should be associated and integrated with a nearby interchange.

A particularly important need that must be met is that of providing the motorist with sufficient advance information on the type of interchange ahead, and the path he must follow to reach his desired destination. This is especially critical if the interchange pattern is unique and unexpected by the motorist. For example, if a series of simple diamond interchanges is followed by a parclo interchange, the driver's expectancy will not be fulfilled, resulting in critical operations. Diagrammatic signs are possible solutions.

3.8.42 Spacing

The Yellow Book (3) suggests a minimum spacing of interchanges for proper signing to be at least one mile between urban crossroads and twice as much along rural sections. Obviously, spacing of less than one mile creates difficulties in fitting in the ramps and maintaining sufficient weaving space. On the other hand, attempting to space interchanges too far apart in the urban area will result in excessive demand on the ramps that are provided. Also, the fewer connections, the less true service that the freeway provides to the urban area. In other words, for a freeway to be functional, there must be frequent access points; we cannot go overboard on this because putting in too many access points will reduce the operational efficiency. Therefore, the spacing of one mile as a minimum should also be considered very close to optimal. Ramp connections in urban areas should not exceed two miles.

3.8.43 Freeway Ramps

Although we have already discussed to some extent the operational requirements of ramps, they are extremely important, and AASHTO has provided a very comprehensive safety treatment of ramps in the new Yellow Book. Therefore, the recommendations listed below have been taken directly from the Yellow Book (3).

Ramps

"Because of the potential for accidents at ramps and ramp terminals, particular concern with geometric design and traffic operation at these locations is advised.

There usually are more accidents on exit than on entrance ramps, especially at cloverleaf interchanges. Damaged delineators and skid marks testify to the accuracy of this statement. Special attention to gore areas as described later in this Chapter is strongly recommended.

Exit ramp capacity and safety can and should be increased at many locations. This is often readily accomplished by the improvement of facilities at or on the crossroad, which will expedite traffic flow generally, or by the addition of vehicle storage space along the ramps and approaches to prevent a back-up on the ramp or even on the freeway itself. Where heavy exiting traffic is combined with heavy through traffic, rear-end accidents are likely to be prevalent. One method to relieve this condition is to add a long auxiliary lane in advance of the exit ramp. Such lanes usually appear as through lanes to the driver and tend to operate as interchange lane drops. Special pavement marking and signing is necessary to prevent the use of such a lane as an added through lane.

The recommendations listed below are intended to supplement and strengthen the recommendations in existing AASHTO publications.

Exit Ramps

1. Exit ramps should normally have a single lane at the gore, although they may be widened for increased capacity and storage shortly beyond the gore. The ramp terminal at the crossroads should have length, capacity, and turning room to accommodate the stored vehicles. Two-lane exits should be used only when warranted by traffic volumes and when an auxiliary lane with adequate length and taper is provided. Two-lane exits are generally reserved for ramps connecting controlled access highways.

2. When the number of lanes leaving a point of divergence is greater than the number of approach lanes plus one, the number of approach lanes should be increased to provide adequate capacity and to permit proper lane assignment.

3. A major consideration in the locations and design of exits is the provision of adequate visibility for the motorist approaching the exit ramp on the freeway mainline. Sight distances of 1,000 feet or more to the exit gore are desirable. Ramps that drop out of sight are a potential hazard and should be avoided wherever practical.

4. Left-hand exit ramps have poorer operational and safety records than right-hand exits and should not be used on new construction. Existing left exits should be signed diagrammatically.

5. On loop ramps a constant radius curve beyond the exit terminal provides for better operation and is preferred to a compound or broken-back curve. On direct connecting ramps, reverse curves without an adequate intervening tangent for superelevation transition should be avoided.

6. Exit ramps should begin where the freeway mainline is on a tangent wherever possible. Where a curve location is necessary, special treatment is required. A parallel speed change lane should be considered, and the exit gore itself should be located well in advance of or well into the curve. Where an exit on a curve to the right is in a cut section with impaired visibility, consideration should be given to cut widening or to adding an auxiliary lane in advance of the exit for orientation.

7. The exit ramp loops of cloverleaf interchanges provide a potential for run-off-the-road accidents as a result of vehicles exceeding the design speed. For this reason, a relatively flat recovery area on the outside of the curve should be provided.

8. To provide adequate distance for maneuvering and for signing at successive exits within an interchange, the following minimum distances between gore points are suggested: 1,000 feet between exits on a freeway mainline and 800 feet between a mainline exit and an exit on a collector-distributor road or a split in the ramp. Multi-lane ramps may require more than 800 feet between sequential exits.

Entrance Ramps

1. In general, entrance ramps should have a single lane at the entrance nose. Where a two-lane entrance ramp is warranted by traffic volume, an additional lane should be provided on the freeway with the outside ramp lane merged into the added lane with a normal taper. The additional lane may be continuous or it may be terminated with a normal taper an appropriate distance down stream (a minimum of 1,500 feet is suggested) depending upon traffic demands.

2. Variations in the width of entrance ramps can cause congestion and hazard. The provision of a two-lane throat at the beginning of the entrance ramp at the crossroad may be needed to accommodate turning maneuvers. A gradual but obvious reduction of this width, however, to a single lane prior to the entrance nose is desirable.

3. Entrance ramps and merging areas should be lengthened to account for the effects of ascending grades in cases where a substantial volume of slow-moving traffic is anticipated.

4. Entrance ramps from local streets should always join the main traffic stream from the right. Where direct connections are provided between two freeways, left-hand junctions are occasionally a necessity. Whether right or left hand, such junctions require added lanes on the mainline with an appropriate lane drop downstream. Drivers must always have full notice of any merging, lane changing, or other action required of them.

5. On urban freeways where ramp metering is used, on-ramps must provide adequate storage for vehicles which may be delayed because of the metering.

Slip Ramps

1. Slip ramps should normally be used only on freeway sections with one-way frontage roads. Slip ramps from a freeway to two-way frontage roads are generally unsatisfactory because they tend to induce wrong-way entry to the through lanes of the freeway and cause accidents at the ramp-frontage road junction.

2. Slip ramps to a frontage road should be long enough to enable traffic to slow down sufficiently to enter the frontage road safely.

3. An intersection of a slip ramp with a one-way frontage road should be sufficiently in advance of a crossroad intersection to provide adequate weaving length for turning traffic plus storage."

Ramp curvature has been considered as an influencing factor in several freeway safety studies. If interchange on-and off-ramps are classified as either straight or curved (excluding scissors, left-side ramps, and direct connections), straight ramps have been found to have a 12 percent lower overall accident rate (5). In a New Jersey study of on-ramps without acceleration lanes, increasing the ramp radius from 40 to 1000 feet

(12 to 305 m) was found to provide little benefit in terms of highway safety. The important factor is not the radius of the ramp but whether or not traffic slows down or stops before entering the freeway (7). The lack of correlation between accident frequency and the horizontal curvature of ramps was also noted in a study of freeways in Texas (8).

Another factor reported in the literature on ramps has been their location on the left- or right-hand side of the freeway. All left-hand entrance and exit ramps in a New Jersey study had poor accident experience (7). This same finding was also reported in an early study of left-hand ramps in California (9).

Although no single ramp design element contributes greatly to accident experience, on- and off-ramps with high accident frequencies are identified by the following combination of characteristics: freeway under the structure; distance back to the structure of less than 750 feet (220 m); freeway on an upgrade at the junction of the ramp with the freeway; and direct-entry maneuvers. Conversely, low-accident ramps are indicative of the following combination of conditions: long acceleration or auxiliary lane; distance back to the structure greater than 750 feet (220 m); and freeway on a downgrade at the junction with the ramp (8).

Entrance Terminals. The merging process is accomplished at the entrance terminal and involves the driver selection of an adequate gap for entry into the freeway traffic flow from an on-ramp. Although the design of the merging area does not provide for the regulation of gaps in the traffic stream of the freeway, the proper arrangement of geometric elements can assist drivers in making safe and efficient merges. The importance of the entrance terminal in highway safety is evidenced by the fact that approximately 53 percent of on-ramp accidents occur in the merging area (5).

The major variable studied in the entrance terminal area has been the length of acceleration lane available for the merging maneuver. If the acceleration lane is less than 800 feet (245 m) long, the accident rate tends to be above average, especially where high entrance speeds are possible. When the merging areas are short, the most common accident in the entrance terminal is the rear-end collision. These collisions usually occur when drivers are forced to look back over their shoulders to find an acceptable gap in the through traffic for merging. When a driver stops or slows down instead of continuing a merging maneuver, a rear-end accident frequently results because the following driver is watching the freeway traffic and does not anticipate a speed change. The process of entering a freeway can be expedited by changing the merging maneuver to the less complex and more common lane-changing maneuver through the use of an adequate parallel acceleration lane. In this situation, the entering driver can select an adequate gap by looking into a rear-view mirror and can change lanes at approximately the speed of the through traffic (7, 8, 10, 11). Entrance terminals with auxiliary lanes, or long tapers, are generally classified in the low accident group, while designs with short tapers, direct entry, or short acceleration lanes are usually in the high accident category (8, 11).

Approximately 23 percent of all through-lane accidents occur in the vicinity of the entrance terminal (8). A recent evaluation of data collected for the Interstate System Accident Research Study II found that as the ratio of ramp to mainline traffic volumes increases, the accident rate also increases, regardless of the length of the speed change lanes. Little difference in accident rates for acceleration lanes of various lengths is evident when the merging traffic is less than six percent of the freeway traffic. As the percentage of merging traffic increases beyond this value, the additional length of acceleration lanes provides a significant reduction in accidents (6).

Another factor considered in the safety appraisal of entrance terminals is sight distance. The restriction of sight distance to either the driver on the ramp and/or the driver on the freeway normally produces an accident-prone design (7, 8). The probability of an accident at ramp locations is related to the vertical alignment of the freeway. The location of entrance terminals at points where the through lanes are on downgrades has a lower accident frequency than merging areas when the freeway traffic is on an upgrade, at the top of a crest, or at the bottom of a sag (11).

Although several studies have reported the effects of ramp merging control, little attention has been devoted to its effect on safety. Reduced accident experience was reported in an Atlanta, Georgia, study after the installation of a traffic signal on a high-accident urban freeway ramp (12). Rear-end collisions were reduced from 78 to 8 in the year following the installation of ramp merging control. Similar results have been demonstrated in Houston and Dallas, Texas.

In summary, the relative safety of entrance terminals is enhanced with geometric designs that provide long acceleration or auxiliary lanes, adequate sight distances for both freeway and ramp drivers, and freeway lanes on downgrades. Attention to these design details also improves the operational characteristics of the merging area.

Exit Terminals. The design of an exit terminal must provide for the efficient and safe transition of traffic from the freeway to the off-ramp. If the traffic volume on the exit ramp is not a controlling factor because of

a traffic spill-back condition, then the diverging process only involves the driver steering the vehicle from the through lane to the ramp. To prevent the development of any discontinuity on the freeway system, the geometry of the diverging area must be arranged so that any necessary speed reductions can be accomplished off the through lane by the exiting traffic. About 44 percent of traffic accidents on off-ramps occur in the exit terminal, and over 50 percent of off-ramp accidents are single vehicle types (5).

The length of deceleration lane available for the diverging maneuver has been the major design parameter considered in the safety evaluation of exit terminals. Off-ramps with speed-change lanes have a lower accident rate than those ramps without speed-change lanes (8).

Roadside signs, which are normally placed on the outside edge of curves and in the ramp nose, are the most vulnerable fixed objects in the exit terminal. Out-of-control vehicles normally leave the roadway on the outside edge of a curve while in entrance or exit terminals. In the case of off-ramps, the gore is extremely vulnerable to those drivers who misjudge the exit location (5). Geometric designs which allow off-ramp gores hidden behind an overpassing structure, out-of-sight over the crest of a vertical curve, or around a curve obscured by an embankment or structure, result in potentially high collision locations (13).

The types of accidents that occur in the area of the exit terminal are summarized by the following conditions: slowed for the exit and hit in the rear; turned from the wrong lane; missed the exit and stopped; hit the island nose; hit in the rear because of sudden stops in traffic (7). In general, these types of accidents can be reduced if the design of entrance and exit terminals provides adequate speed-change lanes, control of access, and proper sight distances to encourage smooth traffic flow at proper operating speeds.

Another important consideration is the design of the gore area on grade rather than elevated. When elevated it is virtually impossible to design a relatively safe gore or bifurcation area without the use of impact-absorbing devices. It is important, therefore, to key the gore or bifurcation points at grade.

Weaving Area. Weaving maneuvers are very common in the interchange area and involve the crossing of traffic streams by vehicles moving in the same general direction through diverging and merging processes. Unfortunately, little information is available in the literature on the subject of weaving accidents as related to various elements of interchange design.

At a weaving section, a vehicle in one traffic stream may collide with a vehicle in the other stream if the movements of the two vehicles converge on the same place at the same time. No matter how abrupt or how gradual the weaving maneuver, there is the possibility of an accident at each merging or diverging movement. As a result, the length of the weaving section has no influence on the possibility of traffic collisions, although length may very well influence the accident probability (14). In a recent analysis of interchange accidents, the accident rate was found to decrease significantly as the length of the weaving area is increased. In addition, as traffic volumes increase, the accident rate also increases. The accident rates level off for weaving lengths in the range of 700 to 800 feet (215 to 245 m) (6).

Geometric designs for weaving maneuvers should provide weaving sections that are at least 1000 feet (308 m) in length. Additional research is needed to correlate the relative safety of weaving areas with other significant features of geometric design and traffic control.

3.8.44 Lane Drops

The manner in which lane drops are designed, marked and signed are fundamental to the safety of freeways and particularly interchanges. The following three paragraphs are excerpts from the second edition of the "Yellowbook" (3).

> "An important factor in providing for safe and efficient traffic operations on freeways is maintenance of a basic number of lanes over a substantial length of roadway. Lane balance at interchanges must also be considered, but too often in the past application of this latter principle has resulted in violation of the former principle, i.e., the abrupt ending of a through lane at an exit. These have been found to be very hazardous locations. Lane drops should be avoided, and auxiliary lanes utilized to obtain lane balance at interchanges.
>
> When a change in the basic number of lanes is made, the lane drop should be provided downstream of an exit gore and not at the exit. Major freeway-to-freeway interchanges are an exception to this rule. Lane reduction designs at major freeway interchanges should be subjected to thorough study to assure that hazardous operating conditions are kept to a minimum.

When lane drops are unavoidable, the primary design consideration is to alert traffic to the unexpected restriction in driving conditions. The lane drop should be located so as to be highly visible to approaching traffic, and removed from other decision points. Sight distance of 1,000 feet (305 m) and a 70:1 taper should be provided. A full width 10 foot (3 m) shoulder, free of obstacles, should be provided along and downstream of the taper for emergency use. Lane drops should not be located on sections of road with near-minimum horizontal curvature and/or crest vertical curvature. Advance signing for lane drops is an essential requirement."

A recent research study (23) on lane drops is reported in NCHRP Report 175. A summary of research findings of that study are quoted as follows:

"A well-designed lane drop will inform an approaching driver, in a timely manner, of three very important facts: 1) that a lane drop situation is ahead; 2) how far ahead; and 3) what action the driver must take. Accordingly, the following eight principles have been developed to serve as guidelines for lane drop design:

1. The lane drop should be placed where the surface of the roadway remains visible continuously for a significant amount of time.

2. The lane drop should be placed away from attention-dividing conditions, such as ramps or complicated direcional signing.

3. The lane drop taper should provide adequate visual cues that inform a driver that his lane is ending and should allow a smooth lane change transition in the taper area.

4. The lane drop should be placed on the side of the freeway that is better with respect to given traffic and geometric conditions.

5. The lane should appear to end on the same side of the freeway as the operational lane drop.

6. When a lane drops at an exit ramp, an escape area of adequate dimensions should be provided to allow for a smooth transition into through lanes.

7. When a lane is added at an on-ramp and dropped at a nearby off-ramp, the entering drivers should be notified that the lane they are traveling in is not a continuous lane for through travel.

8. Consistent and appropriate traffic control devices should be used in advance of a lane drop (23).

Five different lane drop situations, based on design functions, have been identified:

1. Outlying situations - designed to accommodate reduced demand at the perimeter of a metropolitan area.

2. Add-drop situations - designed to accommodate temporarily increased demand at a weaving section.

3. Drop-add situations - designed to accommodate reduced demand through a major interchange.

4. Step-over situations - designed to reduce problems caused by left-hand ramps.

5. Lane split situations - designed to accomplish major route connections.

Applications of the design principles to each of the functional situations are discussed in this report (23)."

3.8.45 Cross Streets

It is essential that designs of ramps and interchanges give adequate consideration to their effects on the local streets to which they connect. Designs which provide good sight distance and avoid operational problems for the local facilities are essential. Exit ramps built immediately adjacent to the ends of overpasses may have sharply restricted sight distances. Consideration must be given to the effect of grades on the crossroad, in order to provide sufficient sight distance for the safety of turning maneuvers to and from the interchange ramps. Close attention should be given in interchange design to providing above-minimum sight distance along crossroads in view of the extra demand placed upon drivers in these areas.

Simple designs for ramp terminals at crossroads induce the fewest wrong-way movements and otherwise improve operations. At too many locations, geometrics at the intersections of ramps with the cross streets are confusing and hazardous.

Complicated intersections require an excessive number of signs and other traffic

control devices. It is strongly emphasized that over-control tends to confuse drivers and generate sight distance problems. Simple, straight-forward design with a minimum of operating controls is desirable for ramp connections with intersecting streets.

3.8.46 Structures

Structures are an integral part of interchange design and permit one roadway to pass over or under one or more roadways without interference to any crossing streams of traffic flow. However, about 10 percent of the fatal accidents on interchanges occur to vehicles passing under the structure; and vehicles passing over the structure are involved in approximately six percent of the fatal accidents (8). The safety problems that are attributable to this design element of interchanges involve the collisions of vehicles with the piers, abutments, parapets and curbs while passing under or over the structure.

The physical obstructions provided by piers, abutments, and bridge rails are more vulnerable in the off-ramp than in the on-ramp situation, because the off-ramp vehicles have a structure straight ahead more often than on-ramp vehicles.

The situation of overcrossing versus underpassing for the main freeway has been investigated in regard to interchange safety. If the ramps are associated with an underpassing freeway, slightly lower accident rates result from the gravity assistance of deceleration on off-ramps.

The distance from the on- or off-ramp to the nearest structure appears to have no statistical significance for accident frequency. However, reports do indicate that high-accident locations result from interchange designs with ramps under the structure proper and with ramp terminals less than 750 feet (230 m) from the structure (15).

Although some benefits accrue from underpassing freeway arrangements, safer designs are produced when the main freeway passes over the minor facility and when the ramp terminals are not located near the structure proper.

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TOPIC 4 SESSION 1
SAFETY IN PLANNING

Objectives:

1. As a result of this session, the participant should better understand the planning process relationship to safety.

2. The participant should gain an understanding of the street design, driveway location and design, layout of the on-site circulation system, and building location interactions as they relate to system safety.

4.1.1 LAND USE-STREET SYSTEM INTERACTION

The transportation system is the predominant man-made, semi-permanent framework around which the land uses and urban activity patterns develop. Land development, in turn, affects the street system in virtue of its location, type, intensity, access design, and site layout. Consequently, the land use and transportation elements are major components of the urban comprehensive plan.

The Comprehensive Plan is the official statement that sets forth the major policies concerning the future physical development of the municipality. The characteristics of the plan are that it is comprehensive, general, and long-range. It is comprehensive in that it encompasses all geographic parts of the community and all functional elements relative to the community's development. The plan is general in that it summarizes policies and proposals and does not indicate specific locations or detailed regulations, long-range in that it addresses the perspective of problems and possibilities 20 to 30 years in the future. While there is only one current comprehensive plan for a city, there may be several functional or categorical plans dealing with the several elements including: Transportation, land use, population, economic base, community facilities, administrative organization, and municipal ordinances.

Experience has demonstrated that the comprehensive plan is used most effectively in guiding city decisions when:

- The plan is formally adopted by the governing body
- There has been a lengthy period of public debate prior to adoption.
- The plan is available to and understandable by the general public.
- The plan is updated periodically to reflect the current development policies and goals of the community.

Provided that a municipality has a current and officially-adopted comprehensive plan, together with the appropriately-drafted city ordinances, city officials can exercise considerable direction over the city's growth and development. A major element of this effort will involve the street system.

4.1.2 THE STREET SYSTEM

The arterial street system should identify the boundaries of neighborhoods as suggested schematically in Figure 4.1.1. An arterial should not divide a neighborhood into two or more parts. Access to the residential neighborhoods and large-scale commercial and industrial development should be provided at a limited number of points which are of a high-quality design. Intermediate intersections might be provided to adjacent streets and access points where the median design limits the number of turning movements. Such a development scheme implies the implementation of a functional street system. In simplest terms, this means that individual streets should be identified by the degree

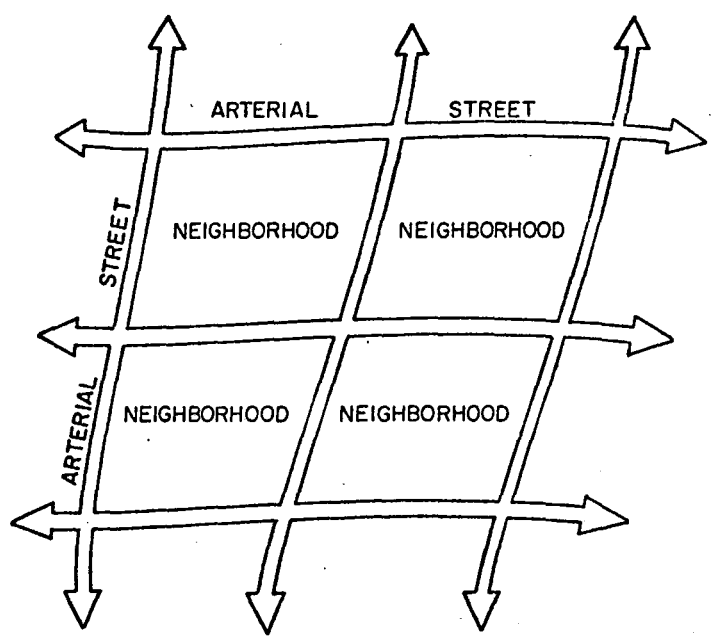
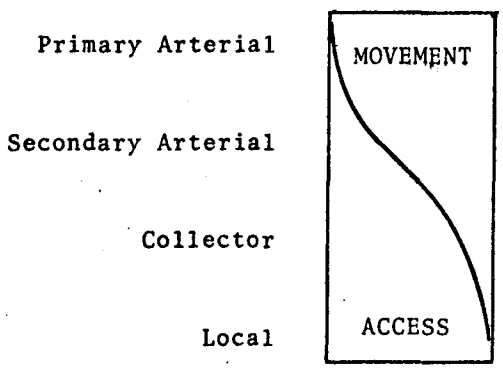


Figure 4.1.1 Arterial Streets Should Define the Boundaries of Neighborhoods

to which they should serve the mutually-incompatible functions of movement and access.

At one end of the spectrum, the primary arterials should accommodate movement; the access function should be permitted only to the extent that it does not interfere with safe, efficient, and relatively high-speed movement. Local streets, at the other extreme, principally serve land access -- movement is provided to a higher classification of street for completion of the trip. The degree to which different functional classes should accommodate movement and access is indicated by the following sketch:



Once the concept of functional classification is accepted, the function and general character of the different street classifications can be identified more specifically as follows:

- Primary Arterial Street

+ Be continuous for substantial distances (10 to 15 miles or more)

+ Connect major activity areas within the urban region

+ Connect with principal intercity highways leading to/from other metropolitan areas

+ Serve trips which are 5 to 7 miles or more in length

+ Safely accommodate traffic at speeds of 40 to 45 mph

- Secondary Arterial Street

+ Serve trips which are 2 to 6 miles in length

+ Be continuous for distances of 2 to 6 miles

+ Intersect with intercity highways that connect to other urban areas within the region not connected by a primary highway and arterial street network.

- Collector Street

+ Continuous for a distance of 3/4 - mile or less

+ Connect with neighborhood shopping, school and recreational facilities

- Local Residential Street

+ Terminate at a T-intersection with a collector street

+ Serve a limited number of dwelling units (perhaps 24 dwellings for a cul-de-sac; 36 to 48 for a loop)

In the case of apartments and commercial development, the circulation isles within the parking lots serve the function of the local street. Hence, the lowest classification of public street in a large commercial or apartment area normally will be a collector street.

Although the idea of functional classification is simple in concept, its relevance in safety design cannot be overemphasized. Traffic safety will result only when efficient traffic operations are achieved through the separation of the movement and access functions. This necessitates a systems design of adjacent land developments in combination with the design of the street elements within the median and at the margin of the traveled roadway.

4.1.21 Median Access Control

The high speeds and large traffic volumes experienced on arterial streets in developed areas suggests that median designs should limit left turn openings to intervals of approximately 450 feet between signalized intersections (1). These openings should be designed to provide limited movements only. Designs such as shown schematically in Figures 4.1.2, 4.1.3, and 4.1.4 will limit the number of conflict points. Alternating the designs as indicated in Figure 4.1.5 will provide access to and from both sides of the arterial. Designs of this type should be implemented on new sections of arterial streets prior to development of the adjacent properties so that the site plans can be designed for compatibility with the median design.

The safety advantages of barrier median designs are indicated in a 1967 report by Paul Box (2). His studies indicated that 60 percent of all driveway accidents in Skokie, Illinois, involved left turn maneuvers. The same study found that the driveway accident rates on arterial streets without control of left turns was three times that of streets having barrier medians.

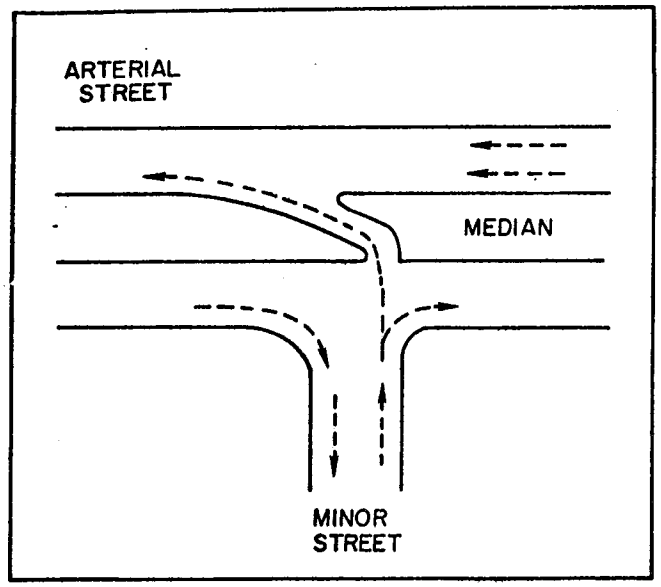


Figure 4.1.2 Left Turn to Arterial Street from A Minor Intersecting Street

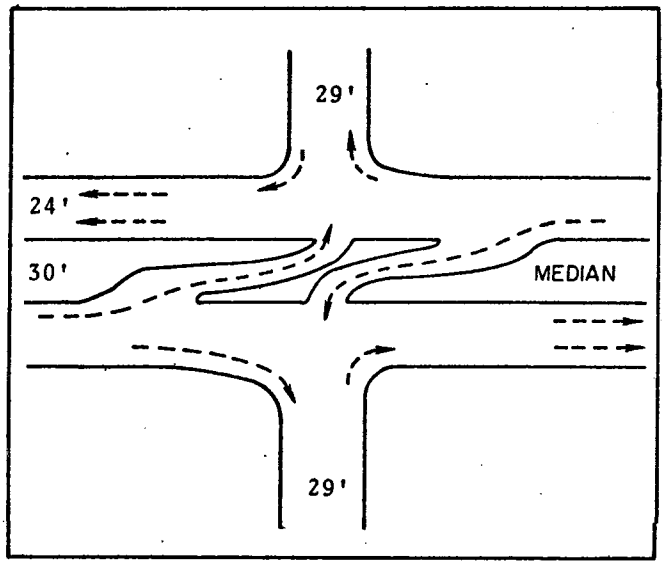


Figure 4.1.4 Opposing Left Turns from An Arterial Street to An Intersecting Minor Street at A Four-Way Intersection

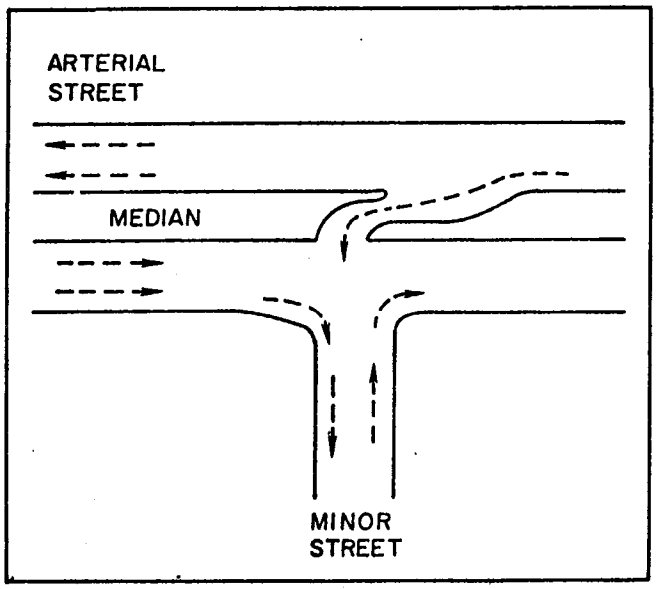


Figure 4.1.3 Left Turn from Arterial to Intersecting Minor Street or Major Access Point

4.1.22 Marginal Access

Analysis of traffic signal progression on two-way streets suggests that major signalized intersections with primary arterial streets should be at uniform intervals of between 1400 and 1800 ft (430 and 550 m) and at intervals of 1200 to 1600 ft (370 to 490 m) for secondary arterials. Such long intervals do not present difficulty for developments such as regional shopping centers, large office/industrial parks and large residential subdivision that frequently have frontages that are greater than these distances. Therefore, the major access points to large-scale developments can conform to the selected signal spacing, and the site plan can be developed accordingly.

Individual tracts having less than 400 to 500 ft (120 to 150 m) of frontage on the arterial can be designed with a common circulation system and common access points to the arterial. Individual parcels, in turn, take direct access from the common circulation system.

Such access requirements on individual properties can be controlled through the zoning subdivision and other development control ordinances in addition to design standards. Restrictions on driveway locations and spacing can be exercised by specifying:

- Minimum frontage that can be developed by subdivision of large tracts
- Maximum number of driveways per length of total frontage
- Maximum number of driveways per property
- Minimum distances from intersecting streets and other driveways

4.1.3 COMMERCIAL DEVELOPMENT

Control of access to single family and duplex residential subdivisions can be more easily achieved than for commercial and apartment development. This in part results from the frequently-held attitude--especially

by many developers--that all frontage on arterial streets is best suited for non-residential uses.

A clear statement of the problem in terms of accommodating efficient traffic movement on arterial streets, rather than as restriction on access, will help overcome objections and facilitate implementation of improved access design practices. This will require a more technically-sophisticated and comprehensive approach in which movement, access, and on-site development are considered to be interdependent elements of a complex design problem.

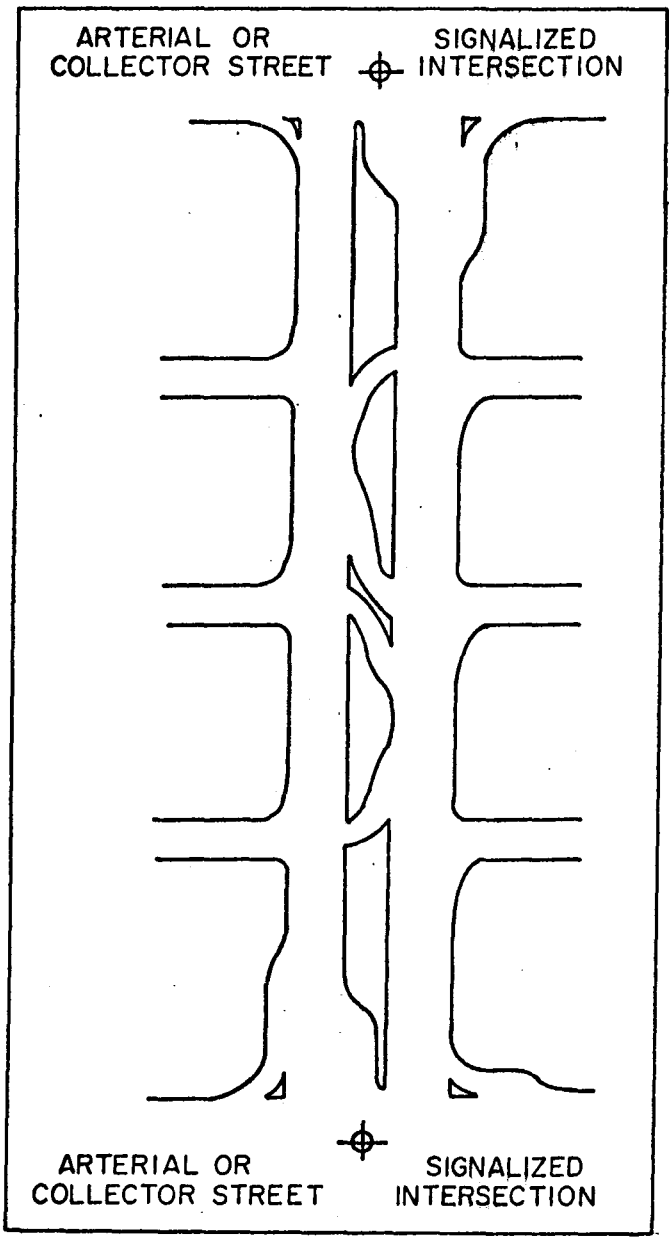


Figure 4.1.5 Access to Primary Arterial Street at Intermediate Non-Signalized Intersections

The initial site planning should anticipate the future traffic conditions and controls that might be imposed as traffic volumes on the arterial increase. This will help ensure the long-term economic stability of the development and can be encouraged by zoning greater depth for commerical uses. The commonly used depth of 200 ft (90 m) is inadequate to develop a site plan that will not result in on-site circulation problems from extending into the adjacent arterial street. Increasing the depth to a minimum of 250 or 300 ft (76 or 61 m) greatly increases the flexibility for improved site development designs.

Increased frontage on the arterial street also is essential in order to develop improved driveway designs and achieve a reasonable separation between a driveway and an intersecting street. In order to minimize conflict between vehicles turning into a direct access driveway and through movement past the driveway, minimum lot frontages such as the following are suggested (7) for consideration:

Functional Class of Street	Intersection Location: Near Corner	Far Corner	Mid-Block
Primary Arterial	900 ft (275 m)	450 ft (140 m)	400 ft (140 m)
Secondary Arterial	300 ft (90 m)	350 ft (105 m)	350 ft (105 m)
Collector	200 ft (60 m)	200 ft (60 m)	200 ft (60 m)

Properties with less frontage should be provided with access via a collector-distributor system rather than having direct access to the main lanes of an arterial street. A service (frontage) road frequently has been used to provide the direct access function--especially where the adjacent land is in commercial uses. While this design eliminates the problem of numerous direct access driveways, it results in a large number of conflict points where the service road intersects the perpendicular street (See Figure 4.1.6). The conflict problems and congestion become acute even with moderate traffic volumes on the service roads. Relocation of the service road to increase the distance between the intersections as indicated in Figure 4.1.7 will improve traffic operations and reduce accident potential.

A rear collector road eliminates many of the undesirable operational features of the service road design. It offers improved intersection control because of the greater separation between the collector street and

the arterial as indicated in Figure 4.1.8. Pedestrian protection and flexibility of design also are enhanced; however, greater design skills are required, and site development costs are increased.

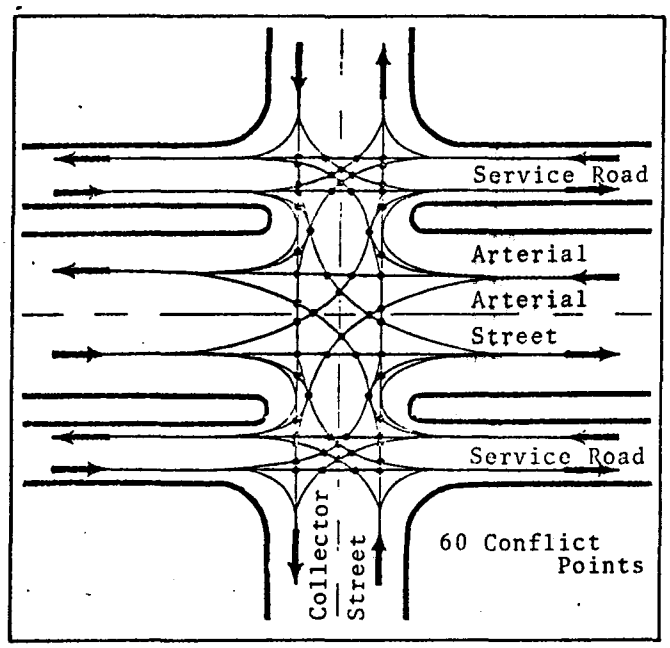


Figure 4.1.6 Conflict Points with Service Road Design

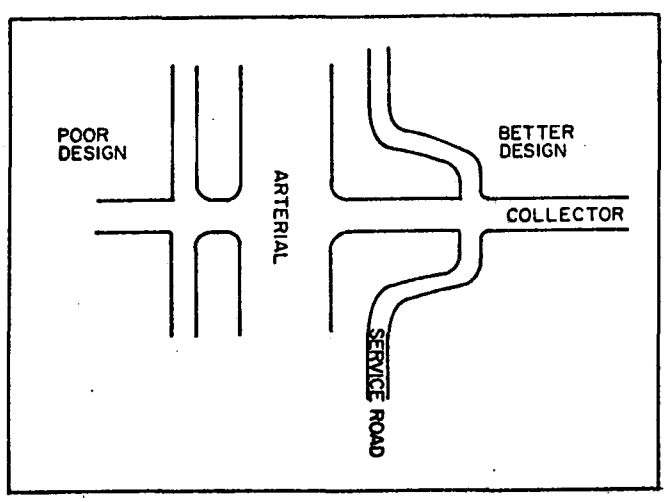


Figure 4.1.7 Relocation of Service Road Intersection

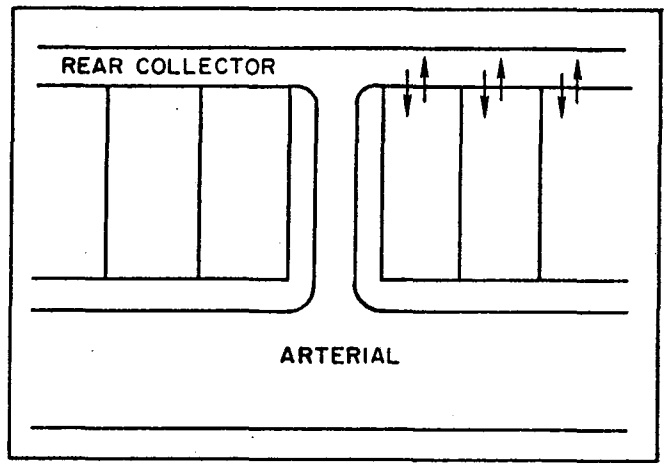


Figure 4.1.8 Rear Collector Design for Stand-Alone Commercial Properties

4.1.31 Driveway Design

The entire site plan is related to the driveway location and design in an effective development design. The entrance must be able to accept the maximum traffic expected to enter the site so that traffic does not back up into the arterial street. The on-site design must provide for a clearly-defined sequence of decision alternatives in order to minimize driver confusion. Interferences from other auto and/or pedestrian traffic in the vicinity of the driveway can be avoided through appropriate attention to site layout.

Entrances to major commercial and industrial developments should be designed similar to an arterial street including right and left turn lanes separate from the through traffic lanes. Free right turn lanes also should be provided to allow traffic to enter the site without interfering with other traffic on the arterial street. A barrier median should extend at least 250 ft (76 m) into the site, and no entrances or exits should be provided to or from adjacent parking areas (1). This median should be of adequate width (8 to 10 ft or 2.4 to 3 m) so that an appropriate sign can be located within it in order to clearly identify the location of the access point.

Driveways to smaller commercial developments can be designed with a direct or parabolic taper as indicated in Figure 4.1.9 so that interference with traffic on the arterial is minimized. Long, unrestricted curb openings should be avoided; Box (2) found that such driveways had an accident rate that is four times that of driveways which are designed to restrict entry/exit to specific points.

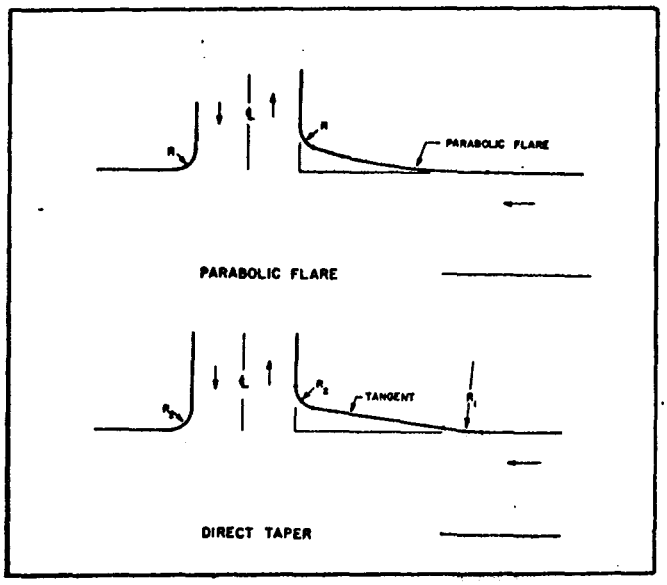
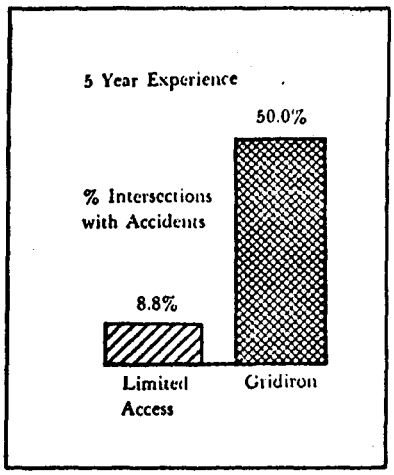


Figure 4.1.9 Commercial Driveway Design

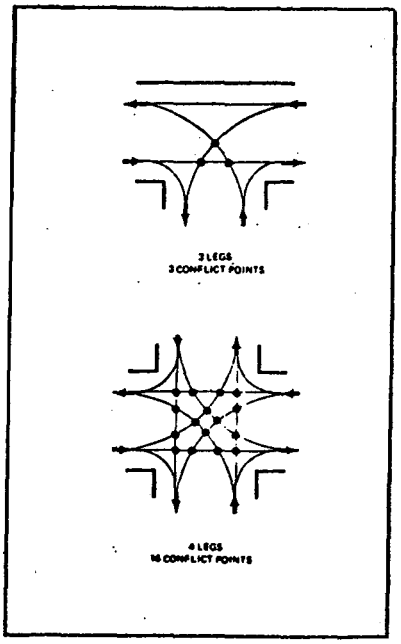
4.1.4 RESIDENTIAL DEVELOPMENT

The limited access subdivision is highly compatible with the hierarchy of functional classification. The discontinuous street pattern, curvalinear alignment, and infrequent intersections with the arterial street system discourage through traffic and high speeds. The local street serves land access, and the collector streets integrate the subdivision with the city by distributing traffic to the surrounding arterials.

The landmark study by Harold Marks (3) clearly identified safety benefits of the limited access subdivision design. Over a five-year period, one or more accidents occurred at 50 percent of the intersections in conventional gridiron subdivisions, compared to less than 9 percent of the intersections in the limited access subdivisions.



Use of the 3-legged or T-intersection greatly reduces the accident potential within the subdivision. The simple nature of the T-intersection results in three major conflict points compared to 16 for the 4-legged intersection as indicated below:



Hierarchial design of the street system so that the minor of the two streets is the stem of the T results in an automatic assignment of right-of-way. Consequently, as indicated in Figure 4.1.10, the accident experience of 3-legged intersections has been found to be substantially less than that for the common 4-legged intersection. It also should be noted that the 4-legged intersection within limited access subdivisions experience about one-third the accident rate (accidents per intersection per year) of similar intersections in the gridiron subdivision street pattern.

These data indicate that accident experience can be minimized through the design of residential subdivisions utilizing the concept of limited access to the development and the use of 3-legged intersections as the principal intersection design within the subdivision. Lots of the perifery of the subdivision can be designed to provide for access control as well as to buffer the residences from the street. Designs which orient the residential development inward toward the neighborhood of which they are a part and eliminate all direct access to the adjacent arterial street are more successful. The inward orientation is often emphasized by the construction of a wall or fence and plantings along the right-of-way line to the separation. Such treatment, with the advantages of visual protection, pedestrian crossing, elimination of parking, and more convenient service are achieved in addition to the elimination of direct access to the arterial street.

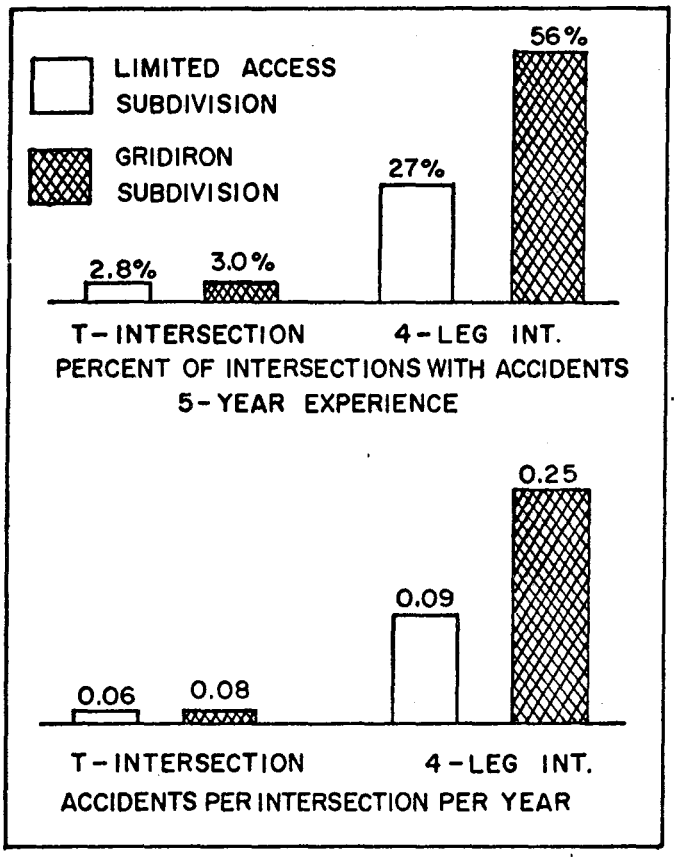


Figure 4.1.10 Accident Comparison of Three- and Four-Legged Intersections [*Source: Marks (3)*]

Reverse frontages and cul-de-sac lot design shown in Figure 4.1.11 are preferred designs. Rear alley, service road, and side-on designs shown in Figure 4.1.12 are not recommended for new development. However, they often can be used to advantage when modifying extension gridiron subdivisions to achieve many desirable features offered by the limited access subdivision.

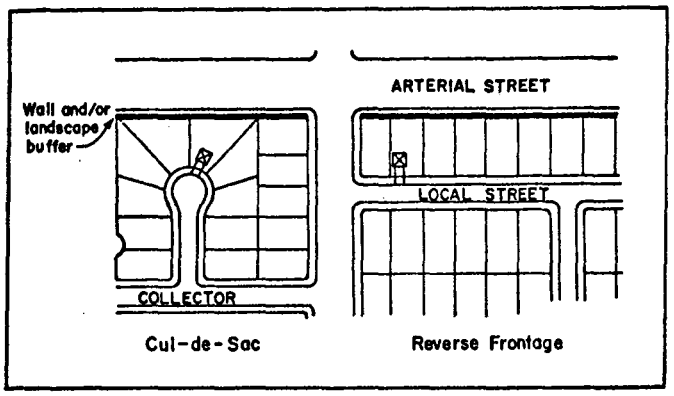


Figure 4.1.11 Preferred Residential Lot Design Adjacent to Arterial Streets

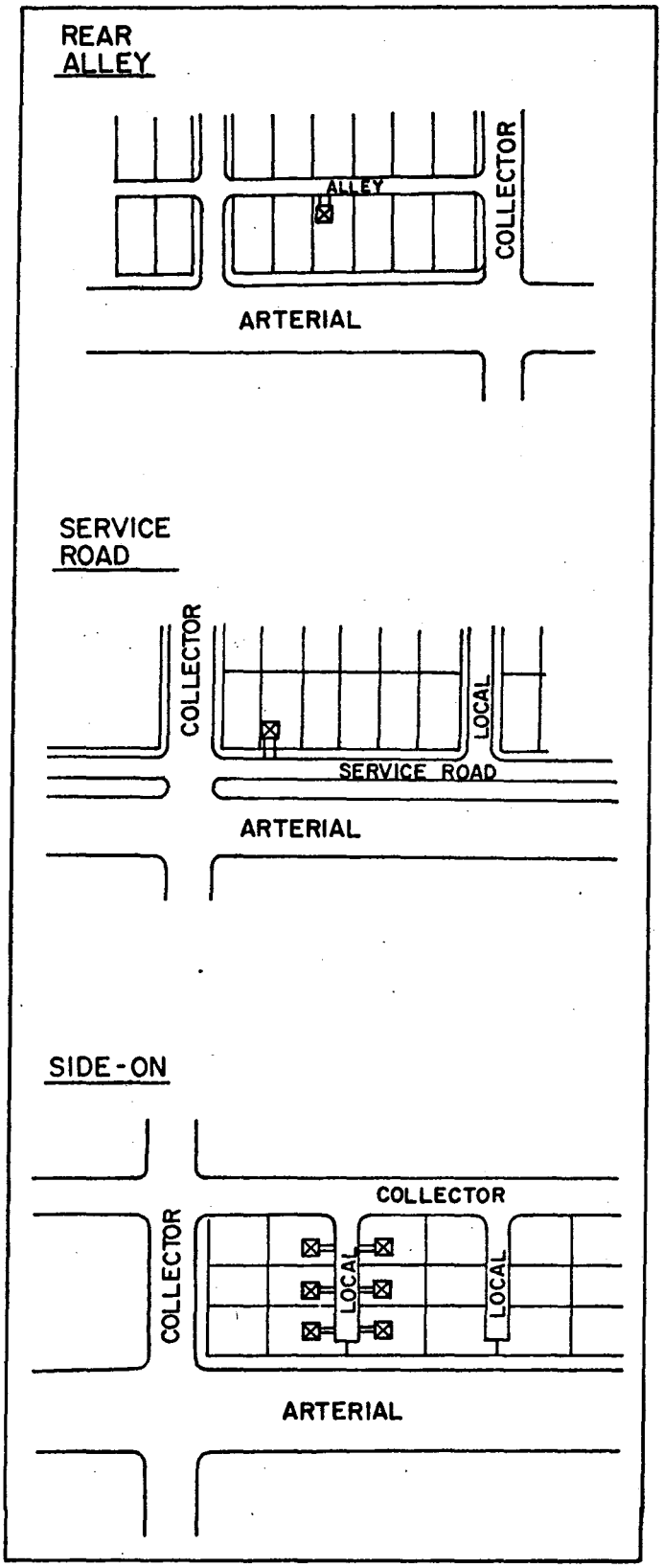


Figure 4.1.12 Alternative Designs for Residential Access

4.1.41 Modification of Existing Subdivisions

Older residential subdivisions which were designed with a gridiron street pattern can be modified to achieve many of the benefits of the modern limited access subdivisions--reduced through traffic, lower speeds, and fewer accidents. Discontinuity of the gridiron pattern can be accomplished through the application of diagonal traffic diverters at 4-legged intersections and abandonment of short sections of street as indicated in Figure 4.1.13. Intersections with the arterial street at the edge of the subdivision can be reduced in number and/or simplified by: redesign and/or the use of a barrier median to limit movements to right turn movements only; abandonment of a portion of the block adjacent to the arterial to create a short, dead-end street with side-on lots, and, construction of cul-de-sacs. It is essential that the concept of modifying the existing neigborhood street system to achieve the advantages of a limited access subdivision is accepted by a majority of the residents in the affected area before the changes are implemented.

Modification of the Stevens Neighborhood in Seattle reported by Orlob (4) resulted in a 50 percent reduction in traffic on east-west streets and a 25 percent reduction on north-south streets. Total cost, including the first year's maintenance of landscaping, for modification of six intersections, was $38,000. Accidents were reduced from an average of 12 per year during the five-year period prior to the modification to only one accident in the two years following the change.

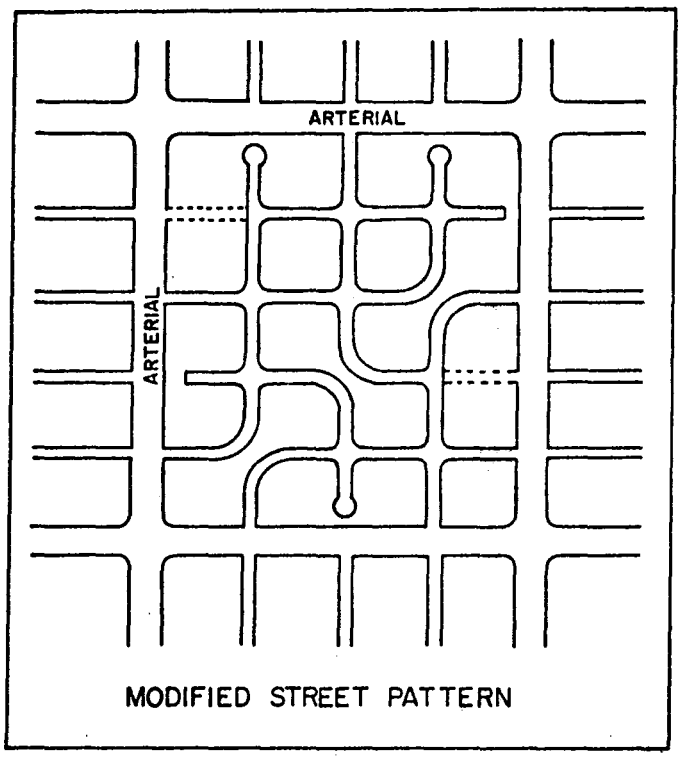


Figure 4.1.13 Modification or Gridiron Street Pattern to Achieve Limited Access Subdivision Characteristic

4.1.5 DEVELOPMENT CONTROLS

The city has a number of tools available for the effective management of the development of the transportation system and the adjacent land uses. These include:

- Zoning ordinance
- Subdivision ordinance
- Major street ordinance
- Official map
- Surety and performance bonds
- Conditional use permit
- Building permit
- Driveway permit
- Occupancy permit
- Administrative procedures

4.1.51 The Zoning Ordinance

Zoning is essentially a means of insuring that land uses in a community are properly situated in relation to one another. It allows the control of the type and density of development in each area so that individual property and neighborhoods can be serviced adequately by the recreation facilities and utility systems. The ordinance provides for regulation of:

- Height and bulk of structures
- Area of a lot which may be occupied
- Density of development
- The use that may be made of the property

The zoning ordinance also might be employed to enhance traffic safety in the following ways:

1. Zone large tracts which can be developed compatible with the arterial system and are of sufficient size to provide flexibility in design of the on-site circulation system. Examples of suggested minimum sizes for different types of development are: 10 acres for apartments and general commerical; and 80 acres for single family residential.

2. Zone for densities of development that are compatible with the street system. Land uses which generate large traffic volumes should be zoned for locations where the arterial street has the capacity to accommodate the traffic volumes, and adequate medial and marginal access design can be provided.

3. Require adequate off-street parking and loading spaces so that parking on streets (other than on local and collector streets in single family and duplex residential areas) can be eliminated.

4. Require permit and establish review procedures for parking lots, of more than some maximum size (say 20 spaces) constructed as part of any apartment, commercial, or industrial development.

4.1.52 Subdivision Ordinance

The subdivision ordinance is a locally-adopted law enacted by each municipality which governs the process of converting raw land into building sites. A developer/owner is not permitted to make improvements or to divide and sell the land until the planning commission has approved the plot of the subdivision. Approval or disapproval is based on compliance or noncompliance with the development standards set forth by the ordinance.

While the subdivision regulations serve a wide range of purposes, they can be utilized to achieve improved safety by the following:

1. Require intersection designs (such as T-intersections within subdivisions) that can safely accommodate expected traffic volumes and prohibit designs (jogs) that are confusing or may present traffic and safety problems.

2. Require sidewalks and bicycle paths to accommodate pedestrians and bicyclists separate from automobile traffic.

3. Require street patterns that discourage through traffic on local residential streets and which have limited access to the adjacent arterial streets.

4. Establish minimum lot size which can have direct access to the arterial and require common access and/or internal circulation via private access easements, or public streets, where arterial street frontage is subdivided into lots.

5. Establish building lines that make it possible for vehicles to maneuver without interference to traffic on the arterial street.

4.1.53 Street Development Ordinance

The major street (thoroughfare) ordinance might be a separate document or it might be incorporated into other ordinances--principally the subdivision ordinance. In any case, the thoroughfare element of the comprehensive plan needs to be translated into specific criteria relative to the planning and design of the street system and the development of adjacent land as it relates to the street system. Areas that need to be addressed in the ordinance include:

1. Establishment of minimum rights-of-way and cross-sections for different functional classes of streets

2. Establishment of basic horizontal and vertical design criteria.

3. Establishment of minimum horizontal and vertical curvature

4. Identification of typical intersection configurations

5. Establishment of signalized intersection spacing

6. Specifications of frequency of access points

Most municipalities currently exercise some degree of control over the above except for areas 5 and 6. Guidelines similar to those indicated in Table 4.1.1 are suggested.

4.1.54 Other Control Mechanisms

The official mapping powers can be utilized to identify the future location of proposed arterial streets and other public facilities. This map indicates the planned extension of the arterial street system; therefore, it can be an effective means of communicating future street and access availability to the private development interests.

Surety and performance bonds can be employed to insure that driveways are located and constructed as specified. Thus, it presents the municipality with a means of achieving driveway designs which allow for safe maneuvering when exiting and entering the street and minimizes the conflict between access and traffic movement.

A conditional use requirement can be written into the zoning ordinance to provide a review and approval procedure to ensure that the site layout (including access, internal circulation, and building location) will provide for convenient and safe traffic flow.

TABLE 4.1.1

EXAMPLE OF ACCESS CONTROL GUIDELINES
FOR URBAN HIGHWAYS AND STREETS

ITEM			FUNCTIONAL CLASSIFICATION					
			PRIMARY ARTERIAL	SECONDARY ARTERIAL		COLLECTOR		LOCAL
NUMBER OF TRAFFIC LANES			4 OR MORE	4 OR MORE	2	4 OR MORE	2	(DESIGN STANDARDS TO INSURE REASONABLE DRIVEWAY DESIGN AND CONSTRUCTION FOR THE PROTECTION OF THE HOME BUYER)
MINIMUM SPACING OF SIGNALIZED INTERSECTIONS			1600 TO 2000 ft	1200 TO 1600 FT.		VARIABLE		
DIRECT ACCESS DRIVEWAYS	RESIDENTIAL		PROHIBIT	SPECIAL CASES ONLY		ONE PER PROPERTY		
	COMMERCIAL AND INDUSTRIAL		MAJOR GENERATORS UNDER SPECIAL CONDITIONS – SEE TEXT	SPECIAL TURN LANES	ONE PER 200 FT FRONTAGE	TWO PER 100 ft FRONTAGE		
ACCESS POINTS WITH OTHER PUBLIC STREETS	EXPECTED ADT ON INTERSECTING ROADWAY	UNDER 500	RIGHT TURNS; LEFT TURNS ONLY WHERE MEDIAN WIDTH PERMITS PROTECTED TURN LANE	RIGHT TURNS; LEFT TURNS WHERE MEDIAN WIDTH PERMITS PROTECTED TURN LANE	RIGHT TURN LANES; LEFT TURN FROM TRAFFIC LANE	DIRECT TAPER FOR RIGHT TURNS, LEFT TURNS FROM TRAFFIC LANES		
		500 TO 2000				RIGHT TURN LANES, LEFT TURNS FROM TRAFFIC LANES		
		OVER 2000	RIGHT TURNS ONLY UNLESS CONFORMS TO SIGNALIZED SPACING		RIGHT AND LEFT TURN LANES	RIGHT TURN LANES, LEFT TURN LANES WHERE PRACTICAL		
GRADE SEPARATIONS	PROVIDE	RAILROAD	ALL CROSSINGS	CROSSING PROTECTION – SEE TEXT				
		HIGHWAY		NOT APPLICABLE				
	PLAN FOR	RAILWAY	ALL CROSSINGS	MAIN LANE CROSSINGS		CROSSING PROTECTION —SEE TEXT		
		HIGHWAY	ESTIMATED PEAK HOUR VOLUMES WITHIN 30 YRS. > CALCULATED INTERSECTION CAPACITY	NOT APPLICABLE				
ABS. MINIMUM MEDIAN WIDTH			14 FT	14 FT	NOT APPLICABLE ON 2-LANE SECTION	NONE	NOT APPLICABLE ON 2-LANE SECTION	NO ACCESS CONTROL
DESIRABLE MIN.			16 FT	16 FT				
MINIMUM SPACING OF MEDIAN OPENINGS			500 FEET PLUS 25 FT./CAR	300 FEET PLUS 25 FT./CAR		NONE		

Rev: 11/77

A conditional use requirement can be written into the zoning ordinance. The objective of such a permit is to provide a review and approval procedure to ensure that the site layout (including access, internal circulation, and building location) will provide for convenient and safe traffic flow. It It also provides the means of ensuring that adequate provision has been made for fire protection, solid waste pick-up, and pedestrian movement.

When the building permit and the driveway permit are processed and issued jointly, similar control can be achieved relative to access design and internal circulation. The occupancy permit could be employed in a simple fashion when a new occupant is proposing to move into an existing developed property.

In any event, administrative procedures must be implemented to ensure compatibility between the several ordinances and coordination in their application if convenient, efficient and safe access designs are to be implemented.

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TOPIC 4 SESSION 2

SAFETY CONSIDERATIONS IN PUBLIC TRANSIT

Objectives:

The participant should be able to:

1. Gain an awareness for public transportation consideration in street design,

2. Establish design guidelines for on-street bus stops,

3. Establish safety and operational design guidelines for reserved lanes and exclusive roadways for buses.

The continued growth in urban areas has generated increased opportunities for conflict between automotive, transit, walking, and other forms of urban transportation. Much of the interest in safety has been directed toward the reduction of automobile accidents. An increased awareness that all the modes of urban transportation need to be coordinated and the recent emphasis on Transportation Systems Management (TSM) is stimulating increased interest in safety considerations in transit services. Bus loading and unloading areas, pedestrian volumes in the vicinity of high-volume bus stops, and reserved lanes are but a few of the areas involving safety considerations in the design of transit services.

Poor safety performance (i.e., high accident experience) results from poor design and/or inefficient operations. Designs and techniques that yield efficient operations also can be expected to provide safe operation. However, the specific accident information that would be most useful in the evaluation of different designs is only sporadically available.

4.2.1 SAFETY IN TERMINAL DESIGN

Terminal areas present unique problems in separation of vehicular and pedestrian traffic due to the variety of access modes - park-ride, kiss-ride, taxi feeder bus, walking, and bicycles. In addition, large areas for the park-ride and sizeable storage for the kiss-ride pickup are required. Inadequate design will increase passenger delays and cause negative impact on surrounding property, as well as create safety problems.

Suggested guidelines for the general design of transit terminals are:

- Access and circulation in the vicinity of the terminal should be in the following priority: pedestrian, bicycle, transit, taxis, kiss-ride, and park-ride.
- The distance from the access points to any intersection should be adequate to provide for maneuvering and to minimize conflicts.
- Adequate capacity should be provided at the access points
- Terminal area layout should minimize conflicts between the modes
- Automobile drivers should not be confronted with multiple decisions at one time (e.g., decision points should be spaced adequately)
- Signing should be simple
- Flexibility to adjust to changes in transit volume and operations should be provided

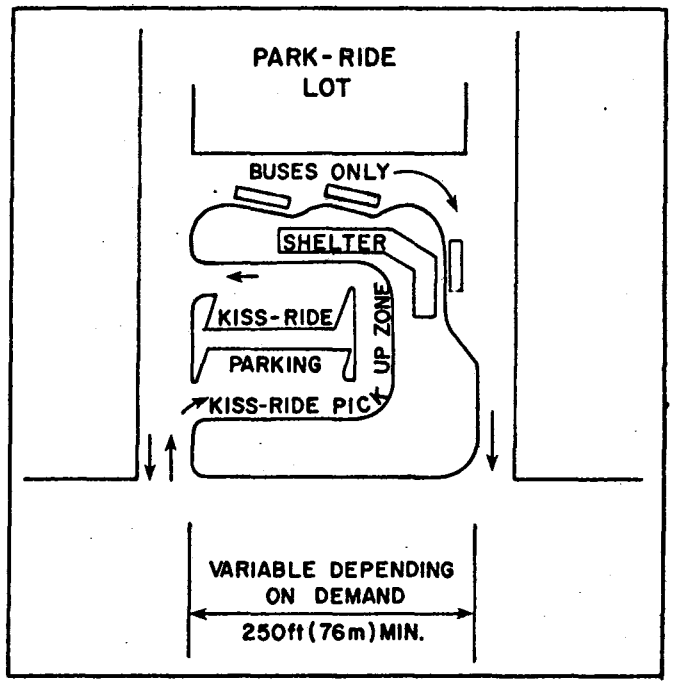

Figure 4.2.1 Schematic Layout of Small Transit Terminal with Common Access Drives for Autos and Buses

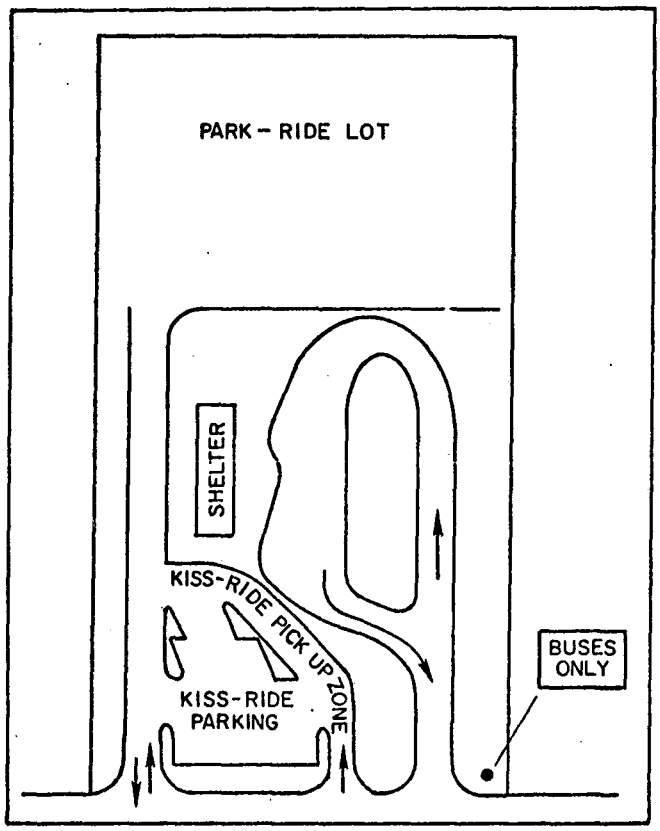

Figure 4.2.2 Schematic Layout of Suburban Transit Terminal with Separate Access for Private Autos and Transit Buses

Where larger transit passenger demands are present and park-ride/kiss-ride operations are involved, the terminal area should be located off-street but with convenient access to/from the arterial. Examples of site layouts that encompass design principles outlined above are shown in Figures 4.2.1 and 4.2.2.

Small terminal facilities might be provided along the freeway frontage road as shown in Figure 4.2.3 or adjacent to an arterial street as shown in Figures 4.2.4 and 4.2.5. The offstreet design shown in Figure 4.2.5 is the better design from a safety standpoint in that it provides a physical barrier between the curb lane of traffic and the buses; it also avoids the long open section of curb resulting with the turn-out design. The turn-out design should be limited to locations where no more than two buses will be stopped at any one time.

4.2.11 Bus Loading Areas

Buses serving particular routes should stop at the same loading location each time in order to minimize confusion and service time. The familiarity developed by regular riders will result in less pedestrian movement and reduced opportunity for persons in a rush to catch their bus to fall or collide with other individuals.

When buses are stopped parallel to the curb, a minimum of 90 ft (27.4 m) (Figure 4.2.6) should be provided for each bus that would be at the stop at the same time. The sawtooth design is more efficient in use of linear space. It also enables passengers to see the route identification on the front of all buses from the downstream end of the platform area.

4.2.2 SAFETY DESIGN FOR ON-STREET BUS OPERATIONS

4.2.21 On-Street Bus Stops

The on-street bus stop is that part of the roadway occupied by the transit vehicle while stopped to load and unload passengers plus the area needed to maneuver to and from the curb. Stops might be located on the near-side of an intersection, or the farside, or at a mid-block location. Location is a function of vehicular traffic, pedestrian movements, and physical characteristics of the specific location. Therefore, an investigation should be made to select the best location for each bus stop.

Sufficient room must be provided to allow the bus to pull parallel to the curb. Non-parallel stops will interfere with traffic movement in the adjacent lane and cause inconvenient conditions for passengers (especially those leaving the bus via the rear door). Both present potentially hazardous situations. Minimum dimensions for near-side and farside stops are shown in Figure 4.2.7; Figure 4.2.8 gives similar dimensions for mid-block stops. An additional 45 ft (13.7 m) are required for every additional bus when two or more buses are at the same stop simultaneously.

4.2.22 Lane Width

Width of the traffic lane required for bus transit operations depends on various factors including:

- Vehicle dimensions
- Speed of operation
- Horizontal alignment
- Traffic in adjacent lanes
- Driver reactions

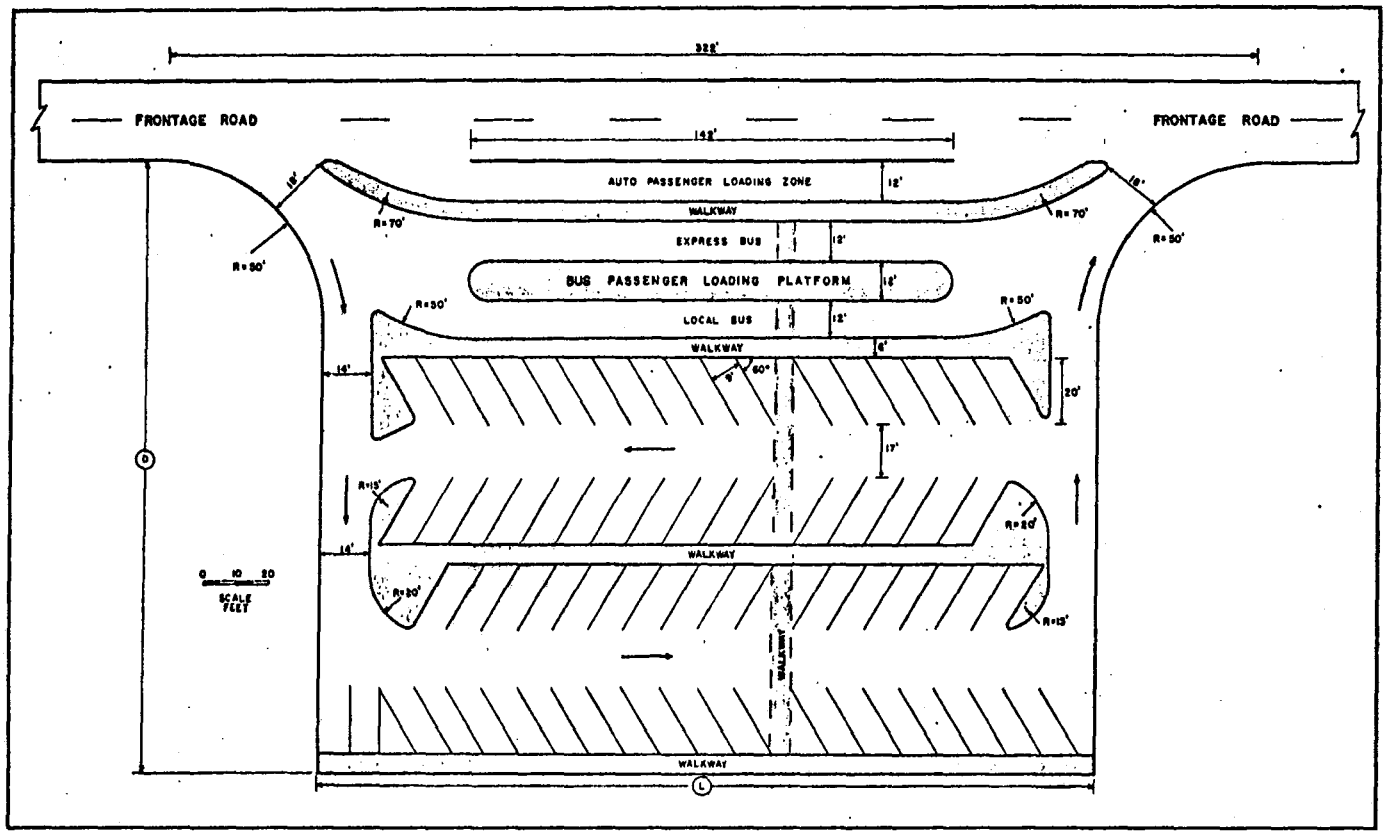

Figure 4.2.3 Park-Ride and Feeder Bus Terminal Adjacent to Freeway Frontage Road

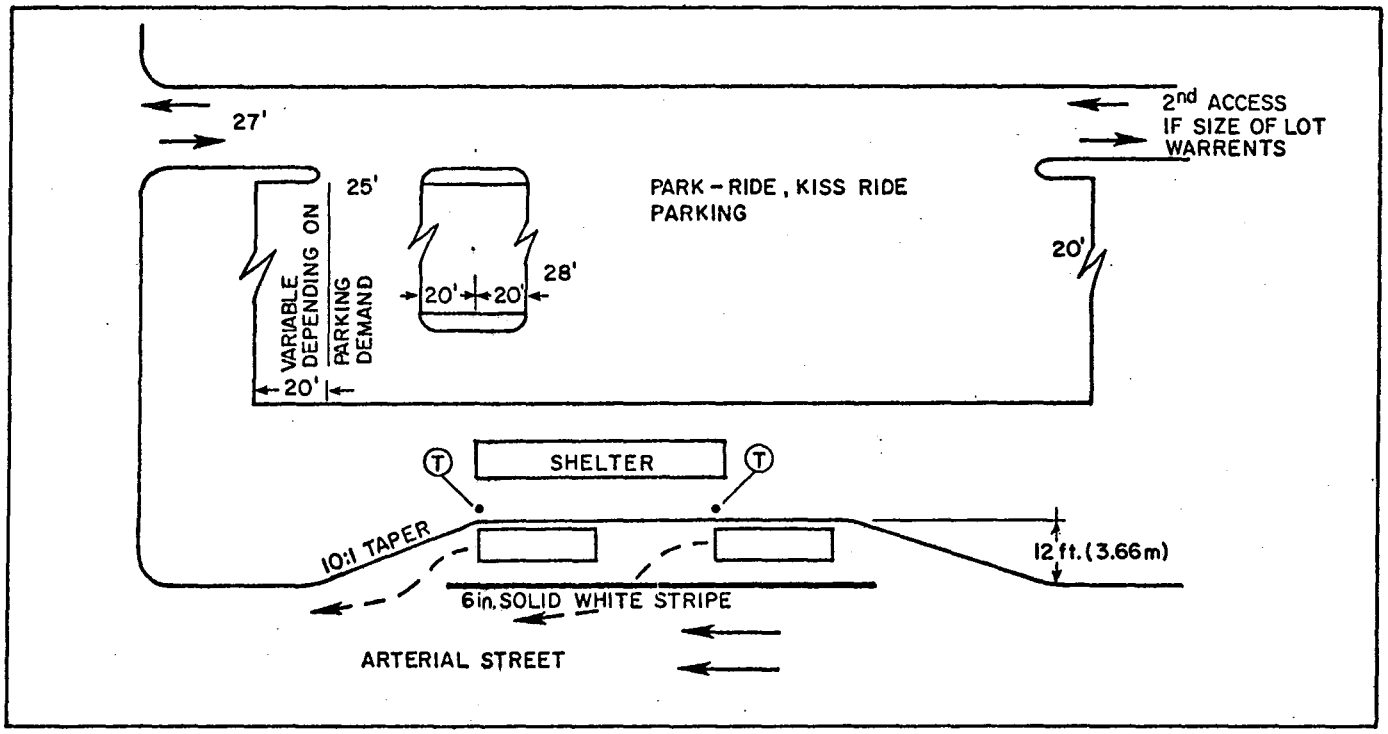

Figure 4.2.4 Schematic of Bus Turnout

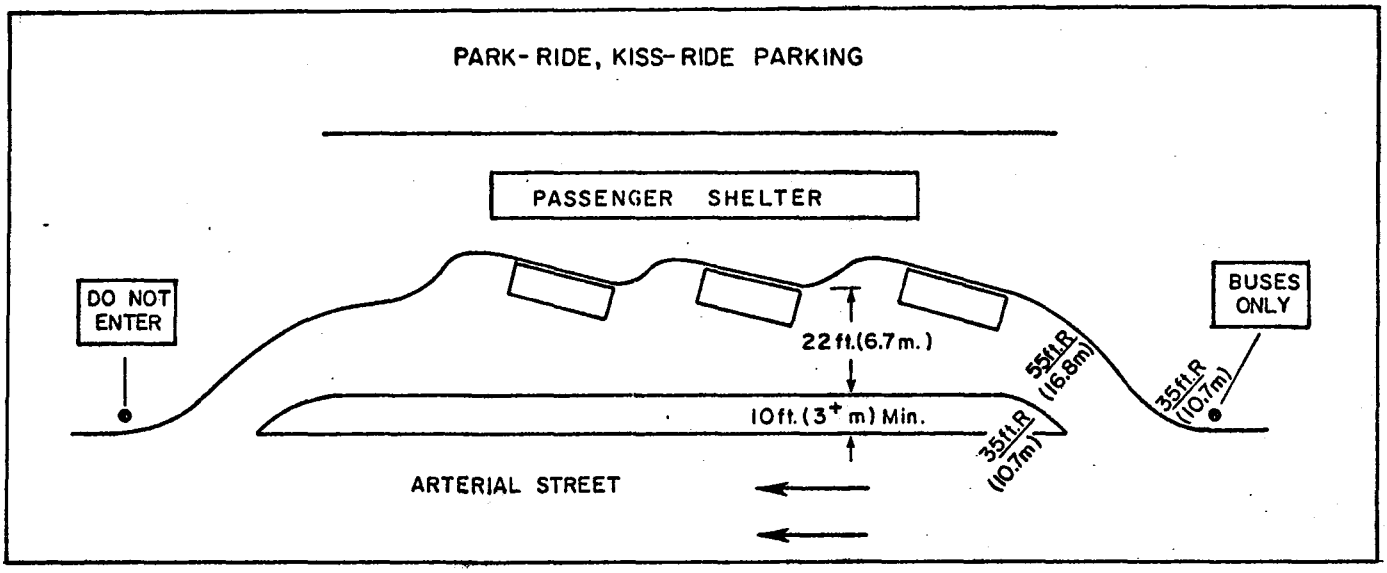

Figure 4.2.5 Schematic of Off-Street Bus Stop

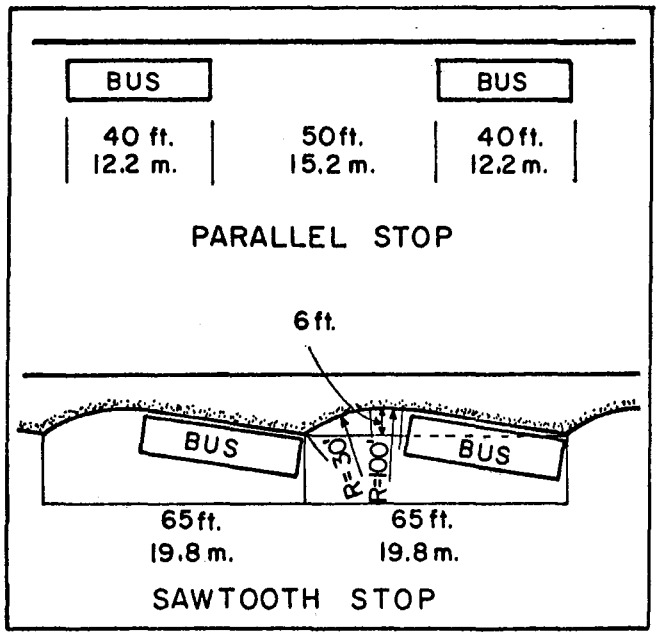

Figure 4.2.6 Bus Loading Areas

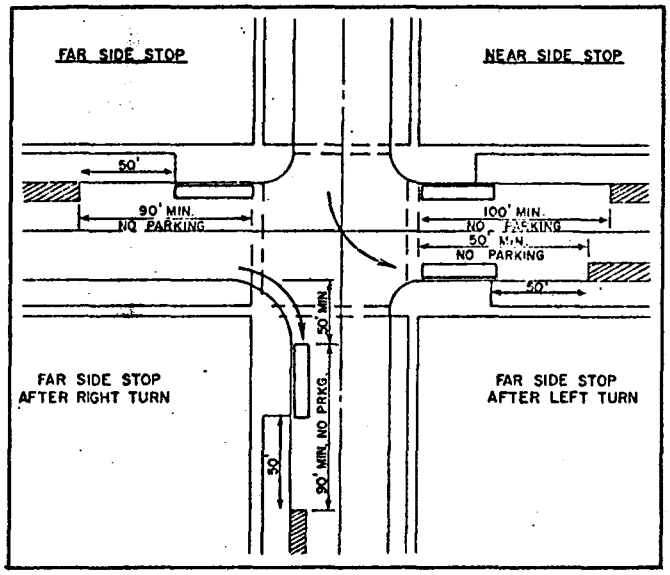

Figure 4.2.7 Minimum Requirements for Near-Side and Far-Side Bus Stops

For purposes of establishing lane width for transit operations, the gutter section should not be considered as part of the lane for three reasons. First, the ride characteristics with the right wheels in the gutter are undesirable. Second, the repetitive impact of the transit coach wheels passing over the grates on drop inlets causes physical deterioration of the drop inlet which requires expensive maintenance. And third, the drainage requirements during wet conditions necessitate that the bus not infringe on the gutter section.

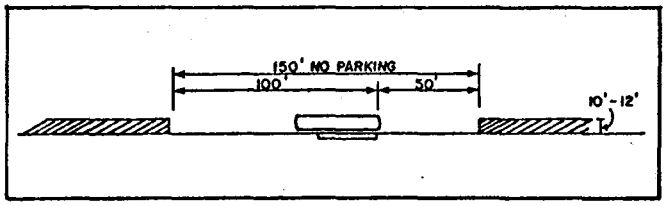

Figure 4.2.8 Minimum Requirements for Mid-Block Bus Stop

The minimum lane width for bus operation should be 12 ft (3.66 m), exclusive of gutter sections, in order to provide for adequate driver reaction at normal transit operating speeds. Where available pavement width is restricted, the lane width for transit operation may have to be reduced; however, it should not be less than 11 ft (3.35 m).

When separate roadways are provided for buses, the recommended widths, exclusive of 1.5 ft (0.46 m) curb and gutter section on each side, are:

- 14 ft (4.26 m) for one-way operation
- 24 ft (7.31 m) for two-way operation

4.2.23 Exclusive Bus Lanes on Arterial Streets

In most cases, inexpensive sign, signal, and marking changes appear to be adequate for implementing exclusive bus use of curb lanes. Accident rates on streets with reserved bus lanes reportedly are not different from those on other arterials carrying similar traffic volumes.

However, the type of accident pattern that might be expected will be different. For example, a high percentage of the auto-bus accidents on the contra-flow curb lane in Indianapolis, Indiana, involved autos approaching the contra-flow lane from the near-side; drivers apparently did not look for southbound buses on a one-way northbound street. Accidents involving pedestrians may also occur because of the buses traveling in an unexpected direction.

The bus-accident rate on NW 7th Avenue in Miami, Florida, was increased several-fold when the median bus lane was implemented. Most of the accidents involved autos that were making illegal left turns or other illegal uses of the bus lane. However, the accident rate per million bus-miles on the northern section of NW 7th Avenue where there was a median barrier were about one-seventh to one-ninth the rate on those sections without a barrier median. This difference suggests that a physical barrier which will prevent midblock left turning is desirable where a reserved bus lane is provided.

Traffic posts or 6 in (15.2 cm) high dividers have been utilized in a few instances to provide separation between auto traffic and the contra-flow bus lanes. Such physical separation probably is most desirable when the volume of buses is relatively low so that an on-coming bus is not in view at all times.

Where median bus lanes are utilized, it is essential that an adequate area for passenger refuge should be provided. When the refuge island is of minimal width - 5 to 6 ft (1.5 to 1.8 m) - a protective barrier fence should be provided in the bus stop area - especially where passengers are leaving through the rear door. Where wider refuge areas - 8 to 10 ft (2.4 to 3.0 m) or more - are provided, additional protection is not necessary.

4.2.3 FREEWAY-RELATED TRANSIT IMPROVEMENTS

Accidents on the Santa Monica Freeway in Los Angeles approximately doubled with the implementation of the diamond lane project. However, there was an indication of a downward trend in accidents during the life of the project. The very large number of accidents during the first week (nearly 5-times the number of accidents per week in the "before" base period) suggests that there is a "learning period" associated with the implementation of preferential treatment of buses and high-occupancy vehicles. Since there was no ready explanation for the increased accidents, there is no apparent action that might be taken to improve the safety performance of such projects.

Tangent section freeway ramps should have a minimum width of 14 ft (4.26 m), exclusive of curb and gutter sections where buses will be entering and leaving the freeway; appropriate widening, of course, must be provided on horizontal curvature. Where buses bypass the queue of autos on the ramp shoulder, as in the Los Angeles area, a 12 ft (3.66 m) pavement, 11 ft (3.35 m) absolute minimum is suggested.

Because of the lower acceleration capability of the city transit coach, relatively long acceleration lanes should be provided where buses enter the freeway system. In most cases, however, it will not be feasible to provide an acceleration lane which would permit the buses to achieve the speed of traffic before merging into the traffic lane. Desirable practice would be to provide sufficient length so that the bus could merge at a 10-mph (15 km/h) maximum speed differential. The minimum length of the acceleration lane, exclusive of taper, should not be less than 500 ft (152 m), however.

4.2.4 SECURITY IN TRANSIT SERVICES

Specifications for bus stop shelters should specify transparent panels so that the interior of the stop is clearly visible from passing vehicular traffic. Lighting within the shelter and in the vicinity of the bus stop should be adequate to provide a high level of illumination at night. Such designs discourage criminal attacks on waiting passengers.

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TOPIC 4 SESSION 3
DESIGNING A SAFE DRIVING ENVIRONMENT

Objectives:

1. The participant should be able to define the driving environment and be able to utilize this definition in design of a safe driving situation,

2. Further, the participant can establish a priority system for evaluation of operational, safety and environmental goals when a conflict arises, and

3. Be able to establish the alternatives for improving pedestrian and bicycle safety.

4.3.1 INTRODUCTION

The driving environment includes all features of the roadway and its environs which are normally visible to the motorist. Thus, all the features which are designed in or added on are combined with naturally-occurring elements to produce the total effect which the driver perceives. Within this context, there are three general areas of concern: 1) Landscaping and roadside beautification; 2) Bicycles and 3) Pedestrians. Each of these three areas will be considered from the safety point of view.

To compound the problem of developing and maintaining a safe driving environment, the combined effect of individual elements each designed to provide a maximum of safety and beautification may result in a less than beautiful facility which is frequently unsafe. This effect is known as the "Gestalt Effect." Stated simply, this principle theorizes that the whole is not necessarily equal to the sum of the individual parts. The effect must be determined for each unique combination of natural and man-made elements. For example, startling, beautiful vegetation can aid in keeping the driver alert in long tangent sections, thus reducing accident potential or distraction of the driver in areas of high driver work load which can contribute to the accident potential. Thus, beauty like safety must be built in with due consideration of the potential safety and operational interactions that are certain to occur.

4.3.2 CONFLICTING REQUIREMENTS

Designing the driving environment almost immediately results in a conflict between the goals of the various groups involved in the design process. A direct straight line between origin and destination is the least distance and hence the least time and fuel consuming route. It may, however, result in excessive cuts and fills, destroy historic or unique features, be monotonous to the driver and frequently are unsafe. Conversely, a narrow twisting roadway matched almost exactly to the terrain and avoiding all environmental sensitive areas provides a minimum of impact on land use but can also result in excessive grades, a reduced level of safety, added travel distance which involves greater fuel consumption and air pollution.

Similarly, large trees can be very beneficial in creating a proper perspective for the motorist, blocking undesirable views and providing a pleasant view. Conversely, if they are placed within the clear zone, trees constitute a substantial hazard to the motoring public. These points bring up the logical question of which criteria should govern the design when conflicts exist. Conceptually, the highest priority must go to operational requirements. References in the operational elements of the system are most frequently the reason for excursions onto the roadside with the associated safety problems. The safety of motorists who encroach upon the roadside is certainly a higher priority than uplifting their spirits. As a basic policy, all three criteria should be equally weighted until it is apparent that a conflict between these criteria exists. At this point the following decision priority is suggested:

(1) Operational Requirements

(2) Roadside Safety Requirements

(3) Beautification Criteria

It should not be interpreted that these criteria are mutually exclusive. It does mean that changes in the nature of the treatments may result from the priorities which are established. For example, when a particular location would require plantings for aesthetics purposes which are to be located fifteen feet from the pavement edge, a shrub with many smaller members would be used in lieu of a tree. Often keeping the public informed of the need for the safety improvement will help ease possible objections to restrictions placed on beautification measures.

4.3.3 SAFETY REQUIREMENTS OF LANDSCAPING AND SCENIC IMPROVEMENTS

4.3.31 Landscaping and Safety

Landscaping involves the use of reshaping the land surface, selected plantings and naturally occurring or selected objects to enhance the visual appearance of the roadside. At points of low driver work load, such treatments may serve to stimulate the driver and help to keep him alert. In areas of critical driving control requirements, the same treatment may constitute a hazard by distracting the motorist. Careful planning can result in a safe, relatively attractive roadway. A few guidelines are available to assist the designer in the selection of landscaping elements so as to enhance the visual quality of the roadway and at the same time result in a safe roadway.

- Avoid treatments involving fine detail. The dynamic driving environment does not permit a detailed review of the roadside elements.

- Select treatments involving vegetation native to the area. Native plants which are well adapted to the local soil and moisture conditions increase the probability of survival and reduce maintenance costs.

- Select plants which change colors with the seasons. Any treatment, regardless of how beautiful it might initially be, will lose its effectiveness with repeated exposure. Selection of plants which change colors with the seasons intermixed with evergreens can provide an interesting yet dynamic roadside.

- Match beautification treatment to driving situation. Areas of high driving task requirements should have relatively low attractiveness treatments such as grass or low ground cover. Areas of low driver concentration can use treatments of high attractiveness.

4.3.4 BICYCLE SAFETY

Bicycle transportation is just one step removed from pedestrian travel. The two modes are similar in many ways. First, they both depend on the individual for propulsion and second, they are both relatively exposed in a mixed traffic stream with motorist vehicles. The bicycle safety statistics parallel those for pedestrians but at a somewhat lower level. In 1975 there were 975 pedacyclist deaths which represents 2.2% of the total fatalities nationally. This is a disproportionally high percentage considering the difference in bicycle travel as compared to auto travel. For a more detailed study, 1975 statistics for one state are considered. Of 2,945 fatal accidents, 57 or 1.9% were pedacyclist. All pedacycle accidents including injury and non-injury incidents were only 0.7% of the total reported accidents in the state. These statistics reflect a very small proportion of the total accidents, but they are characteristically quite severe, and apparently occur at a comparatively high rate. Bicycle-miles travel data are not readily available for a positive comparison.

Some data are available on causative factors relative to bicycle accidents. Table 4.3.1 presents the distribution of 384 accidents in Santa Barbara, California. These are compared to 142 accidents in Davis (Table 4.3.2) with roughly half with and half without bike lanes (1).

A landmark study (1) to improve the quality and consistency of bicycle facility planning and design, conducted by DeLeuw, Cather and Company, was reported in 1975. The key findings of the study as reported in "Safety and Locational Criteria for Bicycle Facilities," FHWA-RD-75-112, are included. For those directly involved in the bicycle planning and design, a review of the complete report is essential. The key findings were as follows:

<u>Bicyclists Perceive Significant Benefits From Bike Lanes</u>

Surveys of bicyclist perceptions indicate that most cyclists believe streets with bike lanes are far safer than they would be without the lanes. On the average, cyclists feel that bike lanes decrease the safety hazard to nearly half what it would be were no bike lanes present on the street. Belief in the relative safety of bike lanes was expressed in a great variety of street situations from commodious suburban streets with wide lanes and no auto parking to auto-impacted urban streets with narrow bike lanes and parked cars. Some sophisticated bicyclists perceive little benefit from bike lanes, being satisfied to depend upon their own riding skills and judgment in traffic.

<u>Bike Lanes Have Positive Impact on Traffic Flow Characteristics</u>

Presence of lane delineation lines normalizes the incidence of extremely close passes and wide avoidance swerves by motor vehicles.

<u>Bike Lanes are Significantly More Effective in Reducing Bike-Motor Vehicle Collisions Than Previously Believed</u>

Recent accident studies have provided data based on accident causal factors and some direct evidence on the effectiveness of bike lanes in reducing collision incidence. These studies show that overtaking and sideswipe collisions occur far more frequently than bike lane critics alleged. Studies indicate that bike lanes are effective in reducing the incidence of a number of other bike-motor vehicle collision types.

<u>Bicycles and Motor Vehicles Should Not Be Mixed in a Single Traffic Stream Except on Streets Where Compatibility can be Achieved</u>

Research has shown that motor vehicles on streets with 25 mph speed limits will exceed

TABLE 4.3.1

BICYCLE-MOTOR VEHICLE ACCIDENTS IN SANTA BARBARA, CALIFORNIA 1971-1973

Type	Percentage
A - Cyclist Exited Driveway Into Motorist's Path	8.59
B - Motorist Exited Driveway Into Cyclist's Path	5.73
C - Cyclist Failed to Stop/Yield at Controlled Intersection	8.33
D - Cyclist Made Improper Left Turn	11.20
E - Cyclist Rode on Wrong Side of Street	14.23
F - Motorist Collided With Rear of Cyclist	4.17
G - Motorist Failed to Stop/Yield at Controlled Intersection	7.81
H - Motorist Made Improper Left Turn	12.76
I - Motorist Made Improper Right Turn	11.20
J - Motorist Opened Car Door Into Cyclist's Path	7.29

TABLE 4.3.2

ACCIDENT DISTRIBUTION COMPARISON

Accident Type	Santa Barbara (No Bike Lanes)	Davis (No Bike Lanes*)	Davis (With Bike Lanes*)	Total
A	8.59	7.89	1.45	3.95
B	5.73	3.95	2.90	2.82
C	8.33	7.89	11.59	7.91
D	11.20	5.26	14.49	7.91
E	14.32	18.42	7.25	10.73
F	4.17	7.89	1.45	3.95
G	7.81	19.74	20.29	16.38
H	12.75	15.79	28.99	18.08
I	11.20	13.16	11.59	10.17
J	7.29	0.00	0.00	0.00
Other	8.60	-	-	18.08
Total	99.99	99.99	100.00	99.99

*Percentages in these columns reflect percent of total accidents which could be classified. Accident percentages in subsequent tables are also based upon total accidents which could be classified.

90 percent of bicyclists in speed. The high incompatibility restricts the number of locations where mixing should be considered. Locations where conditions satisfactory for mixing may occur include streets where motor vehicle speeds are constrained on long downgrades, on approaches to intersections and on lightly traveled streets.

Bidirectional Facilities are Strongly Discouraged

Accident data reveal that riding against traffic is a primary cause of bicycle-motor vehicle accidents. Studies have shown that single direction facilities are most effective from a safety standpoint.

Inconvenient or Indirect Routing is the Primary Reason Given for Non-Use of a Bicycle Facility

Bicycle facilities must connect logical bicycle trip origins with their destination conveniently and directly. Facilities which fail to provide convenient and direct service will simply not be used unless they afford significant recreational benefits.

No Single "Design Cyclist" Can be Identified as a Basis for Design

There is a tremendous range of bicyclist physiological capabilities, bicycling judgment and skill, and trip purposes. Hence, the planner should consider the full range of cyclist types expected to use a facility or, in response to specific planning policy, may tailor the design to the needs and capabilities of a specific user group.

Six Levels of Service for Bicycle Operation can be Defined

These service levels are related to similar levels defined in the Highway Capacity Manual and describe a quality of bicycle flow. Specific speed, volumes, and densities have been ascribed to these service levels.

Specific Bicycle Facility Width Criteria can be Established

Research has established minimum bicycle separation distances which can be used to define lane widths. The recommendations in this report should replace the multitude of conflicting width specifications which have been promulgated previously by various authorities.

Equations for Sight and Stopping Distance for Bicycle Design Have Been Prepared

These equations are of particular use in facility design--particularly locations where bicycles interfere with high speed traffic.

Direct Consideration of Bicycle Traffic Volumes in Warrants for Traffic Control Devices Appears Appropriate

"Recent research has included a major effort to evaluate the role of bicycles in traffic control warrants, and a user's manual including specific warrants has been suggested. Additional research and testing in this area appear desirable."

Fifteen Criteria Measures Have Been Identified Which Should be Considered in the Bikeway Location Planning Process

Principal user-related criteria include potential use, basic width, connectivity, safety, grades, and barriers. Secondary user-related criteria include attractiveness of the bicycling environment, image ability, air quality, surface quality, and truck traffic intensity. Non-user-related criteria include cost and funding, competing use and security. All of these criteria must be considered, to varying degrees, depending on site circumstances and policy objectives. It is particularly important that user travel needs be kept uppermost in weighing the constraints of conflicting criteria categories.

Design and Location Techniques and Criteria Should be Widely Disseminated to Planners and Technicians Active in the Field

The infant state-of-the-art has resulted in many local agencies doing their own "pioneering" with the result that many mistakes have been duplicated and little standardization has taken place in location, design, and graphics. Dissemination of material in this manual and future research efforts, similar to that enjoyed by the Manual on Uniform Traffic Control Devices and the Highway Capacity Manual, will greatly aid professionals in this field.

4.3.41 Bicycle Planning

Bicycle planning is a new activity in many of the urban areas. In some respects, bicycle planning is similar to the planning of the street network. However, there are some major points that should be reviewed briefly.

Most bicycle riding activities may be classified into the following categories:

- Neighborhood riding is done mostly by young children. Except for school trips, riding is often purposeless and is not limited to a specified route.

- Recreation riding is a leisure time activity for all ages. Routes should be considered which provide a minimum of conflict with vehicular traffic. Aesthetics

are important, and attention should be given to providing pleasing visual impressions whenever possible.

- Commute riding is increasing due mainly to parking difficulties, energy conservation, and physical fitness reasons. Routes of this type should be as direct as possible between work and living areas. Aesthetics becomes less important but the need for parking facilities at the work trip end increases.

- Sport riding and touring with sophisticated, lightweight bicycles usually requires facilities built for higher speeds and longer trips. Facilities of this type may not be entirely compatible with those for a more leisurely type of riding.

Various types of facilities may be combined to form the bicycle network much as local, collector, and arterial streets form the urban street network. Various methods of describing these facility types exist; however, the following appear to be the most common (5).

- Class I (Bike Path or Protected Lane)--A completely separated right-of-way designated for the exclusive use of bicycles.

- Class II (Bike Lane)--A restricted right-of-way designated for the exclusive or semi-exclusive use of bicycles; through motor vehicles are not permitted. Vehicle parking and access to property as well as pedestrian access to parked vehicles are allowed.

- Class III (Bike Routes)--A shared right-of-way designated as such by signs placed on vertical posts or stenciled on the pavement. Classes II and III are not appropriate for freeway application.

Bicycle Network Planning. Having established the types of trips to be accommodated and the various kinds of bicycle facilities, the following network planning procedure can be considered for the development of a comprehensive bicycle plan.

Step 1. Conduct inventories of existing facilities. Determine traffic volumes, speeds, and parking conditions on street facilities, and physical dimensions of the street. Explore the availability of semi-private and municipal rights-of-way such as utility and abandoned railroad rights-of-way and areas around lakes and reservoirs.

Step 2. Forecast demand for bicycle facilities. Conduct origin-destination studies or in-home interviews as to bicycle usage according to number of riders in family, number of bicycles, and number and type of trips. Based on areas with similar socio-economic characteristics, project trips between these zones as well as intra-zonal travel. Consider amount of generated traffic by addition of a new facility.

Step 3. Establish planning and design standards.

Step 4. Design bikeway network and facilities.

Step 5. Prepare alternative plans where more than one alternative exists.

Step 6. Evaluate plans. Utilize governmental personnel responsible for city planning, traffic operations, street maintenance and transit, and parking management for review. Involve citizens from bicycling, public service, environmental and other interested groups.

Step 7. Select final plan.

Step 8. Implement plan.

Step 9. Evaluate results. Evaluate use and operations of constructed and marked bicycle facilities as input for future bicycle planning.

The comprehensive bicycle plan should offer similar service to all bicyclists within the confines of the planning area. In addition, the plan should provide continuous routes connecting the smaller community bicycle systems. This is the rationale behind the "honeycomb" system as suggested by the City of Dallas in which Class I Bicycle Paths on exclusive right-of-way for the honeycomb, providing a network for travel between individual zones as well as around the zone. Within each zone, Class II Bicycle Lanes on city streets provide for movement within the zone and outward to the Class I facilities. Class III Bicycle Routes in turn provide access to higher type facilities.

Potential for Reducing Demand. A potential bicycle market for urban areas can be estimated based on the data presented by Everett (6). Everett's data indicate that the commuter bicycle trip is feasible up to six miles one way. Thus, assuming that there is a total demand of 100,000 trips per day in an urban corridor, that 40 percent of the people would be willing to divert to the bicycle, and further that the peak hour is 10 percent of the ADT, the potential peak hour bicycle demand is:

$$(100{,}000)(.4)(.4)(.1) = 1600 \text{ trips}$$

This represents about one freeway lane or 50 loaded buses or 22-75 passenger transit vehicles. Thus, the potential impact of the bicycle on the transportation planning process could be rather significant.

However, bicycle riding is a fair weather activity for most persons. During periods of heavy rain, wind, snow or other adverse

weather conditions, bicycle riding will be reduced substantially. Since the transportation plan is usually based on the tenth highest hourly loading during the year, there are relatively few areas of the country where year around cycling exists. For this reason, the impact of the bicycle on the need for other transportation modes will be nil except in a very few special instances. There is a need for bicycle facilities, but the reduction in vehicular traffic demand will, in most cases, be "icing on the cake" rather than having the effect of reducing the need for other transportation facilities. For this reason, care must be exercised to insure that the vehicular capacity of existing streets not be reduced by the proposed bicycle facilities.

The fact that bicycle use does not reduce the demand for other transportation facilities should not be misconstrued to mean that there is not a real need for bicycle facilities. The increased use of bicycles combined with efforts to get more vehicular capacity from existing streets will undoubtedly lead to greater vehicular-bicycle conflicts and accidents. Additionally, bicycle traffic mixed with motor vehicle traffic impedes flow and reduces capacity.

The transportation manager should include bicycle planning as a part of the overall transportation system.

4.3.42 Bicycle Facility Design

Design Guidelines. The FHWA publication "Bikeways - State of the Art - 1974" presents a detailed discussion of design requirements of all types of facilities. This publication should be available to every agency as a reference document for planning and design of bicycle facilities. Further, the agency should develop its own design standards for bicycle facilities, similar in many respects to city street design standards. Major items to be included in bicycle facility design standards are:

- Design speed
- Horizontal curve controls
- Minimum facility cross sections
- Maximum grades
- Lateral and vertical clearances
- Intersection layouts
- Grade separations

Intersection Design. Regardless of the type of bicycle facility, conflict with vehicular traffic is inevitable at intersections. Turning bicyclists must cross vehicle paths, and turning vehicles must cross bicycle paths. There are two general approaches to partially resolving problems at intersections.

- Channel the bike lane off the street so that it operates as a parallel or adjacent intersection.

- Terminate the protected bike lane immediately prior to the intersection, placing bicycles back into the traffic stream.

- For a more detailed treatment, refer to "Bikeways - State of the Art - 1974" by FHWA. Also, illustrations of other intersection treatments are shown in Figure 4.3.6.

Grade Separations. Where vehicle volumes prevent at-grade crossing by bicycles, grade separations may be constructed. Underpasses are preferred somewhat due to lower vertical clearances for bicycles than for vehicles. Also, the down-grade approach to the underpass allows the bicyclist to gain momentum in order to carry him up the other side. Underpasses, however, need to be well-lighted and should provide line-of-sight throughout, if possible.

4.3.43 Operation and Control of Bicycle Facilities

The operational controls for bicycle facilities serve two fundamental purposes:

- To delineate the bicycle facility
- To insure the safety of the cyclist

For on-street bicycle facilities, the regulatory devices for vehicular control will generally serve for cyclists as well. The primary control requirement is therefore one of delineation of the bicycle lane. For separate bicycle facilities, the safety of the cyclist is the primary concern. Warning signs and intersection right-of-way control devices are the principal controls.

4.3.44 Bicycle Plan Review

The following planning and design points are suggested for special attention in the review of bicycle plans.

Planning

- System Continuity. Considered here should be whether or not the system is indeed a comprehensive system providing for a variety of trip lengths and purposes or whether those improvements being made will only benefit certain areas.

- Implementation Scheduling. All of a proposed bicycle system cannot be constructed or implemented immediately. Where possible, those elements that will benefit the most people first should be given precedence. For example, building a bike path around a lake should probably be done after bicycle access facilities are provided to the area.

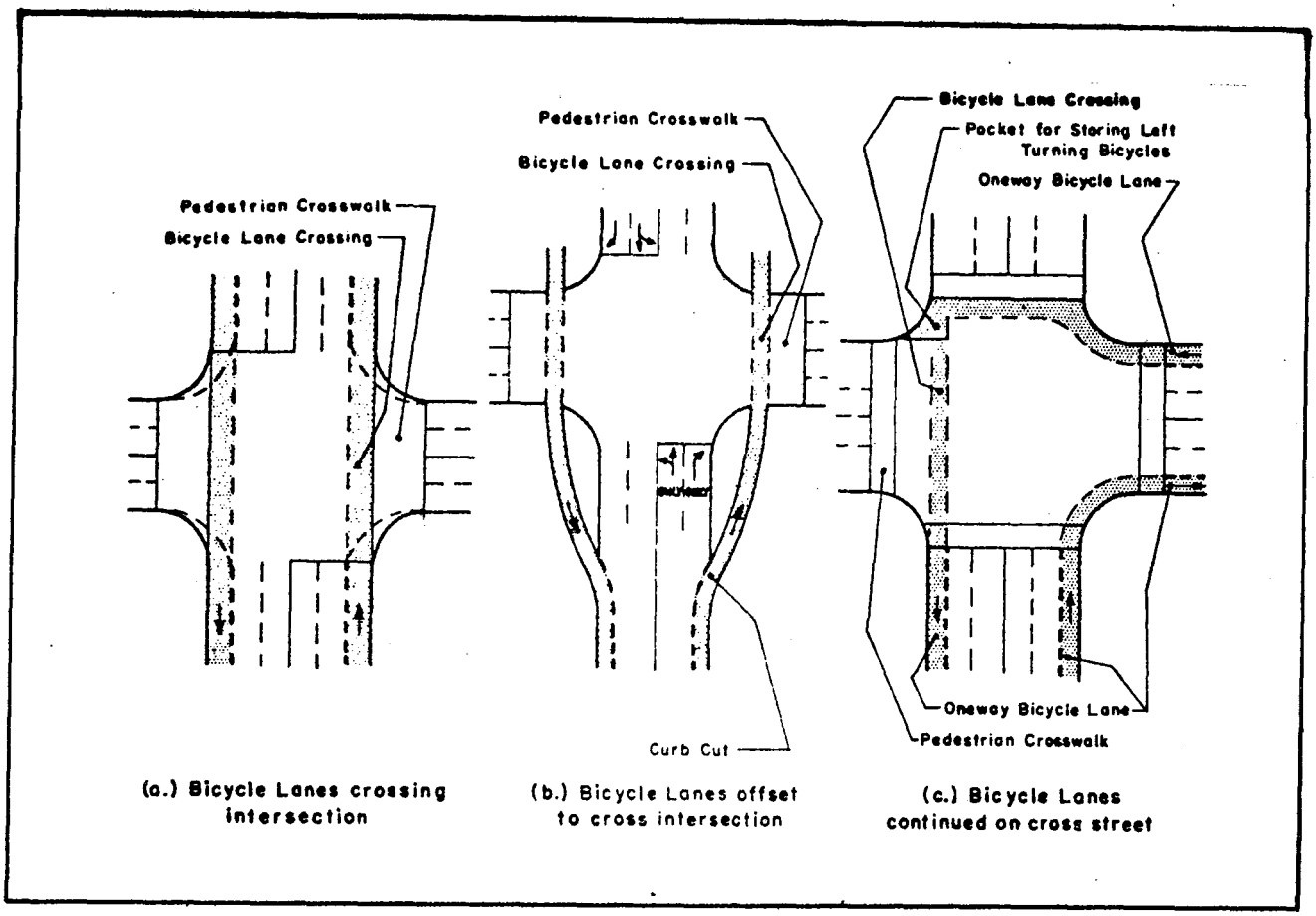


Figure 4.3.1 Typical Bicycle Channelization Arrangements At Street Intersections

- Effects of Vehicular Capacity on Facilities. Either Class II Bike Lanes or Class III Bike Routes reduce the effective street width usable by vehicular traffic. Care should be taken in reviewing bicycle plans to retain sufficient capacity of streets within a given corridor.

Design

- Lane Width and Lane Location for Class II Bike Lane facilities should be carefully reviewed. Lane width must be adequate to prevent bicycles from encroaching upon the traveled roadway. This is especially important in the event that two-way bike lanes must be used. If street width is not sufficient to allow adequate bicycle lanes, route relocation should be considered.

- Grades on Class I Bike Path facilities which parallel street facilities should be less than those on the street facility to encourage use of the bike path. Not only the steepness, but also the length of grades, is important.

Operation

The bicycle plan review should include an operational analysis of the plan with respect to safety of the cyclists. Bicycle facility sections wich are not open to view of passing motorists may be an invitation to criminal acts and should also be considered in the plan review.

4.3.45 Cost of Bicycle Facilities

The cost of bicycle facilities varies dramatically with the type of facility. The cost data presented below in Table 4.3.3 were obtained from the City of Dallas and reflect 1974 costs. Approximately half of the maintenance cost is for security surveillance.

4.3.46 Funding of Bicycle Facilities

The following quote comes from the publication, "The 1973 Federal-Aid Highway Act -- An Analysis."

Bicycle and Pedestrian Facilities--The States may build separate or preferential bicycle facilities and walkways in conjunction with the non-interstate federal aid highway projects and will be financed from funds for the system on which the facility is located. The Federal share of the cost of such projects will be 70 percent, but not more than $2 million per state and $40

TABLE 4.3.3

BICYCLE FACILITY COST PER MILE

Type of Facility	Initial Cost (1975 Base) Excluding Land	Annual Maintenance Costs
Bikeways	$30,000+	$1,500 (Asphalt) $ 950 (Concrere)
Protected Bicycle Lane On Street	$ 6,000	$600
Unprotected Bicycle Lane On Street	$ 3,000	$300
Bike Route	$ 700	$100

million per fiscal year may be used for them. Funds authorized for other federal road programs, e.g., forest and public lands, highways, parkways, etc., may also be used for this purpose.

In addition, Interstate funds can be used for bicycle facilities which are included in new construction. Federal funds for bicycle and pedestrian facilities are limited to $2½ million per year for each state. The non-Interstate funds can be used for 70 percent of the cost with the remaining 30 percent coming from local sources.

All bicycle facilities must meet the following criteria:

- The safety of the bicyclist, pedestrians, or vehicular traffic must not be impaired;
- The proposed facility must be a part of and connect to elements of the existing or planned system;
- The facility must be under the jurisdiction of a public agency; and
- There must be sufficient existing or projected demand to render the proposed facility cost-effective.

4.3.5 PEDESTRIAN SAFETY

Pedestrian travel is the most fundamental transportation mode--virtually every trip begins and ends as a pedestrian movement. Without the protection of a mass of metal surrounding the individual to increase the momentum and serve as armor, the pedestrian is at a great disadvantage. Herein lies the basic safety problem. Just how serious is the safety problem?

Even a cursory review of accident statistics shows that the pedestrian safety problem is very significant. Using recent nationwide statistics, the following significant points are made:

- 20% of all urban motor vehicle accidents involved pedestrians.
- 30% of all urban fatal accidents involved pedestrians.
- 25% of fatal pedestrian accidents involved children 14 or under.
- 25% of fatal pedestrian accidents involved adults 65 or over.
- 70% of all pedestrian accidents occurred while pedestrians were crossing the street.
- 40% of all pedestrian accidents occurred while pedestrians were crossing at the intersection.

From these statistics, it is obvious that pedestrian safety is a major problem. Further, the problem relates to the young and the old, and to pedestrian errors as well as driver errors.

The pedestrian mode is a significant part of the urban transportation system, and there are pedestrian problems that must be solved in order to achieve and maintain a viable urban transportation system. These problems relate to the planning, design and operations aspects of transportation. Safety must be a principal criterion, but like other modes, the transportation manager must consider the general service aspects of pedestrian facilities in satisfying public need.

All things considered, we must view the pedestrian problem in the traditional sense; that is:

- Enforcement
- Education
- Engineering
  - + Planning
  - + Design
  - + Operation

4.3.51 Safety Design of Pedestrian Facilities

The safety design of pedestrian facilities will be discussed in several areas: subdivision standards; city street design practices; and CBD treatments.

4.3.52 Subdivision Standards

Fundamental pedestrian safety begins with the layout of the subdivision. It is at this point that we establish an evironment which is hostile or conductive to the safety and efficiency of pedestrian travel. Some of the features that deserve attention in subdivision planning and design are discussed.

Sidewalks. Contrary to practice in recent years, sidewalks should be required in all residential areas, and on collectors and arterials when pedestrian movements are not restricted. These may be combined with provisions for bicycles where the numbers of either are not so excessive that they are totally incompatible. In residential areas, sidewalks should be placed on both sides of the street. A minimum width of 4 feet is common (2). There are pros and cons relative to the location of the sidewalks in the border area. Some cities place the sidewalk immediately adjacent to the curb, but this results in difficult construction at the intersection of driveways. Most cities place the sidewalk approximately one foot from the property line.

Pedestrian Easements. In the new concepts of subdivision layout, long blocks, cul-de-sacs and discontinuous local streets are recommended to deter vehicular movement. These same practices tend to discourage pedestrian and bicycle movements. With adequate planning of the pedestrian circulation system and with the provision of easements through long blocks and cul-de-sacs, pedestrian flow can be increased and vehicular traffic reduced.

At least one city has established a regulation requiring easements where block lengths exceed 600 feet. An argument against pedestrian easements presents two major points: Noise and security problems created by pedestrians walking alongside private property, and the question of who maintains the easement area. These should be recognized and dealt with satisfactorily in formulating a regulatory policy.

Pedestrian Circulation System. Pedestrian systems should be planned to connect with neighborhood shopping, recreational and public transit facilities. This is accomplished through the rational or practical arrangement of streets and easements. Street layouts can be developed to provide maximum pedestrian circulation while minimizing the need for easements.

Residential Neighborhood Intersections. For vehicular traffic, the T-intersection has proven to be safer and operationally more efficient because of the inherent natural control feature and the fewer number of conflicting paths. For these same reasons, T-intersections are preferred for pedestrian safety.

Location of Pedestrian Generators. Care should be exercised in the location of facilities that attract pedestrian activity. For example, schools, parks, shopping, etc., are typically located for vehicle access. In doing so, we frequently create maximum conflict between those accessing such facilities by foot and by vehicle. Locational planning can likewise be used to minimize this conflict. At least crossings can be controlled through location to the extent that pedestrian access may occur naturally at controlled intersections rather than at mid-block and other uncontrolled points.

4.3.53 Pedestrians on Arterial Streets

The vehicle-pedestrian conflict on arterial streets is especially critical because of the moderate speeds and high traffic volumes on the arterial. There are a number of design and operation features that should be considered in the design and operations procedures to increase the safety of pedestrians.

Sidewalks. Sidewalks should be provided along arterials regardless of the relatively low number of pedestrians. Otherwise, pedestrians are forced into the street. Sidewalk dimensions may be tailored to need. Also, placement may help to eliminate maintenance of grass areas.

Refuge Islands. On wide streets, it is desirable to design medians such that they serve the function of pedestrian refuge. Also, where intersections result in extremely long pedestrian paths, islands may be installed to channel traffic, reduce the exposure of pedestrians to traffic and give pedestrians an opportunity to cross one stream of traffic at a time.

System Continuity. The same concern for system continuity should be exercised for the pedestrian system as for the vehicle system. In fact, even greater consideration may be given to the directness of route because pedestrians tend to operate in that manner whether intended or not. Sidewalks should be designed to approach the crosswalk. Particularly in construction areas pedestrians are channeled into the street for short distances without any protection considerations.

Pedestrian Barriers. Pedestrian barriers should be used more frequently to prevent or deter pedestrians from entering or crossing the street at undesirable locations. It should be recognized, however, that there is a difference between deterrant barriers and prohibitive barriers, and they should be used according to the intent or relative hazard involved. For example, prohibitive barriers should be used on freeways, and rail-type deterrant barriers may be used to block mid-block crossing points on arterial streets.

4.3.54 Pedestrian Grade Separations

One approach to improving pedestrian safety is to physically separate pedestrians and vehicles. This may be done in horizontal arrangements and vertical arrangements. Horizontal systems will be discussed later.

Grade separations, either over or under the roadway, are justified or warranted on the basis of perhaps three principal conditions.

- Policy considerations, e.g., the Interstate system
- Hazardous speed conditions
- Combined vehicle and pedestrian demand exceeds capacity of the facility

On arterials and freeways, grade separations are typically elevated; however, they should be placed above or below grade based on full consideration of the relative merits. Elevated facilities are easier to police and generally pose fewer maintenance problems. Underpasses pose serious enforcement problems when there is not a direct line of sight through the facility, and a general open atmosphere. On arterial streets, underpasses should be limited to situations where a split grade may be utilized--not more than half the height of the pedestrian way should be below grade.

Elevated facilities could be made more effective by providing alternate methods of climbing to the crossing. The accommodations for wheelchairs and bicycles, if permitted, require long grades which are folded or built in a circular form. The added walking distance is objectionable to those who are able to climb stairs and, therefore, it would be desirable to provide a direct stairway as an alternate access method.

There are numerous advantages and disadvantages of elevated and below-grade pedestrian facilities. These are summarized in Figures 4.3.2 and 4.3.3 (3).

4.3.55 Horizontal Separation Systems

Aside from the typical sidewalk system in suburban or outlying areas previously discussed in "Integrated Systems," horizontal separation embraces the concept of separating pedestrians and vehicles in the same plane. This generally constitutes designation of certain areas for exclusive use by pedestrians. Some of the alternatives for horizontal separation are as follows:

Sidewalk Widening. Sidewalks in downtown areas may be widened by the elimination of parking and the utilization of this space for sidewalks, as illustrated in Figure 4.3.4. Such projects can range in magnitude from simply the clinical treatment of reducing the street width, to serpentine treatment with variable widths to facilitate mid-block passenger loading and intersection turn lanes, to the placement of plantings and other decorative treatments to serve as a buffer to traffic. Widening in this manner reduces the confusion, and improves the safety and appearance of the street caused by the congestion resulting from curb parking. The advantages and disadvantages of sidewalk widening are listed in Figure 4.3.4.

Arcade Setbacks. In new construction, or where old construction is being remodeled, the building can be recessed to create additional pedestrian space as shown in Figure 4.3.5. This provides the advantage of sidewalk widening while maintaining street width. It also provides partial cover from the elements. The advantages and disadvantages of arcade setbacks are presented in Figure 4.3.5.

Partial Malls. According to most references, it is difficult to distinguish between the high-quality sidewalk widening project and the partial mall. Perhaps this differentiation should be on the basis of traffic restrictions. For example, Nicollet Mall in Minneapolis limits the vehicular intrusion to buses, taxicabs and emergency vehicles. Whereas, the street was at one time a 4-lane street, it is now limited to two lanes. Cross street traffic on the various intersecting streets is not restricted, but the elimination of turns at the intersections greatly improves pedestrian operations.

Full Malls. Full malls are typified by the exclusion of all vehicular traffic except emergency vehicles. A schematic illustration of the full mall, along with the advantages and disadvantages, are presented in Figure 4.3.6. They provide the opportunity for a full aesthetic treatment which may serve as a stimulus for the urban area. Development is generally funded substantially by the business sector. In the application of the full mall, the major concern of the transportation manager is the integration of the mall with the remainder of the transportation system. Transit, vehicular movement and parking are necessary for the support of the pedestrian system.

Auto-Free Zones. The auto-free zone is principally an extension of the full mall concept where automobile traffic is restricted to give pedestrians exclusive use of an area comprised of multiple street segments. The auto-free zone has the greatest impact on vehicular travel because it concentrates movement, access and parking on the periphery of the area. Because of the problems encountered in servicing the area, it is generally realistic to permit the operation of buses, emergency vehicles and service vehicles in the auto-free zone. In this respect, it is quite similar to the Nicollet

ADVANTAGES

- Separates pedestrian movement from vehicular movement
- Can provide more direct, convenient paths for pedestrians
- Provide elevated visual vantage point
- Provide direct linkage of major activity centers
- Can be built in increments and expanded into comprehensive system
- Particularly applicable to new construction
- May utilize public rights-of-way linking and/or passing through existing buildings
- Allows more compact and efficient arrangement of retailing space
- Improves at-grade vehicular circulation
- Provides cover for at-grade pedestrian movement

DISADVANTAGES

- Expensive to construct
- Requires change-in-grade and numerous entry points
- Difficult and expensive to provide access into existing development
- Could diminish retail activity at the street level
- Coordination of property owners may be difficult to achieve
- Elevated elements form areas at-grade that present security problems
- Difficult to coordinate to at-grade and below-grade transit systems
- Creates potential danger of falling objects if not totally enclosed
- Adds to the already cluttered cityscape
- Difficult to service for emergency, fire, security, etc.

Figure 4.3.2 Advantages and Disadvantages of Above-Grade Systems

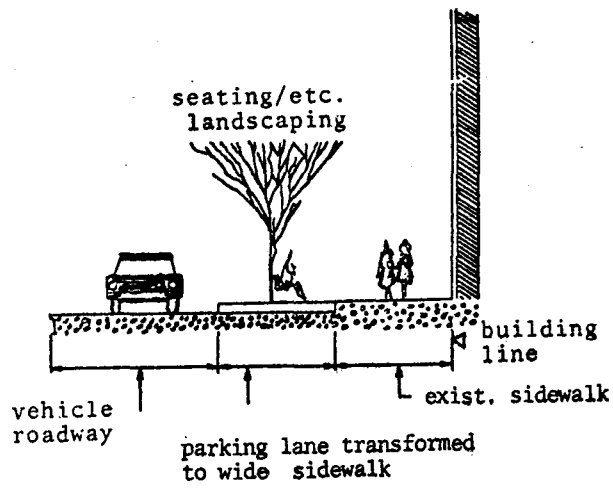
ADVANTAGES

- Separates pedestrian movement from vehicular movement
- Provides built-in protection from sun and inclement weather
- Does not have to follow traditional parallel grid pattern
- Does not visually or physically obstruct the urban landscape
- Can be built in increments
- Particularly applicable to new construction
- Can be linked directly to existing underground systems
- Provide direct linkage between major activity centers
- Improves vehicular circulation at grade

DISADVANTAGES

- Extremely expensive to construct
- Require change-in-grade and numerous entry points
- Difficult to link new and old buildings
- Orientation and coherence are adversely affected due to loss of visual contact with city
- Artificially created environment
- High potential for crime
- Emergency servicing is restricted

Figure 4.3.3 Advantages and Disadvantages of Below-Grade Systems



SECTION

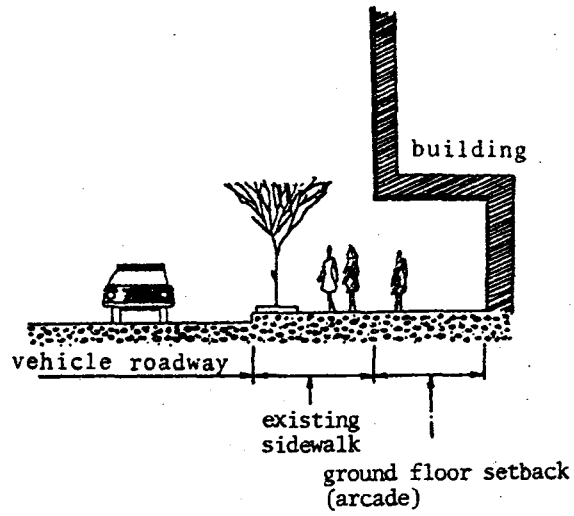
ADVANTAGES

- Increased sidewalk space relieves pedestrian congestion in areas of high volume
- Additional buffer zone reduces potential for conflict and accident
- Annoyance of noise and fumes reduced
- Visual obstruction of parked autos eliminated
- Increases space for pedestrian amenities

DISADVANTAGES

- Reduces width of street available to vehicle
- Increases vehicle congestion on surrounding streets
- Does not solve the problem of conflict at intersections
- Pedestrian exposure to weather is not affected

Figure 4.3.4 Advantages and Disadvantages of Sidewalk Widening



SECTION

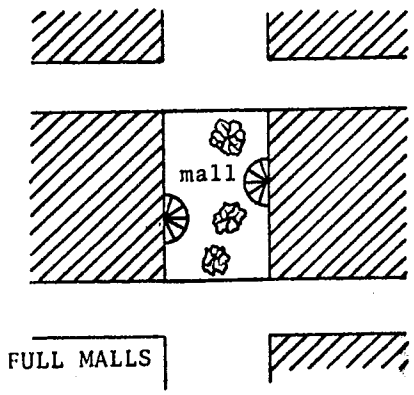
ADVANTAGES

- Relief of pedestrian congestion
- Buffer zone reduces potential for conflict and accident
- Reduced annoyance from fumes and noise
- Increased space for pedestrian amenities
- Some shelter from sun and inclement weather
- Does not reduce vehicle space

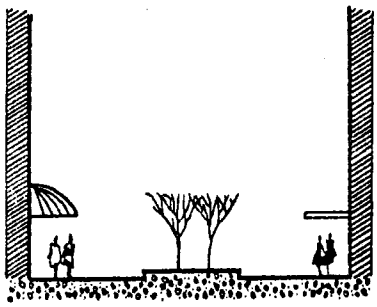
DISADVANTAGES

- Does not solve the problem of conflict at intersections
- Depends on cooperation of builders, developers and other private interests
- Reduces store frontage and retail sales space

Figure 4.3.5 Arcade Setbacks



PLAN



FULL MALL (URBAN STREET)

SECTION

ADVANTAGES

- Eliminates conflict within mall area
- May be integrated with public transit
- Allows use of people-movers, jitneys, etc.
- Can be developed in stages
- Allows a wide range of communal activities (art fairs, craft shows, entertainment, etc.)
- Can integrate with existing parks, plazas, etc. to create "system" of urban open space
- Stimulates retail activity
- Provides freedom from noise, fumes, and usual obstruction of vehicles
- Eliminates on-street servicing of stores

DISADVANTAGES

- High development, operating and maintenance costs
- Requires comprehensive preplanning
- Increases traffic volumes on surrounding streets
- Depends on total cooperation of property owners and other retail interests
- Acts to reduce retail activity on nearby streets
- Creates legal problems with property lines, etc.
- May require extensive utility upgrading

Figure 4.3,6 Full Malls (Urban Streets)

Mall in Minneapolis.

4.3.56 Pedestrian Spot Improvements

Signal Timing. Pedestrian signal displays may be installed, and signals may be timed more efficiently for pedestrian operations. Also, the specialized phasing, the "all-red" interval, may be used to increase the efficiency of pedestrian flow and increase safety through the elimination of pedestrian-vehicle conflicts.

Removal of Obstacles. One of the major problems in high-density urban areas is the normal "clutter" that reduces the capacity and serviceability of pedestrian facilities. This "clutter" consists of functional objects such as newspaper dispencers, mailboxes, litter cans, fire plugs, sign posts, light posts, signal posts and other forms of street furniture. Even though these may be desirable or necessary items, their application or use may be regulated so as to reduce the interference with the pedestrian flow network.

Widening Crosswalks. In the pedestrian system, as in the vehicular system, the intersection is the major capacity-limiting feature. For vehicular flow, intersection widening has proven to be an effective means of increasing street capacity. In the same manner, widening crosswalks may increase the capacity of the pedestrian system.

Pedestrian Lighting. Although lighting may not increase the capacity of pedestrian facilities, its value in the safety and security of pedestrians is obvious. Lighting in other areas such as pedestrian connections to recreational areas, shopping centers and other pedestrian generators permit drivers to see and avoid passengers.

Regarding the application of lighting for the safety protection of pedestrians, one should refer to FHWA Report No. FHWA-RF-76-9, "Fixed Illumination for Pedestrian Protection--Users Manual."

A study in Kentucky (4) listed a number of countermeasures that have been used successfully in reducing the potential for pedestrian accidents. They are:

1. Prohibition of vehicle parking
2. Designation of one-way streets
3. Improvements in overhead street lighting
4. Use of crosswalks
5. Installation of pedestrian signals
6. Use of pedestrian barriers
7. Prohibition of pedestrians (on Interstate highways)
8. Improvements in driver regulations
9. Installation of pedestrian refuge islands
10. Use of reflectorized apparel for pedestrians
11. Installation of special pedestrian signing and markings
12. Widening of shoulders (in rural areas)
13. Installation of sidewalks
14. Grade separation of crossings
15. Construction of pedestrian malls
16. Construction of playgrounds (in urban areas)
17. Conduct of pedestrian education programs
18. Increased enforcement of pedestrian and driver regulations

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TOPIC 4 SESSION 4
ROADWAY LIGHTING SYSTEMS

Objectives:

The participant should be able to:

1. Locate the source documents and determine the application of roadway lighting as a means of improving the safety and operational efficiency of traffic facilities.

2. Select light sources and specify design controls for the achievement of visibility, safety and efficient energy utilization, and

3. Select the type and location luminaire supports to minimize the potential and severity of vehicular collisions with the supports.

4.4.1 INTRODUCTION

Safety is one of the ultimate objectives in roadway lighting. When road users can see better, they presumably operate more efficiently and more safely. Lighting serves all road users - pedestrians and operators of bicycles, motorcycles, automobiles, buses and trucks. It provides night visibility of the roadway, of traffic, and development conditions along the way. It aids some in seeing others, such as drivers seeing pedestrians and bicyclists. From the safety standpoint, none is necessarily more important than the other, except in certain circumstances, drivers greatly outnumber pedestrians and bicyclists; therefore, we will frequently refer to the principal function of lighting as that of providing driver visibility.

The most common method of illuminating the highway is by vehicle headlights. Virtually all of our rural highways and many of our city streets must be, by economic necessity if nothing else, lighted by vehicle headlights. Vehicle headlighting certainly has its limitations, and these limitations have become more critical as vehicle speeds have increased. This has been offset greatly by the effective use of reflective marking and delineation materials. These are used to aid the driver in viewing geometric conditions and physical obstacles beyond the illumination range of the vehicle headlights.

Fixed source roadway lighting becomes important when the vehicle headlights, supplemented by markings and delineation, no longer provide sufficient driver visibility. This normally occurs in urban areas where traffic conditions are more intense, roadways are more complex, and these factors are aggravated by the influence of development along the roadways. Thus, we have identified the factors which justify lighting and, subsequently, we will consider some of the design requirements and characteristics.

4.4.2 WARRANTS FOR ROADWAY LIGHTING

Warrants are conditions that justify the installation of fixed source roadway lighting. Generally, four conditions are embodied in the published warrants:

- Geometric conditions
- Traffic operating conditions
- Environmental or developmental conditions
- Accident experience

Currently, there are two published sources for lighting warrants:

- An Informational Guide for Roadway Lighting, AASHTO, 1976
- Warrants for Roadway Lighting, NCHRP Report 152

The Informational Guide is the "official" warrant source for highway agencies, particularly Federal-aid projects. These warrants are based principally on experience related to roadway classification, traffic volume, traffic maneuvers, interchange spacing, night accident rates and environmental conditions. Unfortunately, AASHTO has addressed warrant applications primarily to freeway and expressway type facilities. Urban arterial streets are treated very superficially and, in effect, little or no direction concerning warrants or guidelines has been given to local agencies.

Warrants presented in the NCHRP report are based on an analytical approach which provides a priority scheme as well as warrants. Using this method the designer is able to determine a numerical score relative to the need for roadway lighting based on geometric, operational, and environmental conditions, as well as accident experience. For a detailed treatment of either of these methods, one should refer to the appropriate document.

4.4.3 TYPES OF ROADWAY LIGHTING SYSTEMS

Types of roadway lighting systems may be described in several ways; however, in terms of safety, the various types of systems required to satisfy the driver visibility needs seem most appropriate. Thus, the following types are presented:

- Continuous Lighting Systems. This refers to the use of uniformly-spaced luminaires along the roadway to illuminate the roadway and the adjacent areas. Continuous systems are used wherever there is a continuing deficiency in headlights providing sufficient driver information. Continuous systems may be used to light freeways, arterials, collectors and local streets. Depending on the character of the facility, continuous systems may be on masts 30 to 50 ft (9 to 15 m) high mounted in the median where applicable; or they may be along the side of the street in a oneside, staggered or opposite arrangement, depending on the width of the street. Also, high-mast lighting, 80 to 150 ft (24 to 46 m) high, may be used in continuous systems on wide, complex freeway sections.

- Partial Lighting Systems. Partial lighting refers to the installation of luminaires at critical points along a roadway, at an intersection, or in an interchange area. The luminaires are generally intended to increase the visibility and target value of critical geometry and operational features. As an example, Figure 4.4.1 shows the application of partial lighting to a freeway interchange. Partial lighting, sometimes referred to as "safety lighting" has been used more commonly in the lighting of interchanges and major intersections in rural or suburban areas.

- High-mast Interchange Lighting. A specialized form of illumination known as high-mast lighting is frequently used to light freeway interchange areas. Because the driver generally needs to see the various elements of the interchange in proper perspective, an area lighting concept aids driver visibility. Masts 80 to 150 ft (24 to 46 m) high and spaced strategically in the interchange are used to support several luminaires which light the entire area.

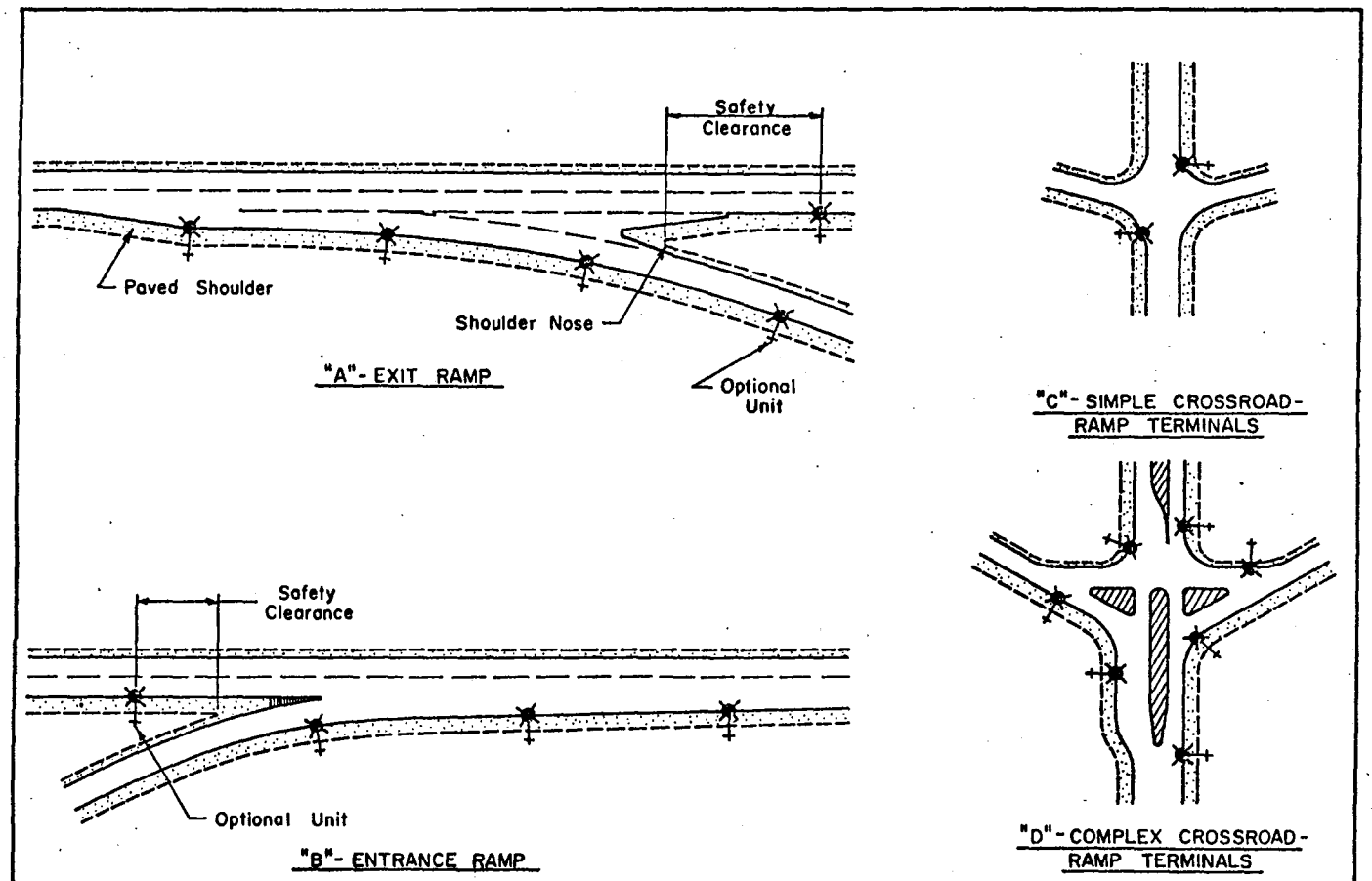

Figure 4.4.1 Partial Lighting for Interchanges

4.4.4 TYPES OF LIGHT SOURCES

The most important element of the illumination system is the light source. It is the principal determinant of the visual quality, economy, efficiency, and energy conservation aspects of the illumination system. Although there are many types of light sources available, major emphasis will be placed on the characteristics of those sources that are currently used for public lighting.

Light sources that are normally used for roadway lighting today are gaseous discharge lamps. Light is produced in this type lamp by passing an electric current through a gaseous medium. The characteristics of the gaseous medium determine the color of the light and the efficacy of light output. For example, a mercury vapor gas produces a blue-white light at approximately 55 lumens per watt, and a sodium vapor gas produces a yellow monochromatic light at approximately 175 lumens per watt.

There are four types of high-intensity discharge lamps used today:

- Mercury
- Metal halide
- High pressure sodium
- Low pressure sodium

4.4.41 Mercury Vapor

The mercury vapor lamp was invented in the 1930's and with development through the years has become the most widely used street lighting source. The principle of operation of the mercury lamp is representative of all gaseous discharge lamps. The characteristics of the gaseous medium determine the color of the light and the efficacy of light output.

Most mercury lamps are constructed with two envelopes, an inner envelope (arc tube) that contains the gaseous medium, and an outer envelope that shields the arc tube from outside drafts and changes in temperature. The inner envelope usually contains nitrogen which prevents oxidation of internal parts. In reflector lamps the outer envelope, by means of a metallic-reflecting coating applied to its inner surface, serves also to direct the light into a beam. Semi-reflector lamps have a portion of the bulb covered with a phosphor coating. The essential construction details shown in Figure 4.4.2 are typical of lamps with fused arc tubes within an outer envelope.

The most common form of mercury lamp is the clear bulb. The clear mercury lamp produces a bluish-white light at efficacies of 50-65 lumens per watt. While the light source itself appears to be bluish-white, there is an absence of red radiation, especially in the low and medium pressure lamps, and most colored objects appear distorted in color rendition. Blue, green and yellow colors in objects are emphasized; orange and red colors appear brownish.

The poor color rendition led to the development of phosphor-coated mercury lamps with efficacies slightly greater than the clear bulbs. Although the phosphor coating corrects the brownish appearance of orange and red colors, it creates a problem with control of the emitted light.

The light is emitted from the arc which, in turn, excites the phosphors that coat the inside of the envelope. Light produced by the phosphors is more difficult to control than light produced from the point source of the clear bulb. For this reason, clear mercury vapor lamps are more widely used for roadway lighting.

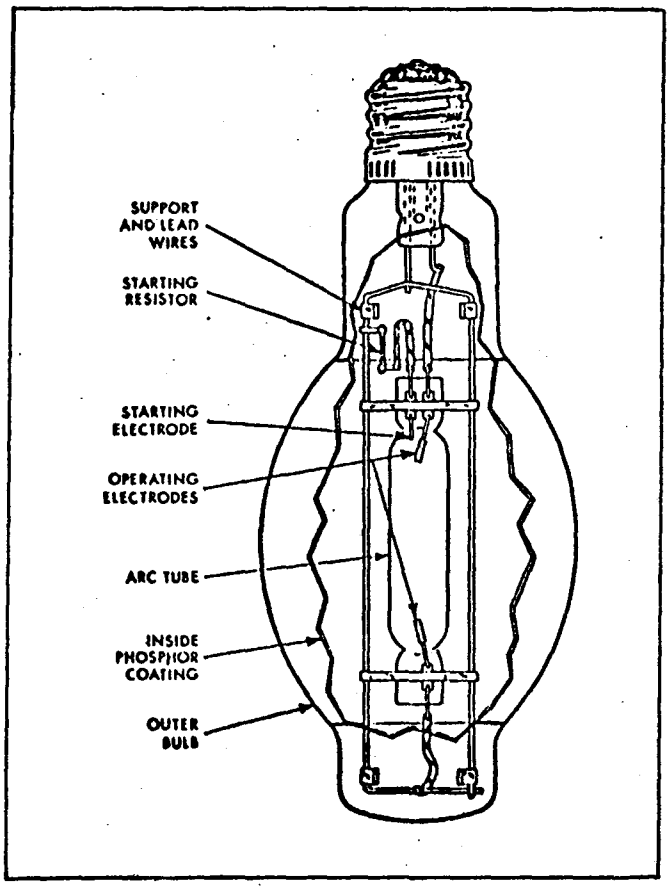

Figure 4.4.2 Construction Details of a Mercury Vapor Lamp

4.4.42 Metal Halide

Metal halide lamps, introduced in the 1960's, provide better color with greater efficacy (75-100 lumens per watt) than mercury lamps through a combination of metallic vapors in the arc tube. Additives to mercury and argon gas in the arc tube are generally iodine compounds of metals such as indium, scandium or thallium.

In general, metal halide lamps are similar in construction to mercury lamps, but a close look will reveal several construction features which are different. The arc tubes in metal halide lamps are usually smaller than in mercury lamps for equivalent wattage, with a coating or reflector at the ends of the arc tube. Some metal halide lamps include a system for either shorting the starting electrode to the operating electrode or opening the starting electrode circuit. This is required to prevent electrolysis in the fused silica between the starting and operating electrodes, especially when a halide such as sodium iodine is used in the lamp.

Almost all varieties of available "white-light" metal halide lamps produce color renditions desired. They are commonly used for lighting sports arenas, commercial business districts, and parks. Instability and short lamp life preclude extensive use of metal halide lamps in roadway lighting.

4.4.43 High-Pressure Sodium

The most recent addition to the discharge lamp family is the high-pressure sodium lamp, with 130 lumens per watt, which is second only to the low-pressure sodium vapor lamp.

In high-pressure sodium lamps, light is produced by electricity passing through sodium vapor. The high-pressure sodium, with a combination of gases in the arc tube, provides a soft, pinkish-yellow light that is generally well-accepted by the driving public. The light source appears to provide better visibility with less glare.

The high-pressure sodium lamp is constructed with two envelopes, the inner envelope (arc tube) being polycrystalline alumina which has the properties of resistance to sodium attack at high temperatures, as well as a high melting point, and good light transmission even though this material is translucent. The arc tube is normally filled with sodium, mercury, and xenon gas. The xenon gas acts as a starting gas, and as the arc tube becomes hotter, the mercury and sodium vaporize and add color to the discharge. Since very little ultraviolet energy is emitted by the high pressure sodium arc tube, it is unlikely that this type will find any advantage in the addition of coatings to the outer envelope.

Because of the small diameter of a high-pressure sodium lamp arc tube, no starting electrode is built into the arc tube as in a mercury lamp. Since the high-pressure sodium lamp does not contain a starting electrode, a high-voltage, high-frequency pulse is used to ionize the xenon starting gas to facilitate starting. Once started, the lamp warms up in approximately 15 minutes during which time the color rendition will change from poor when the lamp first starts, to its normal spectrum when stable operating conditions are achieved.

To achieve the necessary electrical control needed for high-pressure sodium, special ballasts are required. These ballasts incorporate starting circuits that provide pulse voltages in the range of 2250 to 7000 volts in order to strike the arc, since these lamps do not incorporate a starting electrode or heater coil as is found in other types of high-intensity discharge lamps.

4.4.44 Low-Pressure Sodium

In low-pressure sodium lamps, light is produced by passing electricity through vaporized sodium at an efficacy of 150 to 180 lumens per watt. The gas used for starting is neon with small additions of argon, xenon, or helium.

When low-pressure lamps are first started, the light output is the characteristic red of the neon discharge, and this gradually changes to the characteristic yellow as the sodium is vaporized. The yellow color is objectionable to many drivers and, due to its monochromatic character, it produces poor color rendition.

Another disadvantage of the low-pressure sodium lamp is its size. For example, the 180 watt lamp, including the fixture, is 54 inches (1.4 m) long.

4.4.45 Summary Comments - Light Sources

Mercury vapor is certainly not as prestigious in relative efficacy as it was in the past, but it still serves a purpose. It has a very long life and an appealing color quality in a blue-green environment. Thus, for lighting in parks, etc., it is still perhaps the best choice.

The metal halide source was to provide all of the qualities of mercury, plus some additional qualities. The combination of metallic vapors was to increase the efficacy and color quality. The result was, however, a relatively unstable light source of relatively short life. Metal halide sources are most popular in sports lighting, but have never been completely satisfactory on the roadway.

The high-pressure sodium lamp is the latest development of technology, and, therefore, it provides the best in current compromises. The sodium content results in high efficacy, whereas the blend of vapors provides reasonable color quality. It is particularly adaptable for browns and greys, the predominant colors of roadways and buildings. Long life and high efficacy stimulate popularity in a time of economic stress and energy consciousness.

The low pressure sodium source has the highest efficacy but the lowest degree of acceptance of any of the sources presented herein. Its lack of popularity is attributed primarily to its yellow, monochromatic light. Apparently, people prefer light sources that enhance natural surroundings. Applications, however, indicate that low-pressure sodium is not objectionable as a source for freeway lighting. Apparently the harshness and the monochromaticity are reduced by the vehicle headlights, and people are not as closely associated with the freeway environment as they are with the arterial and residential street environment.

These are broad generalizations, and it should be noted that with a wide variety of sources there is no one best solution for a given lighting application. The requirements should be known: the characteristics of the light sources with these requirements in mind should be reviewed, and the sources that are not suitable should be eliminated.

Table 4.4.1 presents various sources and their respective efficacy and estimated lamp life. The given characteristics should aid the designer in choosing the light source best suited for a given application.

4.4.5 DESIGN CRITERIA

In this country, lighting designed according to the illuminance concept utilizes average maintained illumination and uniformity as design criteria. Average maintained illumination, expressed as horizontal illumination in foot-candles (lumens per square foot), or lux (lumens per square meter), is the average illumination on the horizontal road surface at the end of the design life of the lamp, and at the lowest allowable maintenance condition. Uniformity is usually expressed in terms of average to minimum illumination on the roadway. In some instances, designers have utilized maximum, rather than average to minimum. Design criteria are published by AASHTO and IES (1), (3). Tables 4.4.2 and 4.4.3 present the current design criteria.

It has been proposed that lighting in this country should be designed on the basis of the luminance concept. What is the difference between the illuminance and luminance

TABLE 4.4.1. TYPICAL AREA AND ROADWAY LIGHTING LAMP CHARACTERISTICS[1]

	Lumens Per Watt (Includ. Ballast Losses) (2.)	Lumens Per Watt Lamp Only	Lumens	Wattage Range	Rated Ave. Life (Hrs.) (3.)	% Maint Output at end of rated life	Color Rendition	Optical Control	Cost Initial (Lamp)	Cost Operational (Power)
Incandescent	N/A	11-18	655-15300	58-860	1500-12000	82-86	Exc.	Excellent	Low	High
Tungsten-Halogen	N/A	20-22	6000-33000	300-1500	2000	93	Exc.	Ex. Vertical Poor Horiz.	Moder.	High
Fluorescent	58-69	70-73	4200-15500	60-212	10000-12000	68	Good	Poor	Moder.	Moder.
Mercury-Clear	37-54	44-58	7700-57500	175-1000	24000+	62-82	Fair	Good	Moder.	Moder.
Mercury-W/Phosp.	41-59	49-63	8500-63000	175-1000	24000+	50-73	Good	Fair	Moder.	Moder.
Metal Halide	65-110	80-125	14000-125000	175-1500	7500-15000	58-74	Good	Good	High	Low
High Pressure Sodium	70-130	95-140	9500-140000	100-1000	20000-24000	73	Fair	Good	High	Low
Low Pressure Sodium	78-150	131-183	4650-33000	35-180	18000	100(4.)	Poor	Poor	High	Low

Notes:

(1.) All figures show operating ranges typical for lamp sizes normally used in area and roadway applications.

(2.) Ranges shown cover low wattage lamps with regulated type ballasts (worst condition) through high wattage lamps with reactor type ballasts (best condition).

(3.) Rated average life is based on survival of at least 50% of a large group of lamps operated under specified test conditions at 10 or more burning hours per start.

(4.) Low pressure sodium lamps maintain initial lumen rating throughout life, but lamp wattage increases. Considering this change in wattage, the luminous efficacy of these lamps (including ballast losses) at 18000 hours is 67-117 lumens per watt.

TABLE 4.4.2 RECOMMENDATIONS FOR ROADWAY AVERAGE MAINTAINED HORIZONTAL ILLUMINATION

Vehicular Roadway Classification	URBAN					
	Commercial		Intermediate		Residential	
	fc	lux	fc	lux	fc	lux
Freeway	0.6	6	0.6	6	0.6	6
Expressway	1.4	15	1.2	13	1.0	11
Major	2.0	22	1.4	15	1.0	11
Collector	1.2	13	0.9	10	0.6	6
Local	0.9	10	0.6	6	0.4	4
Alley	0.6	6	0.4	4	0.4	4

TABLE 4.4.3 RECOMMENDED AVERAGE-TO-MINIMUM UNIFORMITY RATIOS

For Roadways in -	Recommended Ratios ANSI(1)	FHWA/AASHTO(6)
Commercial Areas	3:1	4:1
Intermediate Areas	3:1	4:1
Residential Areas	6:1	6:1

concepts? The illuminance concept which is almost universally used in the United States is based on the premise that by providing a given level of illumination and a uniformity of distribution, satsifactory visibility will be achieved.

The luminance concept, which is fairly popular in parts of Europe and is promoted by some people in the United States, is based on the premise that visibility is related to the luminance of the pavement and the objects on the pavement. This, in turn, is related to the reflectance properties of the pavement and the objects on the pavement. The primary obstacles to implementing the luminance design concept is that of estimating pavement reflectivity for a wide variation of pavement types and ambient conditions. Estimation of the reflective properties of the objects that drivers must see is also difficult. Information of this type is needed because the luminance measurement is made of light flux reflected from the pavement and the object to an observer. With further development, it is feasible that luminance design procedures could be incorporated into the ANSI recommended practice by the early 1980's.

4.4.6 ILLUMINATION DESIGN PROCEDURE

The steps in the design process are as follows:

- Selection of Type of Light Source
- Selection of Light Source Size and Mounting Height
- Selection of Luminaire Type
- Luminaire Spacing and Location
- Checking For Design Adequacy

The design procedure is far too lengthy for presentation here, but the reader is referred to a detailed treatment in the FHWA Roadway Lighting Handbook (4).

4.4.7 LIGHTING MASTS AND BASES

4.4.71 Masts

There are principally five types of masts utilized for luminaire supports. These are:

- Steel - galvanized or painted
- Aluminum
- Stainless steel
- Wood
- Concrete

The advantages and disadvantages of each are discussed in the following paragraphs.

Steel. Galvanized steel masts are extremely popular because of comparatively low cost and extended life. Painted poles are less popular because of the continual maintenance problem.

Aluminum. Aluminum masts have two principal attributes:

- They are relatively maintenance-free due to their resistance to corrosion when exposed to the natural elements

- They are lighter in weight than other types and thus offer less inertial resistance to impacting vehicles.

Stainless Steel. Stainless steel masts are light weight and corrosion resistant. They employ an integral breakaway base that is of the progressive shear type.

Wood. Wood is perhaps the most economical of lighting masts, particularly in the forest regions. Wood may be treated to resist rotting and deterioration, dyed to appear more attractive; but the major disadvantage is that they may be installed only by the method of embedment. Direct embedment precludes the possibility of utilizing breakaway features.

Concrete. Concrete poles are extremely popular in certain regions where cement and concrete aggregates are plentiful. Thus, the principal advantage of concrete poles is related to economics. Disadvantages are that concrete poles can be installed only by the embedment method, and they are extremely heavy even though they are made by pre-stressing concrete. Collisions with concrete masts cause failure of the concrete; however, the pre-stressing cables may pull the heavy mast down on to the colliding vehicle.

4.4.72 Breakaway Base Requirements

There are several types of bases currently used to support luminaire masts. There are several factors considered in the selection of a particular type of base: the method of construction, type of mast, funds available, agency policy and safety. This latter criterion is one of the most important considerations, and in the past, it has frequently received little or no consideration.

As a principal safety consideration, breakaway or frangible luminaire supports should be used wherever the support is exposed to traffic (5). There are exceptions, however, as outlined in the AASHTO Guide, "Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals." Some of these exceptions are summarized as follows:

- The supports are located well beyond the clear roadside requirements (e.g., high-mast lighting);

- The supports are located upon or behind barriers existing or installed for other purposes;

- The supports are located on retaining walls; or

- The relative hazard of a fallen support is greater than the hazard of the vehicle colliding with a fixed base support (e.g., supports in low speed, high pedestrian volume areas and at intersections where the luminaire support is integrated with the signal mast)

The acceptability of breakaway devices for use on all new Federal-aid projects is determined on the basis of compliance with the AASHTO standard specifications. The dynamic performance of the breakaway supports under automobile impact is the basic measure of satisfactory breakaway characteristics. Satisfactory dynamic performance is indicated when the maximum change in momentum for a standard 2250 pound (1020 kg) vehicle, or its equivalent, striking a breakaway support at speeds of 20 to 60 mph (32 kmh to 97 kmh) does not exceed 1100 pound-seconds (4893 N-sec), but desirably does not exceed 750 pound-seconds (3336 N-sec).

There are other alternatives to full-scale crash testing (10). The specification permits testing by a method "equivalent" to vehicle (automobile) crash testing. Actually, in the future an equivalent test, or test series, will probably become the preferred method because, presumably, it would eliminate the variation in crush characteristics of crash vehicles. Presently there is no universally applicable equivalent test. However, there are nonautomobile test procedures that are considered equivalent for some types of hardware--luminaire supports, slip base and load concentrating sign supports, and, with a bogie only, single post timber and base bending sign supports.

Two equivalent test devices have emerged so far. One is a pendulum, the other is a bogie. In both, vehicle crush characteristics are simulated by the use of an expendable aluminum honeycomb cartridge that is placed between a relatively rigid striking face and the principal mass of the test device. Controlling a relatively few characteristics of such test devices should ensure consistent results between devices.

In determining if an item meets the requirements of the AASHTO specifications, testing and reporting procedures comparable to those given in NCHRP Report 153 should be followed. Acceptance may be based on a single test if

the test change in momentum and the analytically-inferred changes in momentum over the speed range are less than 750 pound-seconds. If the first dynamic test change in momentum is between 750 and 1100 pound-seconds, a second dynamic test will be needed unless assurance that the test results are representative of what would result from further dynamic tests can be demonstrated analytically and statically. The results of the second test must also meet the specification requirements.

4.4.73 Types of Luminaire Support Bases

There are numerous types of luminaire support bases currently in service. Some of the more common types are described in the following paragraphs. Their presentation herein does not imply that they are acceptable for use on Federal-aid projects. In fact several are included to emphasize that they are not acceptable and should not be used where they are exposed to traffic, particularly high-speed traffic.

Butt-Type Bases. Probably the most basic of all support methods is the butt-type base which is embedded directly into the soil. It is by far the cheapest in most instances, and the only method applicable to wood and to concrete. Also, embedment may be used to install galvanized steel and aluminum masts. As mentioned previously, breakaway features are not possible when the embedment method is used. Further, masts get out of plumb during the seasons of high rainfall or spring thaws when the ground is soft and when the masts are subjected to high winds. Obviously, the butt-type bases do not meet the dynamic requirements of the AASHTO specifications.

Flange Bases. Most steel and aluminum poles are fitted with a plate or a flange at the base of the mast to facilitate bolting to a foundation or to some form of base. On steel poles, this is generally a steel plate that is fitted and welded to the base of the mast prior to galvanizing or painting. On aluminum poles, a cast aluminum base plate is fitted over the lower end of the shaft and is welded to the mast.

Although flange bases are frequently are used by bolting directly to a foundation without an intermediate breakaway device, they generally do not meet the dynamic requirements. Such applications should be restricted to locations where they are not exposed to traffic or otherwise excepted in the AASHTO specifications.

Cast Aluminum Transformer Bases. "Serendipity" - the fortune of finding valuable things not sought for - can be used to describe the development of the cast aluminum transformer base as a breakaway device. The T-base orignally was devised to house the transformer or ballast. It was made of cast aluminum as one alternate to reduce the corrision effects. Other T-bases were made of steel plate and galvanized or painted to reduce the corrosion effects. The T-base soon proved to be not so good for housing the ballast because of moisture and insect damage to the electrical components. However, the cast base did prove to be a safety device because it would yield and break apart when struck by a vehicle. Today, virtually no one designs a system to use the T-base to house the ballast, but many agencies have specified T-bases as safety devices.

Cast aluminum T-bases are manufactured in various forms, but generally they are square to fit the flange of the mast and either straight or slightly tapered outward to fit the anchor bolts of a concrete foundation. The earlier T-bases were 20 inches (50 cm) tall, and it has been found that this is an optimum height. It is important that the colliding vehicle strike the T-base rather than the mast. In this manner, the vehicle imparts a certain amount of shock which is desirable in causing a quick, rapid failure and subsequent release of the mast. Experimental versions of shorter bases resulted in extensive damage to the mast and an increased resistance to failure which, in turn, increased the probability of injury to the vehicle occupants.

Cast aluminum T-bases have been used to improve the safety of existing lighting systems. For example, a system which consists of galvanized steel masts flange-mounted to concrete foundations, was modified by simply inserting under the flange mounting a transformer base which had the same bolting configuration, both top and bottom.

Since the issuance of FHWA Notice N5040.20 of July 14, 1976, there have been no T-bases to qualify for use on Federal-aid projects. There are several manufacturers and other agencies that are exploring design modifications that may bring the T-base into compliance with the AASHTO dynamic requirements.

Frangible Couplings. A frangible coupling can be used with flange mounted steel or aluminum supports to greatly improve their impact behavior. The coupling, illustrated in Figure 4.4.3, is simply a short, extruded aluminum connector or sleeve that is threaded internally. It is placed on the foundation anchor bolts, and the flange of the support is attached to the top of the coupling with cap screws. When the support is struck by a vehicle, the coupling breaks; and the intensity of the collision is great-

ly reduced. The fluting of the insert is essential to its satisfactory behavior upon impact.

The frangible coupling has been tested and is approved for use on Federal-aid projects. Another license is developing of slightly different version of this concept.

Multi-directional Slip Base. The multi-directional slip base, illustrated in Figure 4.4.4, operates on the breakaway concept of high resistance to overturning and low shear resistance. The base consists of two identical plates, one welded to a foundation attachment. These plates are slotted in a triangular configuration (See Figure 4.4.4) so that when bolted together they will slip apart regardless of the angular direction of the impact. The multi-direction slip base probably offers the least resistance to collision of all of the breakaway concepts. As an additional advantage, the mast noramlly is not destroyed by the impact of collision. With some minor repairs, it generally can be reinstalled, thus reducing the total cost of the installation when all economic factors are considered. The multi-directional slip base has been adopted by quite a number of state agencies as their standard design for luminaire supports.

The multi-directional slip base has been approved for use on federal-aid projects when designed and installed in accordance with requirements outlined in FHWA Notice N5040.20 (July 14, 1976), as follows:

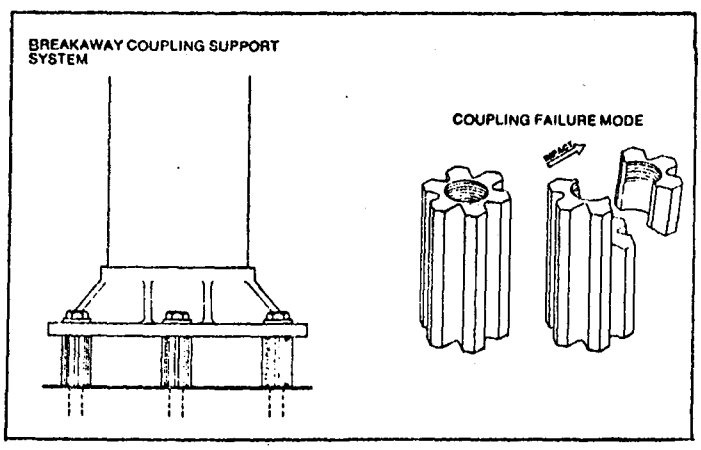

Figure 4.4.3 Frangible Inserts For Luminaire Supports

1. Weigh 600 pounds (272 kg) or less (Supports weighing 1000 pounds (454 kg) have been qualified through testing).

2. Have a total slip face clamping force less than 45 kips. (A force one-quarter this amount or less would be preferred and should give good service under wind loads.) (The clamping force must be controlled by installing bolts with a torque wrench, using torque limiting nuts, or another acceptable method.)

3. Have a 28-gage steel keeper plate or equivalent to prevent "walking."

4. Have washers with the clamping bolts of sufficient strength to prevent the washers' cupping into the vee slots.

5. Have a stub height of 4 inches or less.

There is some hazard associated with snagging on the stub of the breakaway structure when the vehicle suspension system is fully compressed during impact, particularly when the structure is located near a hinge point in the fill slope. We may need to consider some form of deflector on the stub to eliminate the snagging potential.

Stainless Steel Bases. A specially-designed stainless steel base and support has been used extensively as a breakaway design by a number of state agencies. The shaft of the support is fabricated from stainless steel and is welded to a stainless steel base or box as shown in Figure 4.4.5. This base is then riveted to a stainless steel base plate which is bolted to a concrete foundation. The failure mechanism is in the area where the box is riveted to the base plate. When a vehicle strikes the box, the rivets are sheared, and the support breaks away.

Integral Anchor Base. A commercially produced base is available in cast aluminum for 8-, 9- and 13-inch (20.3-, 22.9-and 33.0 cm) aluminum masts. These are applicable to mounting heights up to 50 feet (15 m). These bases, similar to a flange mounting in many respects, are designed to fracture just above the anchor bolts. All three sizes have been tested and are approved for use on Federal-aid projects.

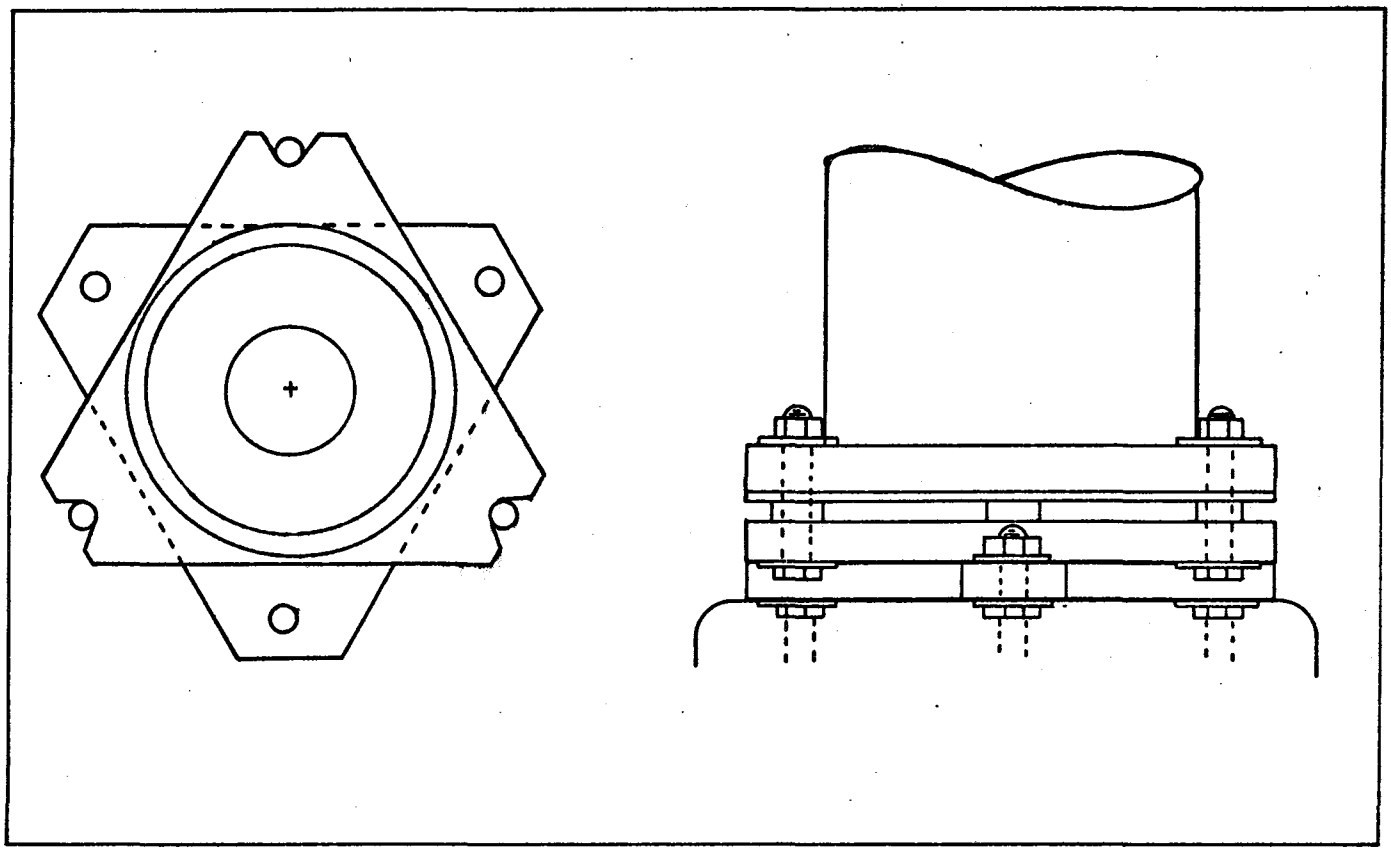

Figure 4.4.4 Multi-directional Slip Base

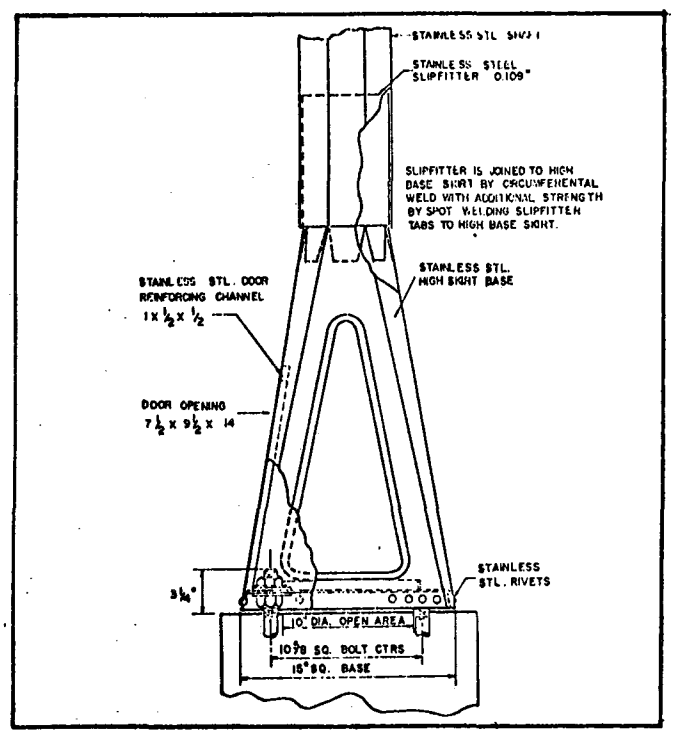

Figure 4.4.5 Stainless Steel Base

4.4.8 MEDIAN LIGHTING SYSTEMS

Because freeways and expressways were, by design, wide multi-lane roadways with a median strip separating the two directional roadways, the conventional one-side and staggered lighting systems were not applicable to the entire system; each roadway was lighted independently from the outside edge of the roadway. Early considerations of placing luminaire supports in the median were rejected because the median was to remain a clear area for reasons of safety. Accident experience on early freeways indicated a severe problem with vehicles crossing the median, particularly the narrow medians in urban areas, and colliding with vehicles in the opposing direction. Physical barriers which were used to reduce vehicle encroachment could then be used to protect the lighting system, or to eliminate or reduce the probability of a collision with a luminaire support. Integrating the luminaire support with the median barrier has become the most common approach to lighting freeways and expressways in urban areas.

The popularity of the median lighting configuration is due to a number of factors. Perhaps the primary one is economy. Placing the luminaire supports in the median reduces the number of supports required for supplemental lighting in the areas of ramp terminals and interchanges. Also, since there is only one row of supports, there is need for only one run of electrical conductor. Savings in material and construction costs are substantial. In addition to the economic advantage of median lighting, there is a service advantage; median lighting simply provides better visibility. The highest level of illumination is along the median and inside higher speed lanes. The illumination level dissipates slowly across the traffic lanes, and out into the areas adjacent to the roadway. The horizontal light component is proportionately high in the border areas and aids the driver's visibility. This effect is compared by schematic (Figure 4.4.6) to show the contrast with side-mounted lighting. In the side-mounted configuration the highest illumination level is along the edge of the roadway, between the driver and the peripheral area. This is, in effect, a brightness curtain that reduces the driver's ability to see beyond the edge of the roadway. The net result is an "illumination tunnel".

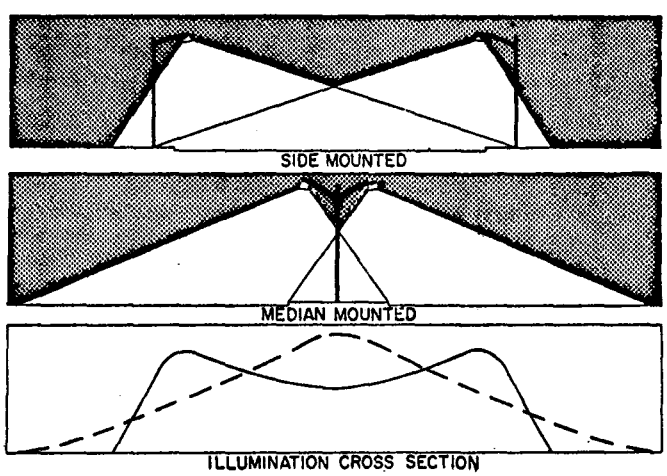

Figure 4.4.6 Comparative Distribution of Illumination for Median vs. Side-Mounted Systems.

Due to the economic and service factors, median lighting should be the first consideration in lighting the main lanes of freeways and expressways. However, there are several conditions where median lighting may not be the most applicable. Among these are:

- Freeways and expressways where the median is to be used for transit vehicles.
- Freeways and expressways where there is no median barrier and the relative hazard of fixed-object collisions would be increased.
- Freeways and expressways where the median is too wide for both directional roadways to be lighted from one mast.

Regarding the first of these conditions, freeways in some large metropolitan areas are being designed or re-designed to provide an exclusive travel way for bus rapid transit in the median area. This may be two-way operation, or one-way reversible to accommodate buses flowing the peak direction only. Regardless, this type of operation may preclude the use of median lighting. The exception, of course, is in the situation where two rigid median barriers are used to separate the busway from the main lanes. In this instance, luminaire supports may be integrated with the median barriers. Because the barriers are not in the center of the median area, the system may require single luminaires mounted on each barrier, or one row with twin mast arms, one arm longer than the other to balance the system. The choice between these two alternatives is largely dependent upon the width of the busway and the capability of the median-mounted system to provide complete coverage.

The second condition which may preclude the use of median lighting is the situation where there is no median barrier, and the placement of luminaire supports would perhaps constitute a greater hazard than if they were side-mounted. The additional hazard one may anticipate is in the case of a luminaire support being driven into the opposing traffic lanes by the impact of a collision. The result of such a collision is largely speculative because speed of the colliding vehicle, the angle of impact, the distribution of the mass of the support and many other factors determine the post-collision behavior of the support. There are two points to consider in reaching a decision on whether to use median lighting without the protection of a median barrier. First, the current Federal Highway Administration policy permits the use of breakaway luminaire supports in medians when the median contains no barrier and is wider than 40 ft (12 m). Thus, on all Federal-aid projects this policy will govern.

Another approach to determining whether median lighting should be used is provided in Figure 4.4.7, where the "relative hazard" of side-mounted vs. median-mounted configurations is compared. In this study conducted by TTI (6), the relative hazard was determined as the product of:

- The probability of a vehicle encroaching on the median or roadside far enough to strike a support
- The probability of the vehicle actually striking a support
- The probability of the support encroaching upon the opposing traffic lanes

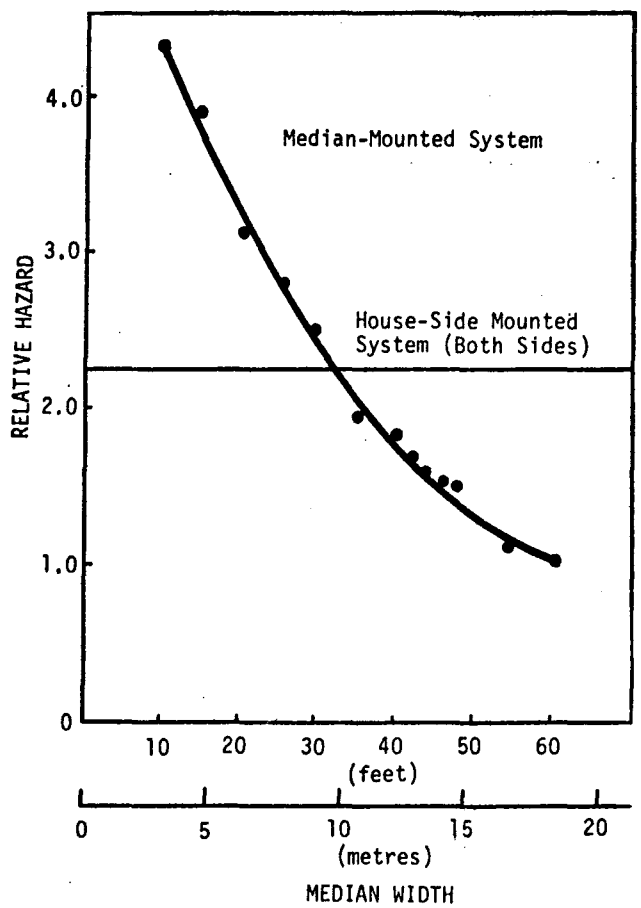

Figure 4.4.7 Relationship of Relative Hazard Index to Median Width for Median-Mounted and House-Side Mounted 50 ft (15 m) Luminaire Supports.

This study was based on a comparison of a median-mounted system and side-mounted system, with the median width as the variable It should be noted that both systems were mounted 50 ft (15 m) high. Longitudinal spacing was 250 ft (76 m) for the median-mounted systems and 225 ft (69 m) for the side-mounted system (considered to be comparable quality). The side-mounted system was mounted 15 ft (4.6 m) laterally from the edge of the travel way. Based on these conditions, it may be noted in Figure 4.4.8 that for any median width greater than about 32 ft (10 m), it is relatively safer to use a median-mounted system.

In the third case, there is naturally a limit to the width of a median where median lighting can be utilized effectively. Mast arm lengths greater than about 20 ft (6 m) are impractical. Thus, if one chooses to place the light source over the edge of an inside shoulder of 6-10 ft (2-3 m), then the practical limit on median width would be 52 to 60 ft (16-18 m). There are a number of installations where median lighting is used in median widths in this range, and they provide excellent service. It should be noted, however, that Type II or Type III luminaires are generally used and they may be tilted upward transverse to the roadway by an angle of 5 to 10 degrees, or to their maximum vertical adjustment in the slip-fitter.

It is a foregone conclusion that breakaway bases should be used when median-mounted lighting is used without a median barrier. Experience throughout the nation has demonstrated an unquestionable safety advantage of breakaway bases on the intermediate to high-speed traffic facilities such as freeways and expressways.

When luminaire supports are integrated with a median barrier, breakaway bases may or may not be applicable, dependent upon the type and characteristics of the median barrier. Median barriers are normally of three types: flexible, semi-rigid and rigid (7, 8) characterized by the amount of deflection permitted during impact. For more detail on median barriers, refer to NCHRP Report No. 118, or to "Guide for Selecting, Locating, and Designing Traffic Barriers," published by AASHTO in 1977. Typically, allowable deflection for the three types are 8-12 ft (2.4 - 3.6 m), 2 ft (.6 m) and 0 ft, respectively (8). Because of the large deflection of the flexible barrier, median-mounted lighting should not be integrated with this type of barrier. The luminaire support would constitute a fixed or semi-rigid object in a flexible system, and the barrier system would simply guide the vehicle into a collision with the luminaire support. If for some reason luminaire supports must be integrated with flexible barriers, they certainly should have breakaway bases.

Luminaire supports frequently are integrated with semi-rigid barriers with a great degree of success (9). The semi-rigid barrier is typically a double-beam section using two flexbeams or standard W-sections on wood or steel posts. Design standards limit deflection to 2 ft (0.6 m), and the system provides a certain amount of beam action to redirect a vehicle and reduce the effect of "pocketing" the vehicle against the luminaire support. Generally, there are two modifications in the barrier design when luminaire supports are included: (1) the barrier beams are spread with additional blocking to provide more space between the beam and the luminaire support and (2) the barrier is stiffened in the vicinity of the support to increase the beam action and further reduce the pocketing effect. For further details, see Reference (9). Some, however, have chosen to continue the normal section of the barrier without widening or stiffening and simply use breakaway supports.

Integrating luminaire supports with rigid concrete median barriers is perhaps the simplest of all such installations. Several different agencies around the country have chosen different approaches. The main problem stems from the barrier design. The standard barrier is 6 inches wide at the top and tapers out toward the base. Because the base plate on luminaire supports is typically about twice the dimension of the top of the barrier, some modifications are necessary for compatibility.

One alternative is to widen the top of the barrier in the vicinity of the luminaire support. At least one state has used this method, and apparently it has been successful. Rather than use localized widening, Texas chose to widen the barriers to 8 inches (20 cm) and re-design the luminaire support to fit the top of the barrier (Figure 4.4.8). The round tapered steel pole is forced into an oval shape to fit a base plate 8 inches (20 cm) in width. This design has been tested in full-scale crash tests and installed in a substantial number of locations. It is the standard design for Texas.

It should be noted that in both situations, a drilled-shaft type foundation is placed under the barrier at the support location, and sufficient re-enforcement of the concrete is provided. Many rigid barriers today are constructed using prefabricated (pre-cast) sections or slip-forming methods. In both instances, the section to contain the luminaire support is formed and poured by conventional procedures, after the other sections have been placed.

In other design variations some, including the state of Minnesota, have left an opening in the median barrier to facilitate mounting the luminaire supports flush with the top of

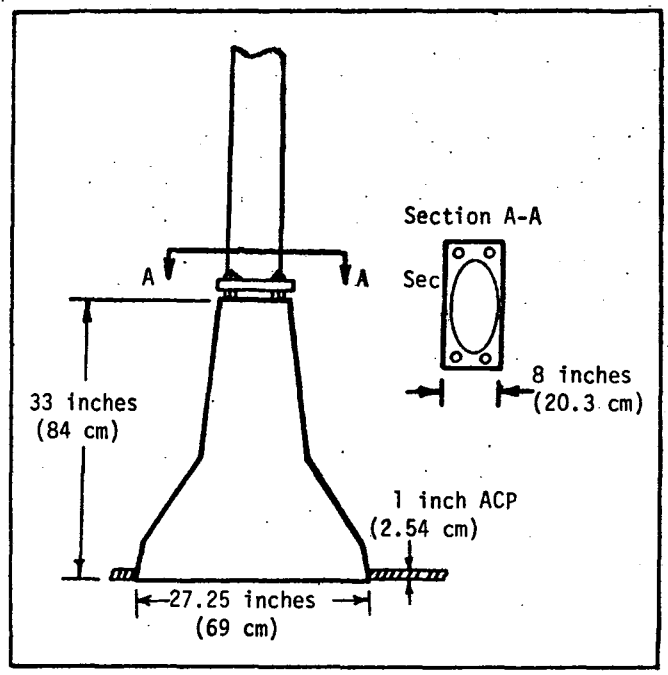

Figure 4.4.8 Luminaire Support Mounted on Concrete Median Barrier.

the median. Continuity across the openings is provided by a steel plate bolted across the opening. In Wisconsin, a split barrier was used to provide protection of luminaire supports as well as sign supports, bridge piers and other fixed objects. The two barrier sections are spaced 36 inches (.91 m) apart, and luminaire supports are mounted flush with the median. It is interesting to note, also, that the electrical conductors are placed within the space between the two barriers, which is not covered. Wisconsin experience has shown that some vehicular collisions result in the vehicle vaulting up onto the top of the barrier and striking the luminaire support. Currently, the strategy is to use aluminum poles which provide some breakaway relief, especially when hit approximately 3.5 ft (1 m) above the foundation. They are considering re-designing future installations to provide breakaway features at the top of the barrier.

4.4.9 SUMMARY

Lighting is an effective tool for the improvement of the night-time operational efficiency and safety of streets and highways. Basically, lighting increases driver visibility, permitting a more orderly identification and execution of driver tasks.

Lighting should be integrated into the overall traffic operations engineering functions in the same manner as signing, delineation and marking the facility. It is simply a tool to be used in providing the driver with needed information.

There is an orderly process of planning, designing and operating roadway lighting systems that should be developed within each operating agency. The major steps of this process are as follows:

1. Analyzing the need for lighting - establishing warrants and priorities

2. Selecting the lighting system configuration

3. Selecting the proper equipment

4. Designing the illumination system

5. Maintaining and operating the system

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TOPIC 4 SESSION 5
RAILROAD GRADE CROSSING SAFETY

Objectives:

The participant should be able to:

1. Identify the various studies necessary for an analysis of traffic control needs at a particular rail-highway grade crossing.

2. Better evaluate the types of materials for grade crossing surfaces that are most compatible with various combinations of railroad and highway conditions.

3. Examine the potential of passive and active warning systems.

4. Review the various types of crossing surfaces and to illustrate the effect of surface condition on crossing safety.

4.5.1 INTRODUCTION

The railroad grade crossing safety problem has existed since the advent of the railroad. Although the number of fatalities resulting from accidents at railroad grade crossings has remained fairly constant over the years, the fact remains that every year considerable injury and property damage result from accidents of this type.

In 1974 there were 219,301 railroad grade crossings in use in the United States. At these crossings, 3079 accidents were reported, resulting in 3155 injuries and 1128 deaths. These accidents, while being a relatively small percentage of total accidents, are usually quite severe and result in severe consequences to the motorist and bring about considerable notoriety.

4.5.11 The Safety Problem at Railroad Grade Crossings

Fundamentally, the rail-highway grade crossing is a simple intersection of two traffic streams. The characteristic that one stream is automobiles, and the other is trains, is the first complicating factor. While there may be several thousand vehicles using the highway, 10 to 12 trains per day is a rather high volume for most crossings. Thus, the major problem with rail-highway grade crossings stems from the fact that, because of the infrequent encounter of many drivers with trains at a particular crossing, the driver grows to expect the absence rather than the presence of trains. For this reason, the approach warning system and the crossing control system must change the driver's expectancy and alert him to actions that may be required of him to assure a safe crossing.

Car-Train Relationship. The fact that the automobile is easily maneuverable, whereas maneuvers made by the train must be planned and deliberate, places the automobile always in the position of yielding to the train traffic. Drivers normally do not question this responsibility placed on them. On the other hand, they develop dangerous expectancy patterns because they seldom encounter a train at a rail highway grade crossing.

Needs of the Driver. Recognizing that the negative expectancy pattern exists, the driver needs very positive advance warning on the approach to a crossing. Since his expectancy must be changed, it is necessary to institute redundancy in approach warning. Unique signs should be placed on both sides of the road, and it is desirable that these be supplemented with pavement markings. When the driver has reached a certain point in his approach to the crossing, he should be given specific information concerning his responsibility at the crossing. If it is an uncontrolled crossing and the burden is placed on the driver for the detection of an approaching train, he should be told. If there is limited sight distance and the safe approach speed is less than the normal highway speed, he should be informed accordingly. Further, if the crossing is controlled by gates or signals, the driver should be advised so that he can develop his expectancies accordingly.

Quite frequently, geometric conditions of the highway or the abutting property influence the traffic operation at the crossing. These conditions should be considered very carefully and the driver advised accordingly. For example, horizontal and vertical alignment of the highway often restrict the view of the crossing itself from the point where the driver must make a decision regarding the safety of the crossing. Also, physical features on abutting property may restrict the view of trains on unprotected crossings. All of these factors should be taken into consideration in the design of the approach warning system.

4.5.2 RAILROAD GRADE CROSSING IMPROVEMENT PROGRAMS

The development and implementation of a railroad grade crossing improvement program may involve various state and local governmental agencies as well as the respective railroad companies. Most improvement programs require the studying and inventorying of crossings, prioritizing of crossings for improvement, and the reviewing of various alternatives for achieving improvement.

4.5.21 Engineering Study

The engineering study should consist of (1) a survey of the physical features of the crossing, its approaches and its general environment, (2) a study of accident history; and (3) an analysis of accident potential.

Study of Physical Features

Roadway Conditions

- Obtain design plans of roadway if available.
- Update plan from field observations, make note of all driveways, intersections and other geometric conditions that may exist.
- Make photographs or color slides at selected intervals along each approach, toward the crossing and along the track approaches.
- Check pavement surface condition.
- Check availability of escape routes for vehicles to avoid a collision with a train on the crossing.

Track and Train Conditions

- Determine the number of main tracks, sidings and other auxiliary trackage.
- Obtain data on train frequency and train speed.
- Determine geometry of track approaches that may have an influence on visibility of an approaching train.

Traffic Conditions

- Obtain traffic volume data for both approaches.
- Obtain traffic speed data similar to that obtained for speed zoning. Where there is a condition of natural decleration in the traffic stream, speed measurements should be made in the approach zone where the driver should be looking first for a train on the track approaches. The speed data should be analyzed to determine the 85-percentile speed for later sight distance computations.
- Determine turning movements near the crossing, particularly within the approach warning system.
- Determine percentage of truck traffic required by law to stop at crossings. This information will be helpful in making a decision as to whether added lanes should be provided.

Crossing Control

- Observe the sufficiency of crossing protection systems while a train is occupying the crossing. Notation should be made of the general appearance, alignment, maintenance and repair, for effective communication with the railroad company.
- A check should be made for fixed object hazards placed unduly close to the traffic lanes, such as pipes or rail sections placed in the ground to protect the signal standard.
- Location and sufficiency of stop lines should be checked.

Approach Warning System

- The sign location and message content should be recorded for all signs on both approach roadways.
- Advance warning pavement markings should be checked for condition and location.

Sight Distance Evaluation

- Train speed and the 85-percentile speed of traffic on each approach should be used to establish a sight triangle that will provide stopping distance on wet pavements for vehicles on each approach.
- If sight distance is not sufficient, action to eliminate the problem, or a determination of the safe approach speed, should be made.
- Determine the maximum distance on each approach that a driver will be able to see the crossing and/or the control devices under day and night conditions.

Accident History

City and State Agency Reports. Accident reports should be obtained from the appropriate records file for at least the last three years.

Railroad Accident Reports. Accident reports should be obtained from the appropriate railroad agency files.

Causative Factors. All accident data should be analyzed to determine if possible causative factors can be isolated for consideration in the approach warning and control treatment.

4.5.22 Environmental Study

Driver Expectancy. This section is intended to determine whether the driver is conditioned to expect a potential vehicular conflict in the approach to the crossing.

- Check to determine whether the driver has been traveling in rural, suburban, or urban conditions prior to the crossing approach.

- Determine the number and locations of intersections or other potential conflict areas on each of the approaches to the crossing.

Competition for Driver Attention

- Determine whether geometric conditions on the approaches to the crossing are such that they demand the attention of the driver during the period in which he must be making a decision regarding the crossing.

- Observe entering traffic conditions at or near the crossing and determine whether it has a significant effect on driver attention to the crossing.

- Determine intersection activity and controls for their effects on driver attention to the crossing.

Other Environmental Effects

- All formal information systems including signs, markings, delineations and other appurtenances should be checked as to their possible effect on crossing safety.

- Other informational systems including advertising signs and lighting should be observed with special emphasis on their effects on driver awareness of the crossing.

- Activity of abutting development near the crossing and crossing approaches should be observed and conditions listed that could affect driver performance at the crossing.

- All other traffic activity, geometric conditions, natural conditions (such as late evening sun), and environmental development should be surveyed.

4.5.23 Inventory Data

Much of the information discussed above currently is being assembled as part of the National Railroad-Highway Grade Crossing Inventory developed cooperatively through the Federal Highway Administration, the Federal Railroad Administration, the Association of American Railroads and the individual states. This national inventory is aimed at placing on computer file pertinent data related to all public grade crossings, grade separations, private crossings, and pedestrian crossings. The data in this file are grouped into four major categories:

- Geographic location of the crossing

- Detailed information for public vehicular crossings

- Physical data at the crossing

- Highway classification and vehicle traffic volume data

Several states also have opted to collect additional information relative to the crossing in order to implement some form of grade crossing improvement priority system. This will be discussed in the following section.

Another advantage of the inventory data file is that it is site specific. That is, a unique number in the form of a plastic tag is attached to the control device at the time that the inventory is made. This allows for the positive identification of the crossing and provides a common link for future accident and inventory data collected by various governmental agencies and railroad companies.

4.5.24 Hazard Index Priority Rating and Warrants

Although it is desirable for all grade crossings to have the optimum level of traffic control or separation, this is rarely possible due to limited funds. Thus, available funds should be allocated to those locations where the provision of control devices and other improvements will have the greatest potential for accident reduction. To aid in this task, most organizations currently employ some form of priority rating or hazard index system. The bases for these systems are discussed below.

Improvements Based on Hazard Ratings. A number of hazard rating formulas have been used to compute a relative index of hazard for individual or groups of crossings. Three basic variables are common to most of these methods:

- The relative effectiveness of various types of traffic control device

- The probability of conflict between trains and vehicles

- Sight distance rating

Bezkovovainy (1) found that 11 of the most popular hazard rating formulas applied to grade crossings in Lincoln, Nebraska, resulted in virtually identical rank ordering of crossings. He also concluded that the New Hampshire Formula provided the best fit for the average of the 11 formulas. The New Hampshire Formula is fairly simple to use and has been included in the software computer program package of the National Inventory Project.

A recent study conducted in Florida (2) developed an accident prediction model that can be used to identify groups of crossings that will have the most accidents if not improved. The model, which also can predict post-modification accident rates, uses traffic, number of trains, vehicle speed, train speed, number of lanes, and presence of control devices as the independent variables.

Improvements Based on Warrants. The techniques discussed above have been developed mainly for ranking a group of crossings as to their priority of need. The determination of exactly which device is needed by these methods requires a "juggling" process to fit other constraints in a trial and error process. Another technique which incorporates the control device requirements into the decision-making process is the "warrants" technique. One study (3) reviewed urban grade crossing accidents in Indiana for 1963-64. "Protection nomographs" were then developed to determine the potential hazard of individual crossings.

The diagnostic team study method has been used quite extensively in the identification of crossing deficiencies and recommending safety improvements.

In any case, the use of hazard rating formulas or warrants requires engineering judgment and should not be viewed as the sole answer to a a question relative to the implementation of improvements.

4.5.25 Improvement Alternatives

Crossing Closure. The first alternative that should be investigated is whether the crossing can be eliminated or not. This alternative may be desirable for the railroad company, especially if it reduces the accident potential and may improve train operations. On the other hand, the re-routing of vehicular traffic may increase travel time, cause economic repercussions and act as a deterrent to emergency vehicles.

Railroad Consolidation and Relocation. An additional alternative for improvement, which involves closure, is the consolidation and relocation of railroads in urbanized areas. The impact of railroad relocation may indeed be widespread and thus require long-range and often complex planning.

Another possible railroad-highway crossing closure alternative is the relocation of the highway. This is often possible where loops or bypasses around urban areas can be built using grade-separated crossings.

Grade Separations. The optimum improvement to a grade crossing is separation of the conflicting vehicular and train movements. Although this alternative requires a large expenditure of funds, the benefits may be very real in terms of reducing accidents, relieving highway congestion and improving train operations. The grade separation alternative especially should be considered in the design of new highway routes and in improvements to railroad facilities.

Traffic Control Devices and Crossing Surface. Once the alternative of closure, relocation, and grade separation have been considered and rejected, the next alternatives for improvement are the selection and placement of adequate traffic control devices and crossing surfaces. This subject is covered in the remainder of this session.

4.5.3 TRAFFIC CONTROL AND WARNING SYSTEM REQUIREMENTS

There are two basic categories of traffic control and warning systems: passive systems and active systems. Passive traffic control systems, consisting of signs, pavement markings, and grade crossing illumination, identify and direct attention to the location of a grade crossing, to permit motorists and pedestrians to take appropriate action. Active traffic control systems inform motorists and pedestrians of the approach or presence of trains; locomotives or railroad cars on grade crossings.

To a large degree the type of control required will depend upon the volume of train and vehicular traffic and the sight distance at the intersection of the roadway and the railroad track. In the following section we will look at the various considerations which pertain to the provision of adequate sight distance or the countermeasures necessary where adequate sight distance cannot be obtained.

4.5.31 Sight Distance

The provision of adequate sight distance at railroad crossings is no less important than the provision of sight distance at major highway intersections. There are three basic sight distance models which are applicable in providing visibility at rail grade crossings (See Figure 4.5.1).

- Visibility of crossing from an approaching automobile. It is recommended that the crossing be visible to the driver on the approach road for a distance equal to the stopping sight distance (for the appropriate design speed) plus a clearance length of 20 ft (6 m) as shown as SD_1 in Figure 4.5.1. Where doubt exists in selecting a design speed, it is suggested that the 98th percentile approach speed or a speed of 10 mph (16 km/h) above the 85th percentile approach speed be adopted as a basis for design.

Curved alignment obscuring the crossing, or gradelines where the crossing is located just beyond a crest, should be avoided. The crossing should not only be visible but should be clearly identified as such.

- Visibility of an approaching train from an approaching automobile. This model is somewhat analogous to the AASHTO Case II model adopted in intersection design in which the driver on the approach road should be able to see down either leg of the railroad a distance, SD_2, so that he can recognize a train in sufficient time to either stop clear of the crossing or proceed ahead of the train. Table 4.5.1 sets out these sight distance requirements (4).

- Visibility of an approaching train from an automobile stopped at the crossing. Adequate sight distance along the tracks (SD_3) should be provided to enable the driver whose vehicle is stopped at the crossing to accelerate and clear the crossing in advance of an oncoming train. This sight distance should be based on the maximum approach speed of trains and should consider long vehicles with low acceleration potential such as school buses and large tractor trailer combinations. Table 4.5.1 sets out this sight distance requirement (0 mph).

4.5.32 Information Needs

The information needs of the driver will generally correspond to the three possible elements of traffic control: initial warning, final warning, and crossing control. Each of these is discussed below.

Initial Warning

Driver Attention. From the engineering and environmental studies, determine the initial warning necessary to demand the driver's attention. Initial warning can include advance warning messages, pavement markings, raised pavement markings for auditory and tactile stimulation, and advance flashers.

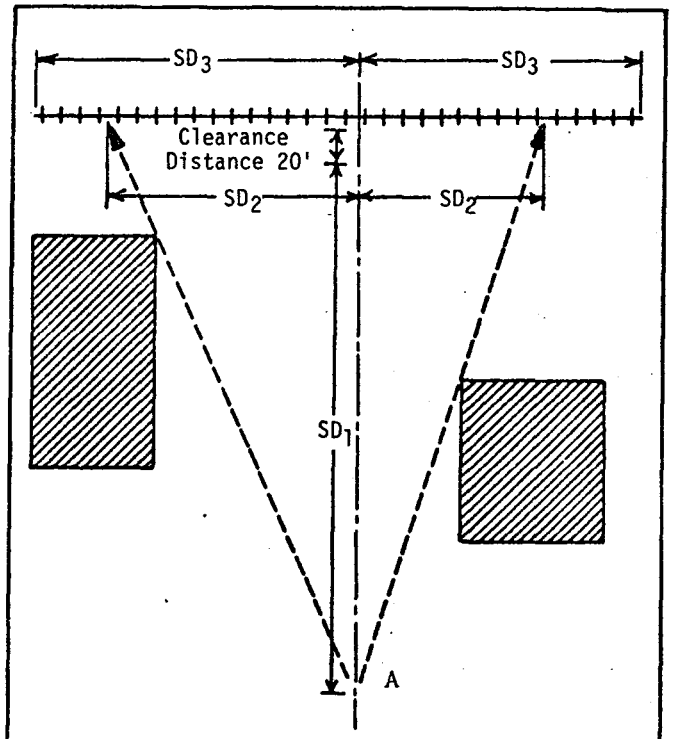
Crossing Location. The initial warning system design should assist the driver in locating the actual crossing. Determination can be made from the engineering and environmental studies of the steps necessary to advise the driver of the crossing location. In some cases, geometric and environmental conditions may be such as to prevent actual visual location of the crossing by the driver. In such cases, the driver should be so advised by the initial warning.

Operating Conditions. A determination should be made from the engineering and environmental studies as to the operating conditions the driver should expect. If modifications in operating conditions, such as a speed change, must be made by the driver, the initial warning should advise the driver of the changes required.

Final Warning

Decision Information. The final warning of the grade crossing should provide the driver with all the information necessary to make a stop or go decision. The engineering and environmental studies will provide the necessary input for determining the information that must be provided for the stop or go decision. For example, if geometric, visibility, traffic or environmental conditions are such at a crossing that the driver cannot be made aware of an approaching train, the final warning system should advise the driver of this situation.

Final Action. The final warning system, if needed, should be designed to alert the driver of the conditions prevailing at the crossing.



SD_1 - Safe Stopping Sight Distance

SD_2 - Distance Along Track Such That Vehicles Traveling at Speed Limit Can Beat Train from Point "A" or Stop

SD_3 Distance Down Track Such That Train Cannot Arrive in Less Than 11.0 Seconds.

Figure 4.5.1 Illustration of Sight Distance

Crossing Control

Hierarchy of Control. In addition to initial and final warning systems, crossing controls such as crossbucks, flashing light signals, and gates used with flashing light signals are installed to identify the crossing location and warn of train presence. Ideally, all crossings should be marked with the highest order of control devices, ie., gates and signals. This, however, cannot be done due to the cost of installing these devices. Thus, there is a need to somehow prioritize those crossings that need upgrading to estimate where financial resources may be used most effectively. Additional information relative to the selection of control devices will be presented later.

4.5.33 Application of Advance Warning Devices

In 1967, the Texas Transportation Institute initiated a research study with the Texas Highway Department in cooperation with the Federal Highway Administration to study safety conditions at public rail-highway grade crossings in Texas (5). Part of this study included the establishment of an interdisciplinary diagnostic team to study a sample of crossings for the purpose of identifying crossing characteristics which contribute to accidents. The study recommendations for 36 particular sites may be generally applicable to a large number of other sites. They were as follows:

- All signs and devices pertaining to advance warning and crossing control should be placed on both sides of the approaching roadways so that maximum effectiveness may be achieved. It was pointed out that vehicles following larger vehicles and vehicles in the act of passing are not afforded an appropriate opportunity to view advance signs and control devices unless they are placed on both sides of the approach.

- Two advance warning signs should be placed on each approach. The first advance warning sign should be located at a distance where the driver could make a comfortable stop before reaching the crossing. This sign should be located at the stopping sight distance, including adequate allowances for perception and reaction time. The sign should be an advance railroad crossing sign, W10-1. Where the signal used for crossing control

TABLE 4.5.1 REQUIRED DESIGN SIGHT DISTANCES FOR COMBINATIONS OF HIGHWAY AND TRAIN VEHICLE SPEEDS

Train Speed (mph)	Design sight distance for highway speed of: 0 mph	10 mph	15 mph	20 mph	25 mph	30 mph	35 mph	40 mph	45 mph	50 mph	55 mph	60 mph	65 mph	70 mph	75 mph
	Distance Along Railroad From Crossing														
10	162	126	104	94	91	94	96	99	101	107	113	118	125	129	138
15	242	189	156	141	137	141	143	147	152	161	169	176	187	194	207
20	323	252	208	118	182	188	191	197	203	214	226	235	250	258	276
25	404	315	260	235	227	235	238	246	253	267	282	293	312	322	344
30	484	378	312	281	273	281	286	295	303	321	339	352	374	387	414
35	565	441	364	328	318	328	333	342	354	375	395	411	436	452	483
40	645	504	416	376	364	376	382	394	406	428	452	470	500	516	552.
45	725	567	468	422	409	422	429	442	455	482	508	528	561	580	620
50	807	630	520	470	454	470	476	492	506	534	564	586	624	644	688
55	886	694	573	516	500	516	524	540	556	588	621	645	685	710	758
60	967	756	624	562	546	562	572	590	606	642	678	704	748	774	828
65	1049	819	676	610	591	610	619	638	657	695	734	762	810	837	895
70	1129	882	728	656	636	656	666	684	708	750	790	822	872	904	986
75	1210	945	780	704	681	704	714	737	758	803	847	879	935	967	1035
80	1290	1008	832	752	728	752	764	788	812	856	904	940	1000	1032	1104
85	1370	1070	885	799	774	779	812	835	861	910	960	998	1059	1097	1172
90	1450	1134	936	844	818	844	858	884	910	964	1016	1056	1122	1160	1240
95	1533	1200	990	890	865	890	910	935	960	1020	1070	1115	1190	1225	1310
	Distance on Highway from Crossing (ft)														
	20	65	95	125	165	215	270	330	395	470	560	640	745	840	965

NOTE: Wet pavement conditions and a 2.5 second think-reaction time is assumed for these calculations.

1 foot = 0.305 metres; 1 mph = 1.61 km/h

is not visible to the driver at the first advance warning sign, it is desirable to use a double flashing amber light in conjunction with the advance warning sign and interconnected with the signal so that it would operate only when the signal is activated.

- The second advance warning sign should be located at a point comparable to the braking distance at the design speed of the facility or the speed limit, whichever is higher. The message included in this sign should inform the driver as to the type of crossing control device and/or the sight conditions at the crossing. The second advance warning sign should indicate the type of protection at the crossing.

- For non-protected crossings, the second advance warning sign should convey a dynamic message such as "LOOK FOR TRAINS" with arrows pointing in each direction that would be suggestive of the dynamic requirements of the driver to actually look for a train. In situations where visibility of trains is restricted, it is recommended that a combination of signs be used to warn the driver of the existing conditions. The first and uppermost sign of the "tree" should be diamond shaped and include the message "LIMITED VIEW OF TRAINS." The second sign should be a square conveying an advisory speed.

- For crossings controlled by flashing light signals, the second advance warning sign should convey the message, RR SIGNAL AHEAD.

- Consideration should be given to increasing the size of the conventional crossbucks used at non-protected crossings to 1 ft x 6 ft (0.3 x 1.8 m) with construction consisting of white, reflectorized sheeting as a background for black letters. These crossbucks should be well maintained and, further, they should be placed on each side of both approaches to the crossing.

- Spot illumination should be considered at non-protected rail-highway grade crossings. Where provision of such illumination is not feasible, some type of delineation device should be used to inform the driver of the presence of a train on a crossing. It was suggested that some unique delineation system be placed on the opposite side of the track so that it would be visible under the train and would indicate motion by the intermittent passage of the train wheels.

- These recommended traffic control systems are summarized in the following figures:

+ Unsignalized Crossing with Unobstructed View, Figure 4.5.2

+ Unsignalized Crossing with Obstructed View, Figure 4.5.3

4.5.4 TRAFFIC CONTROL SYSTEMS FOR RAILROAD-HIGHWAY GRADE CROSSINGS - MUTCD PROVISIONS

On April 1, 1977, FHWA announced the addition of Part VIII - Traffic Control Systems for Railroad-Highway Grade Crossings to the Manual on Uniform Traffic Control Devices (6). This new addition supplements or supersedes parts of the MUTCD and replaces Bulletin No. 7 "Railroad-Highway Grade Crossing Warning Systems - Recommended Practices" published by the Association of American Railroads. Thus, it should be used by all persons engaged in the planning, design and construction of traffic control systems for railroad-highway grade crossings as the basic reference source.

The new Part VIII of the manual has four basic sections: 1) General Provisions, 2) Signs and Markings, 3) Signals and Gates, and 4) Alternatives Process for the selection of Systems and Devices. Subsequent discussion of each of these four sections will pertain to significant changes particularly as they relate directly to safety.

General Provisions. This section addresses the functions of control systems, the use of standard devices, uniform provisions, provisions for crossing closure, and traffic controls during construction and maintenance. It is recommended specifically that control boxes and equipment housing should be located outside a clear zone of 30 feet where practical.

Signs and Markings. This section presents specific information relative to the design and application of cross bucks, advance warning signs, pavement markings, illumination and other supplementary signs. Of particular significance is the provision for placement of the W10-1 Advance Warning Sign 750 feet (230 m) or more in advance of the crossing in rural areas and 250 feet (75 m) in advance of the crossing in urban areas. Also, there is a provision for use of the NO PASSING ZONE pennant sign on the left side of the roadway at the beginning of the no passing zone on the approach to the crossing. Other details relative to signs and markings are illustrated in Figure 4.5.4.

Signals and Gates. This section presents information relative to flashing light signals, both post-mounted and cantilever-mounted, automatic gates, train detection, traffic signals near grade crossings and recommended operating practices for drivers at grade crossings. A key provision relative to safety design is "breakaway or frangible bases shall not be used for cantilever signal supports. Where conditions warrant, escape areas, attenuators, or properly designed guardrails should be provided."

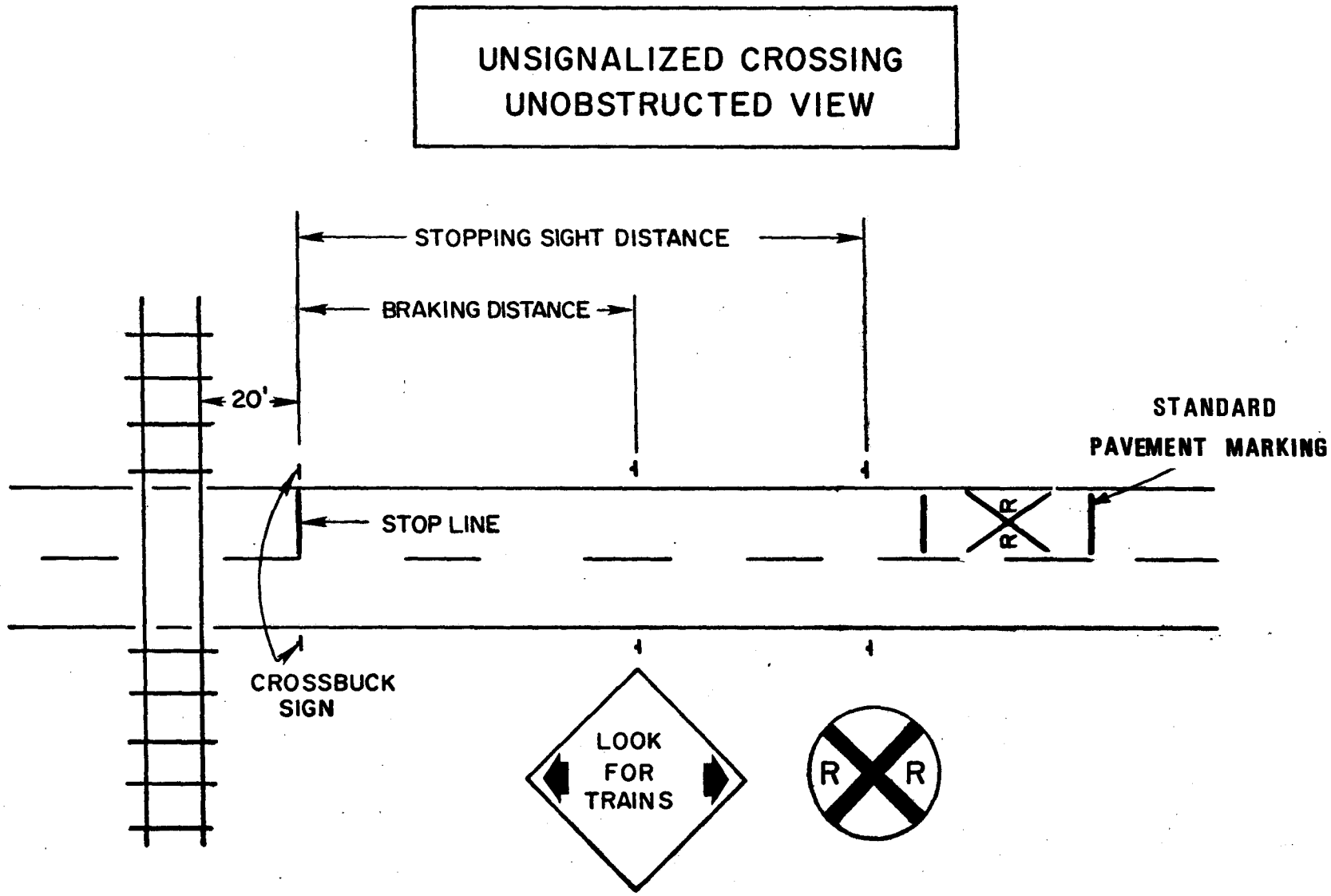


Figure 4.5.2 Alternative Traffic Warning and Control System Recommended in a Research Study for Unsignalized Railroad Grade Crossings with an Unobstructed View of Trains (4)

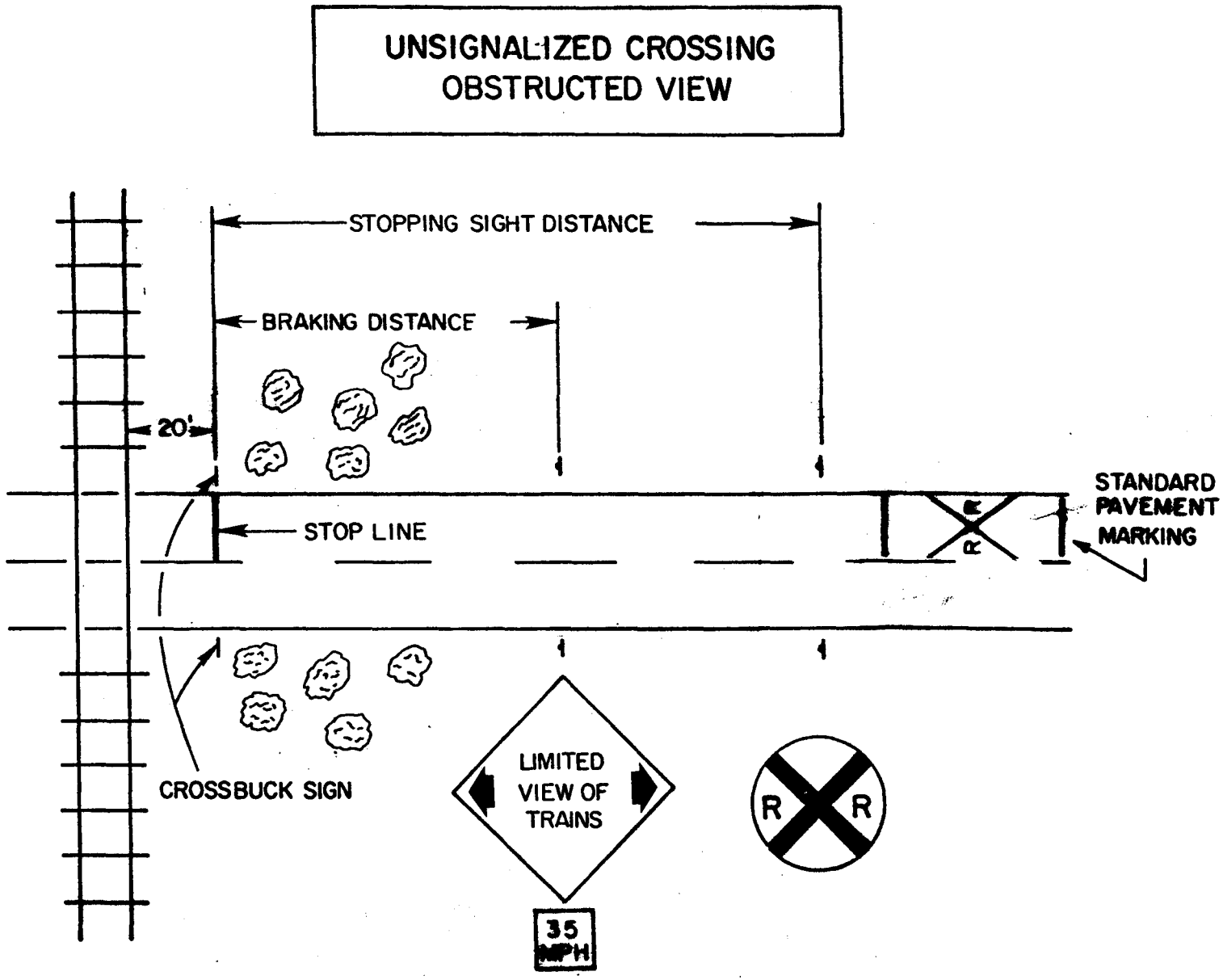


Figure 4.5.3 Alternative Traffic Warning and Control System Recommended in a Research Study for Unsignalized Railroad Grade Crossings with an Obstructed View of Trains (4)

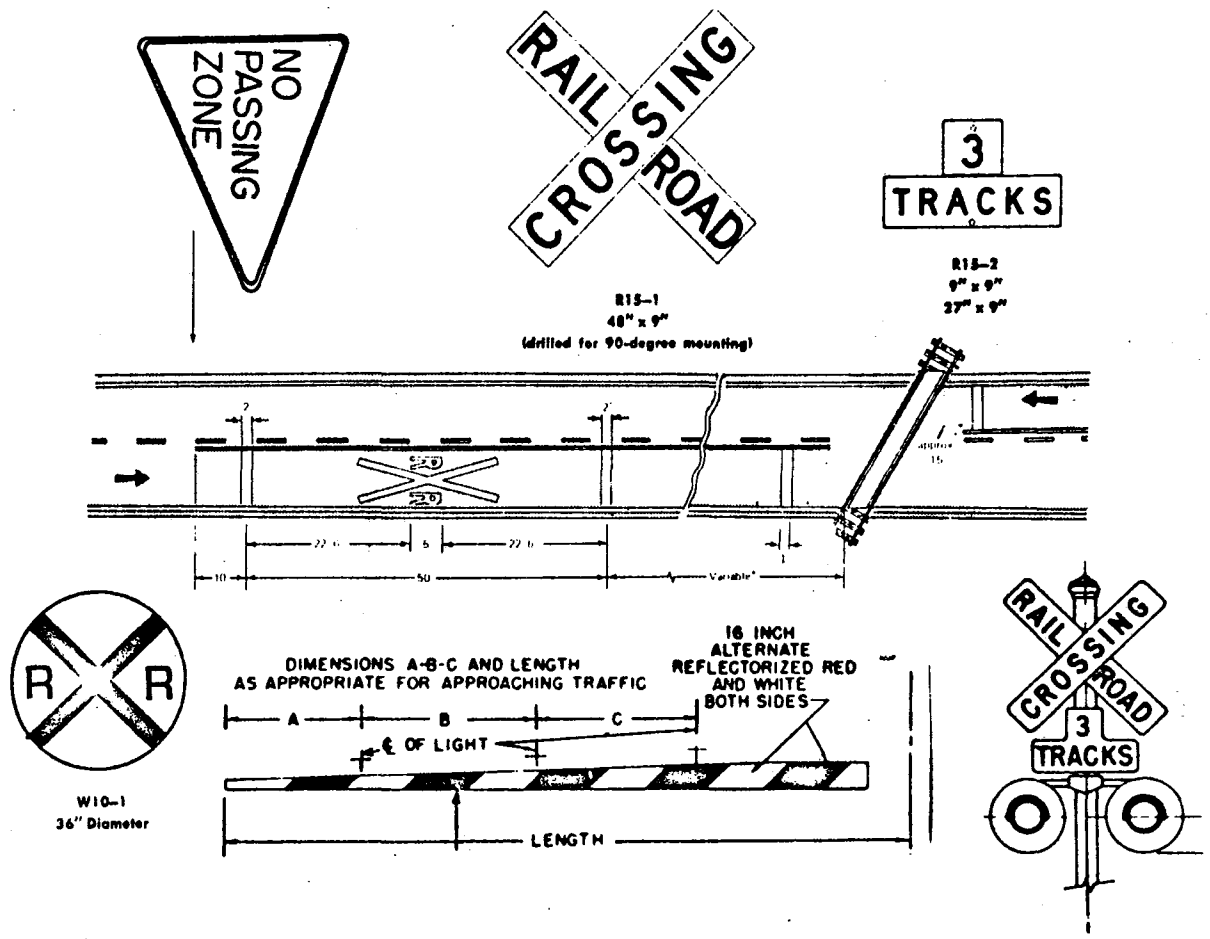


Figure 4.5.4 Traffic Control Devices Used at Railroad Crossings.

Alternatives Process for the Selection of Systems and Devices. This section is quoted in total from the Manual on Uniform Traffic Control Devices for Streets and Highways (6) because of its relevance in this course and because it is completely new material.

"The selection of traffic control devices at a grade crossing is determined by public agencies having jurisdictional responsibility at specific locations. All signs, signals, and pavement markings used in active or passive controls, shall meet requirements of laws, regulations, or standards adopted by State and local agencies, effective at the time of selection.

Active grade crossing traffic control systems range from

1. *Post mounted flashing light signals to*
2. *Automatic gates combined with*
   a. *Post mounted flashing light signals,*
   b. *Cantilever flashing light signals, and*
   c. *Combination of the above*
3. *Any of the foregoing may or may not incoporate a bell.*

Due to the large number of significant variables which must be considered there is no single standard system of active traffic control devices universally applicable for grade crossings. Based on an engineering and traffic investigation, a determination is made whether any active traffic control system is required at a crossing and, if so, what type is appropriate. Before a new or modified grade crossing traffic control system is installed, approval is required from the appropriate agency within a given State.

Numerous hazard index formulas have been developed over a period of 40 years to assess the relative amount of hazard at a

grade crossing based upon some combination of its characteristics but no one such formula has universal acceptance. Nevertheless, use is made of these formulas in the exercise of judgement determination of the system to be installed.

Determination of the need for active traffic control devices should include an analysis of the following factors and their interrelationships:

1. Volume of vehicular traffic - an ADT of less than 1,000 would require other significant warrants.

2. Volume of railroad traffic - less than six trains per day would normally represents light exposure except where passenger train operations exist.

3. Maximum speed of railroad trains - speeds in excess of 50 miles per hour (80 km/hr) in rural areas or 35 miles per hour (56 km/hr) in urban areas deserve careful consideration.

4. Permissable maximum speed of vehicular traffic - speeds in excess of 35 miles per hour (56 km/hr) in rural areas or 25 miles per hour (40 km/hr) in urban areas deserve careful consideration.

5. Volume of pedestrian traffic - pedestrian volume of 150 or more per hour may be a significant determinant.

6. Accident record - occurrance of a train-involved accident within a 3-year period indicates a need for careful analysis.

7. Reduced sight distances - limited view of tracks should be checked for approaching driver reaction.

8. Potential for complete elimination of grade crossings without active traffic control devices - closing lightly used crossings and installing active devices at other more heavily used crossings.

Determination of the need for automatic gates should take into account the following additional factors:

1. Multiple main line railroad tracks.

2. Multiple tracks at or in the vicinity of the crossing which may be occupied by a train or locomotive so as to obscure the movement of another train approaching the crossing.

3. High speed train operation combined with limited sight distance at either single or multiple track crossings.

4. A combination of high speeds and moderately high volumes of highway and railroad passenger or freight traffic.

5. High speed passenger trains, substantial numbers of schoolbuses or trucks carrying hazardous materials, or continuing accident occurrences.

6. Recommendations of a multidisciplinary diagnostic inspection team.

7. Any combination of the conditions listed above.

4.5.5 CROSSING SURFACE CHARACTERISTICS

The railroad grade crossing must be structurally adequate to carry the loadings of both rolling stock and vehicular traffic crossing perpendicularly to the railroad tracks. In essence, the crossing must exhibit some of the characteristics of both of the intersecting roadbeds.

4.5.51 Safety Aspects

The condition of the crossing affects the safety of motorists in two ways:

- At a crossing where the surface is in poor condition, the driver's attention may be diverted as he attempts to find the smoothest crossing path. This detracts from what should be his principal task--checking for the presence of trains.
- A crossing in poor condition also may cause the driver to lose control, thereby striking a fixed object such as a sign or other crossing control appurtenance.

4.5.52 Crossing Surface Types

Various materials have been used for railroad grade crossings. These include:

- Timber
- Bituminous
- Cast-in-place concrete
- Pre-cast concrete slabs
- Rubber panels
- Unconsolidated ballast

Generally speaking, the type of crossing to be used will be determined by:

- Traffic speed and volume
- Train frequency
- Frequency of track repair
- Soil conditions
- Cost

Crossings on high-speed, high-volume train routes should be of the type that can be removed in sections to permit frequent repair of the railroad track. These locations would be best served by a timber or rubber panel type of crossing. On rail lines with less train traffic and fewer track repairs, more permanent, less-easy-to-remove-and-replace crossing types, such as bituminous or cast-in-place concrete, may be used.

4.5.6 SUMMARY

Although the number of railroad grade crossing accidents has remained fairly constant over the years, the accidents which do occur result in considerable injuries and fatalities. Experience indicates that a large number of these accidents are the result of the motorist becoming conditioned to the non-presence of a train at the intersection and, thus, he does not notice an approaching train. This situation is made even worse at crossings where good sight distance of the crossing and approaching train is not available. Therefore, the installation of adequate signs, markings, and flashing light signals is needed to attract the motorist's attention to the crossing location. The condition of the crossing surface also must be maintained if the driver is not to be distracted or lose control of his vehicle due to the rough surface.

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TOPIC 4 SESSION 6

SAFETY DESIGN IN CONSTRUCTION AND MAINTENANCE OPERATIONS

Objectives:

1. The participant should understand the importance of designing detours and temporary roadways to the standards of the approach roadway,

2. Further, he will be cognizant of the need for special safety devices, signs and markings to insure the safety of construction and maintenance personnel and be able to select appropriate safety devices for such situations, and

3. Be capable of planning and scheduling work periods which will minimize the hazard to construction and maintenance personnel including the necessary public information program.

4.6.1 INTRODUCTION

The majority of roadway construction projects are within existing right-of-way lines. Even roadways constructed on a new alignment must somewhere be tied into the old roadway. Because alternate routes do not always exist there is a need for the movement of traffic over a roadway while it is under construction. Also, temporary blockages of one or more lanes on a street or highway may be necessary for maintenance of the road and its appurtenances. Thus, it becomes the responsibility of the public agency in charge to provide the driver with a travel way through the maintenance or construction area which provides at least a minimum level of operating efficiency.

In this session we will review the various classifications of construction and maintenance areas, discuss the design criteria for detours for these areas, and suggest methods of traffic control, personnel protection, and maintenance scheduling which have been used successfully by other public agencies.

4.6.2 CONSTRUCTION AND MAINTENANCE AREA TYPES

The circumstances under which construction and maintenance activities are conducted vary according to the surrounding area. Similar types of activities on rural or urban roadways require different design criteria and traffic control methods. In addition, the roadway configuration, such as two-lane as opposed to multi-lane, may dictate alternate ways of handling traffic. This is the basis for the following four cases of maintenance and construction activities.

4.6.21 Case 1: Urban Multi-lane Facilities

This case includes freeways and arterial streets in urban areas. Normal traffic flow may be disrupted for activities such as roadway resurfacing or widening, underground utility work, or maintenance of signs, signals and other roadside appurtenances. Because facilities of this type are likely to exhibit relatively high traffic volumes, maintaining adequate capacity and a reasonable level of service become a primary concern. Traffic may need to be diverted to the frontage road or detoured over other major arterials or work activities may have to be prohibited during peak traffic periods. During non-peak periods when traffic is flowing more freely, the speed differential between normal traffic and traffic in work areas may become more critical.

4.6.22 Case 2: Urban Two-lane Facilities

Roadways of this type include residential streets and other relatively low volume city streets. Work activities may include traffic control device and street lighting maintenance as well as other utility installation and repair. A major concern is the provision of access to abutting property during street renovation work. Capacity and speed differential problems are relatively minor.

4.6.23 Case 3: Rural Multi-lane Facilities

Highways of this type include freeways and other high-type facilities on which a number of activities may occur. These include maintenance activities such as pavement marking or sign placement or more long-term repairs such as resurfacing, joint repair or bridge deck work. Construction activities would include addition of another lane or regrading. Capacity is not usually severely impaired in rural areas by activities of this type and because level of service remains fairly high, the speed of vehicles through the work area becomes the major problem. On divided facilities where one-half of the roadway is closed and the other is converted to two-way traffic, proper markings and signing must be used to prevent wrong-way movements.

4.6.24 Case 4: Rural Two-lane Facilities

This highway category contains the majority of highway mileage nationwide. A number of work activities may occur on this type of facility including road surface and shoulder repair, bridge widening and overall reconstruction of the roadway. In many cases adequate alternate routes for the diversion of traffic are not present. Thus, while a portion of the roadway is being worked on, traffic is handled through a one-lane section by alternating the direction of traffic flow. Extreme care must be utilized in this situation to separate conflicting traffic streams. As with all rural highways controlling the speed of vehicles entering the work area becomes a major consideration.

4.6.3 DESIGN CRITERIA

A driver on a street or highway builds up a certain expectancy of what lies ahead according to the roadway he has previously encountered. Inclusion of a piece of highway construction equipment, a construction worker, or some obvious discontinuity in roadway characteristic will violate the driver's expectancy. This violation of expectancy is not in itself critical if it does not force the driver into making an erratic or rash maneuver. In order to lessen the effect of such a change we should attempt to accomplish two things. These are:

- Warn the driver as soon as possible
- Make the change in a smooth and uniform manner

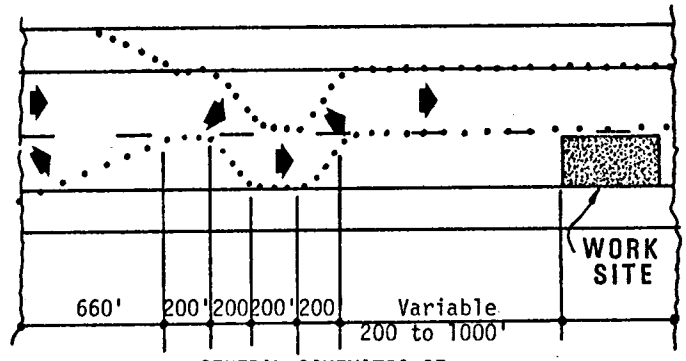
These objectives can best be met by observing the following design criteria.

4.6.31 Design Speed

As a general rule, the roadway through the work area should be designed for not more than a 10-15 mph (16-24 km/h) reduction in speed from the existing roadway. Speed reductions greater than this require more transition distance and place greater demands on traffic control devices.

Reducing speed to a safe operational level approaching construction and maintenance sites is always a problem. The Iowa Department of Transportation has developed a technique for achieving reduced speeds which employs a serpentine path layed out with cones on the approach to the site. This system has been dubbed the "Iowa Weave."

Traffic control for the Iowa Weave begins two miles from the site and continues the typical one mile, one-half mile, 1000 feet and 500 feet warning signs. Two approach lanes are reduced to a single lane (the lane not being closed). The path into the site then involves two lane changes. This system is presented in the sketch below.



GENERAL SCHEMATIC OF THE IOWA WEAVE

Effective speed reductions from 55 mph to 30 or 35 mph have been reported with this system(10).

4.6.32 Roadway Geometry

Vertical and horizontal curvature are primarily governed by the design speed selected for the work area. The cross section of the roadway should be similar to the rest of the roadway in order to provide for continuity through the work area. It is most important that the operational effects of roadway geometry fall within the acceptable range of requirements for satisfactory driving expectancy.

4.6.33 Sight Distance

The driver must have adequate sight distance of the roadway if he is to adequately react to his surroundings and other vehicles. As his speed increases this sight distance must also increase proportionally. For this reason, sight distance may be described in terms of time rather than distance. This measure known as "viewing time" should be in the range of 10 to 15 seconds.

4.6.34 Advance Planning

Early in planning stages of a construction and/or maintenance operation of substantial magnitude, the traffic control plan must be worked out. This plan consists of three basic features:

1. Planning the diversion routes;

2. Selecting construction zone length and activity sequence consistent with reasonable operational criteria;

3. Executing a public information program to advise the motorist of the impending activity.

The planning of diversion routes will identify critical points in the system and suggest an optimal project length consistent with safety. Seldom should the site on high volume facilities exceed two miles and frequently a length of one-half mile will be desirable. Longer construction jobs will necessitate staged activities to insure motorist safety. Construction on both sides of a stream of traffic at one time should never be permitted.

The public information program will most certainly include mass media coverage; however, portable roadside signs at the job site for a period of one to two weeks prior to the beginning of construction has proven most successful in alerting motorists to the impending change.

4.6.35 Storage Areas

Construction or maintenance operations typically have some storage requirement for materials and/or debris. Frequently, space provided for a safe roadside becomes the most convenient location for such storage. While the necessity for on-site storage is recognized, great care in selection of the site must be exercised to insure safety. The following general guidelines are offered in this regard:

1. Avoid gore areas and roadside areas adjacent to ramps.

2. Avoid sites adjacent to horizontal curves.

3. Avoid sites where the geometry is complex or where the driver is fully occupied with the driving task.

4. Provide sufficient separation between the traveled lanes and the storage area. A minimum distance of 20 feet on high-speed roadways and 14 feet on uncurbed lower speed roadways are suggested.

4.6.4 TRAFFIC CONTROL

The control of traffic through construction and maintenance areas is similar to normal traffic control except that there is a greater need to command the driver's attention and warn him of danger. Drivers are conditioned by normally smooth traffic flow to not expect surprises such as sudden changes in alignment, construction equipment, or reductions in operating speed. Thus the primary task of traffic control through work areas is to select the proper devices to communicate with the driver under both day and night conditions.

4.6.41 Control Device Selection Criteria

The type and extent of traffic control depends to a considerable degree on the following factors:

The classification of the roadway and the area in which the work is located. High-speed rural facilities will require considerably more advance warning due primarily to higher operating speeds on these types of highways. On two-lane sections of roadway where one lane is closed and the other lane is operated as an alternately reversible lane, extra care must be paid by traffic control personnel to avoid the simultaneous movement of conflicting traffic streams. On urban streets, traffic control may be complicated by the proximity of intersecting streets, the need to provide access to abutting property, and in general the many more possible movements desired by motorists.

The type of work being performed. Traffic control for large-scale, semi-permanent construction jobs such as highway grading and paving, bridge replacement, or storm sewer installation usually require a number of permanent type signs, barricades and other control devices which are installed when construction is begun and removed when the job is finished. Other less permanent activities such as pavement overlay jobs or shoulder rehabilitation may require traffic control devices which are moved periodically to keep up with the actual work that is going on. Even more portable must be the traffic control devices used with maintenance activities where the work may be completed in a matter of a few hours or less.

4.6.42 Credibility

To be most effective, traffic control devices used in conjunction with construction and maintenance activities must reliably convey the message necessary to attain the desired response from the driver. This means that the message must be accurate and current -- in other words, *credible*. Accuracy involves presenting the correct message to the driver. In general, advising him of a particular hazard at a specified location is preferable to making a general statement. For example, TRUCKS HAULING, NEXT 2 MILES is preferable to SLOW.

Staying current with the work activities also improves credibility. How many times have we seen ROAD CONSTRUCTION AHEAD or MEN WORKING signs when there is no evidence of any construction activity? Another example is the posting of a reduced speed limit throughout the entire length of a highway resurfacing job when conditions on only a short section present a hazard to traffic. Signs that are not applicable during non-work times should be removed or covered with an opaque material.

4.6.43 Maintenance of Control Devices

Traffic control devices are usually installed at the beginning of a construction or maintenance job to meet certain state or local requirements. As the work progresses, little is done to keep these devices current and in good repair. Damage by construction equipment, obliteration of devices by road spray, and accidental respositioning reduce the effectiveness of traffic control devices. Also, certain devices such as pavement markings must be replaced at the end of the work day if they have been obliterated by construction during the day. Removal of non-functional marking immediately is also important to safe operation.

4.6.44 Use of Control Devices

The use of traffic control devices for the movement of traffic through and around road or street construction, maintenance, and utility work is described in detail in Part VI of the 1971 Manual on Uniform Traffic Control Devices (MUTCD). This information covers the following subjects: signs, cones, flagging, and pavement markings. Some of the practices in using these devices which have proven successful are discussed in the following sections.

Signing. Many signs normally used elsewhere will also find application for signing construction and maintenance operations. Other signs with messages unique to these types of activities are also used. Warning signs added in these work areas should be diamond shaped and contain a black text on an orange background. Existing warning signs and other signs in the work area should be allowed to remain if their use is still required.

The location of the first warning sign depends upon the distance at which the sign must be read in order for the driver to make the required maneuver. This distance is in turn related to the driver's approach speed, his perception-reaction time, and the speed at which we want him to enter the work area.

Cones, Barricades and Lights. Once the driver's speed has been reduced to a safe operating speed, the next concern is guiding him through the work area. This is usually accomplished through the use of cones and barricades as illustrated in Part VI of the MUTCD. In general, these devices should be orange in color with white stripes except for traffic cones which are solid orange. Barricades, either post-mounted or portable, are used for longer term work activities while cones are used primarily for short time, maintenance type activities. Vertical panels, 6 to 8 inches (15 to 20 cm) wide and 24 inches (61 m) in height striped and reflectorized in a manner similar to barricades are also used for channelization.

More protection to motorists and workers can be provided by using a safety type barrier. One State uses metal beam guard rail bolted to oil barriers which are partial filled bags of sand for stability. In other instances, sections of concrete median barrier have been used to temporarily separate two-way traffic. The sections of concrete barrier have the potential for reuse at other construction sites.

One of the primary uses of cones and barricades is the transition required to close one or more lanes. The MUTCD suggests that the transition taper be determined as the product of the 85th percentile speed and the width of the transition. This value should be retained as the minimum design for work area transitions. A desirable design would be twice this value as shown in Table 4.6.1.

TABLE 4.6.1 SUGGESTED TRANSITION TAPER LENGTH FOR 12-FT (3.6 M) LANE CONSTRUCTION AREAS

Design Speed mph (km/h)	Minimum Transition Length ft (m)	Desirable Transition Length ft (m)
70 (113)	840 (256)	1680 (512)
60 (97)	720 (219)	1440 (439)
55 (143)	660 (201)	1320 (402)
50 (80)	600 (183)	1200 (365)
40 (64)	480 (146)	960 (293)

In addition to reflectorized cones and barricades, safety through work areas at night may also be improved by the use of various types of lighting devices. Portable floodlights may be used to illuminate particularly hazardous locations, especially those where double work shifts are used and construction vehicles must cross the roadway. Another method which has been tried involves building the lighting system before doing the rest of the construction. This is readily applicable to interchanges where high mast lighting is to be used.

Hazard identification beacons are flashing yellow lights which are typically mounted above a freeway lane(s) that ends or above a sign to draw attention to the sign. Beacons of this type are used primarily in semi-permanent locations.

Barricade warning lights are portable, lens-directed, enclosed lights emitting a yellow light. They may be used in either the steady burn or flashing mode according to their type as defined in the MUTCD and

depending on their specific application. Due to their portability and reasonable cost, barricade warning lights are useful supplements to reflectorized barricades. Where horizontal and vertical curves are involved, random flashing lights used for delineation may actually tend to only confuse the driver more. In this case reflectorized delineators and pavement markings may be less disconcerting to the driver. The use of timber barricades has caused much confusion in recent months. The term does not apply to any barricade made of wood but rather to a specific design composed of a large timber member as a base with a barricade constructed on top of it. Accident experience with this type of design has proven to be unsafe for the motoring public and it is no longer acceptable for use. Any of the standard barricades presented in the MUTCD can be used; however, care should be exercised to insure that the design characteristics of the barricade conform exactly to the MUTCD Standard. Additional members, name plates or similar materials may not be used.

The use of sand bags to provide stability for Type I or Type II barricades is common. When used, the ballast elements must be on the lower members of the barricade system to avoid the possibility of it going through the vehicle's windshield on impact.

Flagging and Traffic Control Practices. Most "on the road" types of maintenance and construction require the use of flag personnel. These people serve the following functions:

- Provide a dynamic warning to traffic
- In some cases alternate the direction of traffic on a one-way section
- Close a section of roadway during dangerous operations such as blasting
- Direct drivers through what may be a somewhat confusing course
- Answer questions about conditions and possible delays

The exact type of flagging activities needed in work ares depends mainly on the roadway location and the type of activity. For example, the direction of traffic on a one-way section may be alternated by giving a metal hoop or other type of baton to the last driver in a platoon who in turn gives it to the flag person at the other end, thereby signifying the section is clear. An alternative is a flag signal between the individuals doing the flagging. Where one end of the section is not visible from the other, two-way radios should be used.

Pace vehicles may also be used to lead traffic through a complicated construction area. Pace vehicles also can be used to create gaps in the traffic during which time construction activities may take place on the roadway main lanes.

Pavement Markings. All construction or maintenance areas which must be maintained overnight should be brought up to the minimum level of traffic control prior to shutting down the job each night. For overlay and seal coat operations, one state requires the contractor to replace all pavement markings at the end of each day's operation.

The pavement markings on detours must include the full complement of markings--the double yellow centerline and both edgelines. In addition, the use of raised pavement markings between the two elements of the centerline will bring greater attention to the hazard. The markings should begin several hundred feet in advance of the beginning of the detour to insure that the driver has an opportunity to adapt to them prior to entering the detour.

When construction is completed, all pavement markings which do not match the finished alignment should be completely removed. Drivers tend to follow the centerline and have been known to drive into a roadside ditch following an old, centerline. Sand blasting, carbide tip grinding or burning can be used to eliminate undesired markings. Care must be exercised to insure that the removal of the marking does not result in the appearance of a line at night. The removal of a line will alter the reflectance properties, and the area may appear very bright against the darker pavement.

4.6.5 HAZARD REDUCTION

In addition to the proper design of roadways in work areas and the use of adequate traffic control procedures, two other areas also exist for the improvement of safety in maintenance and construction areas. These deal with the protection of personnel in the work area and the scheduling of work activities to reduce traffic congestion.

4.6.51 Protection of Maintenance Personnel

In the Roadway. In spite of the traffic control, the use of safety jackets, the use of flagmen, and a great many other safety-oriented techniques designed to protect maintenance personnel working in the roadway, accidents still occur. This is particularly true on high-volume, high-speed facilities. Where maintenance personnel must work under such conditions for extended periods of time, the use of some type of inertia barrier can be effectively utilized. The design of inertia barriers will be discussed in another section of these notes.

Another protection device that can be effectively used on high-volume, high-speed facilities to protect personnel working in

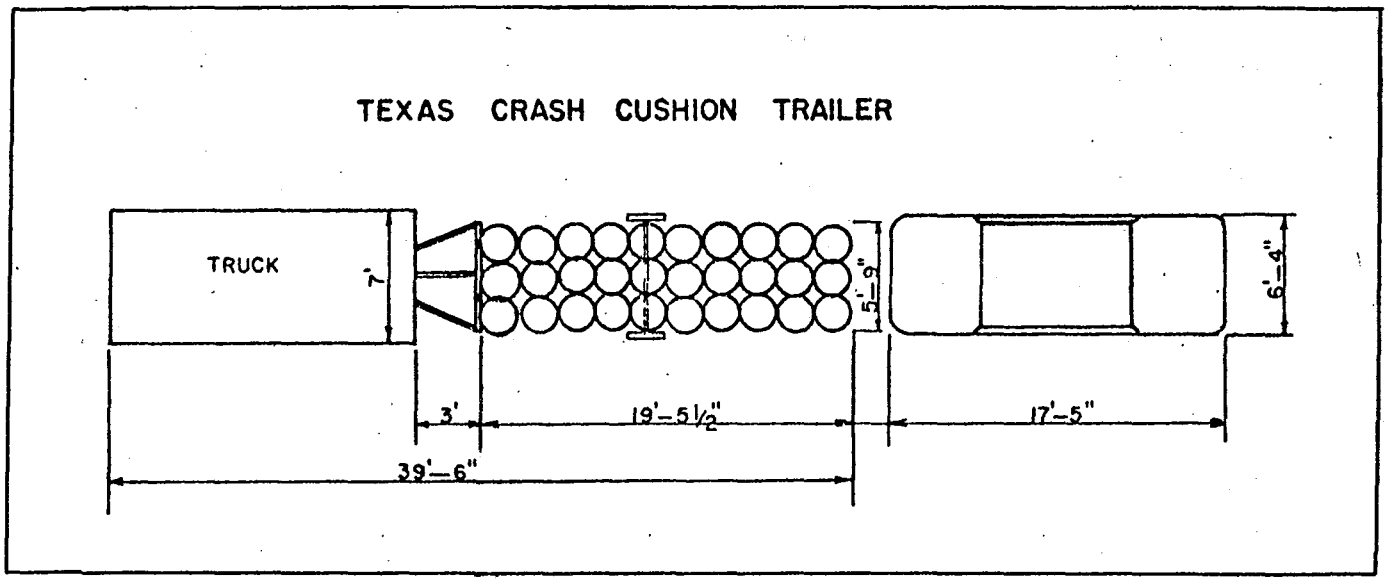


Figure 4.6.2 Crash Cushion Trailer.

the roadway is the portable crash cushion. This unit consists of crash cushions mounted on a trailer which is towed behind a large truck. The design principle of the portable crash cushion is similar to that for the barrel crash cushion as discussed in Session 5.2 of these notes. The truck is placed just in advance of the work area, and the brakes are set to serve as the backup for the crash cushion. Errant vehicles penetrating the work site will impact the crash cushion rather than maintenance personnel. This type of crash cushion is illustrated in Figure 4.6.2.

The portable barrel crash cushion was developed by the Texas Department of Highways and Public Transportation, and is not known to be manufactured and sold commercially. There are, however, portable devices utilizing other energy absorption systems that are commercially available.

Moving Maintenance. Moving maintenance operations require special consideration. In practice the use of arrow boards mounted on a truck which trails the actual operation has proven most successful on high-volume facilities. The use of a portable crash cushion on the trailing vehicle is an added safety feature for the errant driver. Certainly slow-moving vehicle warning panels should be in place on the rear of the vehicle, and vehicles should be equipped with a rotating beacon.

Roadside Maintenance. Advance warning of roadside maintenance operations, i.e., those that move along the roadway, often have the advance notification much too far in advance of the operation. The advance warning is noted and the driver is alerted to possible conflict with maintenance vehicles. However, after a half-mile or more with no indication of the maintenance operation, the driver resumes his normal driving pattern. Advance warning for roadside maintenance operations should be maintained within one-half mile of the actual operation at all times.

Another common accident pattern for maintenance personnel involves maintaining grass on too steep a side slope. While it is commonly recognized that a tractor cannot safely operate on a 2:1 slope, the hazard of a 3:1 slope is not as well recognized. A tractor operating on a 3:1 slope will have wheel drop into an erosion rut and roll the tractor over. A recent study in Texas indicates that a substantial number of accidents involving maintenance were of the type described above. Maintenance vehicles operating on slopes steeper than 4:1 should be equipped with roll-bars and provision to prevent rollover should be considered. A better solution is to maintain these slopes with especially-designed vehicles with telescoping arms.

4.6.52 Scheduling of Maintenance Activities

The accident potential on high-volume roadways can be reduced by judicious scheduling of maintenance operations. On multi-lane roadways the basis of scheduling should be a level of service analysis indicating that the lanes which remain open will be able to accommodate the flows, desirably at level of service C. This would permit some degree of accommodation of the turbulence which is always associated with lane blockage.

Maintenance work on urban freeways frequently involves diverting traffic onto other facilities. Corridor analysis must be used to determine if the corridor composed of other freeways, arterials and collector streets is capable of handling the total traffic load. If it is not, work schedules may need to be adjusted or stage construction may be required to allow more of the traffic to remain on the primary roadway.

The use of planned diversion routes can result in a reduction of the traffic demand in the work area, if the driver is made aware of the problem and the alternative routes. The posting of a temporary sign at the work site several days in advance of the scheduled date should make the driver aware of the situation. The sign should advise the motorist of the date, time and degree of blockage which will result. Also, newspaper advertising indicating the situation and suggesting alternative routes can be effective in combination with the signing of the site. Commercial radio traffic situation announcements can be utilized on the day of the blockage to remind the motorist to take alternative routes.

Some cities have ordinances prohibiting all maintenance activities on city streets and alleys during the peak periods except in emergency situations. In addition, many cities require that a work permit be obtained from the city traffic engineer before any work activities are commenced within the right-of-way lines.

4.6.6 SUMMARY

Practically every maintenance or construction operation may be temporary or permanent depending on how we view it. Individuals responsible for traffic operations through these areas must realize that the situation, whether it exists for an hour or a year, will affect a number of motorists who are probably unfamiliar with the dangers involved in traveling through the area. No matter how temporary these situations may be, it is still a public responsibility to protect the safety of each motorist.

In addition to providing for the safety of the motorist, personnel in the work area must also be protected by the use of proper signing, safety vests and other equipment. Other devices such as inertia barriers and the truck-mounted barrel cushion have also shown good potential for injury reduction.

This session may be summarized by the following techniques quoted from the AASHTO publication, "Highway Design and Operational Practices Related to Highway Safety," Second Edition, 1974:

1. Adequate attention to traffic handling during planning and design stages.
2. Detour alignment and surface that will allow traffic to pass smoothly around the work zones.
3. Long tapers for lane drops or squeezing traffic for transition areas.
4. Adequate maintenance of all traffic control devices and pavement markings for complete day and night effectiveness, including use of temporary marking materials which can be removed when traffic patterns change.
5. Use of roadway illumination and warning lights as necessary. Use of steady burning lights in preference to flashing lights to delineate the travel path through or around a work area. The very short "on" time of flashing lights does not enable motorists to focus on the light and to make a depth perception estimate. Flashers should be limited to marking spot hazards or occasional short lengths of straight-line delineation.
6. Cones, delineators, painted drums or lightweight barricades as a means of channelizating traffic.
7. Removal of signs and markings from job site when they are no longer needed.
8. Use of shadow trucks to follow slow-moving maintenance equipment such as stripers or sweepers.
9. Use of transports to move slow-moving equipment between worksites.
10. Removal of contractor's equipment completely off the roadways and shoulders at night, on weekends, and whenever equipment is not in operation.
11. Holding conferences and job site discussions well off the traveled way and never in the median or on a narrow structure.
12. Protection for personnel exposed to traffic hazards during the performance of their duties, including use of safety vests by all workmen.
13. Possible limitation of working hours to off-peak periods.
14. Parking of workers' private cars well off the shoulders.

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TOPIC 4 SESSION 7
SIGNING AND DELINEATION

Objectives:

The participant should be able to:

1. Recognize the importance of sign placement and design on safe operation,

2. Select sign supports which will provide a maximum of safety for the motoring public,

3. Utilize the latest design criteria in the design of breakaway and yielding devices,

4. Select and apply delineation systems in a manner which will achieve a maximum of safety for the monetary investment.

4.7.1 INTRODUCTION

4.7.11 The Importance of Good Driver Communications

A fully informed driver, given sufficient time to respond to a situation, will rarely make a serious driving error. Conversely, inadequate information or time to respond to the situation results in a high probability of an erratic maneuver and high potential for an accident. Thus, communication with the driver is essential to a safe driving environment.

4.7.12 The Hierarchy of Driver Information Needs

As discussed in Topic 3, Session 1, there are three general levels of driver information needs. These are:

1. Positional information

2. Situational information

3. Navigational information

Traffic signs fall into the latter two categories. This suggests that careful selection of the location of signs so as to offer the greatest potential for complete transfer of the information is necessary. The key factor is the determination that situational and navigational information are lower on the driver's priority list than is positional information.

4.7.13 The Basic Signing Principles

The basic signing principle is locating the needed information in advance of the point where action is required. The distance required will depend on several factors, the dominant one being sign size. The larger the letter size, the closer to the action point the sign can be located. Another fundamental principle is that of sign redundancy. This is particularly true for roadside signs since the driver may not be able to read any given sign due to blockage by a larger vehicle or a situation requiring the full attention of the motorist. Repetition of the sign is a commonly recommended solution to this problem.

A third basic principle is not to locate signs in areas of high driver work load. The location of the device must be close enough to the action point to allow the driver to associate with it, and far enough away to permit adequate time to respond. Often, in a desire to insure adequate response time, signs are located at points where they will not be read (i.e., on the inside of a tight horizontal curve). Failure to fully consider the situation in which the device must perform or adherance to rigid standards regardless of the location of the sign can result in a non-functional traffic control device.

The basic signing principles are:

1. Place the device in advance of the action point;

2. Provide sufficient time to respond appropriately (either by increasing sign size or locating it farther upstream or both);

3. Provide redundant information;

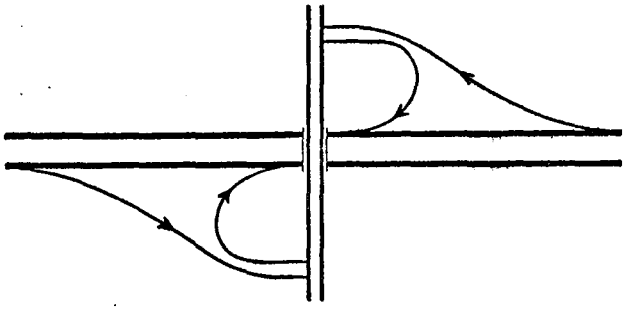
4. Avoid areas of high driver work load.

The safety treatments for signing and delineation must be executed within these basic principles. The traffic control functions must not be comprised as the benefits gained through hardware redesign may well be lost due to driver indecision and confusion.

4.7.14 Driving Expectancy In Signing

Background Color For Freeway Signing. The green background for directional signing on freeways is a dominant feature in the driver's pattern of scanning for information. The use of other colors even on a temporary basis can create a tendency to overlook the entire assembly. This is probably the result of the consequences of failing to exit at the proper point resulting in considerable increase in trip length.

Interchange Element Location. One of the more significant points of confusion regarding signing deals with the identification of unusual maneuver points. For example, a "folded diamond" or partial cloverleaf that requires a left turn in advance of the structure in order to go to the right on the intersecting roadway requires a great deal of preparatory information in order to be driven in a natural manner by the driver.



The expected manuever is to turn right to go right, and advance information must prepare the driver to do the unexpected.

Sign Priority Inversion on Freeways. The priority of traffic control devices normally assuming (i.e., (1) Regulatory, (2) Warning, and (3) Guide, is reversed on modern freeways. Directional signing is the primary concern of the driver following information on lane position and other traffic. Warning and regulatory devices take on a much lower priority. One driver's comments were as follows:

"Those little black and white signs tell you anything: Don't Throw Litter on the Highway, Don't Park on the Shoulders, almost anything. The thing that is important to me is which lane I have to be in, in order to get where I want to go."

This inversion of priority indicates that black and white regulatory signs on freeways are of questionable effectiveness particularly in the vicinity of an overhead sign.

Roadway Division Points. Locations where the roadway divides frequently are not balanced. More lanes go to one destination than to the others. As a general rule, the driver expects the roadway with the larger number of lanes to be the most important roadway. For example, if an Interstate highway and state highway have a directional interchange where the state highway required the greater number of lanes, the driver's normal expectation is violated, and a special effort in the advance signing is required to alter this expectation effectively.

Left Hand Exits. Left hand exits from a divided highway violate the normal exit to the right expectation. This situation requires special treatment such as addition of the "LEFT" complement to the advance signing. This is particularly true when all signing is located on the right side.

Lane Drops. The dropping of a lane that has been continuous for some distance creates many problems for motorists. The yellow tab "EXIT ONLY" has reduced this problem greatly when combined with complementary pavement markings.

The key issue in designing to meet the driver's expectation is anticipating how the driver will react to a given situation. When the normal reaction differs from the maneuver the designer intends, extraordinary traffic control on information systems must be used to correct the driver's expectations.

4.7.2 SAFETY CONSIDERATIONS IN SIGN LOCATION

4.7.21 Overhead Signs

Selecting Safe Locations. Consistent with the highest priority treatment, any sign should first be reviewed to determine that it is indeed an essential part of the traffic control system. If not, it should be removed. Assuming that it is needed, then the location should be one which affords a maximum of safety to the motoring public. The subsequent sections present guidelines for achieving maximum safety from signing and delineation systems.

The priorities for sign location are:

1. On an existing bridge or overpassing structure

2. Behind or integrated with an existing barrier used to protect the errant driver from a critical area

3. Right side of the roadway in area of low vehicle encroachment

4. At the roadside near the pavement edge with adequate redirection or attenuation treatments.

It should be noted that the fourth alternative carries with it substantial safety treatment costs which tend to offset the additional costs of locating the sign support well away from the pavement edge. The design of redirection and attenuation devices is discussed in subsequent sections and therefore will not be discussed at this point.

Overhead signs are an essential part of the freeway driver communications system (1).

Supports for these signs should not be located in areas of high vehicle encroachment. For example, the overhead directional signing for a freeway exit ramp should not be located in the gore area (1). The support is located in an area of high encroachment, and the information is beyond the action point. Observation of driver behavior (2) suggests that drivers tend to execute necessary maneuvers at the location of the overhead sign rather than in advance of it. For this reason, the overhead sign installation should be at or in advance of the theoretical gore as indicated in Figure 4.7.1.

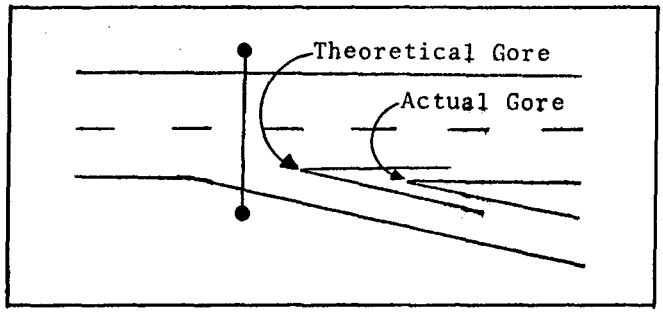


Figure 4.7.1 Location of Overhead Signs for Exit Ramps

Frequently, the information contained on the overhead bridge can be divided into two parts. When this is possible, a cantilever overhead sign from the right side of the roadway can convey effectively the ramp information to the driver.

The tendency to execute driving maneuvers at the location of the overhead sign suggests that horizontal curvature can substantially alter the effectiveness of overhead directional signing since the driver is unable to associate the sign message with the roadway lanes until the vehicle is in close proximity of the sign location. Thus, where overhead signs are located on horizontal curves, consideration should be given to moving the sign location upstream a sufficient distance to permit adequate time to complete the necessary maneuvers prior to reaching the critical point in the roadway. This is particularly true for unusual geometric situations such as left hand exits or double exit designs (i.e., cloverleaf interchange).

The supports for overhead signs, under certain circumstances, can be made breakaway. The requirements are twofold:

1. Three or more supports required for stability, and

2. The support must be separated by sufficient distance to insure that only one support can be struck by an errant vehicle. Design criteria for breakaway overhead signs are presented by Olson (3).

4.7.22 Large Roadside Signs - Multiple Post Supports

Large roadside signs require substantial supports to insure against failure due to wind loading. The most fundamental treatment for signs that cannot be removed or relocated is to make the supports breakaway.

Breakaway Metal Supports. The basic concept of the breakaway sign support is to provide a structure that will resist wind loads yet fail, at preselected locations, when struck by a vehicle. The loading conditions for which the support must be designed are as shown in Figure 4.7.2. Three critical connection locations and their required characteristics under each loading condition are indicated. When these connections are properly designed, the support will be stable and will possess the breakaway characteristics, when struck by a vehicle, as shown in Figure 4.7.3.

The Breakaway Hinge.

Wind Loading. The design of a breakaway sign support and hinge is based on its capacity to resist imposed wind loads. An example will be presented to illustrate the approach. Assume that a 16 by 8 ft (4.8 by 2.4 m) roadside sign panel is desired as shown in Figure 4.7.4.

To compute the wind load, the formula from the 1975 AASHO specifications will be used. The formula is:

$$p = 0.00256\ (1.3V)^2\ C_d\ C_h$$

where: p = Wind pressure in pounds per square foot
v = Wind speed from map (MPH), 50-year Mean Recurrence Interval
C_d = Drag coefficient of sign (see Table 4.7.1)
C_h = Coefficient for height above ground measured to the centroid of the loaded area (see Table 4.7.1)

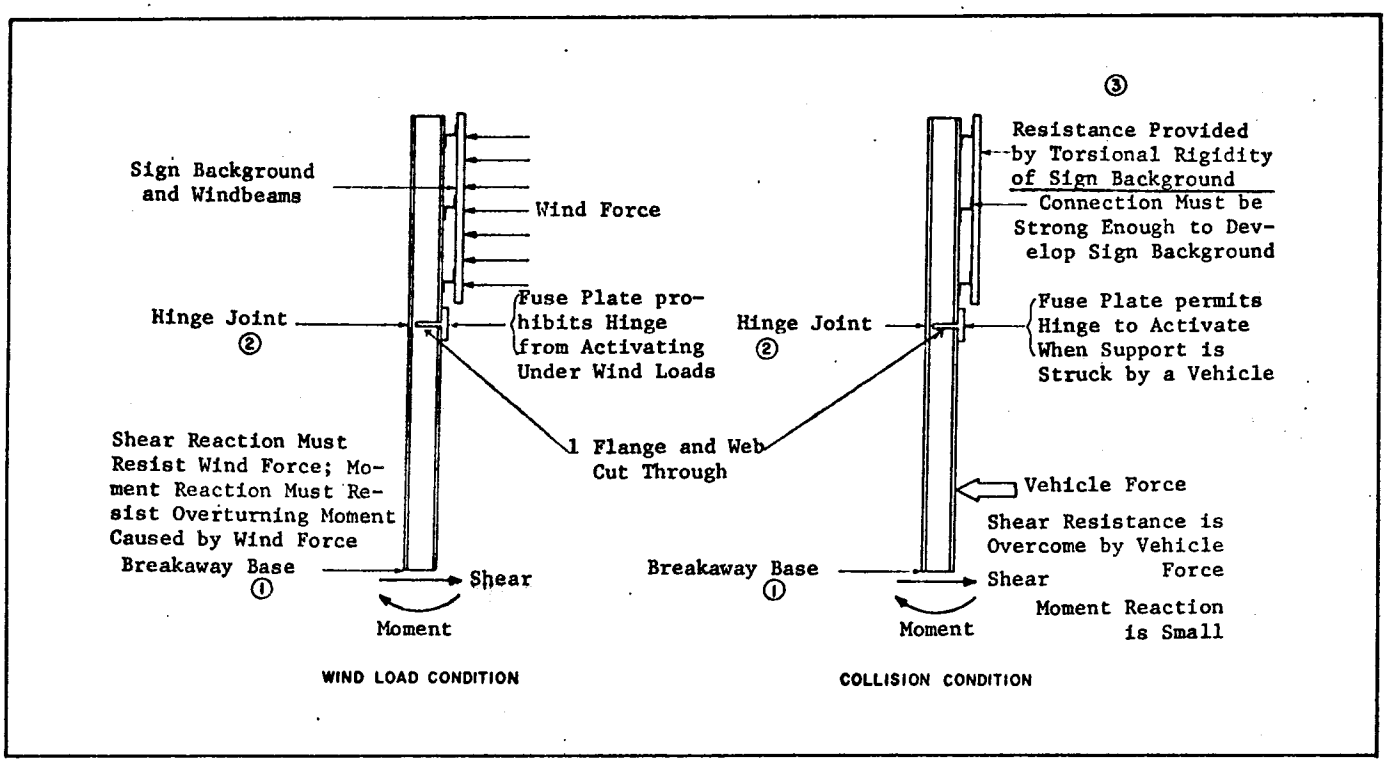


Figure 4.7.2. Breakaway Sign Loading Conditions

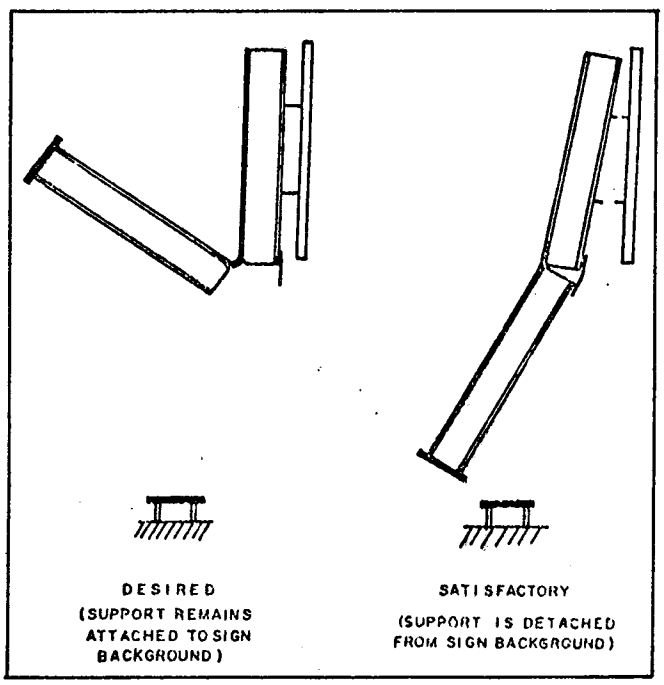


Figure 4.7.3 Collision Behavior for Breakaway Signs

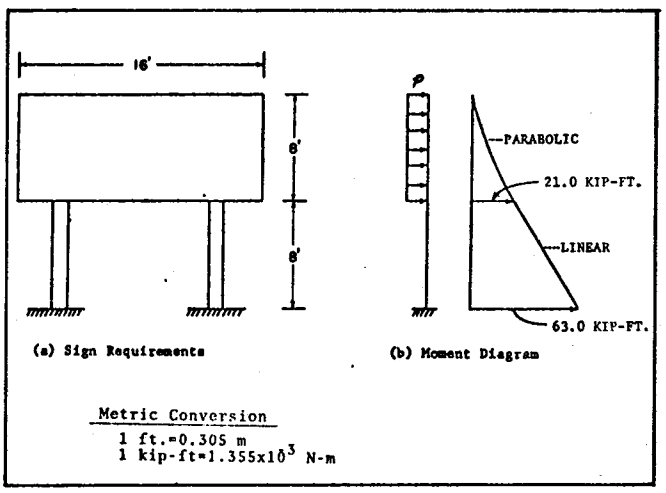


Figure 4.7.4 Wind Loading

TABLE 4.7.1 WIND LOAD COEFFICIENTS

Shape of Member	C_d			
	Single Member or Truss			Two Members or Truss - One in Front of the Other and Trusses Forming Triangular Cross Sections (All Trusses With Small Solidity Ratios)
	$V_d < 32$	$32 < V_d < 64$	$V_d > 64$	
Cylindrical	1.10	$\frac{100}{(V_d)^{1.3}}$	0.45	1.20
Dodecagonal	1.20	$\frac{9.62}{(V_d)^{0.6}}$	0.79	—
Octagonal	1.2	1.2	1.2	—
Flat	1.7	1.7	1.7	2.0
Elliptical (Broadside) (Facing Wind)	$1.7\ (D/d_o - 1) + C_{dD}\ (2 - D/d_o)$			Where: V_d = Wind Speed (mph)
Elliptical (Narrow Side) (Facing Wind)	$C_{dd}\left[\frac{4 - D/D_o}{3}\right]$			D/d_o = Ratio of Major to Minor Diameter of Ellipse C_{dd} = Drag Coefficient for a Cylindrical Shape of Diameter d_o C_{dD} = Drag Coefficient for a Cylindrical Shape of Diameter D

OTHER ELEMENTS C_d	
SIGN PANEL	
L/W = 1.0	1.12
= 2.0	1.19
= 5.0	1.20
= 10.0	1.23
= 15.0	1.30
Traffic Signals	1.2
Luminaires with generally rounded surface	0.5
Luminaires with flat sides	1.2

HEIGHT COEFFICIENT (C_h)	
Height (feet)	C_h
0 - 15	0.8
15 - 30	1.00
30 - 50	1.10
50 - 100	1.25
100 - 150	1.40
150 - 200	1.50
200 - 300	1.60

Let the wind velocity be 100 mph (160 km/h). From Table 4.7.1, the shape factor for the sign is approximately 1.19 and height factor is 0.8 at 12 feet. Using the AASHO formula for the normal wind pressure on the sign panel, the pressure is computed as:

$$p = 0.00256\ (1.3 \times 100)^2\ 1.19 \times 0.8$$

$$p = 41 \text{ psf } (1.96 \text{ kPa})$$

The moment diagram resulting from the wind load is presented in Figure 4.7.4. Loadings on the vertical supports for the sign panel have been neglected. The required section modulus for A441 steel is:

$$S = \frac{M(\text{design moment})}{S(\text{allowable stress})} = \frac{63.0 \text{ kip-feet} \times 12}{25 \text{ kip/in}^2}$$

$$= 30.2 \text{ in.}^3\ (495 \text{ cm}^3)$$

Thus, two 8 by 20 supports having a combined section modulus of 34 in.^{3} (557 cm)^3 may be used. After selecting a support size, the next step is to check them for all other AASHO requirements (see AASHO Specifications). Several types of materials are available that may satisfy the structural requirements of the sign supports, the most common being timber, aluminum, or steel.

Fuse Connection. The fuse connection must be designed to resist the maximum moment produced in the sign support at the hinge area by the wind loads on the sign. This fuse connection is a friction-type joint, with the resisting moment-producing force being a friction force which is developed between the fuse plate and the sign support by tension in the attaching bolts. A slotted fuse plate, illustrated in Figure 4.7.5, is used in the connection.

When a vehicle strikes the support, the slip base is broken loose and, as the support begins to deform in the direction of movement, the two bolts holding the fuse plate to the lower part of the support slip out of the slotted bolt holes, and the hinge becomes activated as illustrated in Figure 4.7.2. A recommended procedure for designing the fuse connection is as follows:

- Determine maximum wind moment at the hinge point. This determines the moment capacity of the fuse connection.

- Determine the required bolt force to develop the friction force between the fuse plate and the support. The initial bolt force can be determined by:

1. Required Force = $\frac{m}{d}$

   Where: m is the moment to be resisted
   d is the depth of the post

2. Individual bolt tension = $\frac{\text{Required force}}{fn}$; $f = 0.2$

   Where: f is the joint friction assumed as 0.2
   n is the number of bolts (usually 2 per post)

3. Select bolt size to accommodate bolt tension.

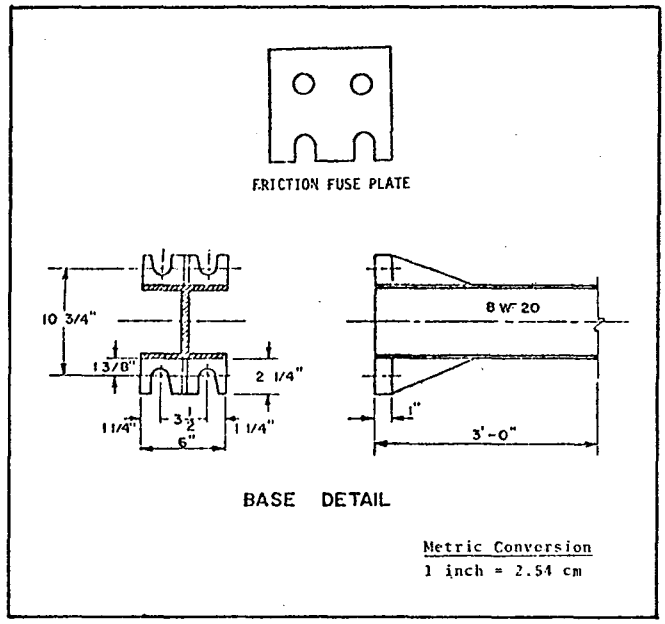


Figure 4.7.5 Breakaway Hardware

- Select the next larger standard bolt size.

- Check the tension that will be produced in the bolts selected. Since the stresses in a bolt will tend to relax over a period of time (creep), the bolt will lose some of the tensile force in it as time passes. In order to produce a friction-type bolted joint, the bolts must have an initial tension force equal to 70% of the specified minimum tensile strength of the bolts. The required tension for various bolt sizes is shown in Table 4.7.2.

- The initial bolt tension must be obtained by the calibrated wrench method. Specifications for these methods are outlined in the aforementioned Specification for Structural Joints Using ASTM A325 or A490 Bolts, and should be closely followed at both the design and construction stages.

Background-to-Post Connection. The maximum connection force anticipated is 10,000 lbs. (44,500 N).

Rotational Stiffness of Sign Background. A minimum stiffness of 100-ft-lb/degree (5,730 ft-lb/radian) [135.6 N · m/degree (7764 N · m/radian)] also has been assumed.

TABLE 4.7.2 FASTENER TENSION*

Bolt Size inches (cm)	Minimum Fastener Tension, Kips (N) A 325 Bolts	A 490 Bolts
1/2 (1.27)	12 (53)	15 (67)
5/8 (1.59)	19 (84)	24 (107)
3/4 (1.90)	28 (124)	35 (156)
7/8 (2.22)	39 (173)	49 (218)
1 (2.54)	51 (227)	64 (285)

* Source: Specification for Structural Joints using ASTM A325 or A490 Bolts, 6th Edition, Sept. 1, 1966.

Sign Supports

Slip Bases. Single support slip base designs and multiple support slip base designs with an 8-foot clearance between supports are acceptable without further testing if they are of a type that has been tested and accepted in the past, provided:

- The post does not weigh more than 45 pounds per foot and the total weight below the hinge does not exceed 600 pounds (Single supports are not expected to have hinges nor is it expected that their posts plus sign panels would weigh as much as 600 pounds).

- The total clamping force at the slip face does not exceed 45 kips. (One-quarter or less of this amount would be preferred and would be closer to current practice.) (The clamping force must be controlled by installing bolts with a torque wrench, using torque limiting nuts, or another acceptable method.)

- Washers used with the clamping bolts are of sufficient strength to prevent the washers' cupping into the vee notches.

- The stub height is no more than 4 inches.

- Where used, the upper hinge is designed to withstand the bending moment at the hinge. (Little or no factor of safety is expected to be provided in this joint.)

Where multiple support slip base designs provide less than 8 feet of clearance between supports, all supports within an 8-foot path must be considered as acting together. The acceptability of such a design may be determined by crash testing a layout that will cause the requisite number of supports to be struck, or acceptability may be inferred by multiplying the number of supports by the energy loss that a vehicle would experience striking a single support and using this value to determine the change in momentum. Criteria for slip base bolt tension are presented in Table 4.7.3.

TABLE 4.7.3

BOLT TORQUE FOR BREAKAWAY SLIP BASES

Bolt Size inches (cm)	Torque inch - lb (N · m)
5/8" (1.6)	600" # (68)
3/4" (1.9)	900" # (102)
7/8" (2.2)	1000" # (113)
1" (2.5)	1500" # (169)

Timber Posts. Wooden posts can be successfully used as a breakaway support for signs. The "Federal Highway Administration guidelines" (4) contain the following statement regarding timber sign supports.

"Timber breakaway supports are quite feasible. However, it is believed that there is no past testing that would qualify designs under the new AASHTO specification. However, available information suggests that soil mounted timber designs (without concrete foundation collars or soil bearing plates) would be acceptable if the posts have uniform cross-sections and if, in an 8-foot (2.4-m) path, there is or are:

A single post with an elastic section modulus no greater than 24 in^3 (393 cm^3) [a full dimension 4" x 6" (10 cm x 15 cm) post]

Two posts, each with an elastic modulus no greater than 18 in^3 (295 cm^3) [full dimension 3" x 6" (8 cm x 15 cm) or 4" x 5" (10 cm x 13 cm) posts]

Three posts, each with an elastic section modulus no greater than 14 in^3 (229 cm^3) [full dimension 3" x 5" (8 cm x 13 cm) or 4" x 4" (10 cm x 10 cm) posts]."

Other designs should be qualified through dynamic testing.

It is apparent that this guideline excludes the use of weakened timber post support

systems. While it is not suggested that immediate projects be undertaken to replace weakened post breakaway devices, the use of such designs on projects involving federal funding is not permitted.

For timber sign supports, the height above the ground line is critical. Sankey (3), based on dynamic tests of wood post sign supports suggests a minimum clearance to the bottom of the sign of 6' -0" (1.8 m) with a more desirable value of 7' -0" (2.1 m).

4.7.3 SMALL ROADSIDE SIGNS

4.7.31 General

Small signs are defined as those supported on a single support or on two supports less than 6 feet (1.8 m) apart. The general expectation is that all supporting elements will be struck by the impacting vehicle. Breakaway devices for this classification of sign may be broadly classified as unidirectional and multidirectional. Both types are discussed below.

Unidirectional Breakaway Posts. The most basic type of unidirectional breakaway sign support is the inclined slip base. The design as depicted in Figures 4.7.6 and 4.7.7, utilizes a 4-bolt slip base inclined at 10° or 20° vertically toward the direction of traffic flow. Inclining the slip base results in a vertical component and ensures that the sign rotates and moves upward sufficiently to allow the impacting vehicle to pass freely under the sign without incurring a secondary collision with the top of the automobile.

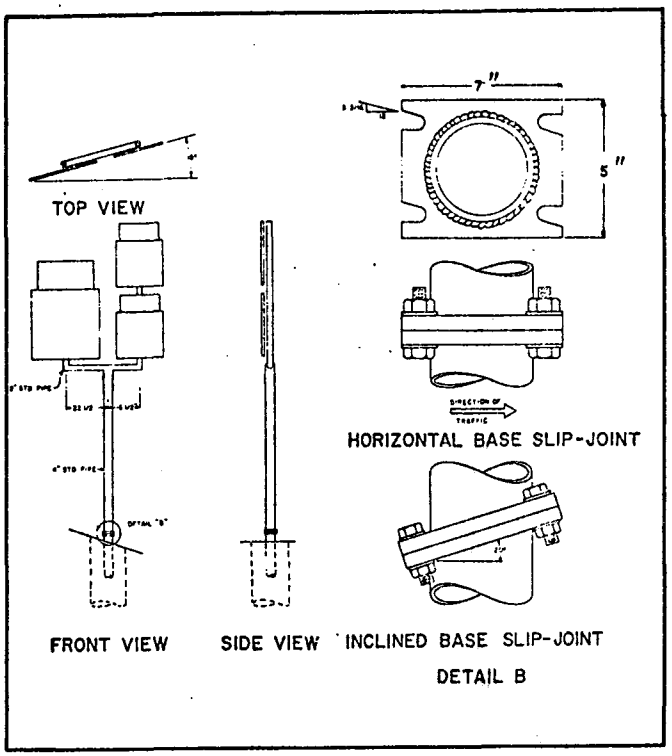


Figure 4.7.6 General Details of Pipe Supports

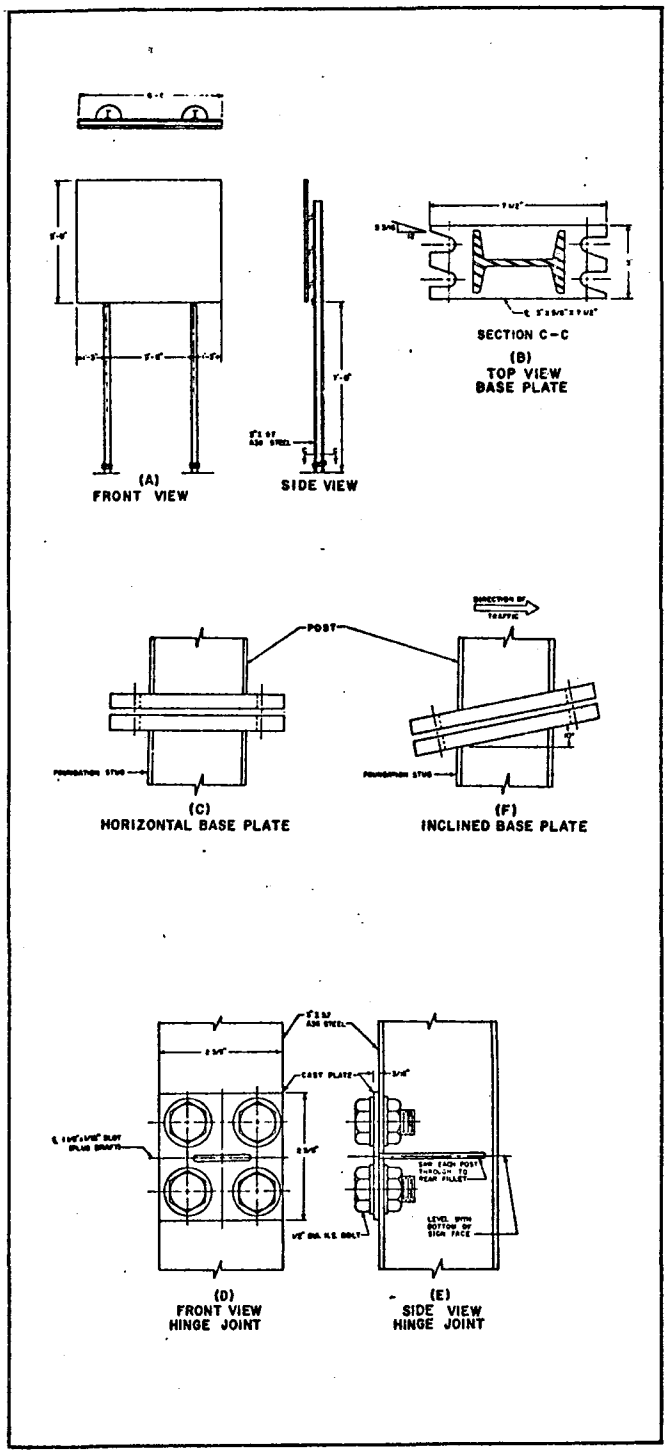


Figure 4.7.7 General Details of Small Sign

Early designs of this type included a hinge in the support. Subsequent testing demonstrated that the hinge was superfluous when used for small signs, and it has been eliminated from later designs. The design is not suited to lightweight, thin-walled pipe supports which may be liable to crushing at the point of impact, thus delaying base release and causing the sign to strike the top of the automobile. A limitation of this type of support is its directional properties. The support is generally not suited to installations in narrow medians or traffic islands separating opposing traffic flows.

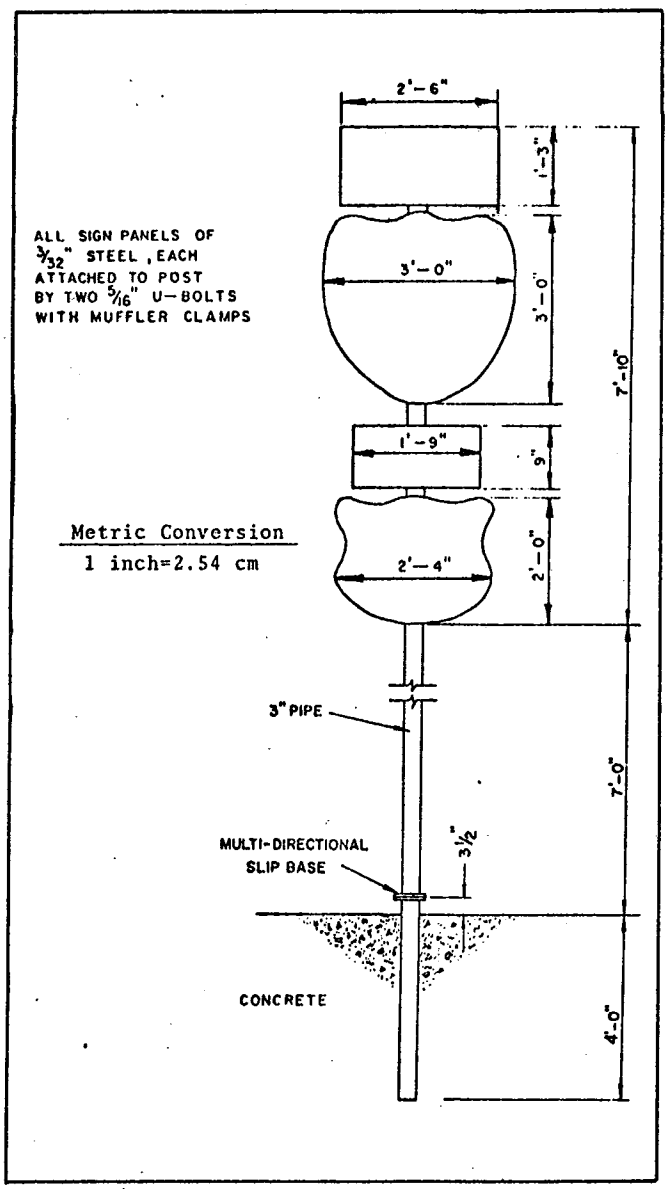


Figure 4.7.8 Route Marker with Multi-Directional Slip Base

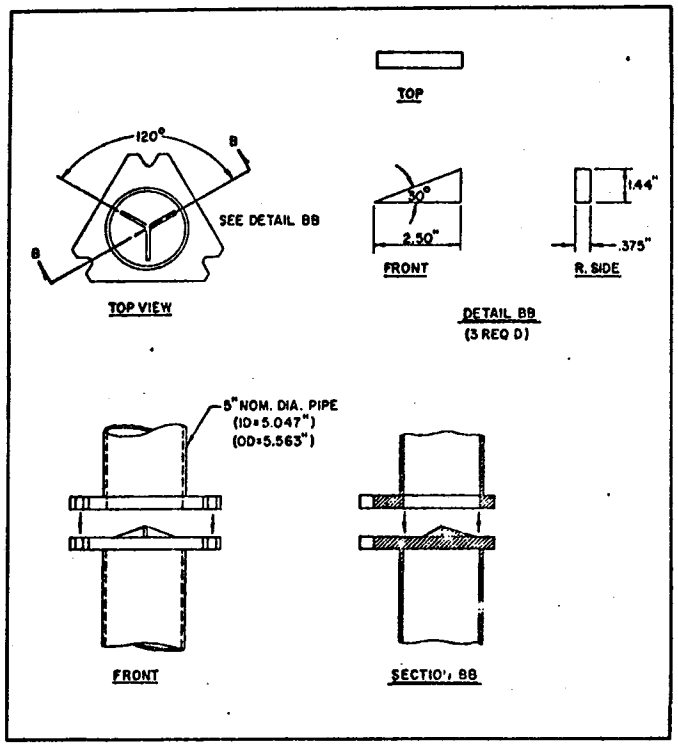
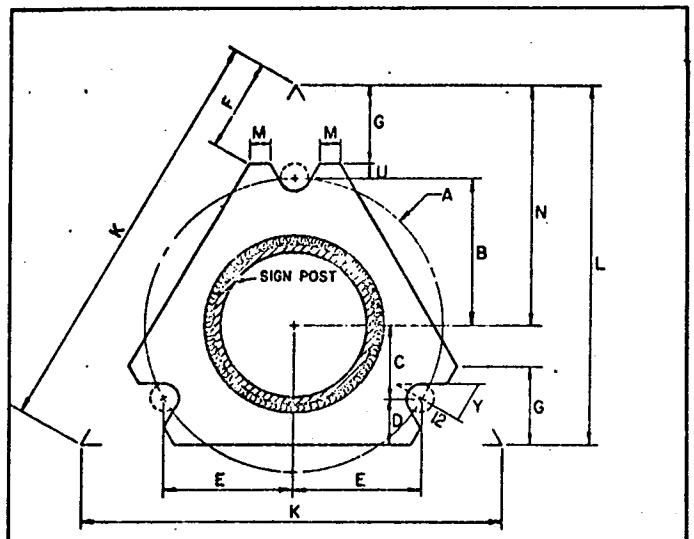


Figure 4.7.9 Recommended Multi-Directional Slip Base with Rise Modification for 5 inch Pipe Support



Metric Conversion
1 inch=2.54 cm

DIMENSIONS NOMINAL PIPE SIZES	BOLT SIZE & TORQUE	WELD SIZE	I	Y	A	B	C	D	E	F	G	K	L	M	U	N
3" DIA. 3 1/2" DIA.	5/8"Φ x 3 1/4" T = 450"	3/8"	5/8"	7"	7"	3 1/2"	1 3/4"	1 1/4"	3"	2 5/16"	2"	10 3/10"	9"	1/2"	1/2"	6"
4" DIA. 5" DIA.	3/4"Φ x 3 3/4" T = 750"	7/16"	7/8"	8 9/16"	9"	4 1/2"	2 1/4"	1/2"	3 7/8"	2 7/8"	2 1/2"	13"	11 1/4"	5/8"	1/2"	7 1/2"

Figure 4.7.10 Details of Multi-Directional Slip Base

4.7.32 Multidirectional Slip Bases

The impact with a unidirectional slip base from the side can result in a severe impact. In locations where the support is expected to be struck from any one of many angles, the multidirectional slip base is appropriate. Figures 4.7.8 through 4.7.10 illustrate the multidirectional slip base concept. The base is designed to pitch up and over the vehicle for any direction of hit. It is suggested for sign locations in medians, in channelizing islands, and at other locations where the sign can be hit from more than one predominant direction.

Another type of multidirectional breakaway base involves the use of a pipe coupling at or just above the ground line. The pipe threads provide a weakened plane on which failure occurs. The sign post pulls out of the coupling and either passes over the vehicle or is hooked on the front of the vehicle depending upon the impact speed (5). Figure 4.7.11 illustrates this basic design. It is particularly effective as an arterial street breakaway device. This system has successfully used pipes up to 2 1/2 inches in diameter. Test data indicate that such systems will meet the FHWA criteria of 1100 pound-seconds. The use of a stub pipe in the soil with the sign support inserted inside the stub has been used successfully for delineator supports. The reader is referred to Dale (6) for the design details of one such system.

4.7.33 Special Problems of Breakaway Devices

Breakaway sign supports are subject to maintenance problems which did not exist for rigid supports. Probably most significant is the wind vibration problem which tends to work the bolts in the slip base loose. In some of the early installations, the bolts actually worked completely out of the slot. A keeper plate of thin (20-gage) sheet metal is now commonly used to restrain this action. Combined with routine maintenance of the bolt torque, this practice virtually has eliminated the vibration problem.

Another common problem is overtorquing of the bolts in the slip base. The slip base operates on the weakened shear plane concept. If the bolts are overtorqued, high friction exists between the slip base elements. A large impact force is required to activate the base, and it is not uncommon for the base to "lock up" under these conditions. The bolt tension specified in Table 4.7.2 should be used as a guide. This can be obtained by the use of a torque as indicated in Table 4.7.3. In the absence of a torque wrench, the one-half turn-of-the-nut method (17) can be used to approximate the proper torque. This specific technique is detailed in reference (17). It is significant to note that the angle of turn of the nut is dependent on bolt size.

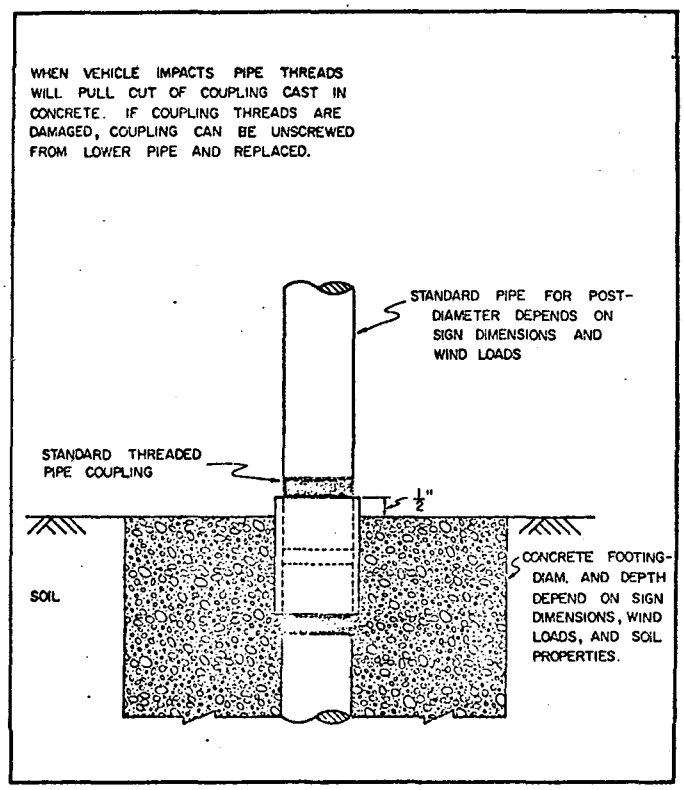


Figure 4.7.11 Threaded Coupling Breakaway Feature

Many field tests have revealed that small traffic control devices located less than 7 ft. (2.1 m) above the ground at the bottom of the panel resulted in secondary collisions with the windshield of the impacting vehicle. Mile post markers and delineator posts almost always are lower than the 7 feet. The secondary collision can take place only if the device is pulled loose from the post during impact. A connection of sufficient strength to insure that the device will not be pulled from the post should be used.

Upon impact the sign is bent down in front of the vehicle in low speed impacts, and breaks and wraps itself around the front of the vehicle on high speed impact. Hirsch, et. al (5) indicate that the MUTCD standard delineator and mile post markers will be safe on impact. Figure 4.7.12 illustrates the coupling application, and Figure 4.7.13 illustrates a variation of the pipe insert concept which has been used successfully.

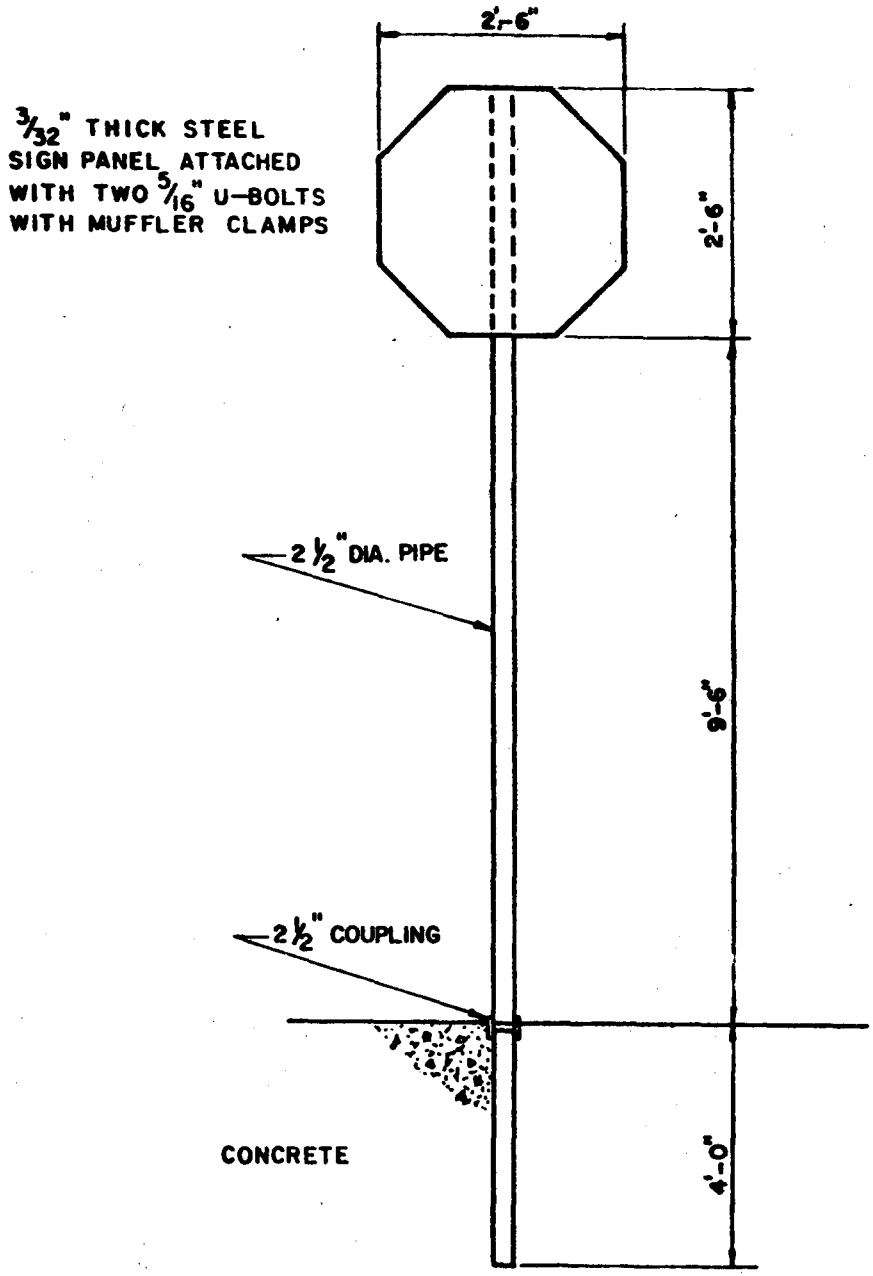


Figure 4.7.12 2½" Diameter Pipe Coupling

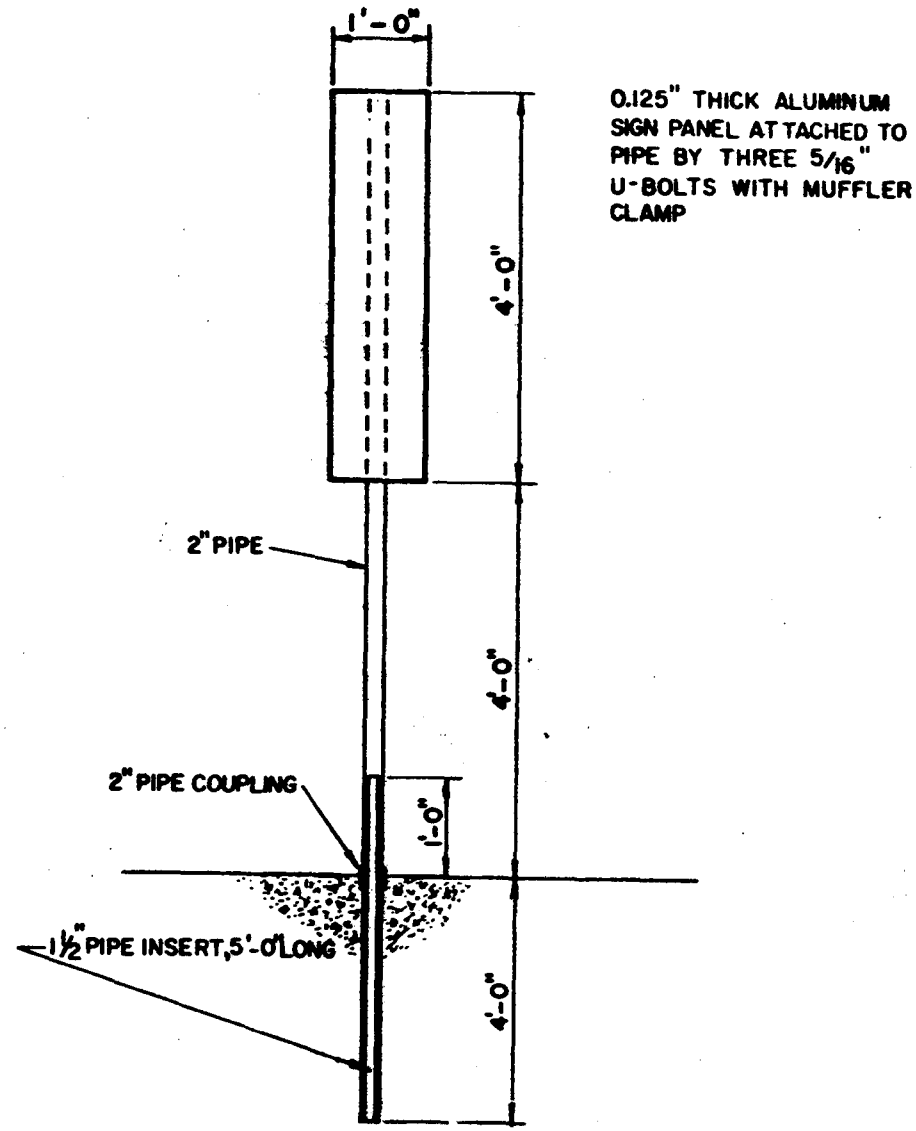


Figure 4.7.13 Pipe Coupling With Pipe Insert

4.7.4 BASE BENDING (YIELDING) SIGN SUPPORTS

Performance of this type of support is much more difficult to predict than other support types. Variations in the depth of embedment, the soil resistance and many other factors influence their dynamic behavior. For this reason, a reasonably conservative approach to design is currently recommended by FHWA (4). Quoting from this source:

"For this type structure, unless acceptability is demonstrated through testing and/or an approved analytical method of which there is none today, posts should be set in soil to a depth no greater than 3.5 feet (without concrete foundation collars, soil bearing plates, or anchors) and, within an 8-foot path, the plastic section moduli should not exceed:

For single post, 1.3 in^3 (21.3 cm^3)

For two post, each 0.7 in^3 (11.5 cm^3)

For three posts, each 0.4 in^3" (6.6 cm^3)

The Federal Highway Administration recommendation also includes an adjustment based on the ultimate tensile strength of the material. Recent testing (June 1977) indicates that this adjustment may result in acceptance of supports which when subjected to impact have unacceptably high decelerations. Therefore, it is recommended that the basic criteria above be used without applying the adjustment.

The plastic section moduli should be computed on the gross section of all the elements included in the basic post. Some manufacturers suggest that two elements combined together actually behave as two independent units and therefore the plastic modulus should be the sum of the two individual moduli rather than that of the composite section. Recent controlled tests suggest that prior to physical separation of the two elements, the deceleration experienced by a 2250 lb (1023 kgm) vehicle is well above the acceptable level when the plastic section modulus of the combined cross section exceeds the values listed above.

4.7.5 DELINEATION AS A SAFETY TREATMENT

Delineation is a general term referring to any method of identifying the limits of the operating area for the motorist. Delineation provides the motorist with a greater sense of security, reduces uncertainty, and thereby increases the time available to search for the lower levels of information needed (i.e., situational and navigational information). In this sense, adequate delineation is an essential part of any safety program. Right and left side edge lines are recommended for all roadways with any substantial traffic volume. The literature contains several references to the effectiveness of edge lines in reducing accident rates (Basile [8], Musick [9], and Tamburri [10]. These findings suggest little or no reduction on continuous sections of open roadway, but a significant reduction at access points. Total accident rates usually were not changed significantly; however, the fatal accident rate generally is reduced (8, 9). Recent reports (15) indicate substantial run-off-road accident reduction when edge lines are used.

The research findings seem to be mixed in regard to the safety benefits of edge lines. Tamburri (10), however, demonstrated a reduction in median-related accidents when yellow left edge lines were used. Thomas (11) observed that edge lines tend to move traffic toward the centerline and demonstrated the overwhelming positive attitude of the motoring public to them. With the potential accident reduction combined with a highly favorable public reaction, the general use of edge lines is deemed to be desirable.

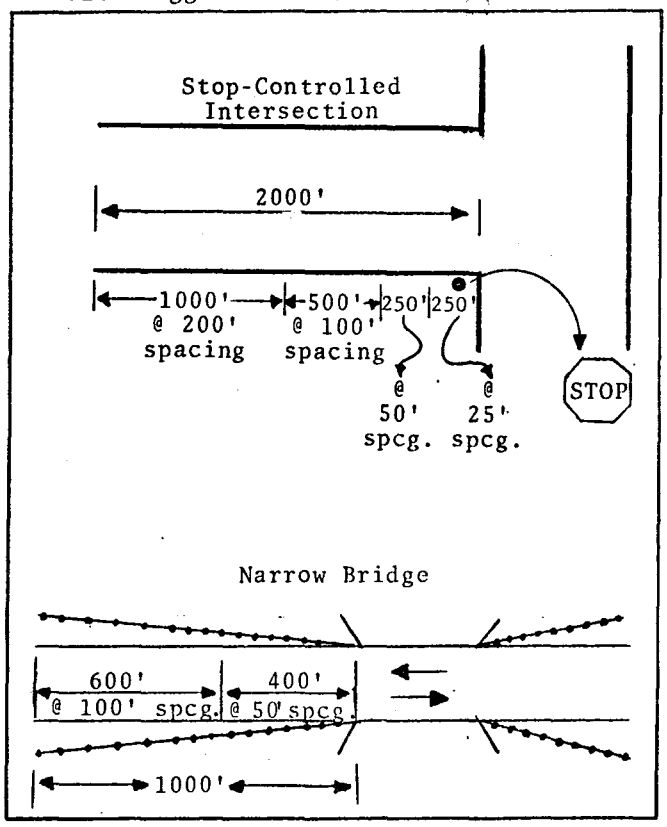
Post-mounted delineation has, on the other hand, not been as favorably accepted. For example, Longenecker (12), Tamburri (10), and Taylor (13) report that post-mounted roadside delineation reduced the accident rate only on relatively sharp curves during periods of darkness. Studies by the Arizona Highway Department (14) suggest that neither edge lines nor post-mounted delineation have any significant effect on the accident rate on open tangent sections, and further that post-mounted delineation was more cost-effective than edge lines for maintenance reasons. From the safety standpoint, it seems apparent that continuous post-mounted delineation is probably not beneficial. Special situations such as restricted width bridges, curved ramps, relatively sharp horizontal curves (D > 5°) and other unusual situations can, however, best be safety treated using post-mounted delineation.

Guidelines for Safety Treatments Using Post-Mounted Delineation (Ref. Taylor [13])

Situation	Treatment
Approach to Stop-Controlled Intersection - Rural Area	Use crystal delineators beginning with 200 foot openings and reducing to 10 foot spacing for the last 500 feet.
Culvert Head-wall Well off the Pavement	Use no delineation if roadside (positive delineation) is used. Use yellow post-mounted delineators otherwise.
Guardrail on Tangent Roadway Sections	No special treatment required.

Narrow Bridge	Pattern of crystal delineators beginning 1000 feet from the structure and funneling traffic between the two edges of the bridge structure.
Horizontal Curves (an accident reduction on the order of 30% can be expected)	Use post-mounted delineators on the outside of the curve only. A spacing of $3\sqrt{R - 50}$ is adequate. All curves of 5° or greater having a central angle of 20° or more should be treated.

4.7.14 Suggested Delineation Patterns



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TOPIC 5 SESSION 1
TRAFFIC BARRIERS

Objectives:

The objective of this presentation is to prepare the participant for the following tasks:

1. Identify roadway and roadside conditions warranting the installation of traffic barriers,

2. Select a longitudinal barrier suitable to the condition or situation identified, and

3. Specify the design requirements of a longitudinal barrier installation to meet the current safety criteria.

5.1.1 INTRODUCTION

Many rigid obstacles along the roadside cannot be removed or made breakaway. These objects must be safety treated in order to provide an acceptable level of safety for the motoring public. The primary method of achieving this safety goal is to redirect the vehicle or soften the impact through the use of vehicle attenuation systems. This session considers the primary requisites for the design of traffic barriers.

5.1.11 Need for Traffic Barriers

Traffic barriers are required when the hazard without the barrier is greater than the hazard of the barrier itself. There are several general conditions which dictate the use of some form of traffic barrier.

- High speed crossover into the opposing traffic lanes or excursions onto the roadside by an errant vehicle
- Control of pedestrian movement
- Control or elimination of certain traffic movements
- Roadside obstacles that cannot be removed or relocated and which are subject to impact by an errant vehicle

The second and third conditions frequently are accommodated by devices that have no major structural requirements. Included in this classification are fences, curbs, earth berms and similar treatments. The first and fourth conditions involve two requirements that frequently conflict. First, the barrier must contain or redirect the errant vehicle without imposing unacceptable conditions on the occupants; and, second, the barrier must satisfy the first criterion for a wide range of vehicle sizes and weights and for a varierty of impact speeds and angles. These two requirements result in design compromises for practical application.

5.1.2 TYPES OF BARRIERS

All traffic barriers may be classified into two major categories: 1) Longitudinal Barriers and 2) Crash Cushions. Longitudinal barriers may be categorized further as a) Roadside Barriers and b) Median Barriers, including all devices that have substantial longitudinal dimensions, and whose primary function is to prevent an errant vehicle from reaching an object or situation. Typically, longitudinal barriers are applied to situations which constitute large-area or linear hazards such as ditches, fill slopes, streams and groups of point hazards. Crash cushions include all devices designed to soften the impact of an errant vehicle with a rigid object. Typically, crash cushions are applied to bridge piers, overhead sign supports, and other fixed objects of a discrete nature.

5.1.21 Dynamic Performance Criteria for Vehicular Traffic Barrier

In satisfying the two requirements discussed above, vehicular barriers must meet three basic criteria: 1) Structural adequacy, 2) Impact severity, and 3) Vehicle trajectory. The AASHTO Guide for Selecting, Locating, and Designing Traffic Barriers (1) suggests dynamic performance criteria for traffic barriers presented in Table 5.1.1.

5.1.3 ROADSIDE BARRIERS

These consist of a longitudinal system used to prevent the errant vehicle from entering a particular area along the roadside due to the inherent hazard of that area. Inherently, traffic operates only to one side of the barrier.

5.1.31 Warrants for Roadside Barriers

Two basic types of hazards are treated using roadside barriers: 1) Embankments, and 2) Roadside obstacles that cannot be eliminated or made to breakaway. Conditions warranting the use of a barrier on embankments relate primarily to the height and steepness of the

embankment. Embankments which are relatively flat and on which full control of the vehicle can be maintained need not have a roadside barrier. Indeed, the use of a barrier in such instances may well increase the hazard rather than reduce it. Steep embankments where the vehicle will go to the bottom of the slope regardless of driver control inputs may require a roadside barrier, depending to a large degree on the nature of the hazard at the bottom of the slope. Since the vehicle will by necessity be forced to the bottom of the slope, the objects or situations at that point should determine the need for protective devices. In the absence of a critical safety condition at the toe of the slope, the criteria presented in Figure 5.1.1 are suggested.

Many design situations dictate the use of a very flat side slope adjacent to the traffic way and a somewhat steeper slope at 10 to 12 ft (3 to 3.6 m) from the pavement edge (i.e., barn roof design). When the difference is excessive, rounding of the slope as indicated in Figure 5.1.2 can eliminate the need for a traffic barrier.

Obstacles on the side slope can create a substantial hazard to the driver of an errant vehicle. It is currently common practice to safety treat all objects located within the clear distance (30 ft (9.1 m) in most instances. The vehicle leaving the roadway onto a steep side slope will, by necessity, travel to the bottom of the slope. Thus, the required clear zone is much greater than 30 ft (9.1 m) in this instance. Figure 5.1.3 is a suggested guideline to assist the decision maker in identifying the situations which do not warrant a roadside barrier. From Figure 5.1.3 it can be noted that for fill slopes flatter than 4:1 the 30-ft (9.1-m) clear distance is appropriate. For slopes steeper than 4:1 the required clear zone increases to about 100 ft (30.5 m) at a side slope of 3:1. The basic 30-ft (9.1-m) clear distance is suitable for all cut-section slopes flatter than 2:1.

Tables 5.1.2 and 5.1.3 provide a listing of warranting conditions for obstacles which cannot be removed or relocated.

5.1.32 Warrants for Side Ditch Treatments

Side slopes typically end in ditches. Vehicles leaving the roadway must be able to safely traverse these ditch sections. Many ditch cross sections are used--for example the vee ditch, the trapezoidal ditch, and the rounded bottom ditch. Based on the forces on the vehicle's center of gravity, the Guide for Selecting, Locating, and Designing Traffic Barriers (1) presents a set of curves to assist in the selection of the preferred ditch design. These curves are presented as a composite in Figure 5.1.4.

5.1.4 DESIGN OF ROADSIDE BARRIERS

A roadside barrier, as indicated in Table 5.1.1 must: 1) redirect the vehicle without penetration or vaulting over the installation; 2) not snag the vehicle or cause an abrupt deceleration, spinout or rollover; and 3) keep the maximum accelerations to tolerable levels. It is desirable that the vehicle trajectory and position of rest not intrude on adjacent traffic lanes.

There are three functional design elements of a roadside barrier: 1) Standard section, 2) End treatment, and 3) Transition section (applicable where the barrier is connected to another barrier of different characteristics, or located in close proximity to a rigid object). All three elements must function properly and compatably to the criteria stated above.

5.1.41 Standard Sections

The Guide for Selecting, Locating, and Designing Traffic Barriers (1) presents details relative to types of roadside barriers that are considered operational at this time. "Operational" implies that the barrier has performed satisfactorily in full-scale dynamic tests, and has demonstrated satisfactory in-service performance. The operational systems are summarized in Table 5.1.4. For further detail, one should refer to the Guide.

The height of roadside barriers ranges from 27" (0.69 cm) to 32" (.81 cm); however the 27" (0.69 cm) height appears to be the most common. Some states have advocated greater heights, but increasing barrier height should be done with considerable caution. Greater heights reduce the probability of vaulting over the rail but they also increase the possibility of snagging the posts. Further, the trend to smaller vehicles constitutes some concern for the performance capability of the greater heights. It appears most practical to assume a "wait and see" position at this point in time.

5.1.42 Transition Sections

Transition sections are necessary to provide continuity when two different roadside barriers join, when a roadside barrier joins another barrier system (such as a bridge rail), or when a roadside barrier is attached to a rigid object. The most common transition is between roadside barriers and bridge rails or bridge abutments.

Several operational transition sections are presented in the Guide in great detail. In all cases, it is fundamental to increase the stiffness of the approach rail to match the stiffness of the bridge rail. This is accomplished by decreasing post spacing.

TABLE 5.1.1 DYNAMIC PERFORMANCE CRITERIA FOR TRAFFIC BARRIERS

Dynamic Performance Factors	Evaluation Criteria	Applicable Criteria: Longitudinal Barriers: Standard Sections and Transitions	Applicable Criteria: Longitudinal Barriers: Terminals	Applicable Criteria: Crash Cushions
I. Structural Adequacy	A. The test article shall redirect the vehicle; hence, the vehicle shall not penetrate or vault over the installation.	XXX		
	B. The test article shall not pocket or snag the vehicle causing abrupt deceleration or spinout or shall not cause the vehicle to rollover. The vehicle shall remain upright during and after impact although moderate roll and pitching is acceptable. There shall be no loose elements, fragments or other debris that could penetrate the passenger compartment or present undue hazard to other traffic.	XXX	XXX	XXX
	C. Acceptable test article performance may be by redirection, containment, or controlled penetration by the vehicle.		XXX	XXX
	D. The terminal shall develop tensile and/or flexural strength of the standard section.		XXX	
II. Impact Severity	A. Where test article functions by redirecting vehicle, maximum vehicle accelerations (50 msec avg) measured near the center of mass should be less that the following values: Maximum Vehicle Accelerations (g's) Lat. / Long. / Total / Remarks 3 / 5 / 6 / Preferred 5 / 10 / 12 / Acceptable These rigid body accelerations apply to impact tests at 15 deg. or less.	XXX	XXX	XXX
	B. For direct-on impacts of test article, where vehicle is decelerated to a stop and where lateral accelerations are minimum, the maximum average permissible vehicle deceleration is 12 g as calculated from vehicle impact speed and passenger compartment stopping distance.		XXX	XXX
III. Vehicle Trajectory Hazard	A. After impact, the vehicle trajectory and final stopping position shall intrude a minimum distance into adjacent traffic lanes.	XXX	XXX	XXX
	B. Vehicle trajectory behind the terminal is acceptable.		XXX	

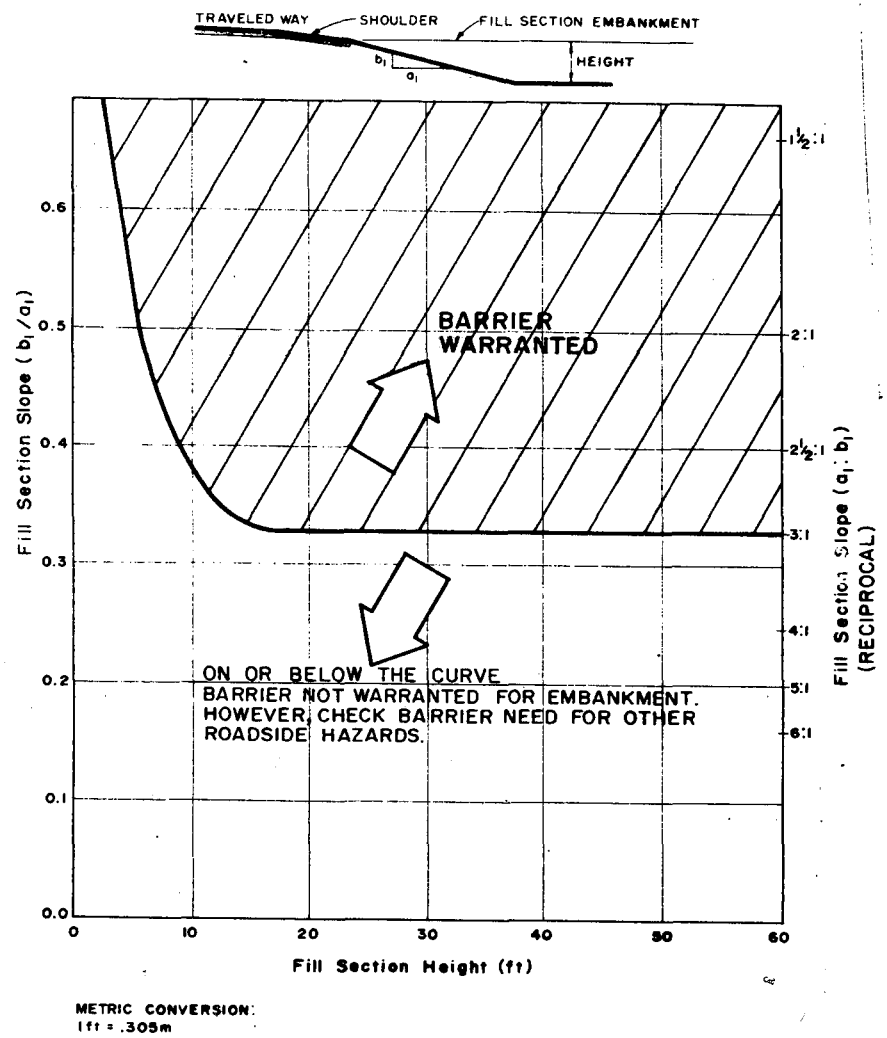
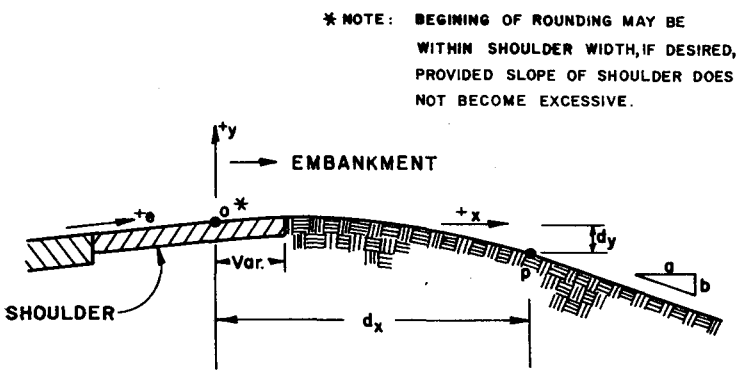


Figure 5.1.1 Warrants For Fill Section Embankments



ROADSIDE ELEVATION

EQUATIONS

	Metric	
$y = x\left[e - \frac{6.9x}{V^2\sin^2\theta}\right]$, $0 \le x \le d_x$	$y = x\left[e - \frac{2.1x}{V^2\sin^2\theta}\right]$, $0 \le x \le d_x$	(III-A-1)
$d_x = \frac{V^2\sin^2\theta}{13.8}\left[e - \frac{b}{a}\right]$	$d_x = \frac{V^2\sin^2\theta}{4.2}\left[e - \frac{b}{a}\right]$	(III-A-2)
$d_y = d_x\left[e - \frac{6.9d_x}{V^2\sin^2\theta}\right]$	$d_y = d_x\left[e - \frac{2.1d_x}{V^2\sin^2\theta}\right]$	(III-A-3)
y, x, d_x, and d_y in feet.	y, x, d_x, and d_y in meters.	

where, e = shoulder slope (ft/ft) [m/m], positive if sloping upward;

$\frac{b}{a}$ = embankment slope (ft/ft) [m/m], negative if sloping downward;

V = vehicle velocity (ft/sec) [m/s]; and

θ = vehicle encroachment angle (deg), or the angle between vehicle heading and tangent to roadway.

"O" = shoulder slope tangent point, and origin of x-y axis.

"P" = side slope tangent point.

Figure 5.1.2a

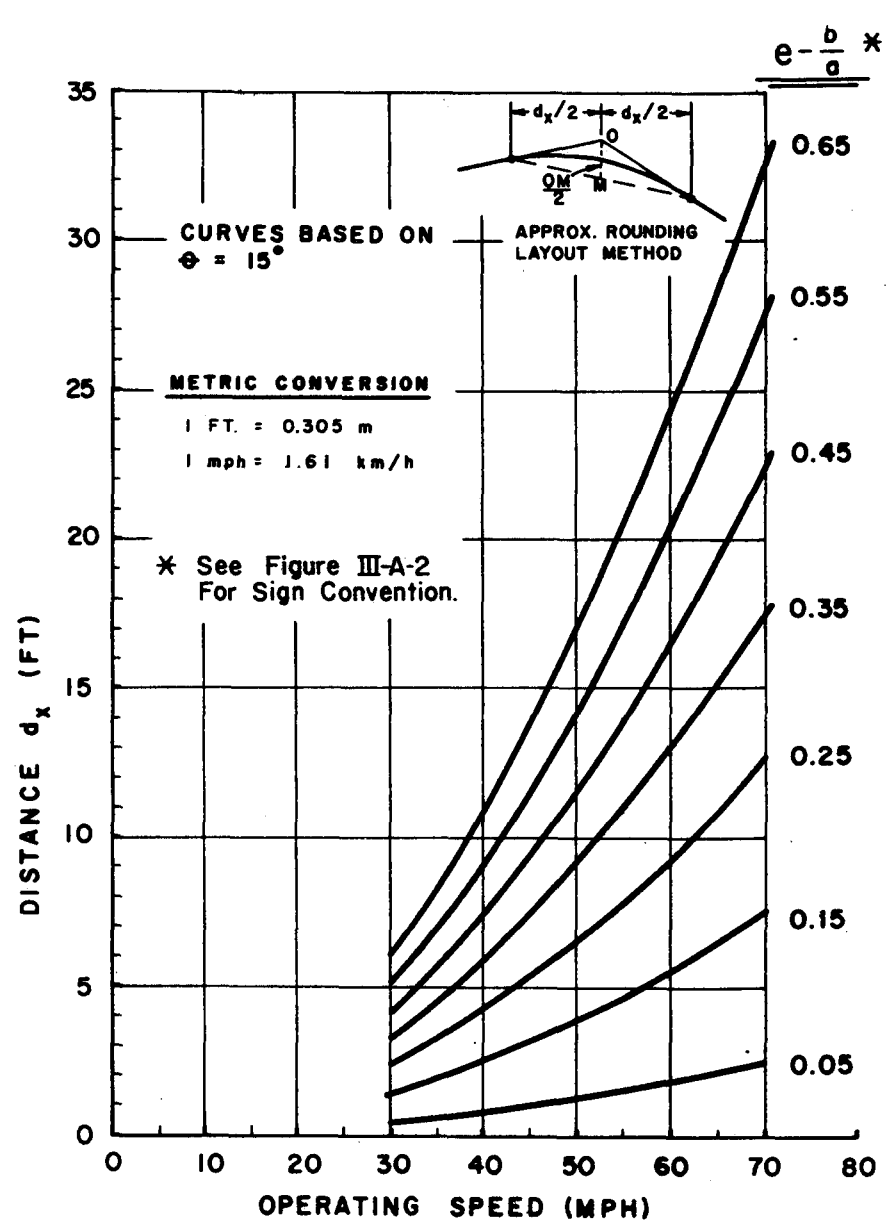


Figure 5.1.2b

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Dimensional Data For Optimum Rounding

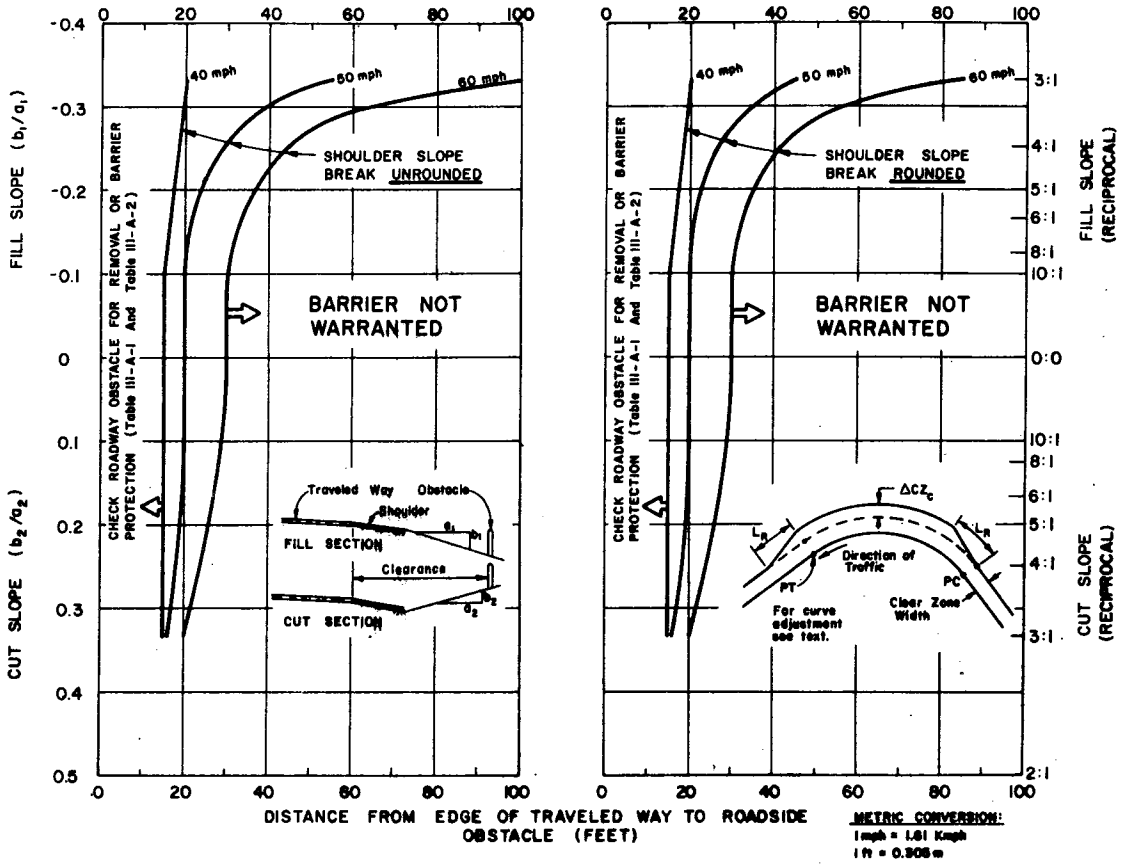


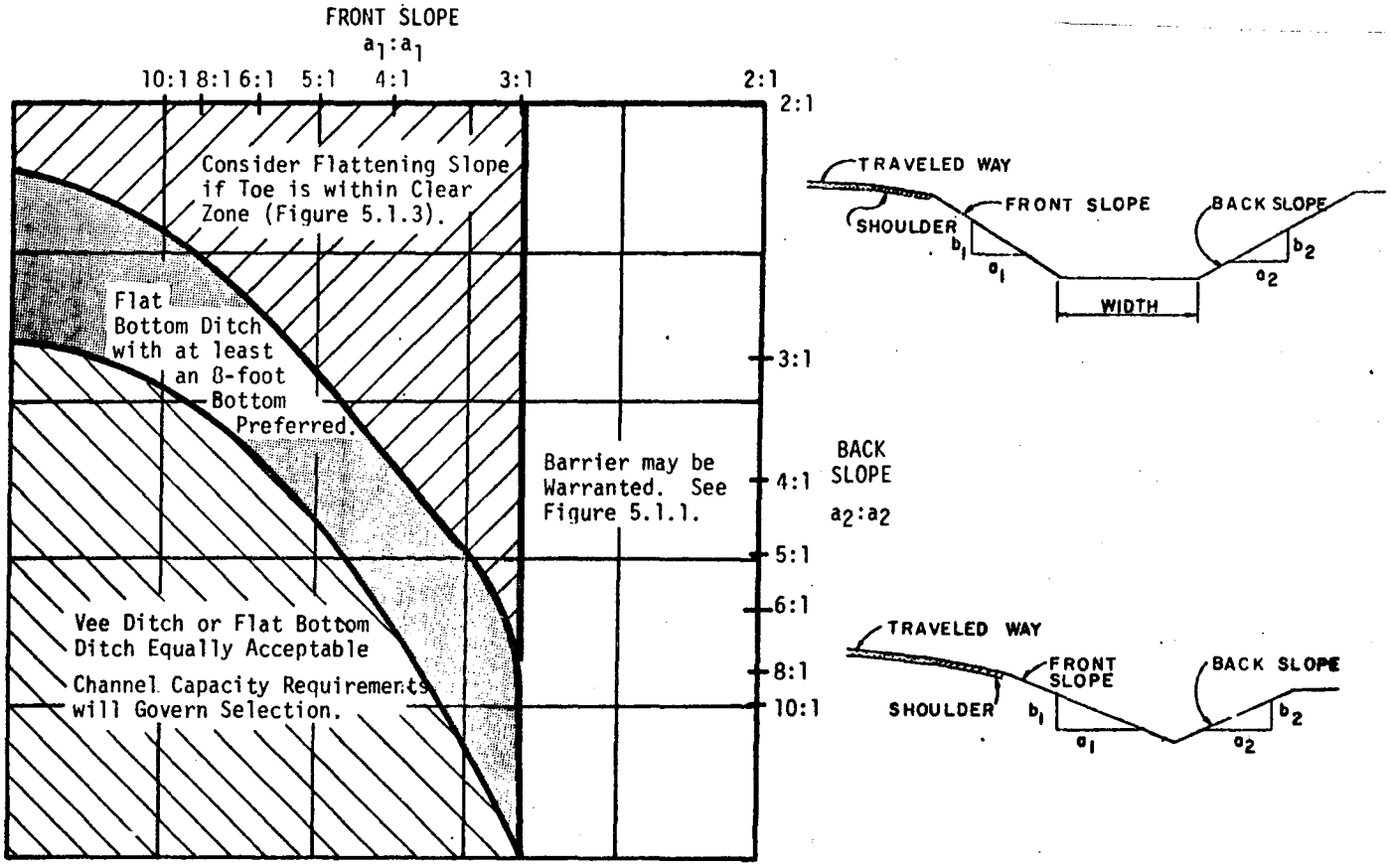
Figure 5.1.3 Clear Zone Width, Speed and Slope Criteria

TABLE 5.1.2 WARRANTS FOR TRAFFIC BARRIERS AT NONTRAVERSABLE OBSTACLES

Nontraversable Hazard within the clear distance	Traffic Barrier Required Yes	No
Rough Rock Cuts	x	
Large Boulders	x	
Streams or bodies of water less than 2 feet deep		x
Streams or bodies of water more than 2 feet deep	x	
Shoulder drop-off with slope steeper than 1:1 and		
a) Height greater than 2 feet	x	
b) Height less than 2 feet		x

TABLE 5.1.3 WARRANTS FOR FIXED OBJECTS

Fixed Objects within the Clear Distance	Traffic Barrier Required Yes	No
Traffic control device support		
a) Breakaway design		
(1) Less than 1100 lb-sec		x
(2) More than 1100 lb-sec	x	
b) Concrete support or greater in height above the surrounding area	x	
Fixed sign bridge support	x	
Bridge piers	x	
Retaining walls and culverts	x	
Trees > 6" in diameter	x	
< 6" in diameter		x
Wooden pole or post within an area greater than 50 square inches	x	



1 ft = 0.305 m

Figure 5.1.4 Preferred Ditch Sections

TABLE 5.1.4 OPERATIONAL ROADSIDE BARRIERS

Barrier System	Dynamic Deflection	Vehicle Acceleration (g's)			Impact Angle	Impact Speed (mph)	Exit Angle
		Lateral	Longitudinal	Total			
Cable Guardrail	11.0'	NA	NA	6	25°	44	15°
"W" Beam-Weak Post	0'	NA	NA	1	6°	57	1°
"W" Beam-Weak Post	7.3'	3.8	3.1	NA	27.8°	59	9°
Box Beam	4.8'	5.8	2.8	NA	26°	58	0°
Blocked out "W" Section (Wood Posts)	2.8'	6.1	3.0	NA	22.2°	60	15°
Blocked out "W" Section (Wood Posts)	2.3'	7.0	6.8	NA	24°	68	14°
Blocked out "W" Section (Steel Posts)	2.6'	6.9	3.8	NA	25°	66	16°
Blocked out "W" Section (Steel Posts)	4.1	6.6	3.9	NA	28.4°	57	8°
Blocked out "W" Beam (Steel "C" Posts)	2.9'	6.8	3.7	NA	25°	59	NA
Blocked out "Thrie Beam" (Steel Posts)	0.6'	4.1	2.9	NA	15°	59	<10°
Blocked out "Thrie Beam" (Steel Posts)	1.5'	7.9	3.9	NA	25°	56	15°
Concrete Barrier	0'	6.0	5.0	NA	15°	61	12°
Concrete Barrier	0'	9.0	7.0	NA	25°	62	7°

Where a 6'3" (1.90 m) spacing is commonly used, the spacing is reduced to 3' 1½" (.95 m) for a distance of 25 feet (7.62 m) immediately preceeding the bridge (See Figure 5.1.5). Also, stiffening the longitudinal elements may be done by doubling the beam or using a thrie beam.

It is of utmost importance that the connection to the bridge-end develop the tensile and flexural strength of the approach rail. Also, the connection should be flared or sloped so that an errant vehicle from either direction will not snag on the connection. Strong post systems must be used on transitions to rigid bridge rails or bridge ends. The rail should be blocked out and rub rails may be required to avoid snagging.

It is suggested that the transition length be 10 to 12 times the difference in dynamic deflections of the two barriers being joined.

5.1.43 End Treatments

The end of a roadside barrier constitutes the most serious hazard of the barrier system. Thus, it is imperative that the rail end be treated satisfactorily. There are two basic requirements of the end treatment. It must be crashworthy if it is within the clear zone, and it must be capable of developing the full tensile strength of the standard rail element. To be crashworthy, the end treatment should not spear, vault, or roll the vehicle for head-on impacts, and vehicle accelerations should not exceed the recommended limits.

The Guide presents only two end treatments that are considered "operational" and both are of the breakaway cable terminal (BCT) type. One employs wood posts while the other utilizes steel posts with breakaway features. Other end treatments that have been used include turned-down-ends, flared, earth berm and anchorage in the back slope.

End treatments of concrete roadside barriers include flaring, tapering, earth berms and tieing into the backslope. Of these, tapering appears to be most questionable because of ramping effects.

Terminals upon which there is long term accident data showing desirable impact performance will usually be considered acceptable.

The cost of safety end treatment of W-beam guardrail has concerned many highway engineers. Reports of excessively large costs on particular systems has limited the general application of some treatments. For this reason, an attempt was made (1, 2) to determine the experience in several states in order to provide a reasonable basis for decision making.

SAFETY END TREATMENT	AVERAGE COST
Flared and embedded rail	$300
Break-away cable terminal	$210
Turned down rail (falldown)	$200
Flared	$300

On the basis of cost, the BCT and Turned Down Rail (Falldown type) are about equal cost while flaring increases the cost appreciably. The reader is reminded that the turned down rail (falldown type) is not currently approved for general use although the dynamic performance upon impact by a compact vehicle appears to be somewhat superior to the BCT (3).

5.1.44 Location of Roadside Barriers

The location of roadside barriers pertains principally to the lateral and longitudinal positioning of the barrier and the length of need to shield the hazard. Lateral positioning (Figure 5.1.6) is based on achieving a clear shoulder width while maintaining a distance from the hazard, particularly a fixed-object hazard, that is greater than the dynamic deflection of the barrier. Dynamic deflections for operational barriers are given in Table 5.1.4.

Where the hazard is a steep embankment (Figure 5.1.6) and dynamic deflection is not so critical, a minimum clearance of 2' (0.61 m) is recommended.

Barriers located on a side slope must be carefully reviewed to insure that an impacting vehicle will not vault over the barrier. As a general guideline, the barrier should not be placed on the lower slope but up at the break point between the slopes. These criteria are illustrated in Figure 5.1.7.

The longitudinal (and lateral) location of the beginning of a roadside barrier shielding an "area of concern" is determined on the basis of the expected angle of departure from the travelled way. Figures 5.1.8 and 5.1.9 illustrate the layout variables for protection for traffic in both directions of travel. The equations for determining the X and Y dimensions to establish the length of needed barrier are as follows:

$$X = \frac{L_H + \left(\frac{b}{a}\right)L_1 - L_2}{\left(\frac{b}{a}\right) + \left(\frac{L_H}{L_R}\right)} \quad \text{and} \quad Y = L_H - \left(\frac{L_H}{L_R}\right)(X)$$

Definitions of variables are given in Figures 5.1.8 and 5.1.9.

Table 5.1.5 is used to determine runout length (L_R), flare rate $\frac{a}{b}$, and shy distance (L_S).

To avoid vaulting over the rail, the side slope in front of the barrier must be controlled not to exceed 10:1. Figure 5.1.10 illustrates the areas in which the 10:1 or flatter slope must be attained.

EXAMPLE:
The following example illustrates the procedure for locating the beginning point of a roadside barrier to protect traffic from a roadside object.

EXAMPLE:

Assumptions
- 4:1 side slope
- 24 inch ϕ pier
- 16 feet from the pavement edge
- 70 mph design
- 5000 ADT

From Figure 5.1.8

L_R = 440' (134.1 m)

L_H = 16' + 2' = 18'(4.9 m + .61 m = 5.51 m) (Distance to back of pier)

L_1 = 0 (Tangent distance in front of the hazard)

L_2 = 14' (4.3 m) (Distance to rail - 2 ft (.61 m) inside pier)

a = 25 (Horizontal run on flare)

b = 1 (Transverse run on flare)

Shy
Distance = 10' (3 m)

Flare Rate = 25:1

$$x = \frac{18 + (\frac{1}{25})\ (0) - 14}{(\frac{1}{25}) + \frac{18}{440}} = \frac{4}{0.29} = 49.4$$

$$\doteq 50 \text{ ft } (15.2 \text{ m})$$

$$y = 18 - (\frac{18}{440})\ (49.4) = 15.98 \doteq 16 \text{ ft } (4.9 \text{ m})$$

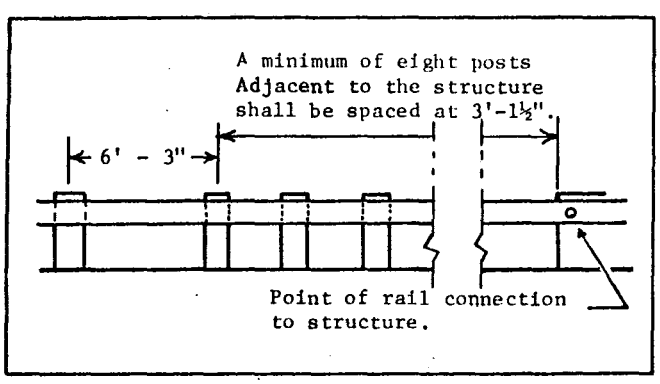


Figure 5.1.5 Barrier Transition Detail

5.1.5 MEDIAN BARRIERS

A longitudinal system used to prevent an errant vehicle from crossing that portion of a divided highway separating the traveled ways from opposite traffic directions constitutes a median barrier.

5.1.51 Warrants for Median Barriers

Figure 5.1.11 presents the suggested warrants for median barriers as related to median width and traffic load. These criteria are based on median crossover accidents, research data and professional judgment. Within the optional area of Figure 5.1.11, the barrier should only be installed following an extended history of crossover-the-median accidents.

5.1.52 Design of Median Barriers

The median barrier is composed of three basic sections: 1) The standard section; 2) The transition section; and 3) The terminal section. Figure 5.1.12 illustrates these three basic elements of the median barrier.

The desirable structural and safety characteristics of median barriers are presented in Table 5.1.1. Standard sections for the more commonly used median barriers are presented in Figure 5.1.13. A summary of the test data on impact performance is included in Table 5.1.6.

5.1.53 The Standard Section

Median barriers are commonly categorized as flexible, semi-rigid, or rigid. Flexible systems generally undergo larger dynamic deflection upon impact and therefore usually result in lower impact forces on the impacting vehicle than do the semi-rigid or rigid systems. Guidelines for the application of rigid, semi-rigid and flexible barriers are given in Table 5.1.6.

The median barrier standard sections that are currently classified as operational are illustrated in Figure 5.1.13. It should be noted that MB2 is a flexible barrier while MB3, MB4W, MB4S, MB7, MB8, MB9 and MB10 are classified as semi-rigid. MB5 is the only operational rigid barrier. Several variations of the Concrete Median Barrier (CMB) particularly the pre-cast variations (MBE1 and MBE3) are very promising but have not had sufficient in-service experience to be classified as operational. For more detail, the designer should refer to the Guide for Selecting, Locating, and Designing Traffic Barriers and all of the FHWA directives that will be forthcoming as additional

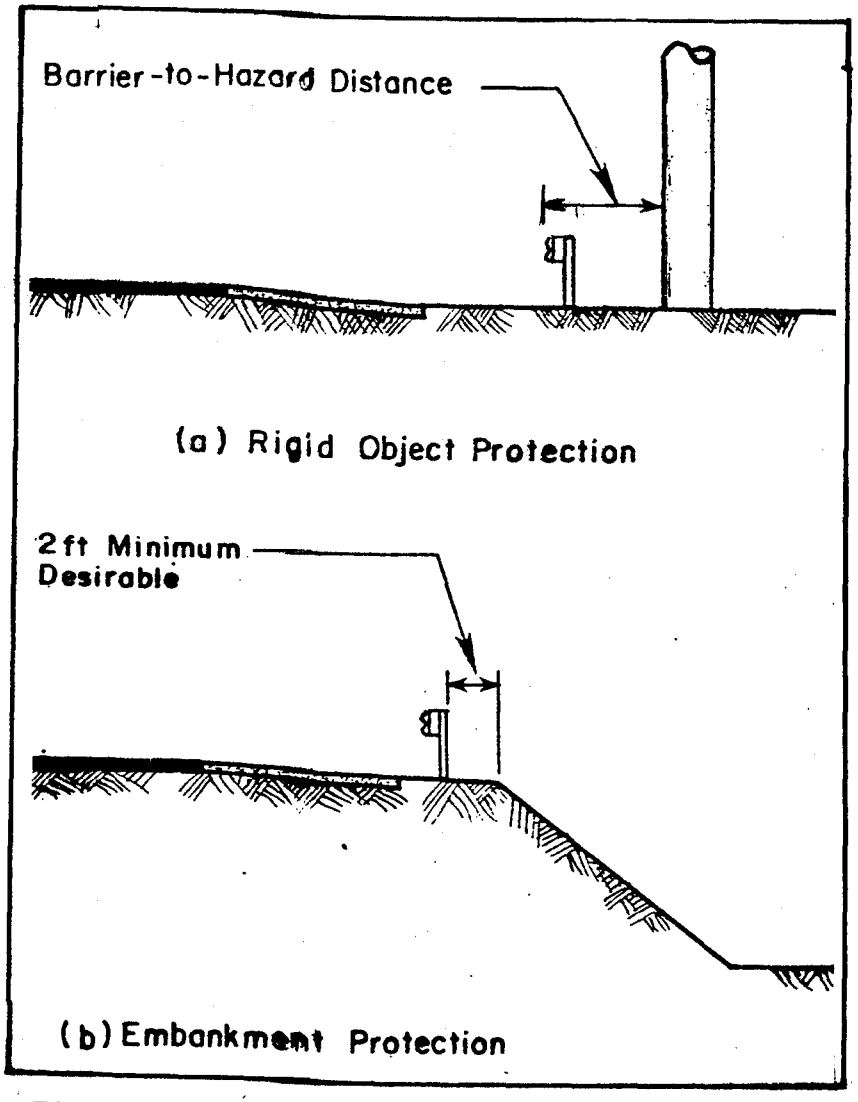


Figure 5.1.6 Barrier-to-Hazard Distance for Roadside Protection

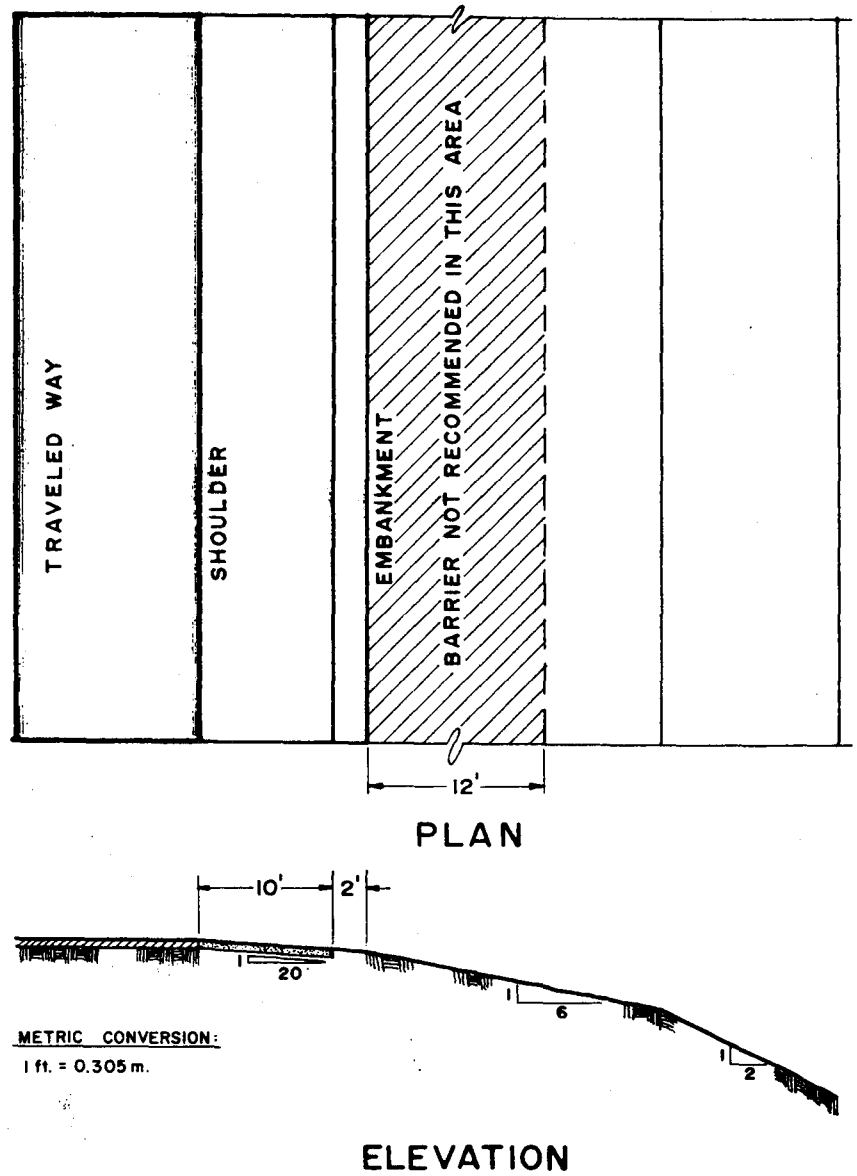


Figure 5.1.7 Roadside Barrier Location on Typical Barn Top Section

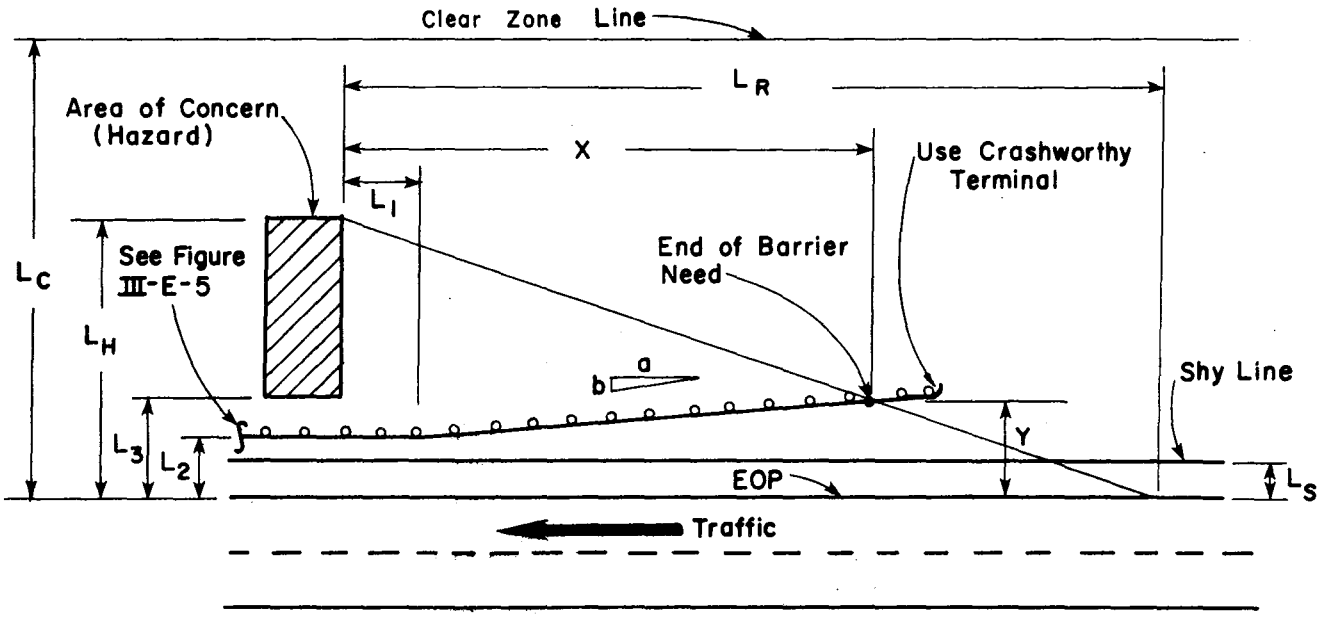


Figure 5.1.8 Approach Barrier Layout Variables

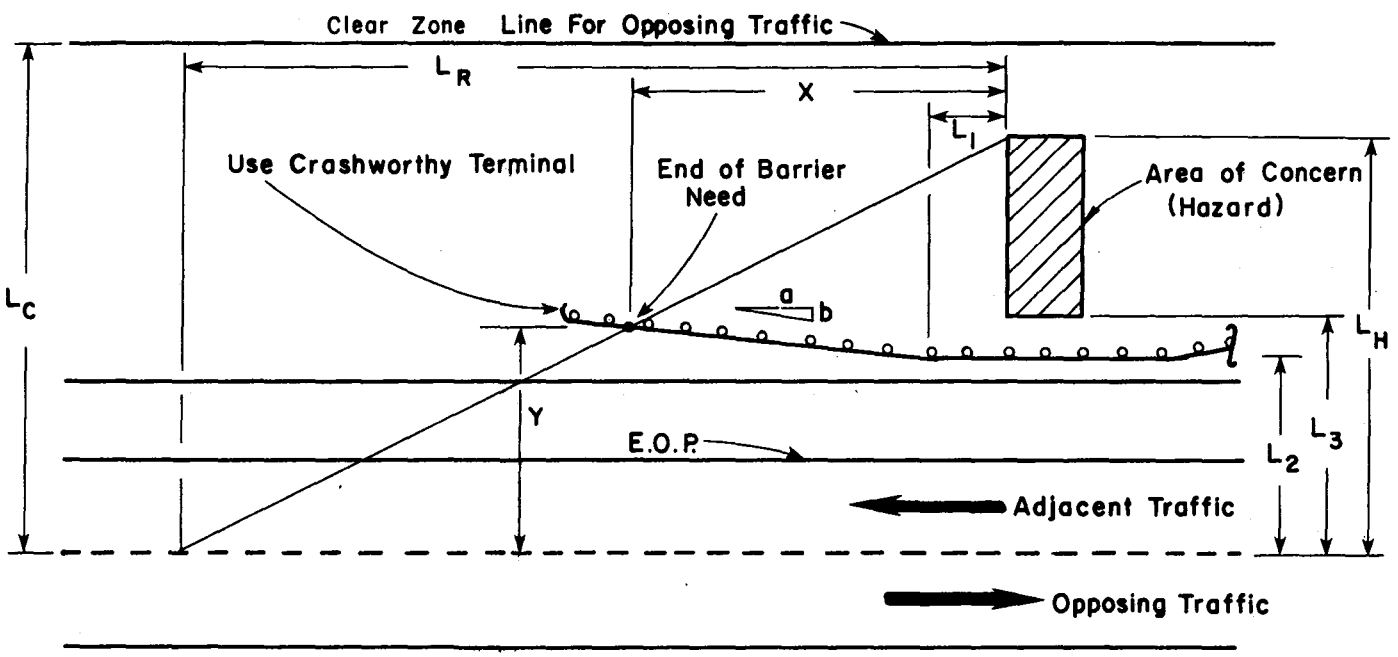


Figure 5.1.9 Approach Barrier Layout for Opposing Traffic

TABLE 5.1.5 DESIGN PARAMETERS FOR ROADSIDE BARRIER LAYOUT

Operating Speed (mph)	Design Traffic Volume (ADT) Over 6000 Runout Length L_R(ft)	2000-6000 Runout Length L_R(ft)	800-2000 Runout Length L_R(ft)	800-250 Runout Length L_R(ft)	Under 250 Runout Length L_R(ft)	Shy Line Offset - L_s(ft)	Flare Rate (a:b)*
70	480	440	400	360	330	10.0	15 **
60	400	360	330	300	270	8.0	13 **
50	320	290	260	240	210	6.5	11 **
40	240	220	200	180	160	5.0	9 **

*When $L_2 < .5L_s$, L_t shall have a minimum length = $\frac{2a}{b}(.5L_s - L_2)$

Where $\frac{a}{b}$ = flare rate from this table

**Values are for yielding barrier (2'> dynamic deflection)

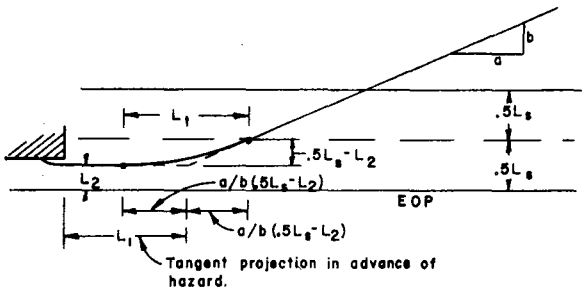
for rigid barriers increase numerator by $.1\left[\frac{V}{10}\right]^2$

V = operating speed (mph)

**Adjustment factor for rigid barriers is $V^2(3.86 \times 10^{-4})$, When V is in Km/hr.

Metric Conversions

1 ft. = 0.305m
1 mph = 1.61km/h



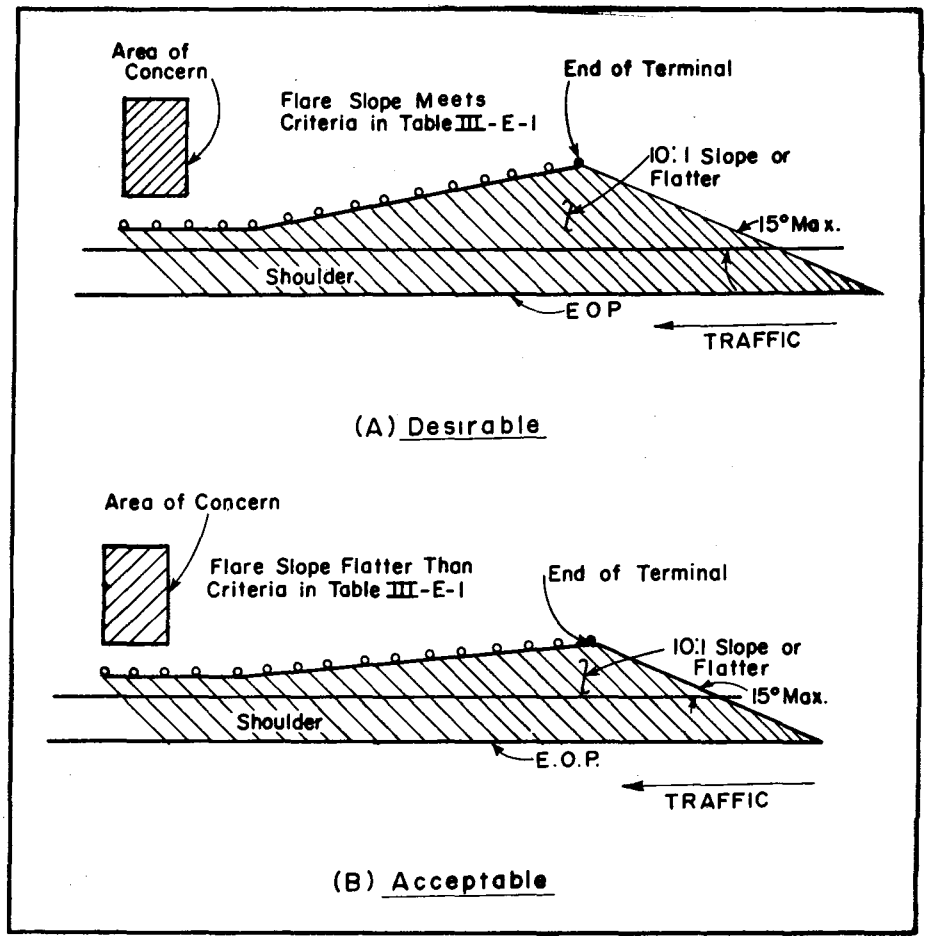


Figure 5.1.10 Suggested Roadside Slopes For Approach Barriers

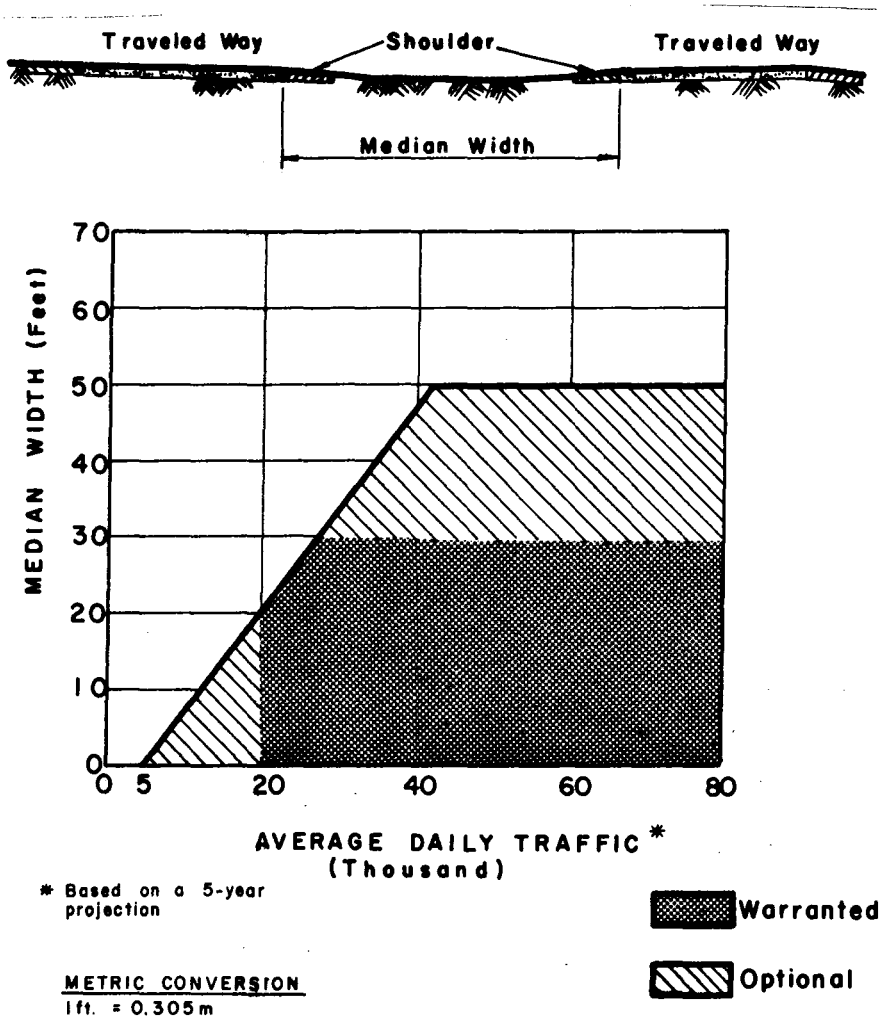


Figure 5.1.11 Median Barrier Warrants

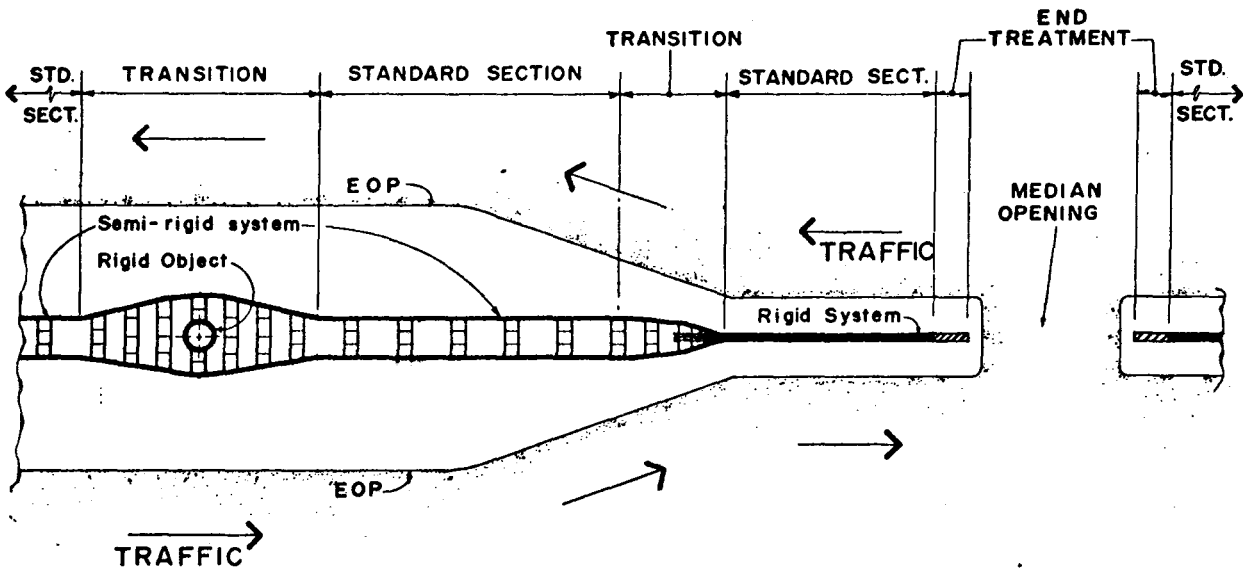


Figure 5.1.12 Definition of Median Barrier Elements

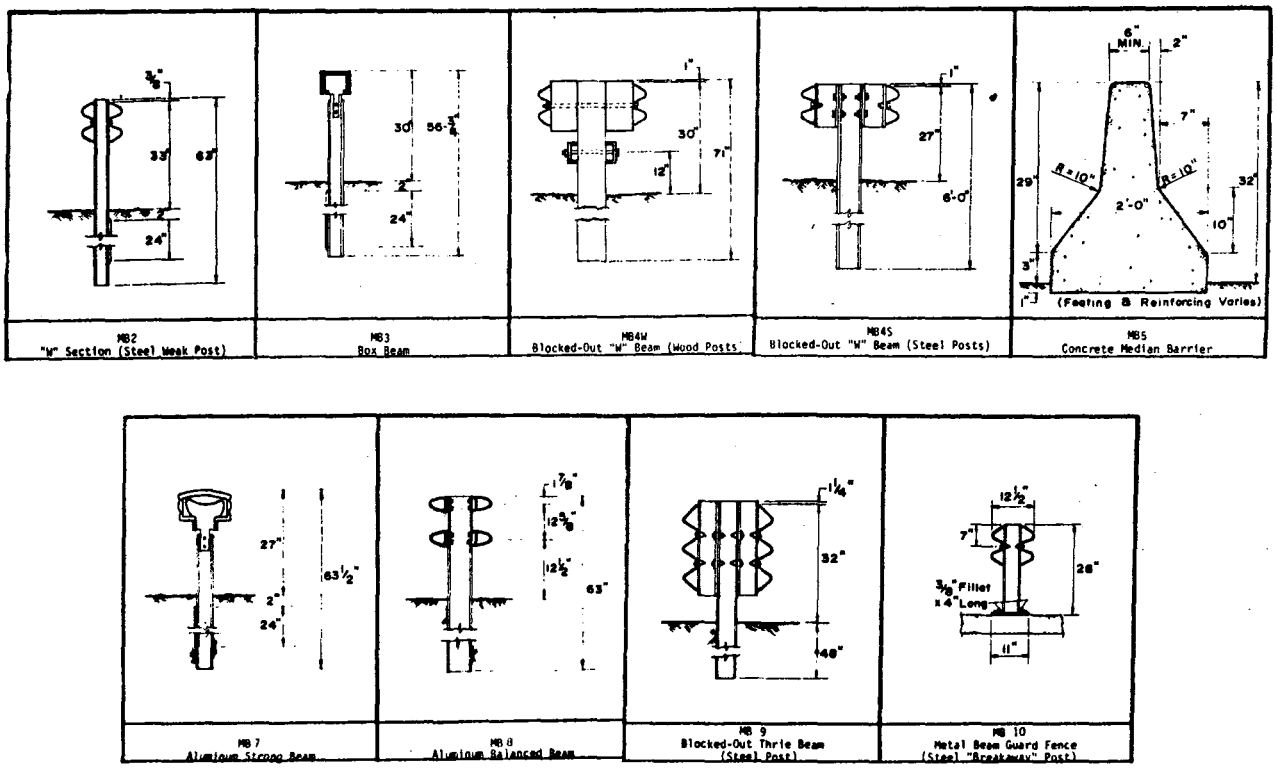


Figure 5.1.13 Operational Median Barrier Systems

information will be gathered following publication of the Guide.

Care should be taken to select a barrier that has dynamic deflection characteristics that are compatible with available median width. Table 5.1.7 presents suggested barrier types relative to available median widths.

In Figure 5.1.13 it is noted that rail heights range from 27 inches (0.69 m) to 33 inches (0.84 m). Barriers utilizing the standard W-Section rail are generally installed at the lower mounting heights, to avoid snagging by the posts. Note that MB4W uses a rub rail to prevent snagging. Concrete median barriers are optimally 32 inches (0.81 m) high to control vehicle trajectory, barrier stability, sight distance, and psychological effects on driver behavior.

5.1.54 Transition Sections

Median barrier transition sections are needed when median barriers of significantly different stiffness are joined, when a median barrier such as a bridge rail, or when a median barrier must be stiffened to shield fixed objects such as luminaire supports or bridge piers. No median barrier transition sections have been determined to fully meet the operational definition; however, the Guide presents details relative to a number of transitions that may be later classified as operational.

Impact performance requirements of median barrier transitions are essentially the same as those for standard sections. Structural requirements of special importantance are summarized as follows:

- All rail splices should be capable of developing full tensile and flexural strength of the weaker rail.
- A flared or sloped connection should be used to avoid snagging of an errant vehicle.
- Strong post median barrier systems must be used in transitions to CMB or bridge rails to parapets or rigid objects.
- Transitions should be stiffened as uniformly as possible over a distance not less than 25 feet (7.6 m).

5.1.55 Terminal Section

An untreated end of a median barrier constitutes an extreme hazard. Impact with the end of a metal beam barrier can result in penetration of the passenger compartment, and impact with the end of a concrete barrier results in a very abrupt stop.

Where the median barrier is terminated within the clear distance of the travel way, a crashworthy end treatment is necessary. There are several methods of developing a crashworthy end treatment including the following:

1) Breakaway Cable Terminal (BCT)
2) Crash Cushion
3) Flaring
4) Earth Berm

There are two variations of the BCT median end treatment, one using wood posts and the other using steel posts with slip bases. Both are considered operational and are illustrated in Figure 5.1.14. One special crash cusion, the G.R.E.A.T. which utilizes hi-dri cell cartridges and steel telescoping thrie beam fender panels, is considered operational. Other types of crash cushions have been used in this application, but are not considered fully operational at this time because of a lack of in-service experience.

Emergency openings through median barriers should be avoided. Where such openings must be provided, possibly the best configuration is the double flared design illustrated in Figure 5.1.15. The ends do not require safety treatment but should be gently sloped for aesthetic purposes.

The barrier also can be buried in an earth berm. This eliminates the blunt end and penetration problems but may result in the vehicle's going airborne or creation of a rollover condition. Since the need for the barrier has been reduced to a point where it can be discontinued, this hazard is considered to be minimal.

5.1.56 Barrier Selection Guidelines

Once it has been determined that a barrier is warranted, a selection must be made. Although the process is complicated by the number of variables involved, the lack of objective criteria and local preferences, there are general guidelines which should be followed.

<u>Goals</u>

- Best protection at the least cost within the constraints of the situation
- Crashworthy transitions to other barriers
- Minimum maintenance requirements
- Aesthetically pleasing appearance

Table 5.1.8 lists many of the pertinent factors in the barrier selection process.

TABLE 5.1.6 SUMMARY OF IMPACT PERFORMANCE OF OPERATIONAL MEDIAN BARRIERS

Barrier System	Code	Dynamic Deflection	Vehicle Acceleration (g's)			Impact Angle	Impact Speed (mph)	Exit Angle
			Lateral	Longitudinal	Total			
Post and Cable	MB1	17.0'	NA	NA	NA	25°	87	NA
Double "W" Section Weak Post	MB2	7.0'	NA	NA	NA	25°	56	NA
Box Beam	MB3	0.75	NA	NA	NA	10°	49	3°
Box Beam	MB3	5.5'	NA	NA	5.3	25°	56	9°
Blocked Out "W" Section with Rub Rails (Wooden Posts)	MB4W	2.0'	NA	NA	NA	25°	69	15°
Blocked Out "W" Section, No Rub Rails (Steel Posts)	MB4S	1.5'	NA	NA	5.7	16°	67	9°
Concrete Median Barrier	MB5	0'	6.0	5.0	NA	15°	61	11.5°
Concrete Median Barrier	MB5	0'	9.0	7.0	NA	25°	62	7°
Thrie Beam	MB9	0.3'	5.3	2.0	NA	17°	54	2°
Thrie Beam	MB9	3.2'	6.3	6.6	NA	25°	66	<10°

TABLE 5.1.7 SUGGESTED MEDIAN BARRIERS AS RELATED TO MEDIAN WIDTH

Median Width	Suggested Barrier
Up to 18 feet	Rigid or Semi-Rigid[2]
18 to 30 feet	Rigid, Semi-Rigid, or Flexible[3]
30 to 50 feet	Semi-Rigid or Flexible

[1] If warranted by Figure IV-A-2.

[2] Semi-rigid system with dynamic deflection greater than one-half of median width not acceptable.

[3] MB1 system not acceptable.

Metric Conversion: 1 ft = 0.3048 m

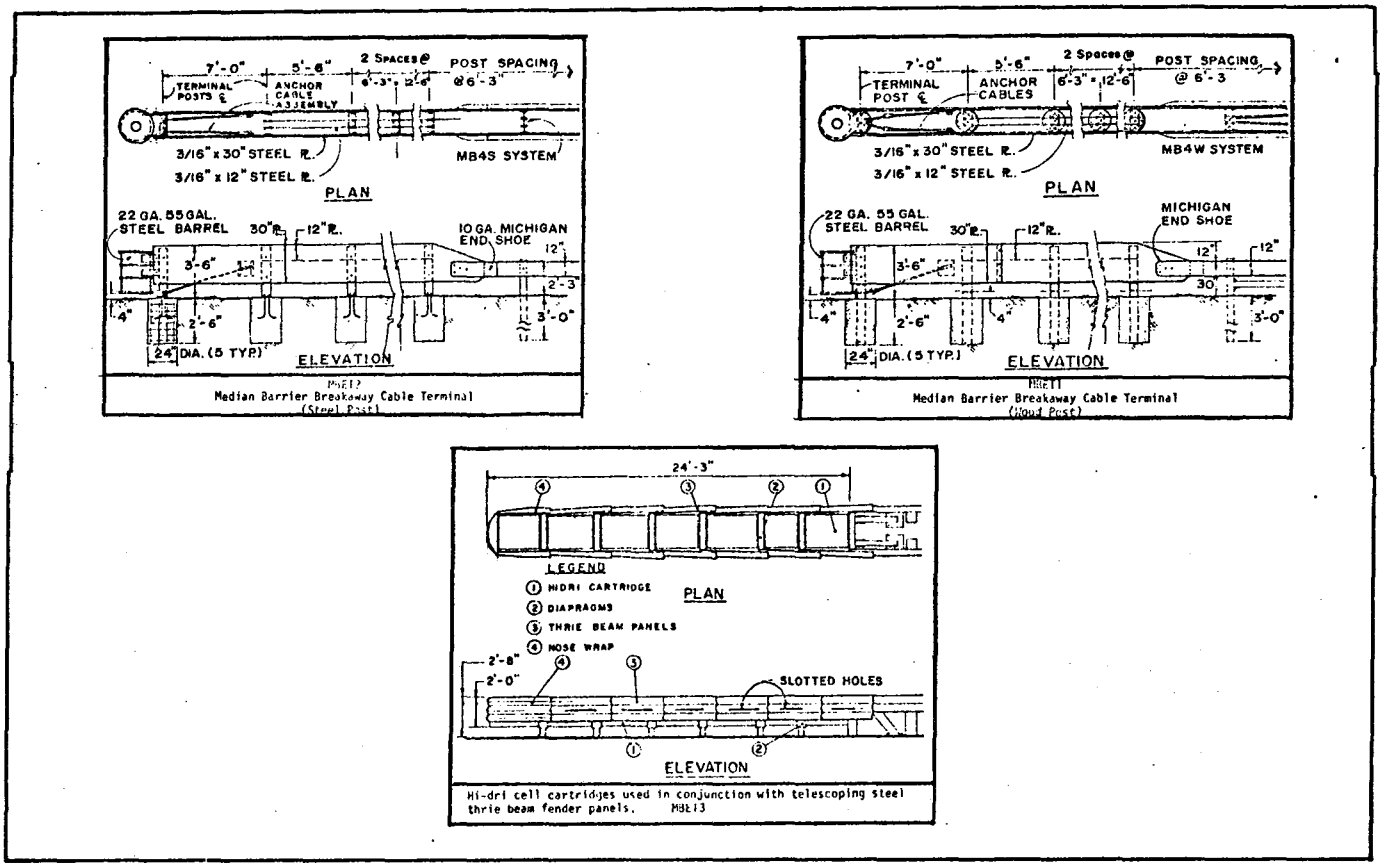


Figure 5.1.14 Operational Median Barrier End Treatments

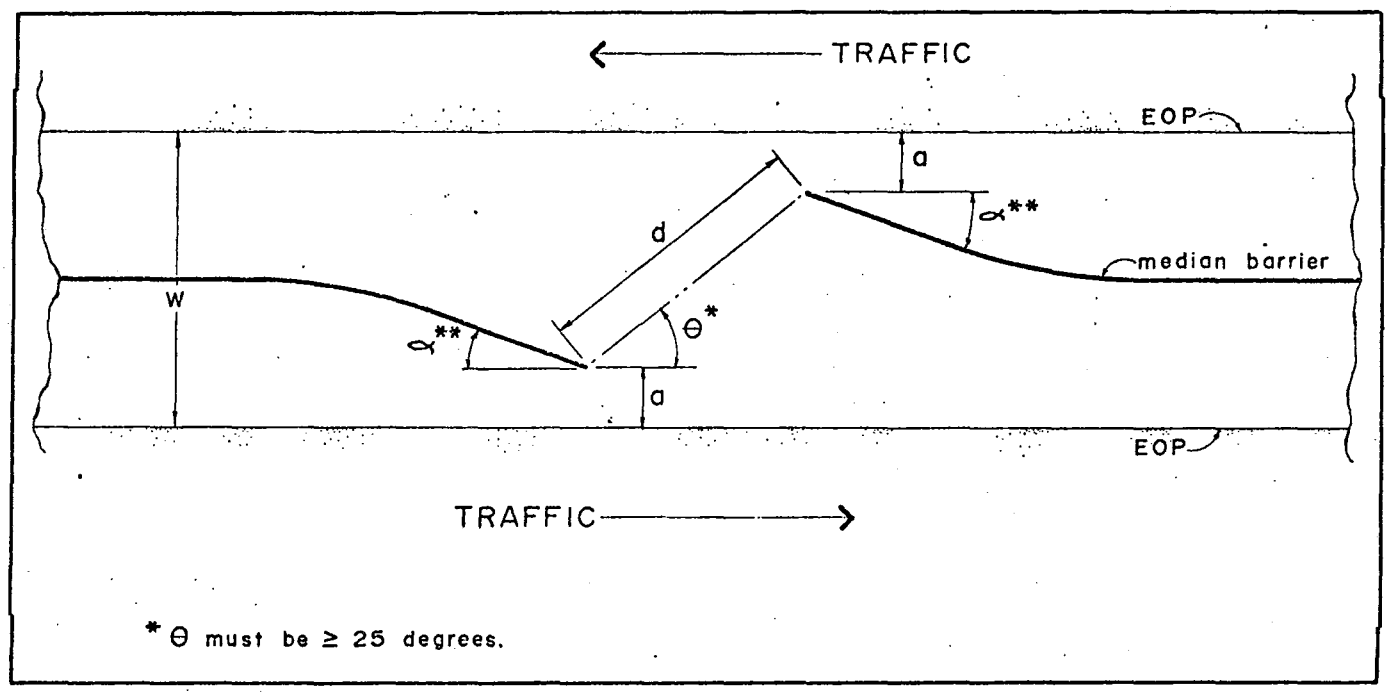


Figure 5.1.15 Suggested Emergency Opening Design for Semi-Rigid or Rigid Systems

Maintenance factors which must be considered are presented in Table 5.1.9.

5.1.57 Maintenance Characteristics of Barriers

The selection of a barrier type should include adequate consideration of the maintenance problem. Table 5.1.9 contains a list of some of the more pertinent considerations. Table 5.1.10 contains a summary of the time and material needs for repair of various barrier systems.

5.1.6 A COST-EFFECTIVENESS SELECTION PROCEDURE

5.1.61 Introduction

Collisions involving vehicles with roadside objects represent a problem inherent to any existing highway facility. Consequently, roadside safety improvement programs have evolved to provide guidance in eliminating those problem locations where attention is vitally needed. For the most part, these programs share the following policy base.

- Obstacles which may be removed should be eliminated.
- Obstacles which may not be removed should be relocated laterally or in a more protected solution.
- Obstacles which may not be moved should be reduced in impact severity. Breakaway devices and flattened side slopes offer such an improvement.
- Obstacles which may not be otherwise treated should be shielded by attenuation or deflection devices.

While the above mentioned points of design summarize the available alternatives, the questions of "where, when or how" are often left unanswered. Limited funds are also a factor most agencies face. The designer is thus confronted with the problem of selecting those alternatives which offer the greatest return in terms of safety benefits.

The purpose of this cost-effective selection procedure is to provide a technique for comparing alternate solutions to problem locations. Present value of the total cost of each alternative is computed over a given period of time, taking into consideration initial costs, maintenance costs, and accident costs. Accident costs incurred by the motorist, including vehicle damage and personal injury, are considered together with accident costs incurred by the highway department or agency. Selection of the alternative with the least total cost would normally be made.

With regard to traffic barriers, the cost-effective procedure can be used to evaluate three alternatives:

1. Remove or reduce hazard so that shielding is unnecessary;
2. Install a barrier; or
3. Do nothing, i.e., leave hazard unshielded.

The third option normally would be cost effective only on low volume and/or low speed facilities, or where the probability of accidents is low. With regard to item 2, the procedure allows one to evaluate any number of barriers that can be used to shield the hazard. Each location and its alternatives should be approached on an individual basis. Through this method the effects of average daily traffic, offset of barrier or hazard, size of barrier or hazard, and the relative severity of the barrier or the hazard can be evaluated.

The procedure presented herein has been adopted from the work of Ross, et.al. (1) and permits objective evaluation of the options at a given site. The procedure included in this document is more generally applicable and is recommended for general use.

5.1.62 Applications

Implementation of the cost-effective procedure primarily involves the determination of several input values. The computations are simple and require only basic mathematics. It should be noted that during the course of the text, the word "obstacle" is used quite frequently. In this context, the term is meant ot apply to either a hazard or improvement, whichever the case may be. The following steps summarize the procedure to be followed in the cost-effective analysis.

1. From existing or proposed geometry determine the following:

   A = lateral placement of the roadside obstacle from EOP

   L = horizontal length of the roadside obstacle

   W = width of the roadside obstacle

2. From volume counts or estimates, determine the average daily traffic, ADT (vehicles per day). This value should represent the two-way volume flow.

3. Determine the encroachment frequency, E (vehicle encroachments per mile per year), from Figure 5.1.16. Figure 5.1.16 was obtained from data discussed previously. Other available data or

TABLE 5.1.8 SELECTION CRITERIA FOR ROADSIDE BARRIERS

ITEM	CONSIDERATIONS
A. Deflection	1. Space available behind barrier must be adequate to permit dynamic deflection of barriers.
B. Strength and Safety	1. System should contain and redirect vehicle at design conditions.
	2. System should be least hazardous available, consistent with costs and other considerations.
C. Maintenance	1. Collision maintenance.
	2. Routine maintenance.
	3. Environmental conditions.
D. Compatibility	1. Can system be transitioned to other barrier systems?
	2. Can system be terminated properly?
E. Costs	1. Initial costs.
	2. Maintenance costs.
	3. Accident costs to motorist.
F. Field Experience	1. Documented evidence of barrier's performance in the field.
G. Aesthetics	1. Barrier should have a pleasing appearance.
H. Promising New Designs	1. It may be desirable to install new systems on an experimental basis.

TABLE 5.1.9 MAINTENANCE FACTORS INFLUENCING ROADSIDE BARRIER SELECTION

ITEM	CONSIDERATIONS
A. Collision Maintenance	1. Typical crew size
	2. Typical man-hours to repair (exposure)
	3. Typical barrier damage
	4. Special equipment
	5. Ability of rail to be repaired or straightened
	6. Salvage value
	7. Level of working knowledge
B. Routine Maintenance	1. Cleaning and painting
	2. Mowing and clearing vegetation
C. Environmental Conditions	1. Snow or sand drifting
	2. Snow or sand removal
	3. Weathering or corrosion due to environment or chemical effects
D. Material and Storage Requirements	1. Dependence on a number of parts
	2. Availability of parts
	3. Storage facilities required

TABLE 5.1.10 COLLISION REPAIR DATA FOR BARRIERS

System	Typical Crew Size	Typical Material Repaired or Replaced		Average Refurbishment Time (Man-Hours/Foot of Rail)
		Rail (ft.)	Posts	
G1-Cable Guardrail	UNAV	112	8	0.30
G2-W-Beam on Steel Weak Posts	UNAV	45	4	0.33
G3-Box Beam	5-6	32	5	0.92
G4(1W)-Blocked Out W-Beam on Wood Posts	4	35	4	0.35
G4(1S)-Blocked Out W-Beam on Steel Posts	3-4	38	4-5	0.32
MB1-Cable Barrier	3-4	75	8	0.10
MB1-Cable Barrier	3-4	75	8	0.13
MB1-Cable Barrier	3-4	75	8	0.055
MB1-Cable Barrier	3-4	75	8	0.083
MB2-W-Beam on Steel Weak Posts	3-4	53	4-5	0.32
MB3-Box Beam	UNAV	36	4	0.61
MB4W-Blocked Out W-Beam on Wood Posts	4-5	25	4	0.36
MB4S-Blocked Out W-Beam on Steel Posts	4-5	57	4-5	0.36
MB5-Concrete Median Barriers	4-5	UNAV	Not Applicable	3.50
MB7-Aluminum Strong Beam[e]	4	66	11	0.48
MB7-Aluminum Strong Beam[f]	4-6	66	11	0.73
MB8-Aluminum Balanced Beam	----------NO DATA AVAILABLE----------			
MB9-Blocked Out Thrie Beam	----------NO DATA AVAILABLE----------			
MB10-W-Beam on Steel Breakaway Posts	5-7	56	2	0.59

Metric Conversion: 1 ft = 0.305 m

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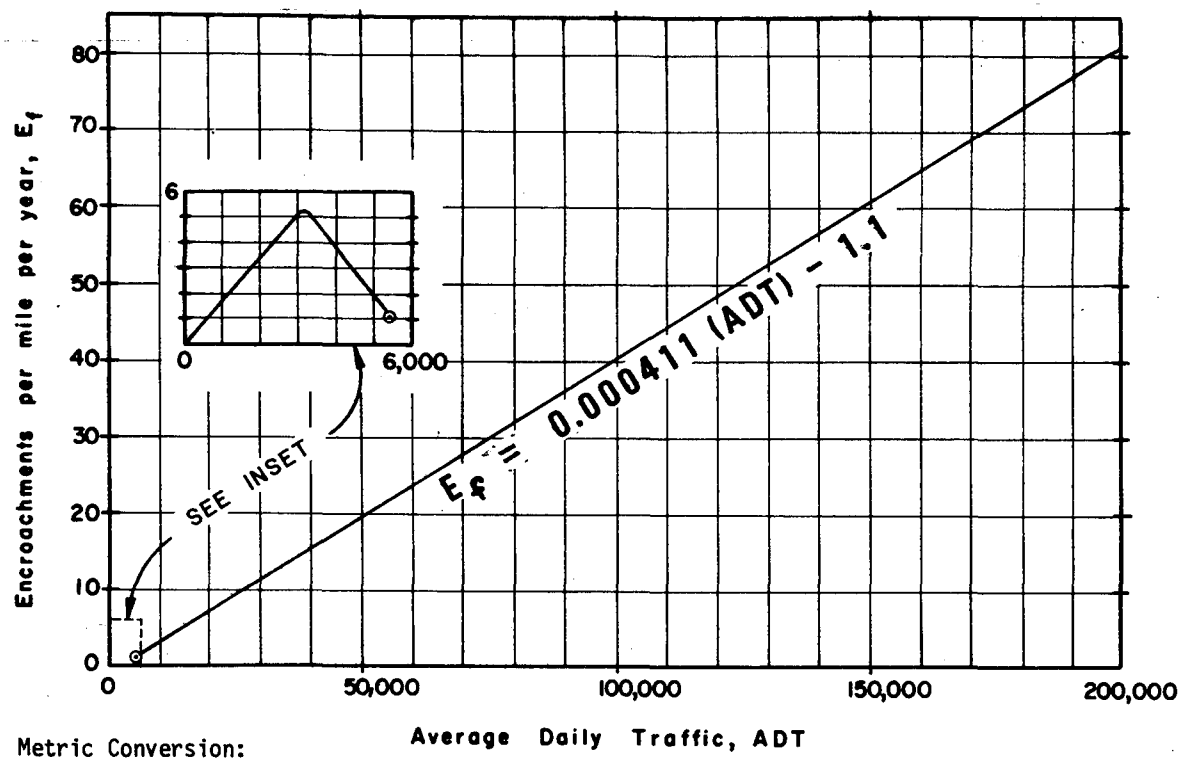

Metric Conversion:

1 Encroachment/mi =

.6214 Encroachments/km

Figure 5.1.16 Encroachment Frequency

adjustments of the above may be used at the discretion of the designer. This latitude offers an option to the user and helps to preserve the generality of the model.

4. Determine the collision frequency, C (accidents per year), from the appropriate nomographs given in Figures 5.1.17 and 5.1.18 (dependent on obstacle length). The nomographs combine the over-all geometry with a given encroachment frequency to yield the collision frequency. Collision frequency, C, is the predicted number of times a given obstacle will be impacted by an errant vehicle per year. The nomographs are used in the following manner.

- Locate and mark the encroachment frequency, E_f, on vertical axis (1).

- On horizontal axis (2) locate the lateral placement, A, and construct a vertical reference line the full height of the graph.

- Locate and mark the point where the lateral placement reference line intersects the width, W, curve in intersection.

- Project a horizontal line to the right from that point to the vertical axis (3) and mark the point of intersection.

- Locate and mark the point where the lateral placement reference line intersects the length, L, curve in consideration.

- Project a horizontal line to the left from this point to the vertical axis (4) and mark the point of intersection.

- Lay a straight-edge across the points marked on (3) and (4) and construct a line to intersect vertical axis (5). Mark the point of intersection.

- From the point determined construct a line to vertical axis (6) keeping approximately parallel to guidelines. Mark the point of intersection.

- Lay a straight-edge across the marked points on vertical axes (1) and (6) and construct a line connecting the two. Read the collision frequency, C_f, where the line intersects the collision frequency axis.

An example demonstrating the application of one of the nomographs is given in Figure 5.1.19. It may be necessary to adjust the collision frequency in locations where the geometry and traffic conditions are critical. Off-ramp gore areas represent such a situation, and an upward adjustment factor of 3 has been suggested. Mathematically, the collision frequency is given in the expression below.

$$C_f = \frac{E_f}{10,560} \left[(L + 62.9) \cdot P[Y \geq A] + 5.14 \sum_{J=1}^{J=W} P\left[Y \geq A + 6.0 + \frac{2J - 1}{2}\right] \right]$$

where,

the variables A, L, W and E are as previously defined

and,

Y = the lateral displacement, in feet (metres), of the encroaching vehicle, measured from the edge of the traveled way to the longitudinal face of the roadside obstacle;

P[Y ≥ ...] = probability of a vehicle lateral displacement greater than some value. These probabilities may be taken from Figure 5.1.20;

and

J = the number of the 1-ft (.3 m) wide obstacle-width increment under investigation. (If the obstacle is not a whole number of feet (metres) wide, the number of increments investigated is obtained by rounding the width down to the nearest whole foot (metre).

5. Assign a severity index to the obstacle of concern. Hazards can be denoted according to the hazard classification codes given in Table 5.1.11. It is suggested that the severity index be chosen on a scale of 0 to 10 according to the criteria given in Table 5.1.12. For example, if it is estimated that an impact with the obstacle will result in injuries or a fatality 60 percent of the time, select an index of 7. Corresponding to the index is an estimated accident cost which includes those costs associated with vehicle damage and occupant injuries and/or fatalities. Figure 5.1.21 is a graphic representation of accident cost versus severity index. Discretion is advised in assigning severity indices and the designer is encouraged to exhaust all available objective data before resorting to judgment.

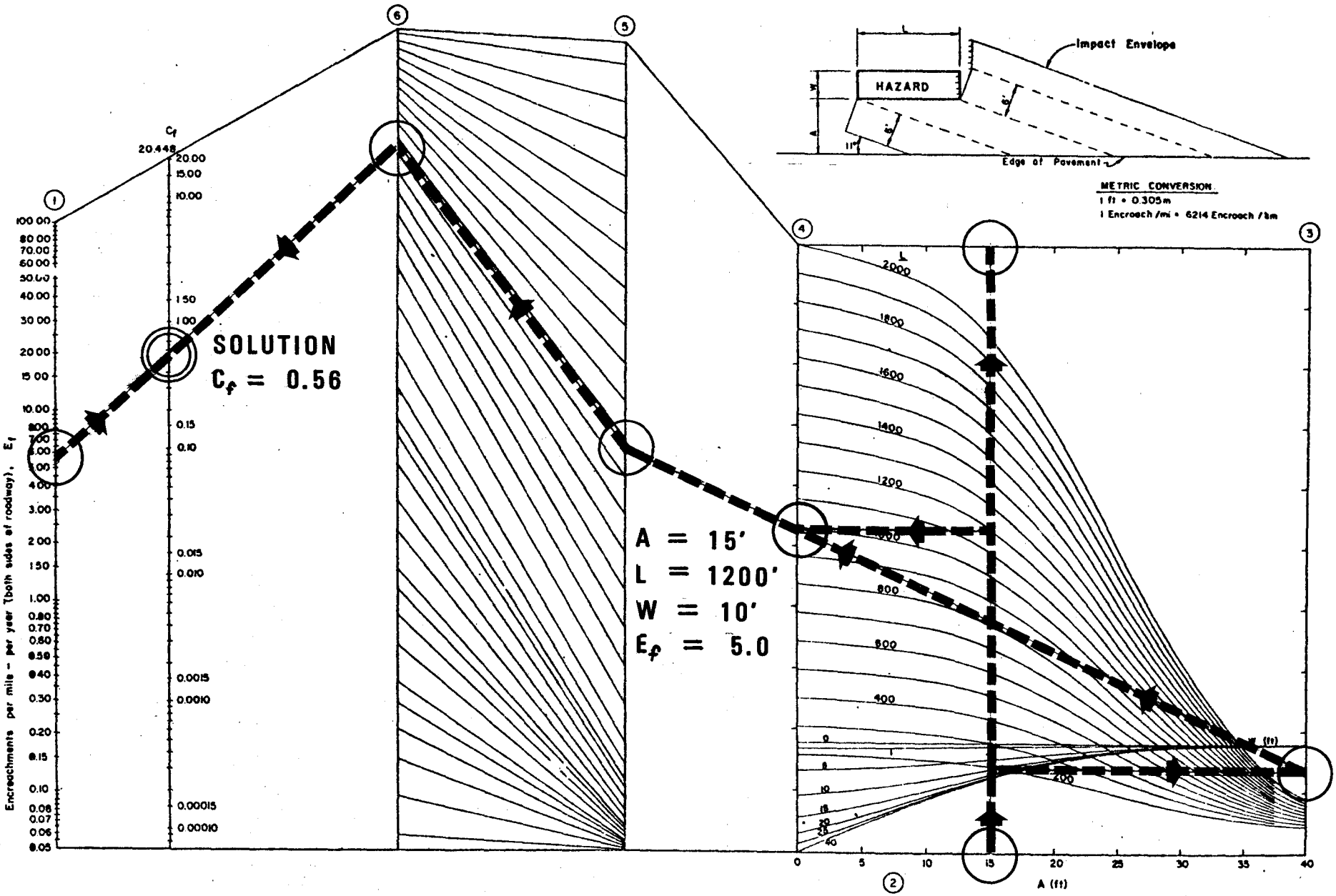

Figure 5.1-19 Collision Frequency Nomograph, Length From 200-2000 Feet (Example Solution)

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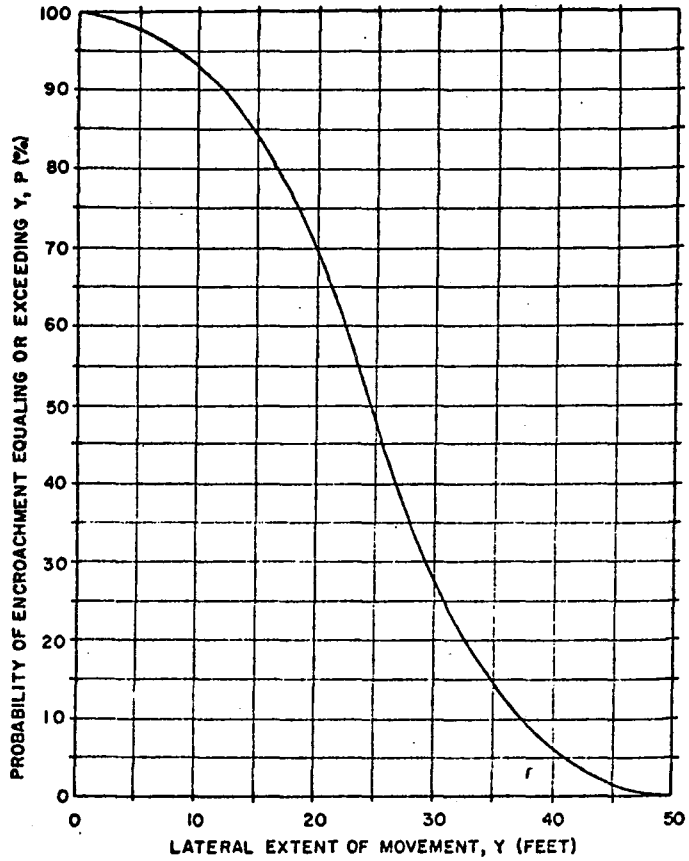

1 ft. = .305 m

Figure 5.1.20 Lateral Displacement Distribution

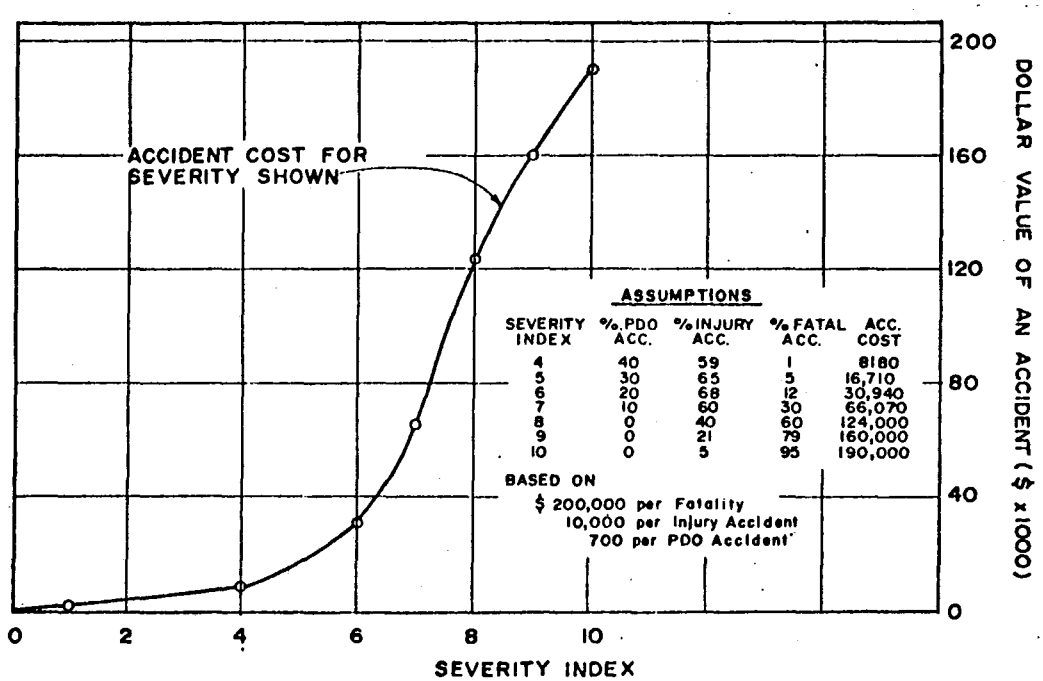

Figure 5.1.21 Average Occupant Injury and Vehicle Damage Costs (2)

TABLE 5.1.11 HAZARD CLASSIFICATION CODES

Note: Circled Codes denote Point Hazard

Identification Code	Descriptor Codes
(01.) Utility Poles	(00)
(02.) Trees	(00)
(03.) Rigid Signpost	(01) single-pole-mounted (02) double-pole-mounted (03) triple-pole-mounted (04) cantilever support (05) overhead sign bridge
(04.) Rigid Base Luminaire Support	(00)
05. Curbs	(01) mountable design (02) non-mountable design less than 10 inches high (03) barrier design greater than 10 inches high
06. Guardrail or Median Barrier	(01) w-section with standard post spacing (6 ft-3 in.) (including departing guardrail at bridge) (02) w-section with other than standard post spacing (including departing guardrail at bridge) (03) approach guardrail to bridge--decreased post spacing (3 ft-1 in.) adjacent to bridge (04) approach guardrail to bridge--post spacing not decreased adjacent to bridge (05) post and cable (06) Metal Beam Guard Fence (Barrier) (in median) (07) median barrier (CMB design or equivalent
07. Roadside Slope	(01) sod positive slope (02) sod negative slope (03) concrete-faced positive slope (04) concrete-faced negative slope (05) rubble rip-rap positive slope (06) rubble rip-rap negative slope

GUARDRAIL END TREATMENT CODES

1 Not beginning or ending at structure-Safety treated
2 Not beginning or ending at structure-Not safety treated
3 Beginning or ending at structure-Full-beam connection
4 Beginning or ending at structure-Not full-beam connection

TABLE 5.1.11 (cont.)

08.	Ditch (includes erosion, rip-rap runoff ditches, etc.--does not include ditches formed by intersection of front and back slopes	(00)
09.	Culverts	(01) headwall (or exposed end of pipe culvert) (02) gap between culverts on parallel roadways (03) sloped culvert with grate (04) sloped culvert without grate
10.	Inlets	(01) raised drop inlet (tabletop) (02) depressed drop inlet (03) sloped inlet
11.	Roadway under Bridge Structure	(01) bridge piers (02) bridge abutment, vertical face (03) bridge abutment, sloped face
12.	Roadway over Bridge Structure	(01) open gap between parallel bridges (02) closed gap between parallel bridges (03) rigid bridgerail--smooth and continuous construction (04) semi-rigid bridgerail--smooth and continuous construction (05) other bridgerail--probable penetration, snagging, pocketing or vaulting (06) elevated gore abutment
13.	Retaining Wall	(01) face (02) exposed end
14.	Miscellaneous Point Hazards	(01) pedestal base > 6 in. above ground, < 1 ft. diam. (02) pedestal base > 6 in. above ground, > 1 ft. diam. (03) historical monument < 1 ft. wide (04) historical monument > 1 ft. wide

TABLE 5.1.12 SEVERITY INDICES

Identification Code	Descriptor Code	End Treatment Code Beginning	End Treatment Code Ending	Severity-Index Survey	Severity-Index Adjusted
1. Utility Pole					
1	0	-	-	7.1	27.5
2. Trees					
2	0	-	-	8.0	50.0
3. Rigid Signpost					
3	1	-	-	4.7	8.9
3	2	-	-	7.2	30.0
3	3	-	-	7.2	30.0
3	4	-	-	7.2	30.0
3	5	-	-	8.1	52.5
4. Rigid Base Luminaire Support					
4	0	-	-	7.5	37.5
5. Curbs					
5	1	-	-	2.4	2.4
5	2	-	-	4.1	4.7
5	3	-	-	3.7	3.7
6. Guardrail or Median Barrier					
6	1	1	1	3.7	3.7
6	1	1	2	4.0	4.0
6	1	1	3	3.6	3.6
6	1	1	4	4.5	7.5
6	1	2	1	5.6	15.2
6	1	2	2	5.7	15.9
6	1	2	3	5.3	13.1
6	1	2	4	5.7	15.9
6	1	3	1	3.3	3.3
6	1	3	2	3.3	3.3
6	1	3	3	3.3	3.3
6	1	3	4	4.6	8.2
6	1	4	1	4.5	7.5
6	1	4	2	4.7	8.9
6	1	4	3	4.5	7.5
6	1	4	4	5.0	11.0
6	2	1	1	3.9	3.9
6	2	1	2	4.2	5.4
6	2	1	3	3.8	3.8
6	2	1	4	4.7	8.9
6	2	2	1	5.8	16.6
6	2	2	2	5.9	17.3
6	2	2	3	5.5	14.5
6	2	2	4	5.9	17.3
6	2	3	1	3.5	3.5
6	2	3	2	3.5	3.5
6	2	3	3	3.5	3.5
6	2	3	4	4.8	9.6
6	2	4	1	4.7	8.9
6	2	4	2	4.9	10.3
6	2	4	3	4.7	8.9
6	2	4	4	5.0	11.0
6	3	1	1	3.7	3.7
6	3	1	2	4.0	4.0
6	3	1	3	3.3	3.3
6	3	1	4	4.5	7.5
6	3	2	1	5.6	15.2
6	3	2	2	5.0	11.0
6	3	2	3	3.9	3.9
6	3	2	4	5.0	11.0
6	3	3	1	3.2	3.2
6	3	3	2	3.2	3.2
6	3	3	3	3.2	3.2
6	3	3	4	4.4	6.8
6	3	4	1	4.0	4.0
6	3	4	2	4.5	7.5
6	3	4	3	3.9	3.9
6	3	4	4	4.7	8.9
6	4	1	1	3.7	3.7
6	4	1	2	4.0	4.0
6	4	1	3	3.6	3.6
6	4	1	4	4.5	7.5
6	4	2	1	5.6	15.2
6	4	2	2	5.7	15.9
6	4	2	3	5.3	13.1
6	4	2	4	5.7	15.9
6	4	3	1	3.3	3.3
6	4	3	2	3.3	3.3
6	4	3	3	3.3	3.3
6	4	3	4	4.6	8.2
6	4	4	1	4.5	7.5
6	4	4	2	4.7	8.9

TABLE 5.1.12 SEVERITY INDICES (cont.)

Identification Code	Descriptor Code	End Treatment Code Beginning	End Treatment Code Ending	Severity-Index Survey	Severity-Index Adjusted
6	4	4	3	4.5	7.5
6	4	4	4	5.0	11.0
6	5	1	1	3.9	3.9
6	5	1	2	3.9	3.9
6	5	1	3	3.9	3.9
6	5	1	4	3.9	3.9
6	5	2	1	3.9	3.9
6	5	2	2	3.9	3.9
6	5	2	3	3.9	3.9
6	5	2	4	3.9	3.9
6	5	3	1	3.9	3.9
6	5	3	2	3.9	3.9
6	5	3	3	3.9	3.9
6	5	3	4	3.9	3.9
6	5	4	1	3.9	3.9
6	5	4	2	3.9	3.9
6	5	4	3	3.9	3.9
6	5	4	4	3.9	3.9
6	6	1	1	4.4	6.8
6	6	1	2	4.4	6.8
6	6	1	3	4.4	6.8
6	6	1	4	5.0	11.0
6	6	2	1	5.6	15.2
6	6	2	2	5.7	15.9
6	6	2	3	5.3	13.1
6	6	2	4	5.7	15.9
6	6	3	1	4.0	4.0
6	6	3	2	4.4	6.8
6	6	3	3	4.0	4.0
6	6	3	4	4.6	8.2
6	6	4	1	4.5	7.5
6	6	4	2	4.7	8.9
6	6	4	3	4.5	7.5
6	6	4	4	5.0	11.0
6	7	1	1	4.2	5.4
6	7	1	2	4.2	5.4
6	7	1	3	4.2	5.4
6	7	1	4	4.2	5.4
6	7	2	1	4.2	5.4
6	7	2	2	4.2	5.4
6	7	2	3	4.2	5.4
6	7	2	4	4.2	5.4
6	7	3	1	4.2	5.4
6	7	3	2	4.2	5.4
6	7	3	3	4.2	5.4
6	7	3	4	4.2	5.4
6	7	4	1	4.2	5.4
6	7	4	2	4.2	5.4
6	7	4	3	4.2	5.4
6	7	4	4	4.2	5.4

Identification Code	Descriptor Code	End Treatment Code Beginning	End Treatment Code Ending	Severity-Index Survey	Severity-Index Adjusted
7. Roadside Slope					
7	1	-	-	3.0	3.0
7	2	-	-	3.0	3.0
7	3	-	-	2.5	2.5
7	4	-	-	2.5	2.5
7	5	-	-	5.1	11.7
7	6	-	-	5.1	11.7
8. Ditch					
8	0	-	-	0.0	0.0
9. Culverts					
9	1	-	-	7.9	47.5
9	2	-	-	5.5	14.5
9	3	-	-	3.3	3.3
9	4	-	-	7.7	42.5
10. Inlets					
10	1	-	-	5.7	15.9
10	2	-	-	3.1	3.1
10	3	-	-	3.3	3.3
11. Roadway Under Bridge Structure					
11	1	-	-	9.3	82.5
11	2	-	-	9.3	82.5
11	3	-	-	2.5	2.5
12. Roadway Over Bridge Structure					
12	1	-	-	7.2	30.0
12	2	-	-	5.5	14.5
12	3	-	-	3.3	3.3
12	4	-	-	3.0	3.0
12	5	-	-	9.3	82.5
12	6	-	-	9.3	82.5
13. Retaining Wall					
13	1	-	-	3.3	3.3
13	2	-	-	9.3	82.5
14. Miscellaneous Point Hazards					
14	1	-	-	7.5	37.5
14	2	-	-	9.3	82.5
14	3	-	-	7.5	37.5
14	4	-	-	9.3	82.5

Metric Equivalent Equation

$$C = \frac{E_f}{2,000} [(L + 19.2) \cdot P[Y \geq A] + 5.14 \sum_{J=1}^{J=W} P[Y \geq A + 1.8 + \frac{2J - 1}{2}]]$$

E_f in Encroachments/km/yr

L, Y, A, and W in metres

(The width of J may be taken as 1 metre with the number of J units equal W rounded to the nearest whole number.)

This equation may be implemented directly into the cost analysis or used as a double-check for the collision frequency nomographs. Computation of the collision frequency for multiple objects requires special procedures.

6. Determine the initial cost of the obstacle, C_I. If it is already in place, its initial cost may be assumed to equal zero. For example, if a group of median bridge piers had been in existence for ten years, then the initial cost of a no improvement alternative would be taken to be zero. On the other hand, improvements to such a hazard would require initial expenditures which should be so designated.

7. Determine the average damage cost to the obstacle per accident, C_D (present dollars).

8. Determine the average maintenance cost per year, C_M, associated with the upkeep of the obstacle (present dollars).

9. Determine the average occupant injury and vehicle damage cost per accident, C_{OVD}, which would be expected as a result of a collision (present dollars). Table 5.1.12 or Figure 5.1.21 may be used to determine C_{OVD} in the absence of mroe definitive date. Direct interpolation of the cost table is suggested to increase the occurrence of the estimate.

10. Determine the useful life, T, of the obstacle (years).

11. Determine the capital recovery and sinking fund factors, CRF and SF for the useful life. T, and a current interest rate come from Tables 5.1.13 and 5.1.14.

12. Estimate the expected salvage value of the obstacle, C_S, at the end of its useful life (future dollars).

13. Calculate the total annual cost, C_{A_T}, from the following equation:

$$C_{A_T} = C_I [CRF] + C_D C_f + C_M + C_{OVD} C_f - C_s (SF)$$

or, to determine those costs which are directly incurred by the highway department (or implementing agency), (C_{A_D}), use the equation below:

$$C_{A_D} = C_I [CRF] + C_D C_f + C_M - C_s (SF)$$

These total annual costs represent an estimated value related to some appurtenance/barrier. Any number of locations or alternatives may be evaluated by utilizing this method, and a priority listing may be established. This weighting scheme provides some insight as to where the greatest return in safety may be realized.

Summary of Variable Definitions

A = lateral placement of the roadside obstacle from EOP (feet) [metre]

L = horizontal length of the roadside obstacle (feet) [metre]

W = width of the roadside obstacle (feet) [metre]

ADT = average daily traffic (vehicles per day, two-way)

E_f = encroachment frequency (encroachments per mile per year) [encroachments per kilometre per year]

C_f = collision frequency (accidents per year)

SI = severity index

CI = initial cost of the obstacle (present dollars)

C_D = average damage cost per accident incurred to the obstacle (present dollars)

C_M = average maintenance cost per year for the obstacle (present dollars)

C_{OVD} = average occupant injury and vehicle damage cost per accident (present dollars)

C_S = estimated salvage value of the obstacle (future dollars)

C_T = total present worth cost associated with the obstacle (dollars)

C_{TD} = total present worth direct cost associated with the obstacle (dollars)

CRF, SF = economic factors for some current interest rate

TABLE 5.1.13 CAPITAL RECOVERY FACTORS (CRF)

Useful Life T (years)	Interest Rate i (Percent)												
	0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
1	1.000	1.010	1.020	1.030	1.040	1.050	1.060	1.070	1.080	1.090	1.100	1.110	1.120
2	0.500	0.508	0.515	0.523	0.530	0.538	0.546	0.553	0.561	0.567	0.576	0.584	0.592
3	0.333	0.340	0.347	0.353	0.360	0.367	0.374	0.381	0.388	0.395	0.402	0.409	0.416
4	0.250	0.256	0.263	0.269	0.275	0.282	0.288	0.295	0.302	0.302	0.315	0.322	0.329
5	0.200	0.206	0.212	0.218	0.225	0.231	0.237	0.244	0.250	0.257	0.264	0.271	0.277
6	0.167	0.173	0.179	0.185	0.191	0.197	0.203	0.210	0.216	0.222	0.230	0.236	0.243
7	0.143	0.149	0.155	0.161	0.167	0.173	0.179	0.186	0.192	0.199	0.205	0.212	0.219
8	0.125	0.131	0.137	0.142	0.149	0.155	0.161	0.167	0.174	0.181	0.187	0.194	0.201
9	0.111	0.116	0.123	0.128	0.134	0.141	0.147	0.153	0.160	0.167	0.174	0.181	0.188
10	0.100	0.106	0.111	0.117	0.123	0.130	0.136	0.142	0.149	0.156	0.163	0.170	0.176
11	0.091	0.096	0.102	0.108	0.114	0.120	0.127	0.133	0.140	0.147	0.154	0.161	0.168
12	0.083	0.089	0.095	0.100	0.107	0.113	0.119	0.126	0.133	0.140	0.147	0.154	0.161
13	0.077	0.082	0.088	0.094	0.100	0.106	0.113	0.120	0.127	0.134	0.141	0.148	0.155
14	0.071	0.077	0.083	0.089	0.095	0.101	0.108	0.114	0.121	0.128	0.136	0.143	0.150
15	0.067	0.072	0.078	0.084	0.090	0.096	0.103	0.110	0.117	0.124	0.131	0.139	0.147
16	0.063	0.068	0.074	0.080	0.086	0.092	0.099	0.106	0.113	0.120	0.128	0.136	0.143
17	0.059	0.064	0.070	0.076	0.082	0.089	0.095	0.102	0.110	0.117	0.125	0.132	0.140
18	0.056	0.061	0.067	0.073	0.079	0.086	0.092	0.099	0.107	0.114	0.122	0.130	0.137
19	0.053	0.058	0.064	0.069	0.076	0.083	0.090	0.097	0.104	0.112	0.120	0.128	0.136
20	0.050	0.055	0.061	0.067	0.074	0.080	0.087	0.094	0.102	0.110	0.117	0.126	0.134
21	0.048	0.053	0.059	0.065	0.071	0.078	0.085	0.092	0.100	0.108	0.116	0.124	0.132
22	0.045	0.051	0.057	0.063	0.069	0.076	0.083	0.090	0.098	0.106	0.114	0.122	0.130
23	0.043	0.049	0.055	0.061	0.067	0.074	0.081	0.089	0.096	0.104	0.113	0.121	0.129
24	0.042	0.047	0.053	0.059	0.066	0.072	0.080	0.087	0.095	0.103	0.111	0.120	0.128
25	0.040	0.045	0.051	0.057	0.064	0.071	0.078	0.086	0.094	0.102	0.110	0.118	0.127
26	0.038	0.044	0.050	0.056	0.063	0.070	0.077	0.085	0.093	0.101	0.109	0.118	0.127
27	0.037	0.042	0.048	0.055	0.061	0.068	0.076	0.083	0.091	0.100	0.108	0.117	0.126
28	0.036	0.041	0.047	0.053	0.060	0.067	0.075	0.082	0.090	0.099	0.107	0.116	0.125
29	0.034	0.040	0.046	0.052	0.059	0.066	0.074	0.081	0.090	0.098	0.106	0.115	0.125
30	0.033	0.039	0.045	0.051	0.058	0.065	0.073	0.081	0.089	0.097	0.106	0.115	0.124

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TABLE 5.1.14 SINKING FUND FACTOR (SF)

Useful Life T (years)	Interest Rate i (Percent)												
	0.1	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
1	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
2	0.500	0.498	0.495	0.493	0.490	0.488	0.486	0.483	0.481	0.477	0.476	0.474	0.472
3	0.333	0.330	0.327	0.323	0.320	0.317	0.314	0.311	0.308	0.305	0.302	0.299	0.296
4	0.250	0.246	0.243	0.239	0.235	0.232	0.228	0.225	0.222	0.219	0.215	0.212	0.209
5	0.200	0.196	0.192	0.188	0.185	0.181	0.177	0.174	0.170	0.167	0.164	0.161	0.157
6	0.167	0.163	0.159	0.155	0.151	0.147	0.143	0.140	0.136	0.132	0.130	0.126	0.123
7	0.143	0.139	0.135	0.131	0.127	0.123	0.119	0.116	0.112	0.109	0.105	0.102	0.099
8	0.125	0.121	0.117	0.112	0.109	0.105	0.101	0.097	0.094	0.091	0.087	0.084	0.081
9	0.111	0.106	0.103	0.098	0.094	0.091	0.087	0.083	0.080	0.077	0.074	0.071	0.068
10	0.100	0.096	0.091	0.087	0.083	0.080	0.076	0.072	0.069	0.066	0.063	0.060	0.056
11	0.091	0.086	0.082	0.078	0.074	0.070	0.067	0.063	0.060	0.057	0.054	0.051	0.048
12	0.083	0.079	0.075	0.070	0.067	0.063	0.059	0.056	0.053	0.050	0.047	0.044	0.041
13	0.077	0.072	0.068	0.064	0.060	0.056	0.053	0.050	0.047	0.044	0.041	0.038	0.035
14	0.071	0.067	0.063	0.059	0.055	0.051	0.048	0.044	0.041	0.038	0.036	0.033	0.030
15	0.067	0.062	0.058	0.054	0.050	0.046	0.043	0.040	0.037	0.034	0.031	0.029	0.027
16	0.063	0.058	0.054	0.050	0.046	0.042	0.039	0.036	0.033	0.030	0.028	0.026	0.023
17	0.059	0.054	0.050	0.046	0.042	0.039	0.035	0.032	0.030	0.027	0.025	0.022	0.020
18	0.056	0.051	0.047	0.043	0.039	0.036	0.032	0.029	0.027	0.024	0.022	0.020	0.017
19	0.053	0.048	0.044	0.039	0.036	0.033	0.030	0.027	0.024	0.022	0.020	0.018	0.016
20	0.050	0.045	0.041	0.037	0.034	0.030	0.027	0.024	0.022	0.020	0.017	0.016	0.014
21	0.048	0.043	0.039	0.035	0.031	0.028	0.025	0.022	0.020	0.018	0.016	0.014	0.012
22	0.045	0.041	0.037	0.033	0.029	0.026	0.023	0.020	0.018	0.016	0.014	0.012	0.010
23	0.043	0.039	0.035	0.031	0.027	0.024	0.021	0.019	0.016	0.014	0.013	0.011	0.009
24	0.042	0.037	0.033	0.029	0.026	0.022	0.020	0.017	0.015	0.013	0.011	0.010	0.008
25	0.040	0.035	0.031	0.027	0.024	0.021	0.018	0.016	0.014	0.012	0.010	0.008	0.007
26	0.038	0.034	0.030	0.026	0.023	0.020	0.017	0.015	0.013	0.011	0.009	0.118	0.007
27	0.037	0.032	0.028	0.025	0.021	0.018	0.016	0.013	0.011	0.010	0.008	0.117	0.006
28	0.036	0.031	0.027	0.023	0.020	0.017	0.015	0.012	0.010	0.009	0.007	0.116	0.005
29	0.034	0.030	0.026	0.022	0.019	0.016	0.014	0.011	0.010	0.008	0.006	0.115	0.005
30	0.033	0.029	0.025	0.021	0.018	0.015	0.013	0.011	0.009	0.007	0.006	0.115	0.004

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5.1.53 Example 1 - Roadside Slopes

In the first example, it is desired that criteria be established to indicate when it is cost-effective, in terms of ADT and side-slope, to shield an embankment. It is assumed that an operating speed of approximately 60 mph (96.6 km/hr) exists. The general geometry of the roadside is illustrated in Figure 5.1.22. For purposes of analysis, both the average daily traffic, ADT, and the roadside slope will be considered as variables. Values assigned to other variables are assumed to fall within a reasonable expected range. The following analysis will consider shielding with a roadside barrier first and then the alternative of no shielding.

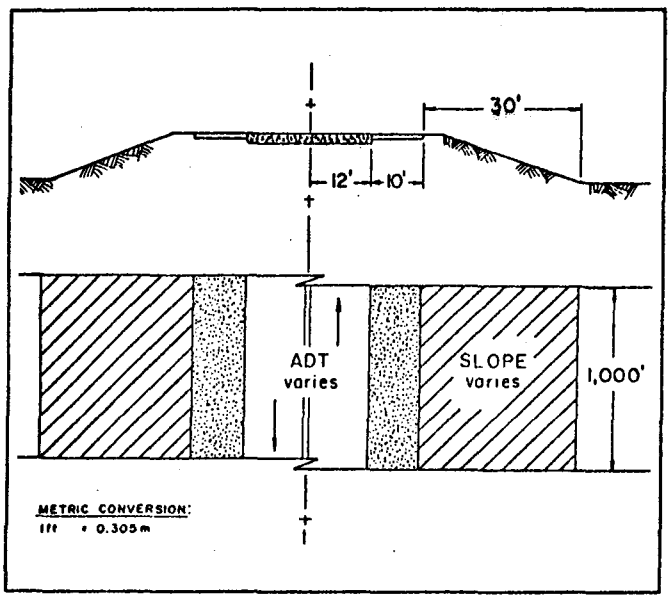

Figure 5.1.22 Roadside Slope Geometry

Roadside Barrier

Before this alternative can be considered in the cost-effectiveness procedure, the flared end-treatment geometry should be established by implementing the barrier flare criteria set forth in Section B. On the basis of these criteria, the flared sections were assumed to exhibit the following general geometry:

- The average offset equals 15 ft (4.6 m).
- The horizontal length of the flared sections equals 256 ft (78.0 m).
- And the total rail length needed equals 257 ft (78.4 m).

These lengths represent the total length of need of the flared section plus a breakaway cable terminal treatment.

In continuing, the roadside barrier analysis involves two distinct computations. In the first case, costs associated with the flared portion of the barrier are computed. Then, costs associated with the barrier proper or the tangent section are computed. The two are then combined to determine the total cost. However, a minor adjustment must be made in determining the collision frequency since the flared portion and the barrier proper are joined at a common point. The following general rule applies in this and other such cases:

For two objects joined together, use the actual length (L) of the object with the highest severity index (SI) and subtract 31.4 (9.6 for metric equivalent) from the length of the other object when determining their respective collision frequencies.

This rule is illustrated in the following example: Note that the cost determination steps follow the format previously outlined.

Flared End Treatment

1. A = 15 ft (4.6 m)

   L = 256 ft (78.0 m)

   W = 1 ft (.305 m) (rail width)

2. ADT = 10,000 (assumed)

3. E_f = 3.2

4. C_f = 0.078 (Actual length is used to determine C_f because SI for flared section is higher than for barrier proper.)

5. Code 06-01-1; SI = 3.7

6. C_I = $13.00 (assumed) per foot at 257 ft (78.39 m)

   C_I = $3,341

7. C_D = $225

8. C_M = $1.50 per foot per year (assumed) at 257 ft (78.4 m);

   C_M = $386

9. C_{OVD} = $7,192 at SI = 3.7 (Figure 5.1.21)

10. T = 15 years

11. CRF = 0.117
    SF = 0.037 at an assumed rate of 8%

12. C_S = $3.00 per foot (assumed) at 257 ft (78.4 m)

    C_S = $771

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13. CA_T = 3341 (0.117) + 225 (0.078) + 386 + 7192 (0.078) - 771 (0.037)

CA_T = $1,327

CA_D = 3341 (0.117) + 225 (0.078) + 386 - 771 (0.037)

CA_D = $766

Barrier Proper

1. A = 10 ft (3.05 m)

   L = 1000 ft (305 m)

   W = 1 ft (.31 m)

2. ADT = 10,000

3. E_f = 3.2

4. C_f = 0.29 based on L - 31.4 or 968.6 ft (295 m) (See Example 1)

5. Code 06-01-3-2; SI = 3.3 (See Table 5.1.10)

6. C_I = $13.00 per foot (assumed) at 1000 ft (305 m);

   C_I = $13,000

7. C_D = $225 (assumed)

8. C_M = $1.50 per foot per year (assumed) at 1000 ft (305 m);

   C_M = $1,500

9. C_{OVD} = $5,874 at SI = 3.3

10. T = 15 years

11. i = 8%

    CRF = 0.117

    SF = 0.037

12. C_S = $3.00 per foot (assumed at 1,000 ft (305 m);

    C_S = $3,000

13. C_{A_T} = 13000 (0.117) + 225 (0.29) + 1500 + 5874 (0.29) - 3000 (0.037)

    = 1521 + 65 + 1500 + 1703 - 111

    C_{A_T} = $4,678

    C_{A_T} = $2,975

TOTAL CA = 1327 + 4678 = $6,005

TOTAL CD = 766 + 2975 = $3,741

These two total costs represent values associated with an average daily traffic equaling 10,000 vehicles per day. The above steps are repeated for higher values of ADT until enough data points are determined to plot CA_T versus ADT. Ultimately, the total barrier values as a function of average daily traffic will be used in the alternative comparison.

Unprotected Slopes

Another alternative which should be considered involves no shielding at all. This alternative requires no direct expenditures since it is assumed that the problem involves existing roadways. Consequently, only the total costs (to include occupant and vehicle damage) can significantly indicate the benefits/disbenefits associated with no shielding of the embankment.

For purposes of analysis, four slopes have been considered as variables in addition to the average daily traffic control. These slopes and their respective estimated severities for assumed site conditions are as follows:

- (3.5:1) slope - severity index equals 3.5
- (3:1) slope - severity index equals 4.0
- (2.5:1) slope - severity index equals 4.5, and
- (2:1) slope - severity index equals 5.0

(Note that for fills steeper than about 3:1 the height of fill should be expected to influence severity.)

Although the slope severities are not specifically identified in the hazard inventory information, a severity index is listed for a negative slope. Assuming that this negative slope represents an average situation and that a 4:1 slope is approximately average, then the severity index of a 4:1 slope would be found to equal 3.0. Furthermore, since the severity index of the roadside barrier is greater than that of the 4:1 slope, then in no way can the barrier be more cost-effective. By taking the average slope as a base, the severities of the other gradients were estimated, and occupant and vehicle damage costs were assigned. The initial, damage, maintenance, and salvage costs were all taken to be zero since it is assumed that the existing geometry requires no direct expenditures. By choosing the average daily traffic again to equal 10,000 vehicles per day and considering a 3.5:1 slope, the costs may be determined by the following steps:

1. $A = 10$ ft (3.05 m)

   $L = 1{,}000$ ft (305 m)

   $W = 30$ ft (9.15 m)

2. $ADT = 10{,}000$

3. $E_f = 3.2$

4. $C_f = 0.30$

5. $SI = 3.5$

6. $C_I = \$0$

7. $C_D = \$0$

8. $C_M = \$0$

9. $C_{OVD} = \$6{,}533$ at $SI = 3.5$

10. $T = 15$ years

11. $CRF = 0.117$, $SF = 0.037$ } at an assummed interest rate of 8%

12. $C_S = \$0$

13. $C_{A_T} = 0 + 0 + 0 + 6535\ (0.30) - 0$

    $= \$9{,}961$

    $C_{A_D} = \$0$

Total costs for the four slopes and varying volumes are calculated in a similar manner to provide the basis of comparison for the no protection alternative.

Comparison

The various situations can best be compared by plotting curves of total present cost versus average daily traffic. Such a set of curves is shown in Figure 5.1.23. By interpreting the data the following conclusions may be drawn:

1. Unprotected slopes of 3:1 and flatter are more cost-effective than the barrier for an average daily traffic up to and in excess of 50,000 vehicles per day; i.e., the barrier is not warranted;

2. The 2.5:1 slope, unprotected, (assumed severity 4.5) becomes less cost-effective than the barrier for an average daily traffic equal to or above 7,500 vehicles per day; and

3. The 2:1 slope, unprotected, (assumed severity 5.0) becomes less cost-effective than the barrier for an average daily traffic equal to or above 7,500 vehicles per day.

This analysis serves to provide some insight as to where roadside barrier protection of slopes may or may not be more cost-effective. General design guidelines or policies may be established and, more importantly, justified in terms of the highest returns in safety.

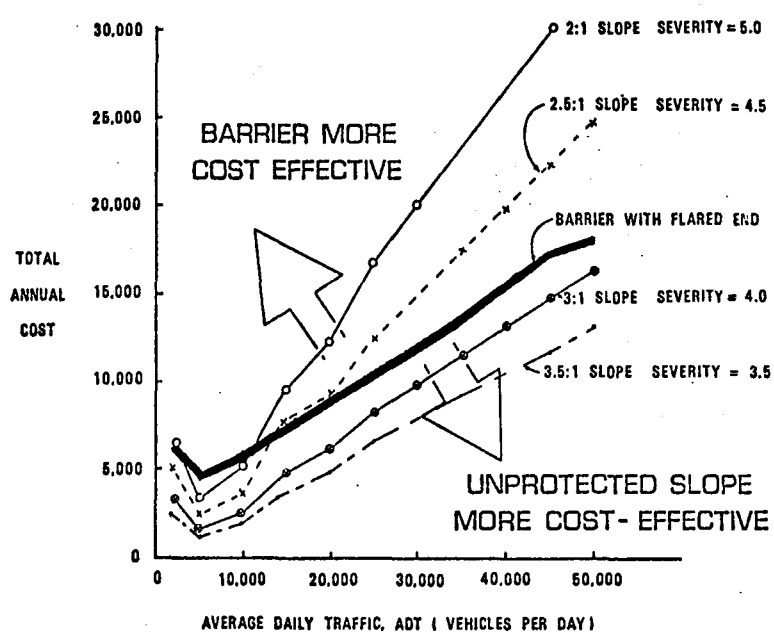

Figure 5.1-23 Cost Comparison Curves

General Comments

1. The analysis, as presented in this problem, involves only those costs associated with one side of the highway facility. If the same conditions exist on the opposite side, then the total costs for both sides would be double those previously determined.

2. The average daily traffic should represent the two-way volume flow since the volume split is built into the analysis procedure. This adjustment is effected by the collision frequency nomographs.

3. The useful life of a roadside slope is taken to be 15 years, which is obviously not the real case. However, it is necessary to consider an equal time span for each alternative in order to make the comparison legitimate.

4. This example illustrates how the procedure can be used to determine the cost-effectiveness of two basic options, i.e., barrier shielding versus no shielding of slopes, for a given location. Although not considered here, the next desirable step may be to establish a priority or ranking system for reducing hazards within a given roadway system. The objective would be to make improvements that offer the greatest return in terms of safety. The following equation may be used for determining a ranking factor, R:

$$R = \frac{C_{A_H} - C_{A_I}}{C_{A_{D_I}}}$$

where

C_{A_H} = annual cost associated with the unshielded hazard over the period T;

C_{A_I} = annual cost associated with the improvement over the period T; and

$C_{A_{D_I}}$ = annual cost to the highway department or agency associated with the improvement.

Improvements should be made to those hazards having the highest value R first. Note that if the numerator is negative, the improvement would not be cost-effective. In Example 1, the ranking factor for placing a roadside barrier to shield the 2:1 slope (assumed severity 5.0) for an ADT of 25,000 would be computed as follows:

C_{A_H} = \$16,710 (Slope) (From Figure 5.1.21)

C_{A_I} = \$10,612 (Barrier) (From Figure 5.1.21)

$C_{A_{D_I}}$ = \$3,530 (From previous calculations)

thus

$$R = \frac{16,710 - 10,612}{3,530}$$

or

$$R = 1.7$$

5.1.54 Example 2 - Bridge Piers

Figure 5.1.24 shows a typical bridge pier hazard. Three alternatives will be considered in the cost analysis as follows:

1. No protection of the bridge piers

2. Protection of the bridge piers with a roadside barrier rail

3. Protection of the bridge piers with a combination roadside barrier rail and crash cushion system

Subsequent to the cost calculations, a comparison of the three operations will be made based on a present worth basis, and the most cost-effective design will be identified. Note that the steps in the analysis correspond to those described in the introduction of the section above.

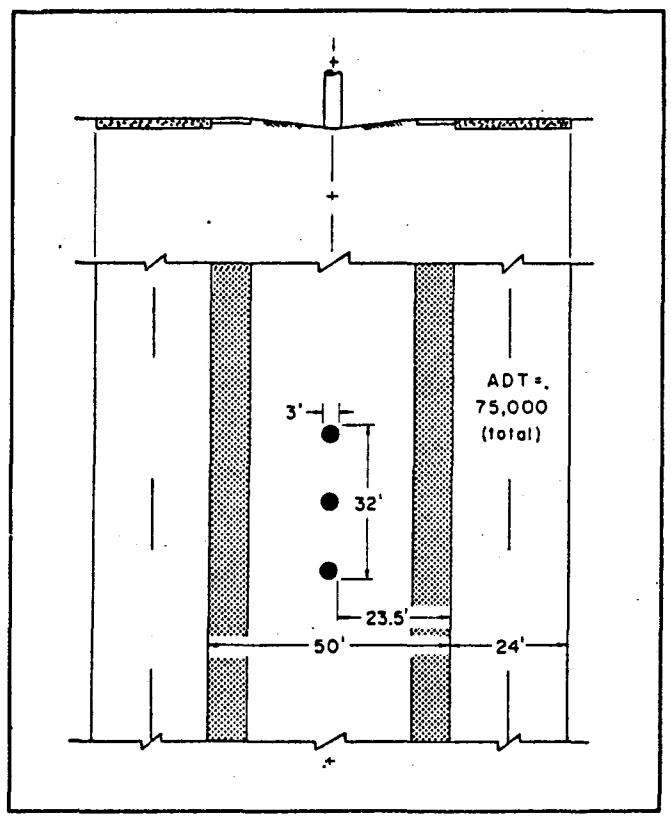

Figure 5.1.24 Bridge Pier Hazard

No Protection

1. A = 23.5 ft (7.17 m) or approximately 23 ft (7.02 m);

   L = 32 ft (9.75 m) and:

   W = 3 ft (.92 m)

2. ADT = 75,000 (assumed)

3. E_f = 31.0

4. C_f = 0.17

5. Code -01; SI = 9.3 (See Table 5.1.10)

6. C_I = $0 (since the piers are existing)

7. C_D = $0 (assumed)

8. C_M = $0 (assumed)

9. C_{OVD} = $169.340 at SI = 9.3

10. T = 20 years

11. CRF = 0.102 } at an interest rate of 8%

12. SF = 0.022 }

C_S = $0

13. C_{A_T} = 0 (.102) + 0 (0.17) + 0 + 169.340 (0.102) - 0 (0.022)

= $17,273

C_{A_D} = $0

or considering collisions with both ends of the bridge pier hazard,

C_{A_T} = $34,545

C_{A_D} = $0

These figures represent the present costs associated with no protection to the roadway hazard. The total cost, as would be expected, is quite substantial due to the severity associated with impacting a fixed bridge pier, while the total direct cost is zero since no improvements are involved. Although the existing geometry may not offer the best alternative, it must be calculated for use as a basis in comparison.

Roadside Barrier

Before the cost analysis can be implemented for this option, specific attention needs to be directed toward identifying the barrier flare geometry. From the barrier flare criteria outlined previously, the placement values to be used in the cost procedure were assumed to be the following:

1. The average offset for the flared sections equals 16 ft (4.88 m)

2. The projected longitudinal length of the barrier flare equals 151 ft (46.01 m)

3. The actual length of the barrier flare equals 153 ft (46.67 m).

In determining the total costs associated with roadside barrier protection, two separate calculations will be made - one considering collisions with the barrier flare and the other involving impacts to the barrier proper. The sum of these two costs will represent the total value associated with the roadside barrier alternative. Note that costs for one direction of travel are computed, then doubled, to obtain costs for both directions of travel. It is assumed that a crashworthy end treatment is used at the upstream terminal.

Barrier Flare

1. A = 16 ft (4.88 m),

   L = 151 ft (46.01 m)

   W = 1 ft (.31 m)

2. ADT = 75,000

3. E_f = 31.0

4. C_f = 0.52 (Actual length is used to determine C_f, because SI for flared section is higher than for barrier proper.)

5. Code 06-01-1-1 SI = 3.7 (Table 5.1.10)

6. C_I = $13.00 per foot (assumed) at 153 ft (46.67 m), thus

   C_I = $1,989

7. C_D = $225 (assumed)

8. C_M = $1.50 per foot per year (assumed) at 153 ft (46.67 m);

   C_M = $230

9. C_{OVD} = $7,192 at SI = 3.7

10. T = 20 years

11. CRF = 0.102 } at 8%

    SF = 0.022 }

12. C_S = $1.50 per foot (assumed) at 153 ft (46.67 m)

    C_S = $230

13. C_{A_T} = 1989 (0.102) + 225 (0.52) + 203 + 7192 (0.52) - 230 (0.022)

= $4,285

C_{A_D} = $545

Barrier Proper

1. A = 13.5 ft (4.12 m);

L = 32 ft (9.76 m); and

2. ADT = 75,000

3. E_f = 31.0

4. C_f = .17 Based on L - 31.4 = 0.6 ft (0.2 m) (See rule in Section 5.1.52.)

5. Code 06-01-3-2 SI = 3.3 (Appendix E)

6. C_I = $13.00 per foot (assumed) at 32 ft (4.12 m); thus, C_I = $416

7. C_D = $225 (assumed)

8. C_M = $1.50 per foot per year (assumed) at 32 ft (4.12 m); thus

C_M = $48

9. C_{OVD} = $5,874 at SI = 3.3

10. T = 20 years

11. CRF = 0.102

SF = 0.022

12. C_S = $1.50 per foot (assumed) at 32 ft (4.12 m); thus C_S = $48

13. C_{A_T} = 416 (0.102) + 225 (0.17) - 48 + 5874 (0.17) - 48 (0.022)

= $1,078

C_{A_D} = $79

The total barrier costs may now be found by totaling the values for the flare and the barrier proper. Furthermore, the total amounts considering shielding for both sides may be attained by doubling the costs associated with collisions from one side.

Therefore, for protection to one end:

Total C_{A_T} = 4285 + 1078 = $5,363

Total C_{A_D} = 545 + 79 = $624

for protection of both ends:

Total C_{A_T} = $10,726

Total C_{A_D} = $1,248

Roadside Barrier/Crash Cushion System

The third alternative considered in the bridge pier analysis will be an integrated crash cushion - longitudinal barrier system. The crash cushion will be utilized as an end treatment to shield the end piers and the ends of the roadside barrier. The roadside barrier is placed along the 32 foot length (9.8 m) to shield the interior pier. Costs for each of the subsystems may be determined given their respective geometrics, and a total present worth may be fixed.

Crash Cushion - End Treatment

1. A = 21 ft (6.4 m),

L = 25 ft (7.6 m),

W = 8 ft (2.4 m)

2. ADT = 75,000 (assumed)

3. E_f = 31.0

4. C_f = 0.12 Based on L - 31.4 = -6.4 ft (-2.0 m) (See rule in Section 5.1.53)

5. Code 15-00-0-0 SI = 1.0 (Table 5.1.10)

6. C_I = $5,000 (assumed)

7. C_D = $1,000 (assumed)

8. C_M = $150 (assumed)

9. C_{OVD} = $2,095 at SI = 1.0

10. T = 20 years

11. CRF = 0.102, SF = 0.022 } at an assumed interest rate of 8%

12. C_S = 0.0

13. C_{A_T} = (5000) (0.102) + 1000 (0.12) + 150 + 2095 (0.12) - 0 (.022)

= $1,031

C_{A_D} = $780

Roadside Barrier

1. A = 21 ft (6.4 m),

   L = 32 ft (9,8 m),

   W = 1 ft (0.305 m)

2. ADT = 75,000

3. E_f = 31.0

4. C_f = 0.19 (Actual length is used to determine C_f because SI for roadside barrier is higher than for crash cushion.)

5. Code 06-01-3-3 SI = 3.3 (Table 5.1.10)

6. C_I = $13.00 per foot (assumed) at 32 ft (9.8 m); thus C_I = $416

7. C_D = $225 (assumed)

8. C_M = $1.50 per foot per year (assumed) at 32 ft (9.8 m); thus,

   C_M = $48

9. C_{OVD} = $5,874 at SI = 3.3

10. T = 20 years

11. CRF = 0.102, SF = 0.022 } at an assumed interest rate of 8%

12. C_S = $1.50 per foot (assumed) at 32 ft (9.8 m); thus C_S = $48

13. C_{A_T} = 416 (0.102) + 225 (0.19) + 48 + 5874 (0.19) - 48 (0.022)

    = $1,248

    C_{A_D} = $132

Considering both the costs for the attenuator and the longitudinal barrier, the total system present worth values may be compared as follows:

For protection of one end:

Total C_{A_T} = 1031 + 1248 = $2,279

Total C_{A_D} = 780 + 132 = $912

and for shielding for both sides:

Total C_{A_T} = 2 (2279) = $4, 558

Total C_{A_D} = 2 (132) = $2[illegible]

Comparison

Table 5.1.15 summarizes the results of this example. By collectively reviewing the three proposed alternatives, several observations and conclusions may be outlined. However, the significance of these observations must be weighed in light of the assumptions made and the values assigned to the various parameters. While these values are thought to be typical, they may not be representative of all areas.

1. While the no shielding alternative requires no direct expenditures, it does represent a very substantial total annual cost in terms of accident losses.

2. On an annual cost basis, the roadside barrier/crash cushion system offers the best alternative. However, it does require a somewhat higher direct expenditure.

3. The ranking factor indicates that of the two improvements, the crash cushion/roadside barrier combination would provide the greatest return per dollar spent.

TABLE 5.1.15 EXAMPLE COMPARISON

OPTION	Direct Annual Cost, C_{A_D} ($)	Total Annual Cost, C_{A_T} ($)	Ranking Factor, R
1. No Shielding	0	$34,545	--
2. Shielding by Roadside Barrier	$1,248	$10,726	19.1
3. Shielding by Crash Cushion/ Roadside Barrier	$ 264	$ 4,558	113.6

General Comments

1. Practically speaking, the main interest in comparing alternatives two and three is to objectively decide whether the shorter, more expensive and less severe crash cushion would/would not enjoy an advantage over the longer, lower cost and higher severity barrier rail.

2. The main purpose of this example is to demonstrate the use of the cost-effectiveness approach in weighing several alternative solutions for one problem location. Other roadside hazard locations may be evaluated in a similar manner to organize a complete facility inventory and a set of ranking factors.

5.1.55 Example 3 - Elevated Gore Abutment

In this example, an elevated gore abutment has been chosen for analysis, and both costs for the hazard and an improvement will be determined. By referencing the layout shown in Figure 5.1.25, those inputs necessary for the calculations may be obtained, and the procedure may be initiated. Also, higher than normal encroachments that are common to such a location will be considered in the analysis, and adjustments will be made accordingly. Furthermore, the evaluation will consider only collisions with the exposed gore and crash cushion, whichever the case may be. Also, the equation for C_f will be applied in lieu of the nomographs to demonstrate its use.

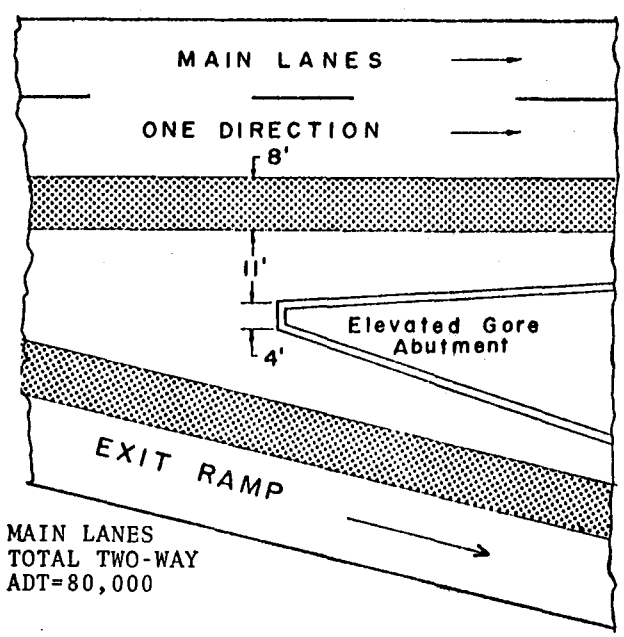

Figure 5.1.25 Elevated Gore Abutment

Existing Hazard

1. A = 19 ft (5.8 m);

   L = 1 ft (.305 m); and

   W = 4 ft (1.2 m)

2. ADT = 80,000

3. E_f = 33.5

4. C_f by using equation may be determined as below:

$$C_f = \frac{33.5}{10,560} (1 + 62.9) (.73) + 5.14 (0.455 + 0.405 + 0.360 + 0.325)$$

C_f = 0.17 and by applying an adjustment factor of 3.0 for higher than normal encroachments (assumed),

C_f (adjusted) = 3 (0.17) = 0.52

5. Code 12-06-0-0 SI = 9.3 (Table 5.1.10)

6. C_I = \$0

7. C_D = \$0 (assumed)

8. C_M = \$0 (assumed)

9. C_{OVD} = \$169,412 at SI = 9.3

10. T = 15 years

11. CRF = 0.117, SF = 0.037 } at an assumed interest rate of 8%

12. C_S = \$0

13. C_{A_T} = 0 (0.117) + 0 (0.52) + 0 + \$169,412 (0.52) - 0 (0.037)

    = \$88,094

    C_{A_D} = \$0

Crash Cushion Improvement

1. A = 17 ft (5.2 m);

   L = 25 ft (7.6 m); and

   W = 8 ft (2.4 m)

2. ADT = 80,000

3. E_f = 33.5

4. C_f by using the equation may be determined as below:

$$C_f = \frac{33.5}{10,560} (25 + 62.9) (0.79) + 5.14 (0.550 + 0.505 + 0.455 + 0.405 + 0.360 + 0.320 + 0.290 + 0.260)$$

C_f = 0.27 and by applying an adjustment factor of 3.0 for higher than normal encroachments (assumed)

C_f (adjusted) = 3 (0.27) = 0.81

5. Code 15-00-0-0 SI 1.0 (Table 5.1.10)

6. C_I = $5,000 (assumed)

7. C_D = $1,000 (assumed)

8. C_M = $200 (assumed)

9. C_{OVD} = $2,095 at SI = 1.0

10. T = 15 years

11. CRF = 0.117, SF = 0.037 } at an assumed interest rate of 8%

12. C_S = $0 (assumed)

13. C_{A_T} = 5000 (0.117) + 1000 (0.81) + 200 + 2095 (0.81) - 0 (0.037)

= $3,292

C_{A_D} = $1,595

By comparing the total costs related to each of the two situations, it may be seen that from a safety standpoint the advantage obviously lies with the improvement alternative. The ranking factor for this site would be 53 which further points out the benefits, in terms of increased safety, that can be realized by installing a crash cushion at such a zone.

In those locations where the traffic-geometric relationships become critical, the collision frequency may be adjusted upward at the discretion of the designer. A factor of 3.0 has been proposed for gore areas, and this seems to be a legitimate number; however, in locations where the variables are not so critical, possibly a lower factor would be appropriate. The decision on such an adjustment would rely strictly on the user's knowledge of the field and his engineering judgment.

5.1.56 Example 4 - Isolated Roadside Obstacles

As has been emphasized throughout this section, the most desirable roadside is one that is relatively flat and free of roadside hazards. If ample recovery room is provided, a driver of an errant vehicle will be able to return to the traveled way or safely stop the vehicle. Removal or relocation of hazards, or the installation of a breakaway device should always be the first option considered. However, various exigencies may sometimes dictate that isolated obstacles such as small trees or small utility poles be located within the desirable recovery area. In such cases, the designer often is faced with the question: Should the obstacle be shielded by a barrier, even though it is obvious that the hazard potential of the barrier is less than the obstacle? The following example illustrates how this question can be answered by the cost-effectiveness procedure.

Existing Hazard - No Protection

Assume that the existing hazard conditions are the same as those in Example 2 except that instead of three bridge piers the obstacles are three small trees located on the roadside instead of the median. All of the parameters defined under no protection of Example 2 therefore apply here,[1] with one exception and that is the SI of the trees which is assumed as 5.0. It will be further assumed that the SI of the trees does not change over the 20-year period. Should not be the case, the procedure presented herein would not be applicable. Selection of an SI for such obstacles must be based primarily on engineering judgment due to an absence of objective criteria. From Figure 5.1.21:

C_{OVD} = $16,710

Thus,

C_{A_T} = 16,710 (0.102)

C_{A_T} = $1704

and

C_{A_D} = $0

Protection by Roadside Barrier

All of the parameters from the Example 2 Roadside Barrier Section apply here.

Thus,

C_{A_T} = $10,726

and

C_{A_D} = $1,248

Comparison

The most cost-effective alternative in this case is to leave the trees unshielded (assuming they cannot be removed) since the numerator of the ranking equation "R" is negative. Although the trees would have a greater hazard potential per accident, the considerably greater target area of the barrier and its closer proximity to the traveled way would result in considerably more barrier impacts than tree impacts. However, as the length of the line of trees increases, the difference in the cost of the two alternatives decreases. At some length of unshielded trees the barrier would become more cost effective. The reader should also remember that the size of the tree is very significant in this analysis. Repeated solution similar to the one above for different lengths of unshielded trees will reveal the break-even point where the barrier will be cost-effective.

REFERENCES

1. AASHTO, Guide For Selecting, Locating and Designing Traffic Barriers, 1977.

2. Weaver, Graeme D. and D.L. Woods. Cost-Effectiveness Evaluation of Roadside Safety Improvements on Texas Highways. Research Report 15-2F, Texas Transportation Institute, 1976.

TOPIC 5 SESSION 2

CRASH ATTENUATION SYSTEMS

Objectives:

The participant should be able to select and locate attenuators in a manner which will insure an economic and safe operating environment for the motoring public and to be able to effectively utilize the cost-effectiveness analysis procedures to evaluate alternative safety improvements.

5.2.1 CRASH CUSHIONS

Definition - Crash cushions are protective systems which prevent errant vehicles from impacting hazards by either smoothly decelerating the vehicle to a stop when hit head-on, or by redirecting it away from the hazard for glancing impacts. These barriers are used to shield rigid objects or hazardous conditions that cannot be removed, relocated, or made breakaway.

5.2.2 WARRANTS FOR CRASH CUSHIONS

Crash cushions have proven to be a cost-effective and safe means of shielding rigid objects. Their use is therefore warranted to shield rigid objects within the clear distance that cannot be removed or shielded by more cost-effective means. Studies indicate that crash cushions are considerably more cost-effective than conventional longitudinal barriers in many instances.

The most common application of a crash cushion is in the ramp exit gore wherein practical design for the site calls for a bridge rail end in the gore. Where site conditions permit, a crash cushion should also be considered as an alternate to a roadside barrier for shielding rigid objects such as bridge piers, overhead sign supports, abutments, and retaining wall ends. Crash cushions also may be used to shield roadside and median barrier terminals.

Since limited resources may preclude the shielding of all rigid objects, a priority system should be established for crash cushion installation. In the absence of a more definitive procedure, the following equation may be used to establish priority:

$$RF = \frac{(1 + NOA) \times ADT \times S}{10{,}000}$$

where

RF = ranking factor

NOA = number of accidents at the site over a given period of time (the same period should be used for all sites);

ADT = average daily volume of traffic; and

S = operating speed of roadway.

Locations with the higher ranking number are considered the most hazardous and should be the first to receive crash cushion protection.

Long steep downgrades present a unique type of problem with regard to traffic barriers. Loss of brakes on a vehicle on such a grade quickly produces a hazardous condition to its driver and to other motorists. Where such problems exist, special consideration should be given to the installation of a roadside decelerating device. An experimental device which shows considerable promise is the gravel bed attenuator. Some states have installed gravel bed attenuator systems, and the results are very encouraging.

Another special condition for which crash cushions are warranted concerns the protection of maintenance personnel, and the motorist, during maintenance operations. It has been shown that a portable crash cushion can be used effectively to provide this type of protection (2). Further studies have been made to establish recommended design configurations (3). Also, a portable "truck-mounted attenuator" has been developed and marketed commercially (4).

A crash cushion or a vehicle arresting device also may be warranted at the end of a dead-end street or beyond a "T" intersection. Need should be based on an evaluation of the probability and consequence of an errant driver's going beyond the intersection.

5.2.3 VEHICLE IMPACT ATTENUATION CONCEPTS

Presently available vehicle crash cushions basically use one of the following two concepts to stop a speeding vehicle before it strikes a rigid obstacle.

5.2.31 Kinetic Energy Principle - Newton's 2nd Law

The first concept involves absorption of the kinetic energy of the speeding vehicle by use of "crushable" or "plastically" deformable materials or structures, or by the use of hydraulic "dashpots" or energy absorbers placed in front of the hazard. Devices of this type need a rigid backup or support to resist the vehicle impact force and deform the energy-absorbing material or structure. Figure 5.2.1 illustrates this principle applied to a compression type barrier.

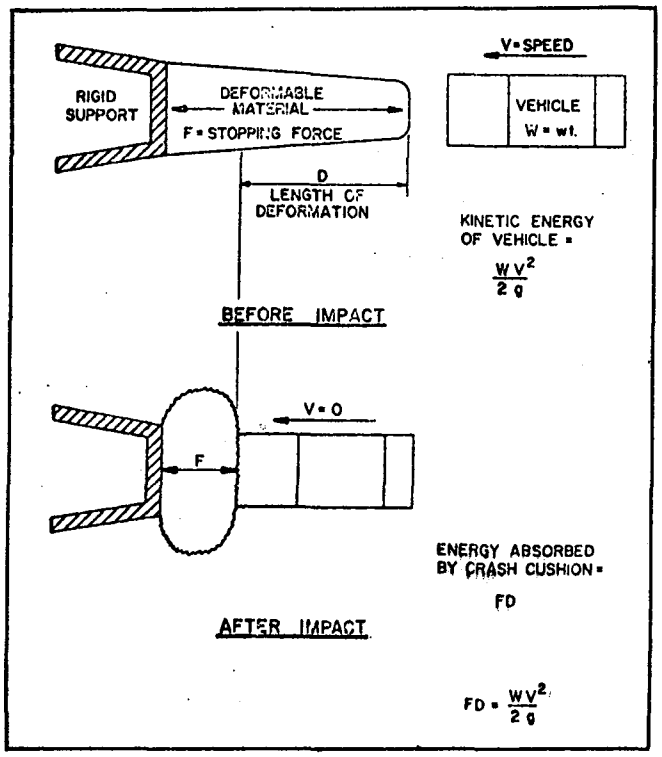

Figure 5.2.1 Principle of Absorbing Vehicle Kinetic Energy - Compression Device

As illustrated in Figure 5.2.2, the stopping force (F) need not be constant, but the area under the force (F) vs deformation (D) graph of the crush cushion should equal the kinetic energy of the impacting vehicle. The crash cushion should be designed so that it will stop a small 2,000-lb vehicle traveling at 60 mph (96 km/h) with D equal to or greater than the minimum required stopping distance of 10 ft (3 m). Additional material and distance also should be provided so that the device will be capable also of stopping a 4,500-lb (2040 kg) vehicle traveling 60 mph (96 km/h).

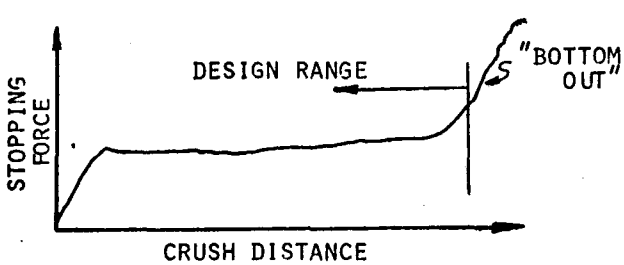

Figure 5.2.2 Typical Relationship Between Stopping Force and Distance of Crush

The kinetic energy of the impacting vehicle is:

$$E_{veh} = \frac{MV^2}{2} = \frac{WV^2}{2g}$$

where:

M = mass of the vehicle

V = Impact speed

The energy absorbed by the attenuation device is:

$$E_{device} = \sum_{d=o}^{d=D} F_{STOP} \; \Delta D$$

where:

F_{stop} = Stopping force on the vehicle over distance ΔD

ΔD = Distance over which stopping force is applied

D = Total crushing distance

The stopping force vs. distance of deformation relationship will build up very quickly upon impact to the level required to crush the initial elements of the device. The force level then will remain relatively constant until all of the energy of the vehicle has been absorbed. However, if the impacting vehicle has more kinetic energy than the attenuation device can absorb, then the device will "bottom out," and a much higher force will be acting against the vehicle. This action is illustrated in Figure 5.2.1 and 5.2.2.

Assuming that the build-up of force occurs over such a short time period that the crush during this interval can be ignored and that the stopping force on the vehicle remains constant throughout the crushing period (i.e., no "bottoming out"), the critical deformation relationship can be developed for this type of device.

$$E_{device} = F_{stop} \times D$$

This is equated with the kinetic energy of the impacting vehicle:

$$E_{veh} = \frac{WV^2}{2g} = F_{stop} \times D$$

or

$$D = \frac{WV^2}{2g\,F_{stop}}$$

Let the maximum average deceleration (a_d) that can be tolerated without an excessive risk of injury to vehicle occupants be 12

g's. The design vehicle has an impact speed of V and a weight of W_d. Thus, a maximum deceleration of 12 g's can be developed from:

$$F_d = ma_d = \frac{W_d}{g}(12g)$$

$$F_d = 12\ W_d$$

where:

F_d = the design stopping force

W_d = the weight of the design vehicle

Then setting the stopping force, F_{stop}, equal to the design stopping force, F_d, the critical crush distance can be computed as follows:

$$F_{stop} = F_d = 12\ W_d$$

$$D_{crit} = \frac{W_d V^2}{2gF_d} = \frac{W_d V^2}{2g(12W_d)} = \frac{V^2}{773}$$

where:

D_{crit} = Critical crush distance in feet to obtain an average deceleration on the design vehicle of 12 g's

V = Design vehicle impact speed in feet per second

A summary of the critical crush distances for kinetic energy attenuation devices for various design impact speeds is presented in Table 5.2.1.

The critical crush distance is independent of the design vehicle's weight but the resisting force is not. Thus, a vehicle which is traveling at speed V but has a weight $W = fW_d$ will stop in a distance D_f as determined by:

$$D_f = \frac{WF^2}{2gF_d} = \frac{WV^2}{2g(12W_d)}$$

but since $W = fW_d$ where f is the ratio of W to W_d

$$D_f = \frac{fW_d V^2}{2g(12W_d)} = f\frac{V^2}{773} = fD_{crit}$$

Hence, knowing that

$$V^2 = 2a_d D_{crit} = 2(12g)\ D_{crit}$$

$$\frac{V^2}{2D_{crit}} = 12g$$

Solving for D_{crit} in the D_f Equation and substituting for D_{crit} in the Equation above, it follows that

$$\frac{V^2}{\frac{2D_f}{f}} = 12g$$

$$\frac{V^2}{2D_f} = a_f = 12g \cdot \frac{1}{f} = 12g\ \frac{W_d}{W}$$

TABLE 5.2.1

CRITICAL CRUSH DISTANCE FOR VARIOUS VEHICLE SPEEDS (IF a IS TO BE 12G OR LESS)

Impact Speed		Critical Crush Distance	
mph	(km/h)	feet	(metres)
30	(48)	2.51	0.8
40	(64)	4.45	1.4
50	(80)	6.96	2.1
60	(96)	10.02	3.0
70	(115)	13.64	4.2
80	(130)	17.81	5.4

5.2.32 Conservation of Momentum Concept

Newton's 3rd Law. The second concept involves transfer of the momentum of the speeding vehicle to some expendable masses of material located in the path of the vehicle. The expendable masses (or weights) are usually containers filled with sand although water and other materials can be used. Devices of this type need no rigid backup or support to resist the vehicle impact force since the kinetic energy of the vehicle is not absorbed, but merely transferred to the other masses. This type of crash cushion sometimes is referred to as an "inertia barrier."

Figure 5.2.3 illustrates this principle applied to a speeding vehicle impacting a series of five masses or containers filled with sand.

By the Law of Conservation of Momentum, the vehicle speed after first mass impact (assuming rigid body plastic impact) is

$$V_1 = V_o \left[\frac{W}{W + W_1} \right]$$

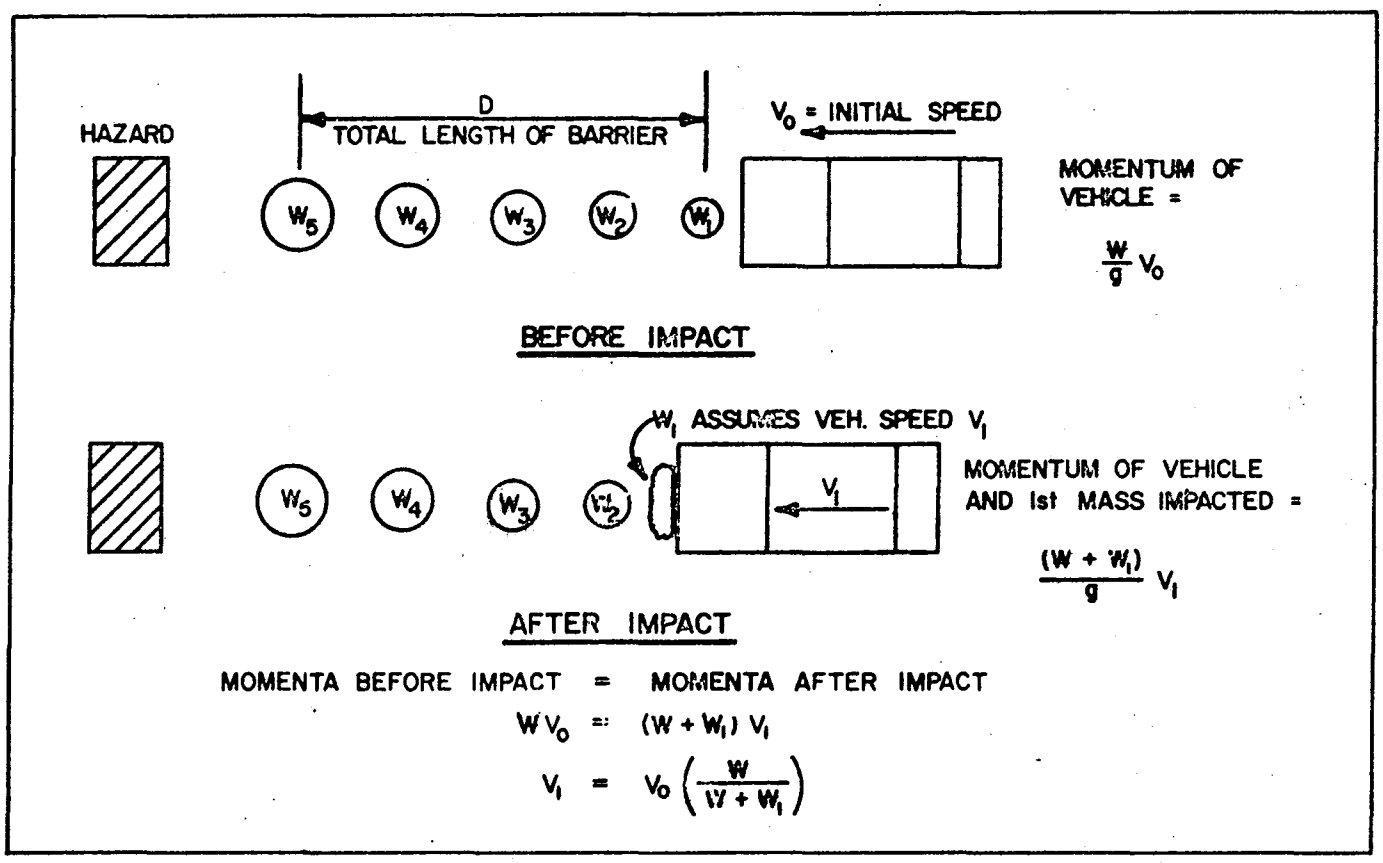

Figure 5.2.3 Principle of Transferring Vehicle Momentum to Expendable Masses Assuming Plastic Rigid Body Impact

The vehicle speed after second mass impact is

$$V_2 = V_1 \left[\frac{W}{W + W_2} \right]$$

The final speed after fifth mass impact will be

$$V_5 = V_4 \left[\frac{W}{W + W_5} \right]$$

To obtain a constant change in velocity as the vehicle strikes each container (W_1 through W_5) it can be seen that containers must increase in weight (or mass) as they get closer to the hazard.

Thus

$$V_1 = V_0 - V_1 = V_0 \left[1 - \frac{W}{W + W_1} \right]$$

and

$$V_2 = V_1 - V_2 = V_1 \left[1 - \frac{W}{W + W_2} \right]$$

and so forth. It is apparent that theoretically the vehicle cannot be stopped completely by this principle. Practically, however, it is usually adequate to design the inertia barrier to reduce the vehicle speed to about 10 mph (16 km/h) after the final container is impacted.

5.2.4 VEHICLE IMPACT ATTENUATION - GEOMETRIC AND DESIGN DETAILS

In the preceding sections, the basic design criteria and concepts used in the development of most presently available vehicle impact attenuation devices were presented. To enable a crash cushion to perform as intended by the design, however, careful attention must be given to several other geometric and design details.

Figure 5.2.4 illustrates how a vehicle may ramp and vault over the vehicle impact attenuation device if the resultant stopping force provided by the crash cushion is considerably lower than the vehicle center of gravity (C.G.). The energy absorbing material may deform more at the top than at the bottom and thus form a ramp for the vehicle. Figure 5.2.5 illustrates how a vehicle may also flip end-over-end due to the couple formed by the eccentricity of the resultant stopping force and the vehicle inertia force.

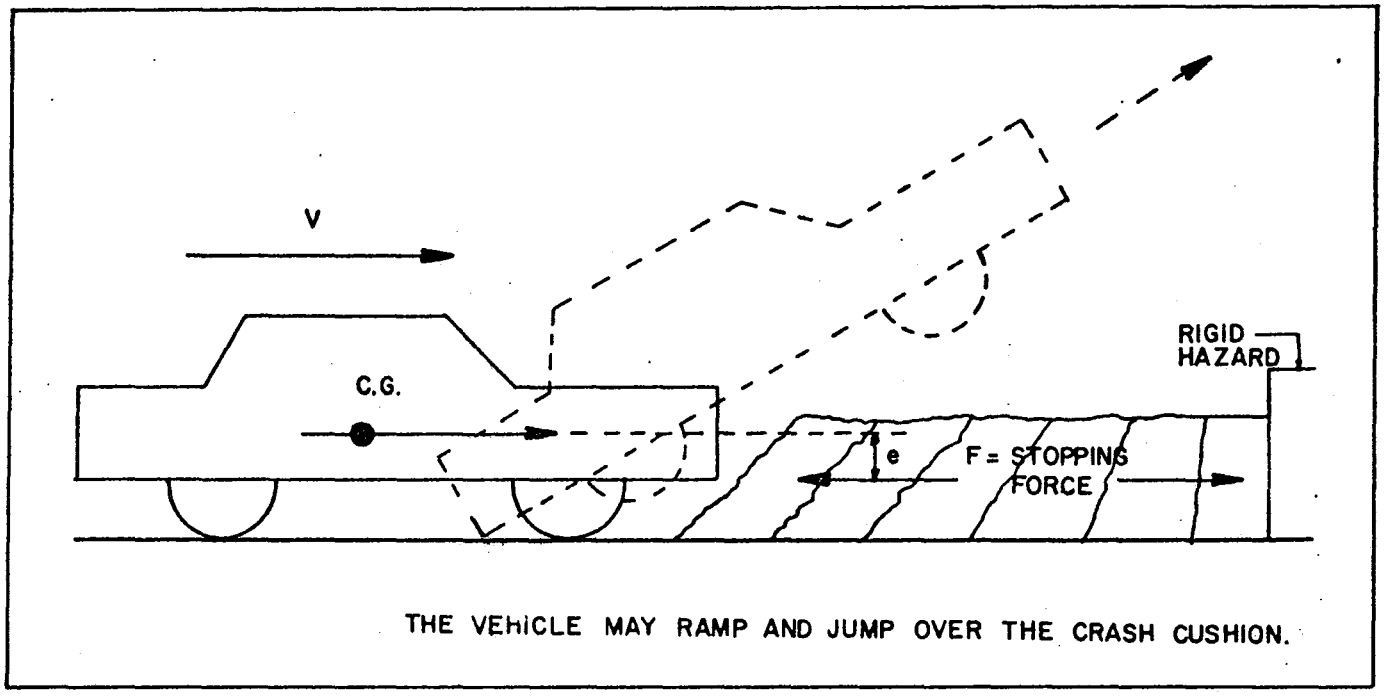

Figure 5.2.4 Resultant Stopping Force Lower than the Vehicle Center of Gravity (C.G.) Head-on Impact

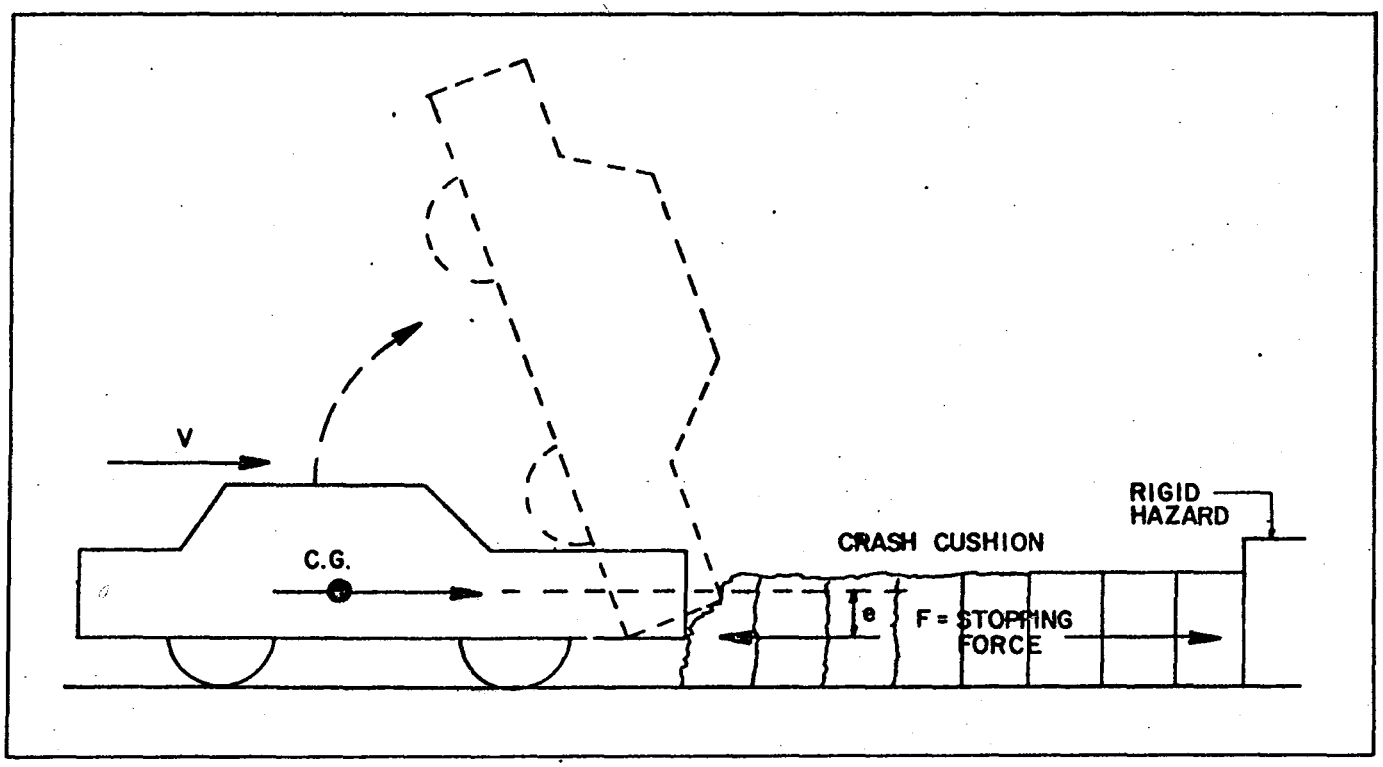

Figure 5.2.5 Resultant Stopping Force Lower than the Vehicle Center of Gravity (C.G.) Head-on Impact

On the other hand, Figure 5.2.6 illustrates how a vehicle may submarine under the impact attenuation device if the resultant stopping force is considerably higher than the vehicle center of gravity. To guard against such behavior, as shown in Figures 5.2.4 and 5.2.5, the resultant stopping force provided by the energy absorbing material or inertia masses should be located approximately 22 to 24 in (55.8 to 61 cm) above the roadway or ground (this is the approximate location of a passenger vehicle's center of gravity). In addition, the energy absorbing crash cushion materials are usually stabilized by a cable or other anchoring system to prevent the material from moving up, down, or sideways during the collision.

Figure 5.2.7 illustrates how a vehicle may "pocket," "spin-out," or even "roll over" in a head-on off-center impact. This type behavior can occur if the vehicle crash cushion is extremely massive and/or stiff, thus generating a large eccentric stopping force and rotation couple on the vehicle.

Thus far we have discussed vehicle impact attenuators (VIA) when hit head-on. Of

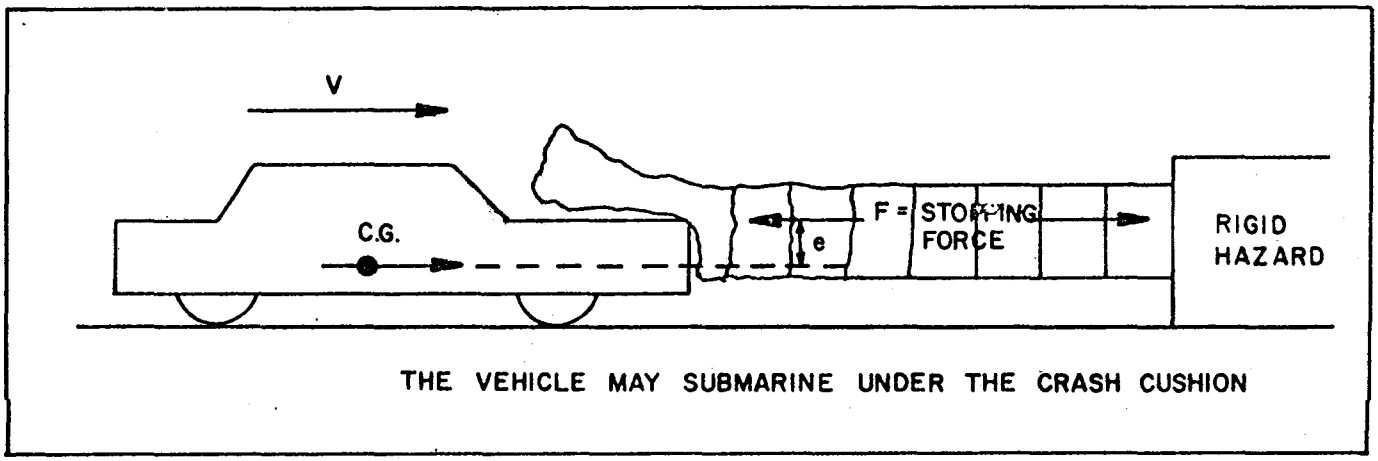

Figure 5.2.6 Resultant Stopping Force Higher Than The Vehicle Center of Gravity (C.G.)

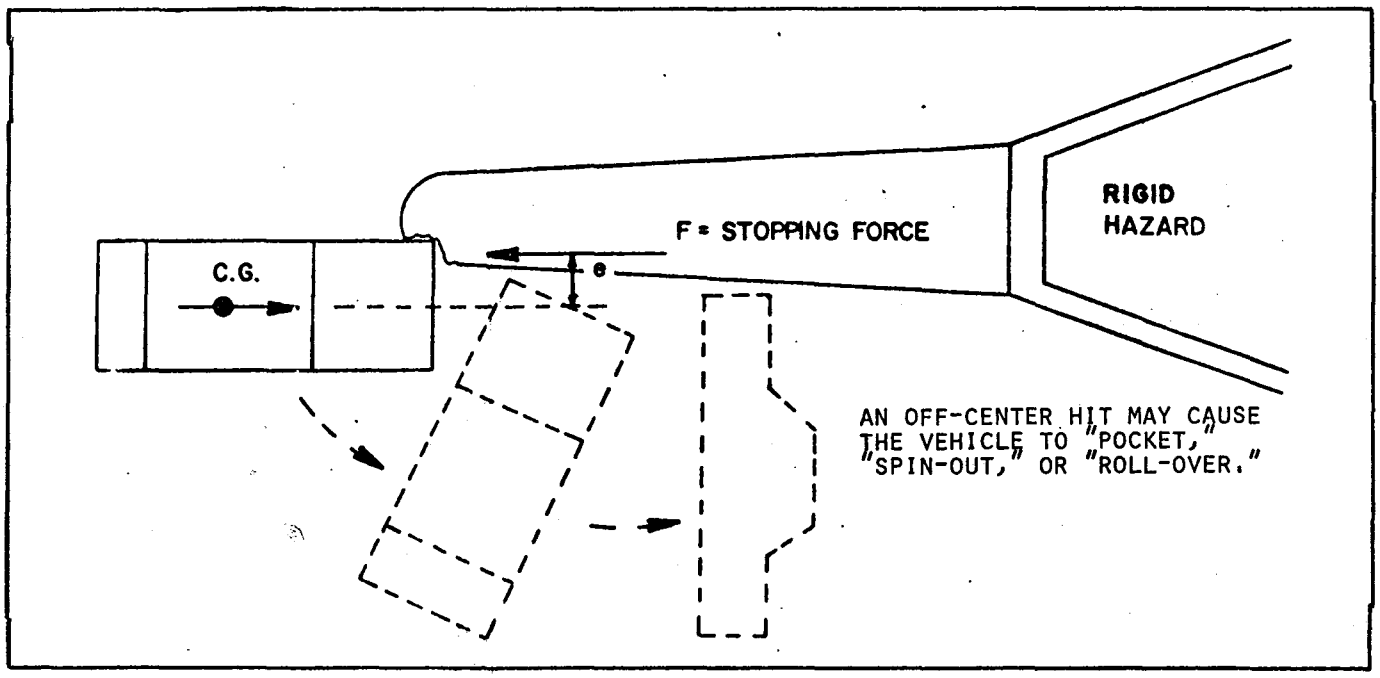

Figure 5.2.7 Vehicle Impact Attenuator is too Massive and Stiff (Stopping Force too Large)

importance also is the behavior of these devices when the vehicle impacts them at an angle with respect to the VIA's longitudinal axis. Figure 5.2.8 illustrates how a typical vehicle impact attenuator will behave under an angle impact near the nose. In this case sufficient distance and energy absorbing material are usually available between the point of impact and the rigid hazard to stop the colliding vehicle safely. In such cases it is satisfactory to allow the vehicle to "pocket" and come to a complete stop, short of the rigid hazard.

Should the vehicle impact the VIA at an angle at a point near the rear of the VIA, a severe collision may occur when the vehicle strikes the rigid hazard. Figure 5.2.9 illustrates this potential problem. In such a collision, distance and energy absorbing material are usually insufficient to stop the vehicle safely before it strikes the rigid hazard. In an attempt to remedy this potential hazard, many VIA designers are cladding the sides of the vehicle impact attenuators with hard, stiff, and smooth panels which will prevent the vehicle from

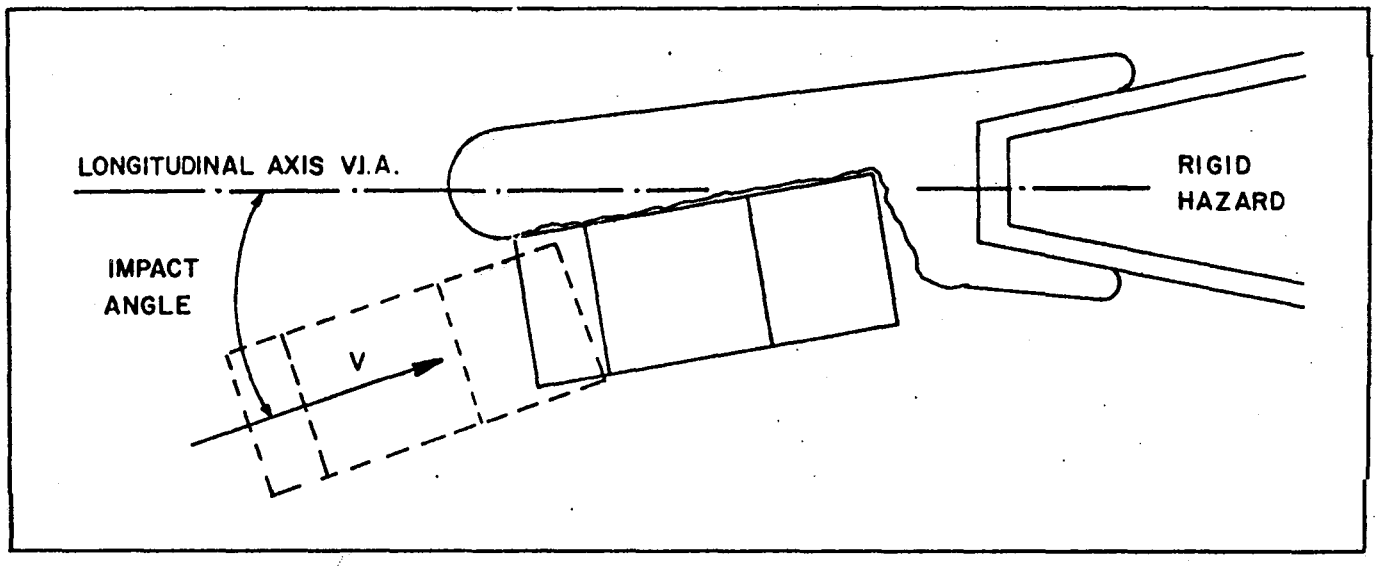

Figure 5.2.8 Vehicle Angle Impact Near the Nose of the Impact Attenuator

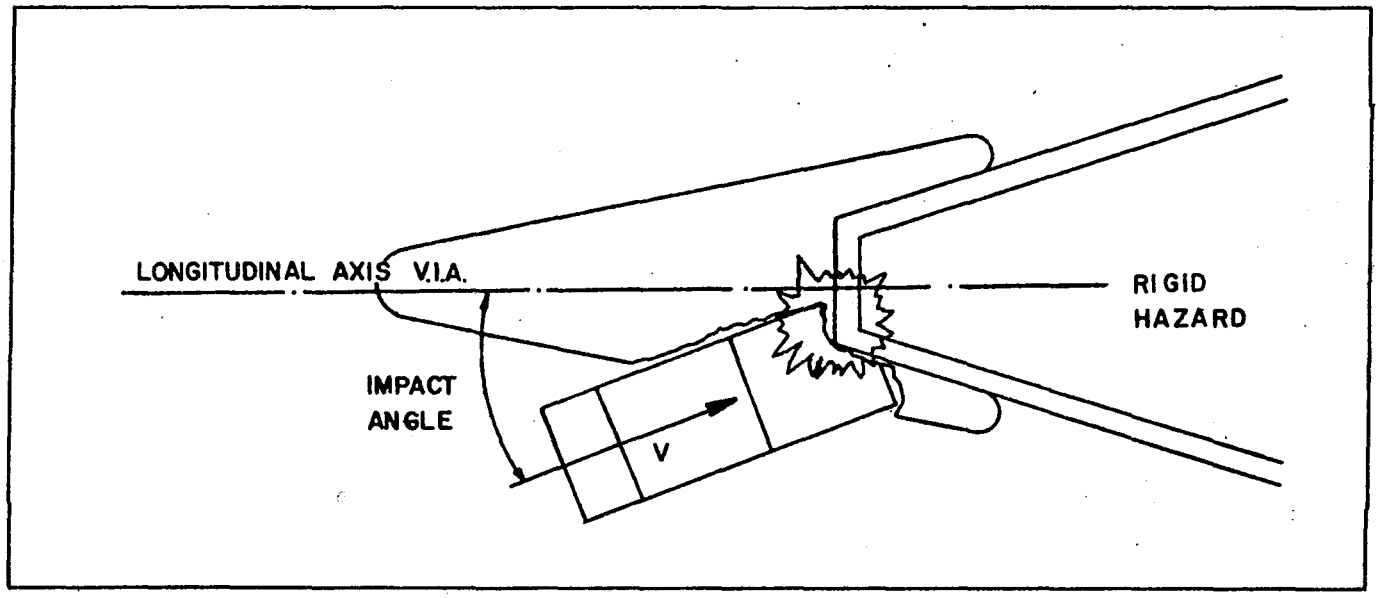

Figure 5.2.9 Vehicle Angle Impact Near the Rear of the Vehicle Impact Attenuator

"pocketing" and thus redirect it as shown in Figure 5.2.10. This figure illustrates typical cladding using 4' x 2' x 3/4 (1.2 x .61 x .19 m) plywood panels. The provisions for redirection must be such that the VIA has lateral stability and still maintains the relatively "soft" crush characteristics under head-on impacts.

5.2.5 SUMMARY OF DESIRABLE IMPACT ATTENUATION BARRIER CHARACTERISTICS

For impact attenuation barriers to be effective and acceptable for use on the nation's highways, test results and experience gained through field installations indicate that it would be desirable for such barriers to have the characteristics described below.

5.2.51 Vehicle Impact Attenuation Barriers (Crash Cushions Without Redirection Capabilities)

- A crash cushion should smoothly stop a vehicle impacting it head-on. The vehicle should not vault over the barrier and should not become unstable and roll over. (It would be desirable for simple crash cushions to have the capability of stopping a vehicle impacting anywhere along its length and at any angle up to the maximum design conditions of impact speed, vehicle weight, and impact angle).

- A crash cushion should minimize vehicle decelerations in such a manner that occupants restrained by seat belts can survive, preferably uninjured.

- A crash cushion should remain essentially intact during and following a vehicle collision. A vehicle impact should not dislodge any hazardous elements into the traveled way.

- A crash cushion should be compatible with the roadway and fixed object it is guarding. It should not protrude into the traveled way or the shoulders provided for emergency or evasive maneuvers by a vehicle.

- A crash cushion should be capable of quick repair. All elements of a barrier should be so designed that when repairs are necessary they can be done quickly and with a minimum of special equipment.

- A crash cushion should be mechanically reliable and dependable. It should be durable and withstand extreme environmental exposure--heat and cold, wet and dry, and corrosive elements expected under service conditions.

- The foregoing requirements should be met by giving emphasis first to safety, second to economics, and third to aesthetics.

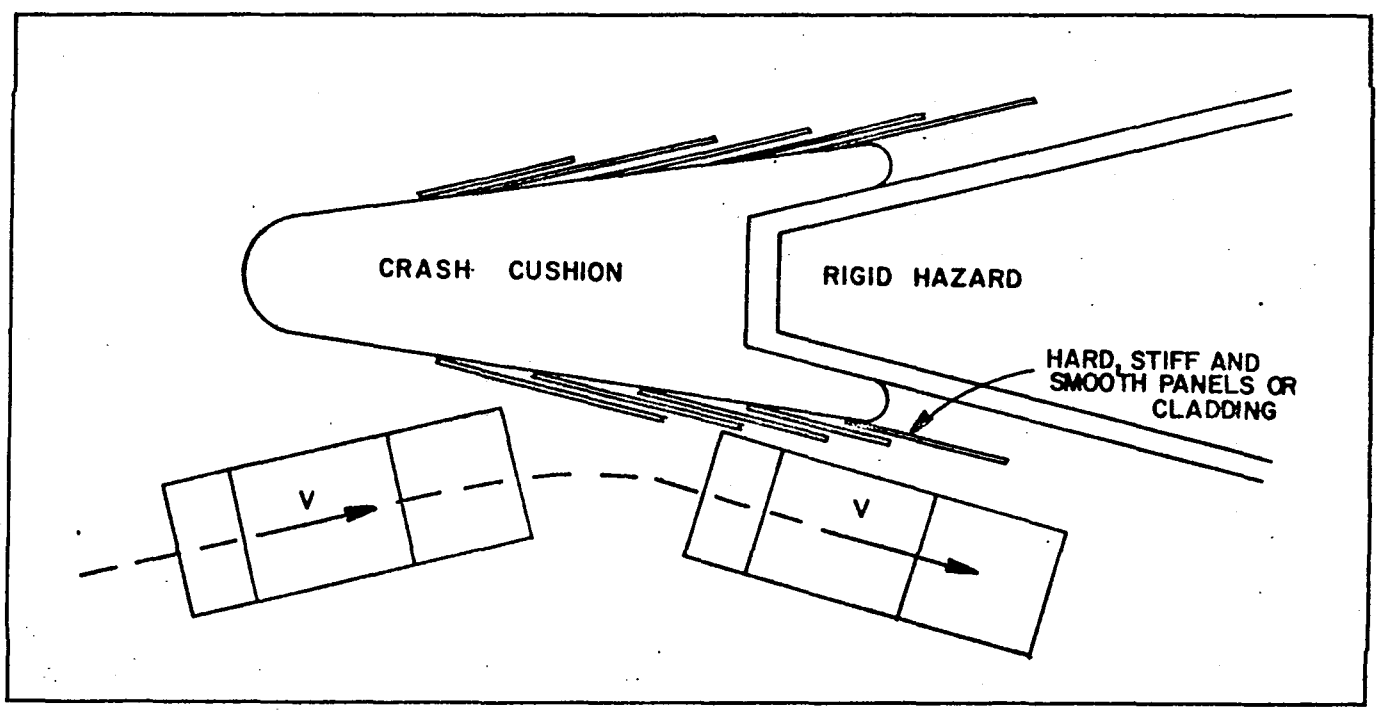

Figure 5.2.10 Vehicle Angle Impact Into Vehicle Impact Attenuator Designed to Redirect Vehicle Rather Than Stop It

5.2.52 Vehicle Impact Attenuation Barriers (Crash Cushions with Redirection Capabilities)

- A crash cushion with redirection capabilities should satisfy all the service requirements of a simple crash cushion of item 5.2.51 when a selected vehicle impacts it head-on.

- A crash cushion with redirection capabilities should restrain and smoothly redirect a selected vehicle which impacts it along its length (or side). The impacting vehicle should not penetrate or vault over the barrier. The vehicle should not snag or pocket under side angle impacts.

- A crash cushion with redirection capabilities should be compatible with adjoining or abutting longitudinal barriers (guardrails, bridge rails, or median barriers) in order to prevent collisions with the ends of the adjoining or abutting barriers. A smooth redirection should be obtained at the transition point between the two barriers.

5.2.53 Characteristics of Available Systems

Table 5.2.2 summarizes the most common forms of attenuation systems.

5.2.54 Costs of Attenuation Systems

Table 5.2.3 summarizes approximate installation and replacement costs of a number of alternative systems.

TABLE 5.2.2

SUMMARY OF STRUCTURAL AND SAFETY CHARACTERISTICS OF CRASH CUSHIONS

	G-R-E-A-T Systems	Hi-Dro Cell Sandwich (C2)	Energite Inertial (C4)	Hi-Dri Cell Sandwich (C5)	Hi-Dro Cell Cluster (C6)
1. Tolerable accelerations?	Yes	Yes[1]	Yes[1]	Yes[1]	Yes[2]
2. Redirection capabilities?	Yes	Yes	No	Yes	No
3. Back-up structure required?	No	Yes	No	Yes	Yes
4. Debris produced upon impact?	No	No[3]	Yes	No	No[3]
5. Anchorage required?	No	Yes	No	Yes	Yes

[1]For any reasonable design speed.

[2]For a speed of 45 mph (72.4k/h) or less.

[3]Except water. Water on the roadway can increase the potential for accidents by reducing skid resistance of pavement. Some anti-freeze agents may increase this potential.

TABLE 5.2.3

SUMMARY OF APPROXIMATE COST FOR CRASH CUSHIONS [HIGH SPEED HIT WITH A 4500# (2045 Kg) VEHICLE]

Type of Cushion	Initial Cost	Maintenance Cost Per Hit
G-R-E-A-T	8000	300
Inertia Barrier	3000	900
Hi-Dro Cell	13000	200
Hi-Dry Cell	17000	300

An analysis of these costs reveals that the G-R-E-A-T system is the least expensive when less than ten hits are expected in the life of the system. Inertia systems are less costly for all frequency of hits in the life of the system below ten.

5.2.6 DESIGN OF THE KINETIC ENERGY CRASH CUSHION

The modular crash cushion dissipates the kinetic energy of a colliding vehicle by utilizing the concept of plastically deforming a set of crushable modules. These modules are arranged such that the resultant force they exert on a vehicle very nearly coincides with the height of the vehicle's center of gravity. The modules are constrained vertically and laterally by cables but are free to move rearward during crushing. They are rigidly supported at the rear. The modules are either bolted or welded together so they will remain fastened to each other during a crash; thus, they will not be thrown onto the roadway to create a new hazard. The cushion itself is designed to exert g-forces well within the range of human tolerance levels.

5.2.61 Basic Equations - Kinetic Energy Cushion

$$D_s = \frac{V^2}{2gG}$$

where:

- D_s = Distance required to completely stop a vehicle at the average g-force
- V = Impact velocity
- g = Pull of gravity
- G = Average deceleration of vehicle in g's; Center of gravity

$$N_m = \frac{WG\ D_s}{KF_s\ D_c}$$

where:

- N_m = Number of modules in the crash cushion
- G = Average deceleration in g's
- K = Ratio of dynamic to static crush energy (usually 1.5)
- F_s = Average static crush force
- D_c = Crush distance of module
- F_s = Average static crush force
- D_c = Crush distance of module

5.2.62 Design Methods

Two design philosophies have evolved during the development of the modular crash cushion. One philosophy utilizes a single type module of a given crush strength and has a varying number of modules per row to give an increasing cushion crush strength. Cushions using this philosophy are referred to as "monomodular." The other philosophy utilizes several different strengths of modules to achieve this increasing cushion crush strength while maintaining a constant cushion width. Cushions using this philosophy are referred to as "polymodular."

The Monomodular Cushion. The monomodular cushion has had extensive experimental and field crash experience.

Figure 5.2.11 shows a plot of the data representing the static performance test for the one module.

After solving for the stopping distance, D_s, the number of rows of modules, n_r, can be found by dividing D_s by the crush distance each row can furnish, or

$$n_r = \frac{D_s}{D_c}.$$

Substituting the value of D_s from above n_r equals

$$n_r = \frac{V^2}{2gGD_c}.$$

From Figure 5.2.11, the load at 18" is approximately equal to the first peak load, and at this crush distance the row of modules behind will begin to crush, provided both rows have the same number of modules. Having design values for g, G, and D_c now gives

$$n_r = \frac{v^2\ (\frac{22}{15})^2}{2(32.2)(6)(1.5)} = 3.71\ V^2\ (10^{-3})$$

where V is in miles per hour (km/h). It is obvious that at points in the cushion where the number of modules per row increases, the preceding modules can crush more than 18" (.46 m). Allowing for only 18" (.46 m) makes the design equation conservative, however, and since the saving of human lives is the ultimate constraint on this design problem, conservatism is an appropiate attribute.

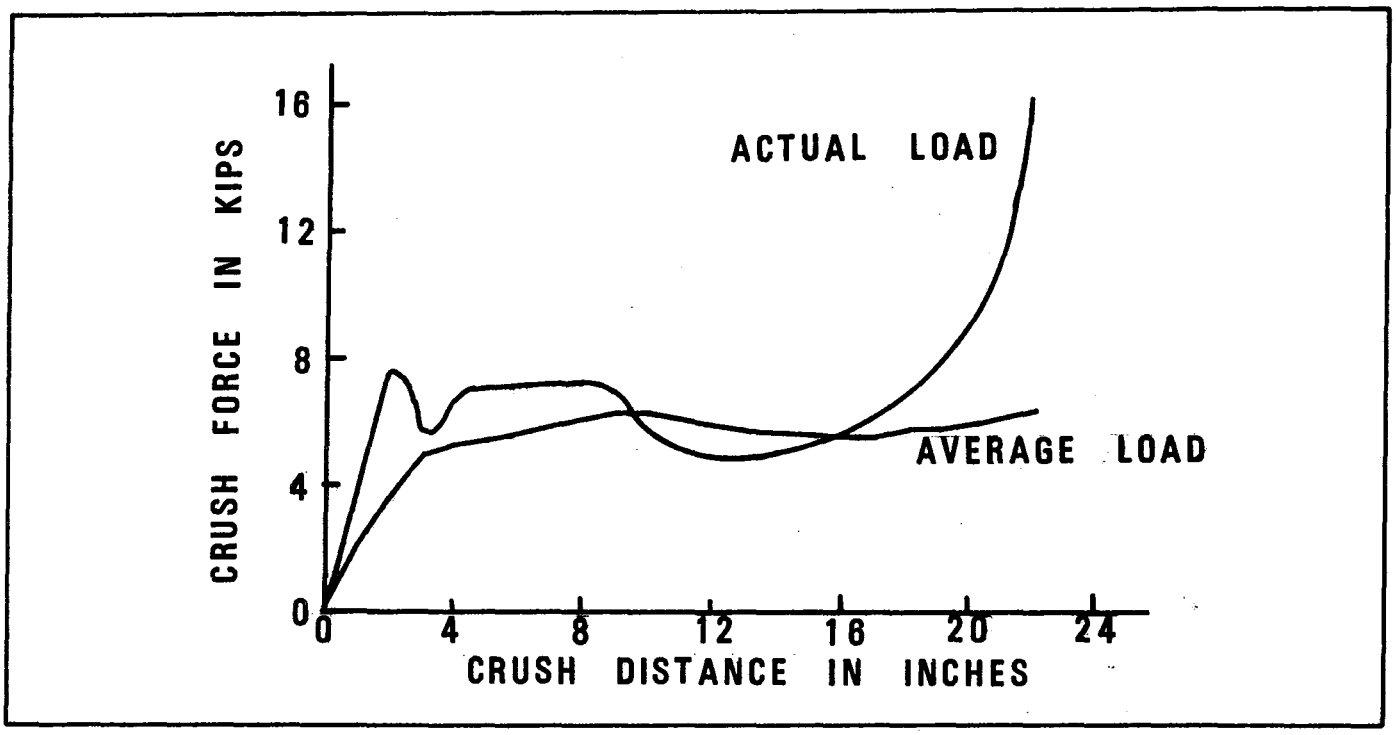

Figure 5.2.11 Static Performance Data For One Module

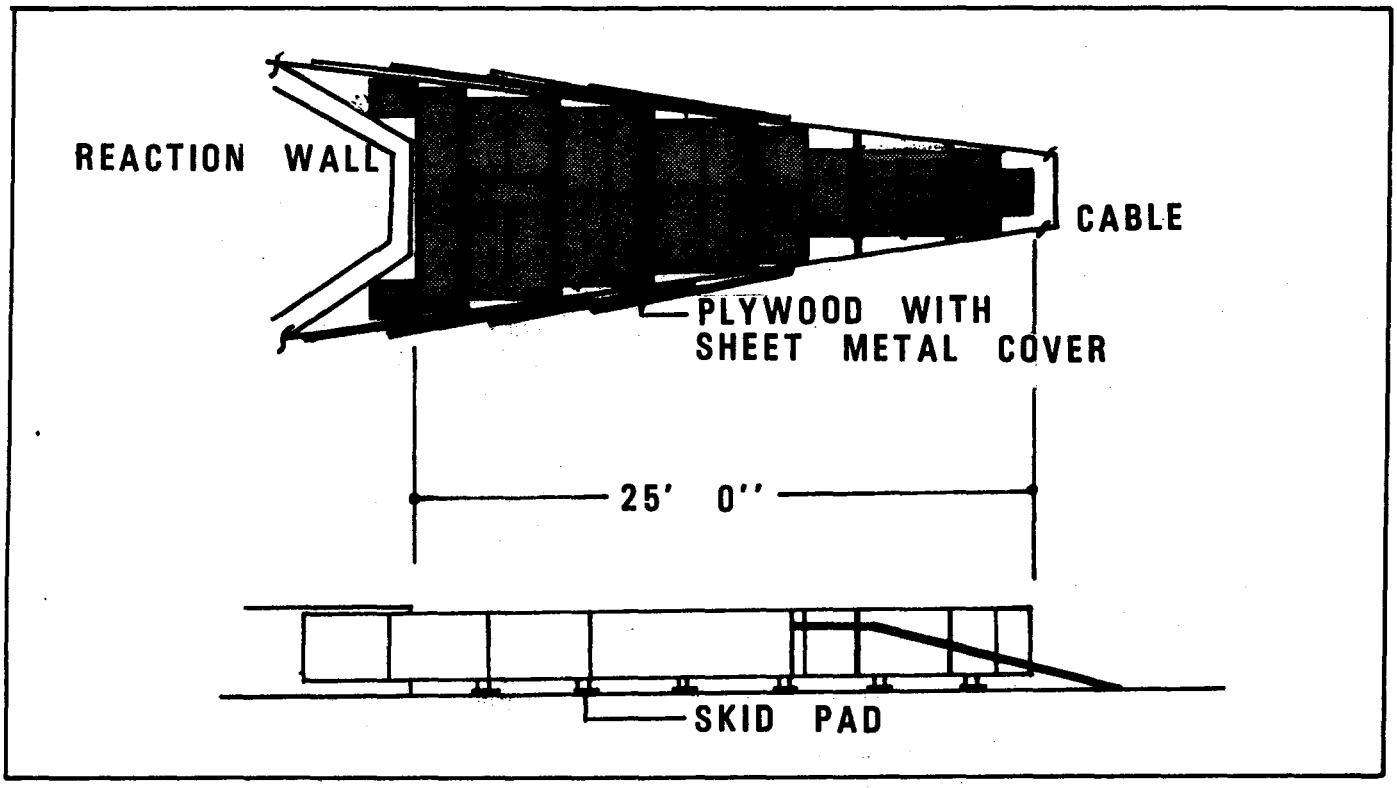

Figure 5.2.12 Kinetic Energy Crash Cushion Schematic

Combining the equation for N_M and n_r, one obtains

$$N_M = \frac{WGn_r}{K\,F_s}$$

and using design values for G, K, and F_s gives

$$N_M = \frac{W(6)n_r}{1.5(5791)} = \frac{Wn_r}{1448}.$$

The n_r and N_M found above are minimum values, and larger values may be used to add a factor of safety. Most present day crash cushions usually are designed for a 4,500 lb (2041 kg) car going 60 mph (96 km/h). Solving for n_r and N_M gives

$$n_r = 3.71\ (3600)\ (10^{-3})$$

and

$$N_M = \frac{13(4500)}{1448}$$

$$= 41$$

The Polymodular Cushion. The polymodular cushion design philosophy entails a discrete element solution of a collision on a row by row basis. That is, whereas the monomodular cushion design philosophy begins by designing the overall cushion, the polymodular cushion philosophy begins by designing one row and then another and another until the kinetic energy of the vehicle is dissipated. The weight of the modules and the range of design vehicle weights are included in the design, thus making this design philosophy cover a broader range of safe performance than the monomodular design which is primarily designed for heavy vehicles with allowances made for lighter vehicles.

5.2.7 DESIGN OF INERTIA BARRIER SYSTEMS

The use of inertia barrier systems in the past ten years has increased dramatically. Analysis procedures using the conservation of momentum principle have been included in the highway safety literature for several years (5). The decelerations associated with inertia barrier module impacts have not been so generally available. For the most part, the design of inertia barrier systems has been dependent upon the recommendation of the manufacturers or upon direct field tests.

This section has been prepared to provide the designer with a comparatively simple, yet logical, approach to designing inertia barrier systems with respect to the deceleration associated with impacting various modules of the system.

5.2.71 Basic Theory

Due to the transferral of kinetic energy resulting from impact with an inertia barrier module, a vehicle's velocity will change by a finite amount in accordance with the principle of the conservation of momentum assuming a completely plastic deformation as shown in Figure 5.2.13.

$$V_i\left[\frac{W_v}{g}\right] = V_{i+1}\left[\frac{(W_v + W_s)}{g}\right]$$

$$V_{i+1} = \frac{V_i\ W_v}{W_v + W_s}$$

$$\Delta V = V_i - V_{i+1} = V_i - \frac{V_i\ W_v}{W_v + W_s} = V_i\left(1 - \frac{W_v}{W_v + W_s}\right)$$

where:

V_i = initial velocity of the vehicle
V_{i+1} = final velocity at the end of module interaction
W_v = weight of the vehicle
W_s = weight of the inertia module
g = gravitational acceleration

For a given module, $\frac{W_v}{W_v + W_s} < 1.0$ and ΔV is finite. Since a change in velocity occurs, the deceleration of the vehicle may be expressed as

$$a_v t = V_i - V_{i+1}$$

where:

a_v = average deceleration of the vehicle
t = time

It is assumed that all of the mass of the module is accelerated to the speed of the vehicle in a distance equal to the diameter (D) of the container. Thus, the time in which the vehicle experiences deceleration occurs over a distance "D" can be expressed as:

$$t = \frac{D}{\frac{V_i + V_{i+1}}{2}} = \frac{2D}{V_i + V_{i+1}}$$

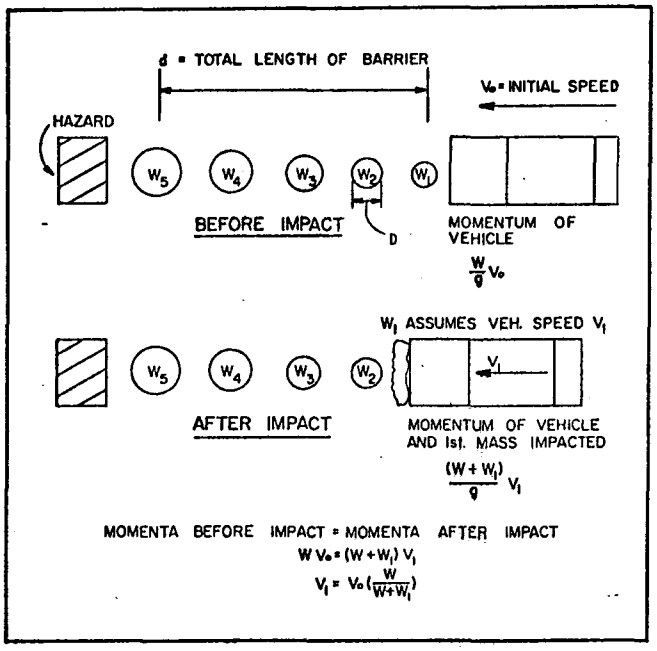

Figure 5.2.13 Principle of Transferring Vehicle Momentum to Expendable Masses--Assuming Plastic Rigid Body Impact

where:

t = time of deceleration

D = diameter of the container

$\frac{V_i + V_{i+1}}{2}$ = average velocity of the vehicle during impact, assuming a uniform deceleration rate.

Substituting t into the $A_v t$ equation, the change in velocity becomes

$$V_i - V_{i+1} = A_v t = A_v \left[\frac{2D}{V_i + V_{i+1}} \right]$$

Solving this equation for A_v:

$$a_v = \frac{V_i^2 - V_{i+1}^2}{2D}$$

Stating V_{i+1} in terms of V_i, the deceleration can be expressed in terms of the initial velocity, the weight of the vehicle and the weight of the inertia sand mass for a given deceleration distance "D."

$$V_{i+1} = V_i \frac{W_v}{W_v + W_s}$$

$$a_v = \frac{V_i^2 - \left(V_i \frac{W_v}{W_v + W_s}\right)^2}{2D} = \frac{V_i^2 \frac{V_i^2 W_v^2}{(W_v + W_s)^2}}{2D}$$

$$a_v = \frac{V_i^2 \left[1 - \frac{W_v^2}{(W_v + W_s)^2} \right]}{2D}$$

From this basic equation it is desirable to convert a_v to "g's," as the basic tolerance criteria are expressed in "g's;" to express the velocity in miles per hour ; and the diameter of the container in feet for dimensional agreement.

$$G = \frac{(1.47)^2 \; V_i^2 \left[1 - \frac{W_v^2}{(W_v + W_s)^2} \right]}{2Dg}$$

where;

$G = \frac{a_v}{g}$ = Average deceleration in "g's" over the diameter of the container

With this equation, the average deceleration per module may be calculated for any combination of design vehicle and module elements.

5.2.72 Development of the Design Charts

For convenience, the equation for "G" was solved for V_i. Using a given maximum design deceleration, the associated initial velocity prior to contact with an individual module is found by solving for V_i.

$$V_i^2 = \frac{2DgG}{(1.47)^2 \left(1 - \frac{W_v^2}{(W_v + W_s)^2}\right)}$$

$$V_i = \left[\frac{2DgG}{(1.47)^2 \left(1 - \frac{W_v^2}{(W_v + W_s)^2}\right)} \right]^{\frac{1}{2}}$$

This equation is plotted in Figures 5.2.14 to 5.2.17 for 2250-lb (1020 kg), and 4500-lb (2041 kg) vehicles impacting 3.0 ft (.91 m) diameter modules. The family of curves reflecting the deceleration values between 2 "g's" and 12"g's" is shown in relation to the weight of the mass in an individual module and the initial velocity of the vehicle prior to impact with the module in question.

As stated above, the conservation of momentum principle has been used in this analysis to derive the equation for the change in velocity. It has been assumed that all of the mass in each module flies clear of the vehicle and does not contribute additional weight to the vehicle as it continues through the subsequent elements of the barrier. The change in velocity due to this transferral of kinetic energy from the vehicle to the mass in a 3.0 ft (.91 m) diameter module is plotted in Figures 5.2.14 to 5.2.17 for vehicle weights of 2250 lbs (1020 kg), respectively.

The weights of the standard modules offered by one manufacturer are plotted. These modules are 200 lbs (91 kg), 400 lbs (182 kg), 700 lbs (318 kg), 1400 lbs (640 kg), and 2100 lbs (960 kg). It is advisable to use a module slightly less than the weight of mass that will produce the desired deceleration for the initial modules of the system to assure a conservative design. The use of a module of slightly greater mass than the theoretical value necessary to produce the desired deceleration is permissible for the latter elements of the system. The final cluster of modules is not designed by the momentum principle. Consideration of available space, economics, and other factors will determine the mass designed to allow the slowed vehicle to "plow" to a halt. It should be said, however, that as large a mass as possible should be made available to completely stop a vehicle in dangerous locations Precluding this, a clear distance approximately equal to six-tenths of the length of the barrier system (0.6 d) should be provided between the last barrier module and the hazard.

5.2.73 Example Problem Solution

Design of an inertia barrier system consisting of 3.0 ft (.91 m) diameter modules to stop a 2250 lb (1020 kg) vehicle veering from the roadway and impacting the attenuator at an initial velocity of 58 mph (93 km/h) is presented below. Seat belts are assumed to be in proper use; therefore, a maximum deceleration of 10"g's", somewhat less than the 12 g's maximum suggested by FHWA (Table 5.1.1) has been assumed as the safe limit for the driver and passengers.

From Figure 5.2.14, for the 2250 lb (1020 kg) vehicle, since V_o = 58 mph (93 km/h) and "g's" $\leq$ 10

$$W_s = 280 \text{ lbs } (127.3 \text{ kg})$$

$$\text{Use } W_{so} = 200 \text{ lbs. } (91 \text{ kg})$$

From Figure 5.2.15, the change in velocity of the vehicle after transferral of a finite amount of kinetic energy to the module of sand is

$$V_o = 58 \text{ mph } (93 \text{ km/h}) \text{ and}$$

$$W_{so} = 200 \text{ lbs. } (91 \text{ kg})$$

$$\Delta V_o = 5 \text{ mph } (8 \text{ km/h})$$

Again, from Figure 5.2.14 but with a new velocity

$$V_1 = V_o - \Delta V_o$$

$$V_1 = 53 \text{ mph } (85 \text{ km/h}) \text{ and "g's"} \leq 10$$

$$W_w = 480 \text{ lbs } (217.7 \text{ kg})$$

$$\text{Use } W_{s1} = 400 \text{ lbs. } (181 \text{ kg})$$

From Figure 5.2.15, ΔV for V_1 = 53 mph (85 km/h) and W_{s1} = 200 lbs (91kg):

$$\Delta V_1 = 8 \text{ mph } (12.8 \text{ km/h})$$

$$V_2 = V_1 - \Delta V_1 = 45 \text{ mph } (72.2 \text{ km/h})$$

Iteration between Figures 5.2.14 and 5.2.15 can be facilitated by using a table (Table 5.2.4).

TABLE 5.2.4

EXAMPLE PROBLEM

Conditions: Initial Velocity = 58 mph (93 km/h)
Design Deceleration < 10 g's
Design Vehicle Weight = 2250 lbs (1020 kg)
Reference: Figures 5.2.20 and 5.2.21

iteration i	V_i (mph)	Ws Lbs of Sand for Max g's	W_{si} Wt of Modules lbs	ΔV_i (mph)	$V_i + 1)$ $(V_i - \Delta V_i)$
0	58	280	200	5	53
1	53	480	400	8	45
2	45	740	700	11	34
3	34	1400	1400	13	21
4	21	——	1400	8	13
5	13	——	1400	5	8*
6	8	——	2100	4	4*

* Somewhat below a speed of 10 mph (16 km/h). The frictional forces (plowing action) becomes the decisive factor in bringing the vehicle to a complete stop.

DESIGN CHART FOR 36" DIAMETER INERTIA BARRIER
2250 LB. DESIGN VEHICLE

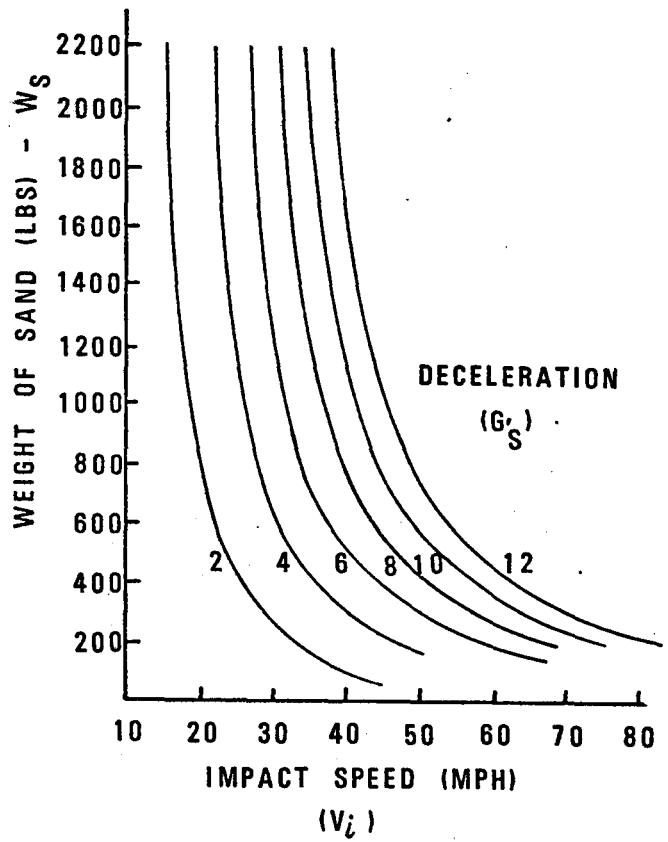

Figure 5.2-14

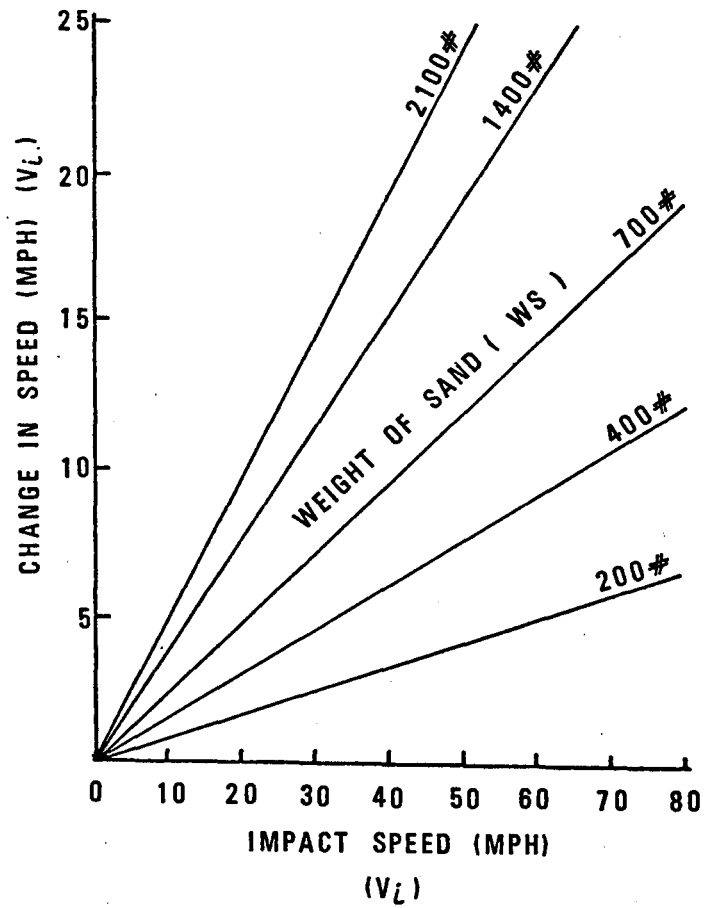

Figure 5.2-15

DESIGN CHART FOR 36" DIAMETER INERTIA BARRIER
4500 LB. DESIGN VEHICLE

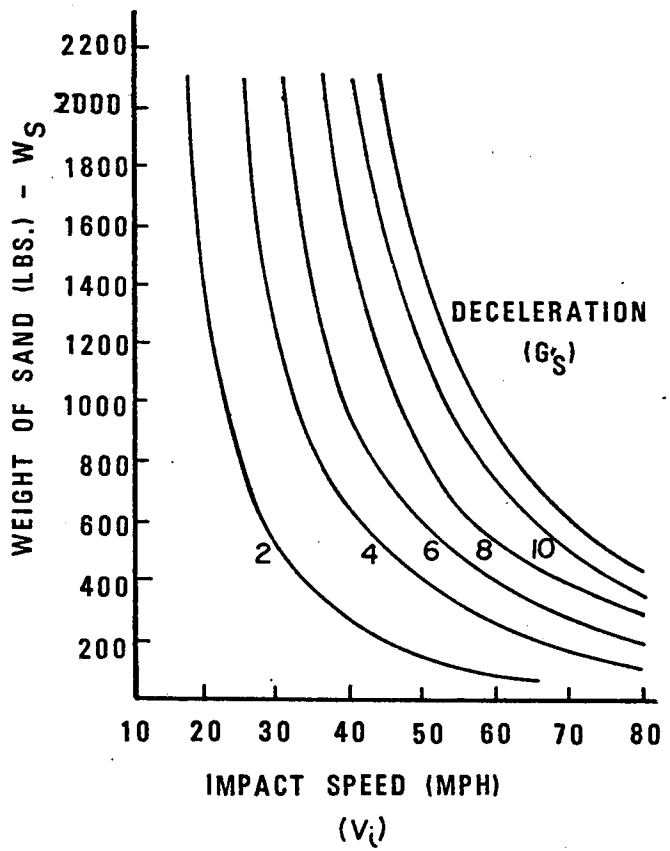

Figure 5.2-16

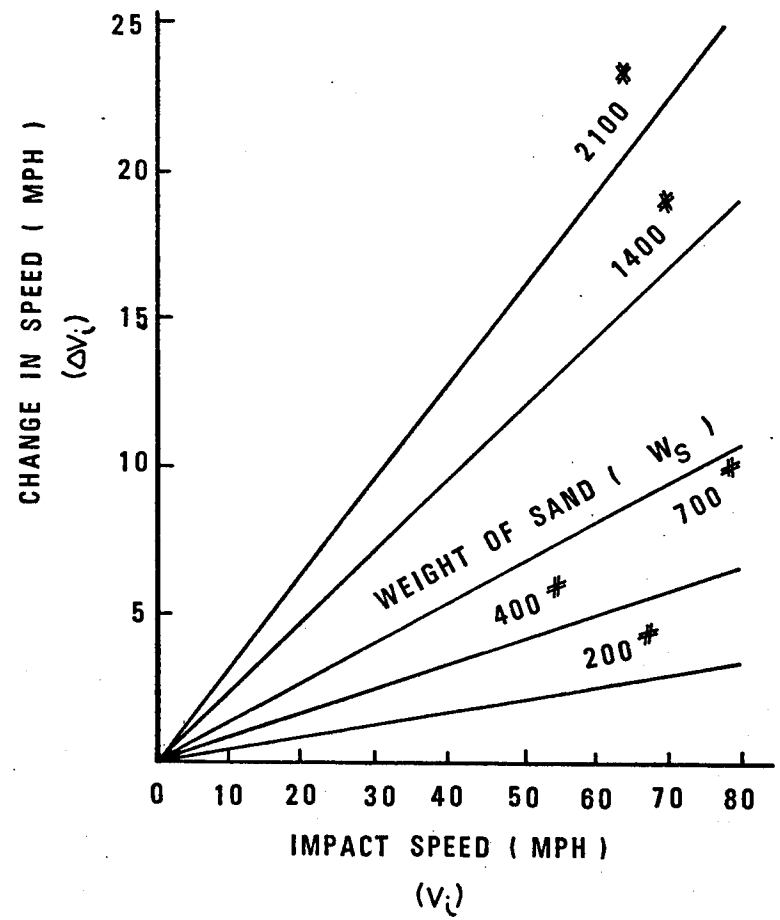

Figure 5.2-17

5.2.74 Validation of the Theory

In order to validate the theory, actual test data were compared to the theoretical values of deceleration obtained from Figures 5.2.14 to 5.2.17.

The results of this analysis are shown graphically in Figure 5.2.18. The figure visualizes the tendency of the theoretical values to form a slightly conservative design averaging about five percent of the measured values.

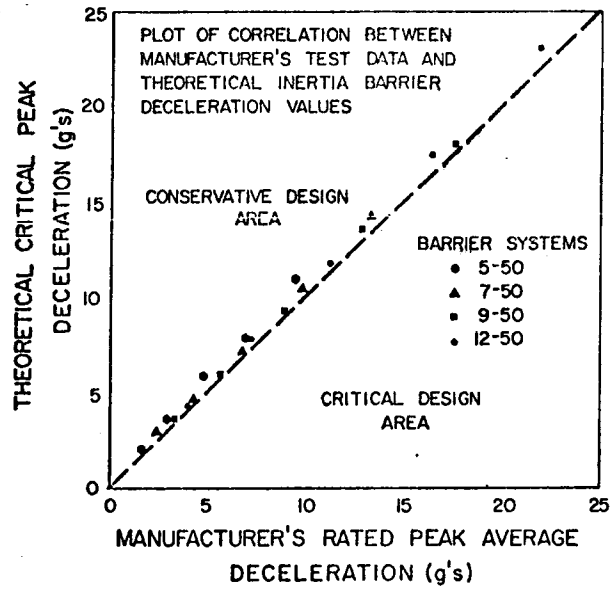

Figure 5.2.18 Comparison of Theoretical and Measured Deceleration

The tire-sand inertia barrier system does not allow the assumption of an average deceleration throughout the diameter of the module. The stiffness of the tire causes the entire mass to accelerate prior to the complete penetration of the vehicle. The accumulation of weight by the vehicle during impact has been demonstrated in several tests. To a somewhat lesser degree, this occurs with plastic containers. Nevertheless, a comparison of the deceleration due to the impact of the first single module of the barrier has been presented. In Table 5.2.5 the deceleration values from tests, conducted by the Texas Transportation Institute for the NCHRP Project 20-7, are compared to theoretical values. The decelerations from the full-scale tests were recorded on a tri-axis impact-o-graph attached to the vehicle.

The average peak deceleration over the diameter of the container could not be theoretically predicted for all but the first module due to the partial absorption of kinetic energy by the tires as the barriers compressed together into one mass penetrating into the front end of the vehicle.

5.2.8 DESIGN OF HI-DRO CUSHION CRASH MODERATION SYSTEM

The Hi-Dro Cushion Crash Moderation System dissipates a colliding vehicle's kinetic energy by transferring the energy to an expendable mass of water. The water is contained in flexible modules with orifices to relieve and regulate pressures incurred by a colliding vehicle. The flexible modules, called cells, are positioned such that they apply a resultant force at approximately the vertical height of a vehicle's center of gravity. The modules require a rigid support in the rear, connection between modules, and vertical and lateral restraint with a cable and plywood diaphragm system as shown in Figure 5.2.20.

Two types of cells are in present use, the vinyl cell and the cartridge cell. The vinyl cell is a hollow plastic cylinder with a nominal six-inch outside diameter and a one-forth inch thick wall, whereas the cartridge cell is made of vinyl-impregnated nylon fabric with the same nominal 6-in (15.24-cm) diameter. The cells have one end closed and the other end open. The open end has a glued-in insert containing sharp-edged orifices to regulate the release of water (See Figure 5.2.19).

The vinyl cells are fastened together in groups and generally are used for speeds under 50 mph (31 km/h). They also are used on the nose of the cell-sandwich system.

The cartridge cells are used in conjunction with plywood diaphragms, interior panels, cable restraints, and fiberglass-coated plywood fendering panels, and comprise the cell-sandwich system. The fender panels are intended to deflect vehicles impacting on the side at an angle, providing redirection instead of a complete stop.

A crash cushion made of Hi-Dro cells depends on two physical phenomena for attenuating the energy of a high-speed vehicle:

- Momentum transfer between vehicle and fluid and

- The kinetic energy loss associated with the orifices

Combining these two concepts with a relatively soft plastic cell produces a design which has very little elasticity.

Their units are reusable after impact and need only be refilled to regain their full protective value. This suggests that the system could be made to restore itself, if

TABLE 5.2.5

COMPARISON OF THEORETICAL
DECELERATION TO FIELD MEASUREMENTS FOR
TIRE-SAND INERTIA BARRIER

TTI Project RF 846-3 Test Programs	Impact-O-Graph	Theoretical	% Difference**
I-1 [1961 Ford: Wt. 4170 lbs (1895.4 kgm) V_i = 59 mph (36.6 km/h)]	5.0 g's	5.6 g's	8.9
I-2 [1961 Renault: Wt. 1950 lbs (886 kgm) V_i = 61.6 mph (38.3 km/h)]	10.0 g's	11.9 g's	15.9

** The difference is on the conservative side.

a self-controlled system can be provided to replace the water in the cushion.

Several criticisms have been leveled at the Hi-Dro cell concept. Most notable of these are:

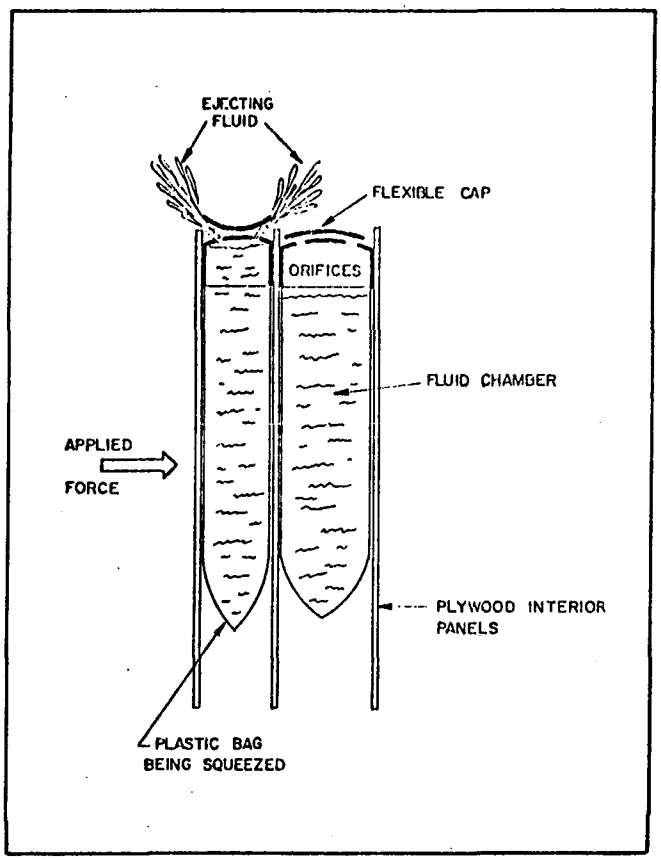

Figure 5.2.19 Function of a Hi-Dro Cell

1. Low temperature performance of cell

- Plastic will become brittle
- Water will freeze, or anti-freeze must be used

2. The onset rate provided by the cushions commonly used is well above the recommended 500 g's/second recommended by the Federal Highway Administration.

3. Partial contact with the barrier can cause the vehicle to spin and/or roll.

In response to these criticisms, the manufacturers indicate that the plastic currently being used is suitable for the temperature range from -40° F to +140° F. Since the plastic is not affected by salt, the use of sodium chloride as an antifreeze is feasible where low temperatures are expected.

The onset rate can be controlled by altering the size and shape of the orifices and/or using empty or partially-filled cells near the front of the barrier system. The onset rate problem does not appear to be a critical one.

5.2.9 OPERATIONAL CRASH CUSHIONS

Table 5.2.6 summarizes the operational crash cushions available at this time. Impact performance data are also presented. As can be seen in the table the impact conditions were not consistent. Also, the as-tested designs would not all necessarily be used for the same site conditions. Design and functional characteristics are discussed below.

None of the crash cushions have been standardized. Also, all of the operational crash

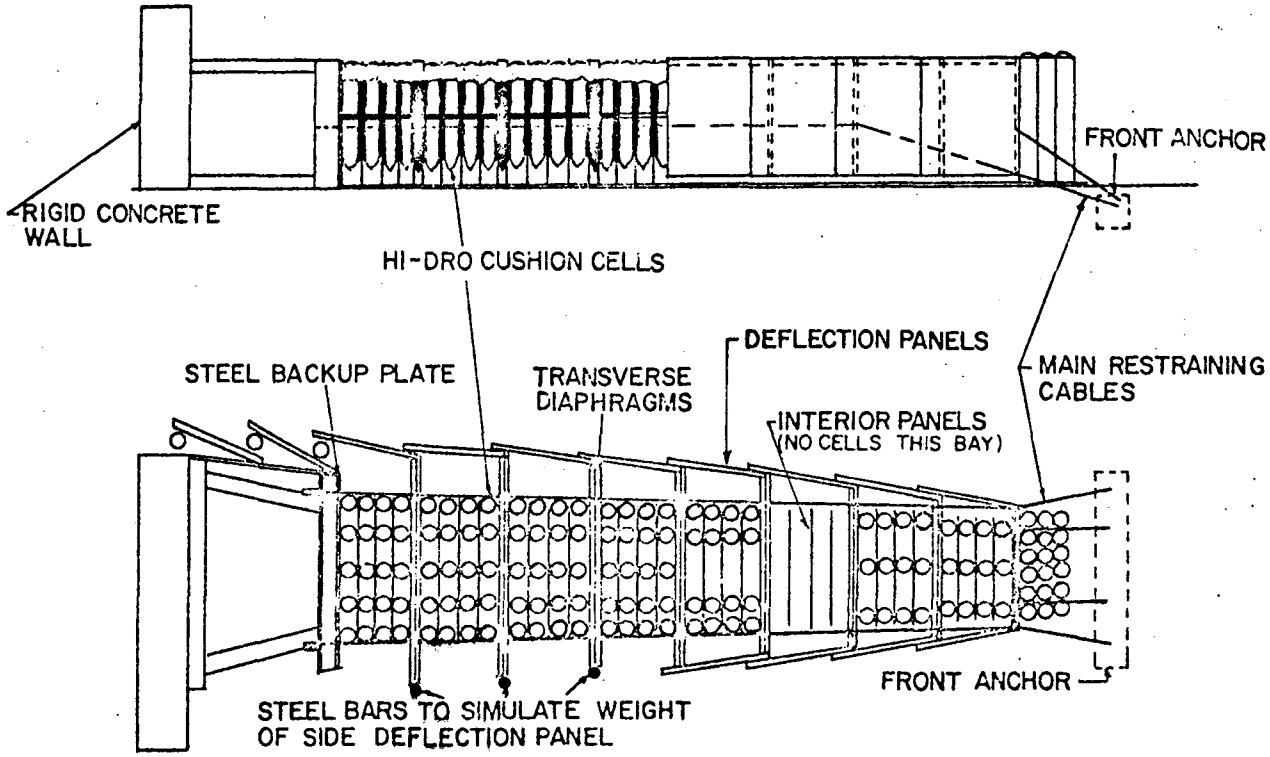

Figure 5.2.20 Hi-Dro Cell Sandwich Unit

TABLE 5.2.6
SUMMARY OF OPERATIONAL CRASH CUSHION SYSTEMS

System	Barrier Deformation		Vehicle Accelerations (Reab gs)			Impact Speed (mph)	Vehicle Exit Angle For Side Impacts
	Head-On Impact	Side Impact	Lateral	Longitudinal	Total		
Hi-Dro Cell Sandwich	18.0	--	NA	9.8	NA	62	--
Hi-Dro Cell Sandwich	---	NA	5.2	8.4	NA	57	<10°
Inertia Barrier	19.0'	--	NA	8.7	NA	59	--
Inertia Barrier	--	NA	NA	7.9	NA	57	No Redirection
Inertia Barrier	35.0	--	NA	3.3	NA	58	--
Inertia Barrier	--	46	6.0	8.0	NA	59	No Redirection
Hi-Dro Cell	14.5	--	NA	7.2	NA	56	--
Hi-Dro Cell	--	NA	4.5	4.0	NA	60	9°

cushions are patented.

Recommended structural and safety criteria for crash cushions are presented in Table 5.1.1 of these notes.

5.2.91 Placement Recommendations

It must be recognized that all of the crash cushions were designed and tested for relatively level terrain conditions. Adverse and unacceptable performance can be expected if the barrier is placed on or behind certain terrain conditions. It is highly desirable that the crash cushion be placed on a relatively flat surface (5 percent slope or less preferable) and that there be no appurtenances between the traveled way and the barrier.

Two prominent roadside features which the designer must often contend with are curbs and slopes. Tests and computer simulations have shown that both of these features can cause an errant vehicle to rise above the terrain and become airborne and reach undesirable roll and pitch angles. For new projects, curbs should not be built where crash cushions are to be installed. Existing curbs where cushions are to be installed should be removed, if feasible, in particular those that are higher than approximately 4 inches (0.1 m).

For roadside or median installations, it is desirable that the shoulder be extended to provide a relatively flat approach area to the cushion.

Unanchored crash cushions (Inertial Systems), when placed on elevated gores, may walk or crack due to vibration of the structure. However, at this writing there is no clear pattern of such occurrences.

The location of the crash cushion must allow for the penetration of the vehicle into the system. As illustrated in Figure 5.2.21, the last three modules of the inertia barrier should be outside the projected line of the hazard. Greater space should be provided where practical.

Hazardous gore areas have received the greatest attention with regard to crash cushion installations. It cannot be refuted that these areas have a higher potential for serious accidents than any other area of similar size along the roadway. Treatment of these areas should be given top priority. It now appears, however, that other areas, which heretofore had been shielded by conventional roadside barriers, can best be shielded, cost effectively, by crash cushions.

Figure 5.2.21 shows examples of median and roadside hazards which can be shielded either totally or partially by crash cushions. The approach areas should be flat and have no appurtenances between the traveled way and the cushion. If these conditions do not exist and cannot be provided, a roadside barrier placed near the shoulder is the recommended system. Selection of the barrier angle, Θ, should be based on the probable impact angle of encroaching vehicles. Impact angles will be dependent in most part on operating speeds, roadway alignment, and lateral distance from the traveled way to the cushion. For most roadside conditions, an angle of approximately 10 degrees or less is suggested.

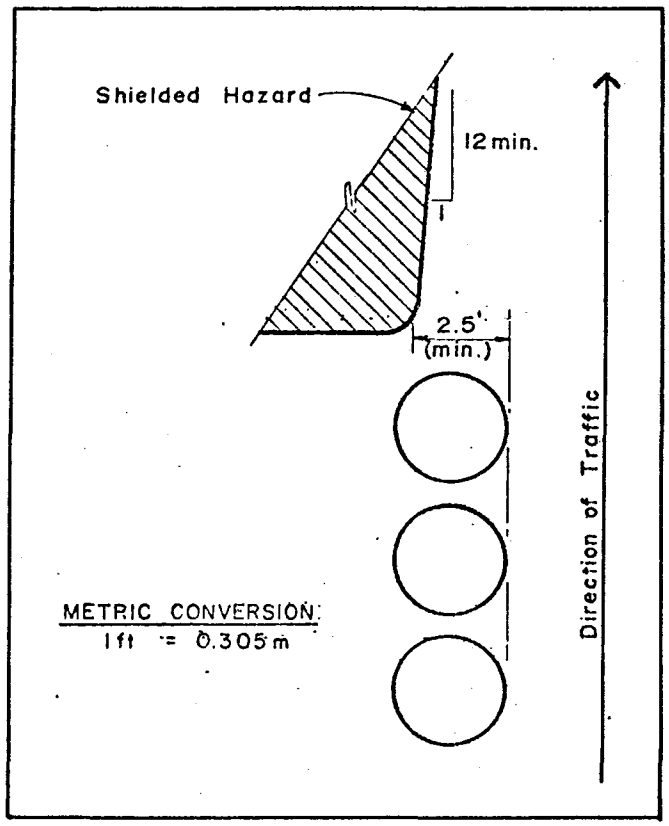

Figure 5.2.21 Suggested Layout for Last Three Exterior Modules in an Internal Barrier

All of the operational crash cushions can probably be adapted to shield rigid objects such as those shown in Figure 5.5.22. However, with the possible exception of the median barrier end treatments, the inertia barriers are more easily adapted to shield rigid objects. First, they do not require a back-up structure. Secondly, if exposed, the rear part of a non-inertial barrier system may itself be a significant hazard. Such problems would arise for median installations. It is likely, however, that the non-inertial systems could be adapted by careful design of transition and attachment details.

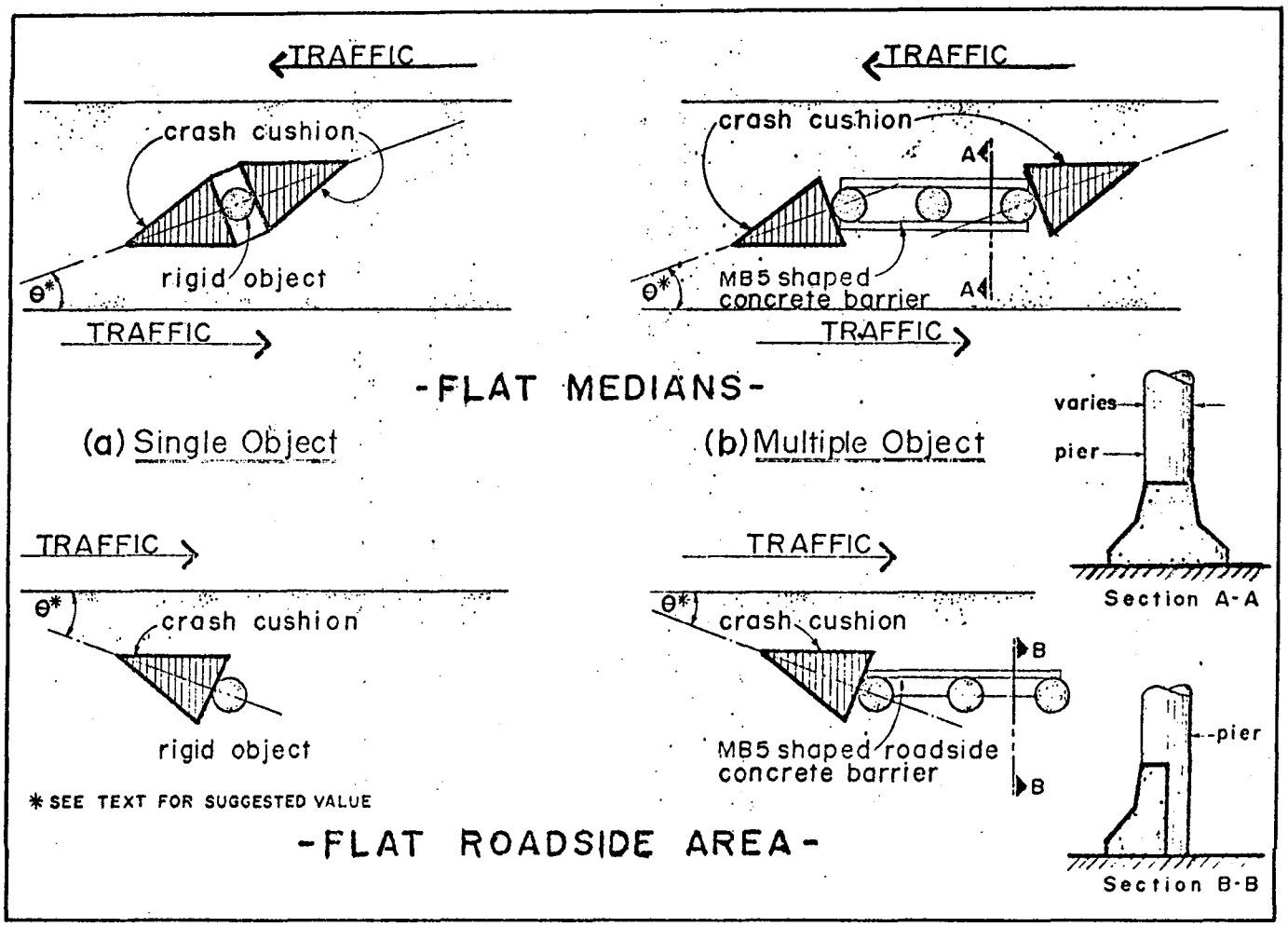

Figure 5.2.22 Examples of Possible Crash Cushion Application on the Roadside or in the Median

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TOPIC 5 SESSION 3

CROSS SECTION: NARROW BRIDGE TREATMENTS

Objectives:

The participant should be able to:

1. Classify narrow bridges on the basis of width relative to traffic operational requirements.

2. Identify the features of the bridge affecting the probability and severity of bridge accidents.

3. Select and implement treatments to improve the safety of narrow bridges.

5.3.1 INTRODUCTION

Some have defined a narrow bridge as any bridge that restricts the lateral movement of vehicles relative to the adjoining roadway section. In this context, virtually any bridge is narrow because it restricts drivers from leaving the roadway even though the bridge itself may be quite wide. This may be academic, but it does serve to illustrate the point that all bridges may deserve some special consideration. However, it is obviously those that severely restrict the lateral movement of vehicles that are of greatest concern to us. Here again, restriction is an inverse function of the width of the bridge.

"Narrow" bridges also may be defined or described on the basis of their effect on operational characteristics. The most common effects are driver behavior, reduction in capacity, and accident frequency.

Driver Behavior. Driver behavior effects may be used to identify and describe the degree of a narrow bridge if the bridge causes driver apprehension. This apprehension may be manifested in lateral vehicle displacement and/or a reduction in speed as the vehicle approaches and traverses the bridge. It has been observed that vehicles approaching and traversing a narrow bridge without opposing traffic simply move over the center line and continue through at a uniform speed. If, however, the vehicle is forced by on-coming traffic to remain in its lane, the most common effect then is the reduction in speed.

Reduction in Capacity. Although it is measurable in terms of fewer vehicles per hour, a reduction in capacity is actually a result of driver behavior, specifically driver apprehension. Due to the restricted lane width in which the driver must travel and due to the close proximity of the physical barrier to the right (the bridge rail) drivers frequently react by slowing their vehicle. A more consistent reaction of drivers, however, is the increase of time headway between their vehicle and the vehicle in front of them. This appears to be a natural reaction of the driver when he is placed under a stressful condition.

Increases in Accident Frequency. The close proximity of the bridge rail generally tends to increase the frequency and the rate of vehicular collisions. Rear end accidents may also increase.

Thus, when either or all of these operational characteristics are present, a bridge or a class of bridges may be termed "narrow."

5.3.11 Classification of Narrow Bridges

Because of the degree of the narrow bridge problem and the need for priorities in treatment, it is necessary that we establish a classification of narrow bridges and work from that classification in identifying and formulating safety treatment alternatives. For the purpose of presentation, narrow bridges are classified as follows:

CLASS I	Bridge Width	$\geq$ Crown Width
CLASS II	Roadway Width	$\leq$ Bridge Width $\leq$ Crown Width
CLASS III	Roadway Width	$\geq$ Bridge Width (2-lane operation)
CLASS IV	Roadway Width	$\geq$ Bridge Width (1-lane operation)

Crown width is the pavement width plus shoulders and a minimum two-foot clearance on each side.

5.3.2 ACCIDENT EXPERIENCE

Contrary to first thought, there are several ways in which an accident may occur at a bridge. Some vehicles may strike the end of the bridge rail; others may strike the approach rail to the bridge; some may pocket between the approach rail and the bridge rail; and others may snag on posts within the bridge; and still others may be redirected or spin out and collide with opposing

Figure 5.3.1 is an illustration of the probable distribution of accidents relative to the bridge rail and the approach end.

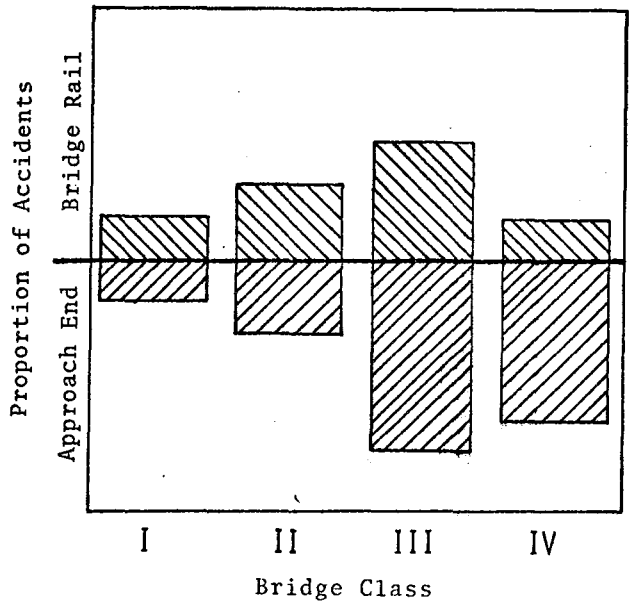


Figure 5.3.1 Conceptual Distribution of Accident Types Relative to Bridge Class.

vehicles. Data relative to the character of bridge accidents are not readily available. One study in Texas (1) on a primary 2-lane with frequent 26-ft (8-m) bridges showed that 50% of the bridge-related accidents involved vehicles striking the bridge rail, and the other 50% striking the bridge end or approach rail. While improvement of barriers at narrow bridges may enhance delineation and thus reduce accident frequencies, the principle benefit to be expected is a reduction in the severity of accidents. It is believed that the character of the accident may be a function of the class of bridge.

5.3.3 ELEMENTS OF NARROW BRIDGE SAFETY IMPROVEMENTS

There are three principal areas in which physical improvements to the bridge system will improve the overall safety of the bridge. These may be made singularly to effect safety improvements but, most generally, the improvements within all three areas or elements will be necessary to achieve a satisfactory level of narrow bridge treatment. These three basic elements are: 1) the approach rail system 2) the bridge rail system 3) traffic control and warning system.

5.3.31 Approach Rail

The purpose of the approach rail of the bridge is to prevent errant vehicles from going down the embankment and into the river, stream, railroad, or whatever the bridge may be crossing. Also, the approach rail must protect the errant driver from striking the rigid end of the bridge rail, or the bridge parapet, or becoming impaled on a loose section of the bridge rail. Further, the approach rail must perform this function in such a manner that it does not become a greater hazard than the one that it is shielding. Thus, the approach rail has the same basic requirements as roadside barriers, ie., a satisfactory approach end treatment, ribbon strength for structural integrity and compatibility with the end of the bridge rail to avoid pocketing of a colliding vehicle (2).

5.3.32 Terminal Connection

The requirements of a good terminal connection are: 1) it should provide anchorage to the barrier rail so that full ribbon strength may be developed immediately, 2) it must not impale the vehicle, and 3) it must intercept the vehicle without causing it to vault or overturn. Although there are four basic types of roadside barriers used for approach end treatment of bridges, this presentation of bridge treatment presumes the W-section type of rail because it is the most common. A more detailed and broader treatment is provided in Section 5.2 dealing specifically with traffic barriers (See Reference 2 Page 49).

There are four methods of effecting a satisfactory terminal connection. These are:

- Flared and embedded rail
- Breakaway cable terminal
- Turned-down rail (Texas Twist)
- Flared rail

Flared and Embedded. A very popular end treatment at the moment is flaring and embedment in the backslope of the highway or, in some instances, a man-made mound. Obviously, the best treatment for the end of a traffic barrier or approach rail is to flare it away from the roadway gradually, until it intercepts a natural backslope to the roadway where it may be anchored securely. This method frequently requires more length than other methods, but this may be an asset rather than a liability. At least it protects those vehicles that could otherwise have gone behind the rail. Additional costs may be offset by reducing the special treatment to avoid the ramping effect. The state of Virginia in 1976 initiated a major program along Interstate 81 to extend approach rails and tie them into the backslope.

Other states such as Oregon have built earth mounds that conceal the end of the approach rail. This is particularly applicable where rigid approach rails such as the concrete barrier are used.

Breakaway Cable Terminal. The breakaway cable terminal is a means of providing immediate ribbon strength to the barrier rail to resist interior collisions and at the same time to provide an accordion effect that will reduce the probability of impalement. In this method, the first post is made to break away, and the end of the rail is blunted so that the vehicle may force the rail to buckle. Then, the rail tends to function as an attenuation device. More detail is given on the breakaway cable terminal in Session 5.1.

Turned-down Rail. The turned-down rail, commonly known as the Texas Twist has gone through an evolution of design which may account for much of the varied reaction to the system. The idea of burying the end of the rail and tapering it upward to full height was first developed by GM Proving Grounds back in the 1950's. Texas adopted the idea and added to it the concept of twisting the rail in order to achieve a low profile, full strength anchorage to a concrete foundation. Unfortunately, Texas also designed an intermediate support post, built to support the first 25-ft (7.6-m) section of rail at the midway point at an angle of 45° and at 1/2 normal rail height. This post constituted a very hard spot in the rail. This hard spot intercepted the vehicle, giving it an upward trajectory, and sending the vehicle out of control. Texas soon realized the problem with their design and eliminated the intermediate post. Thus, the first 50 ft (15.2 m) of approach rail are secured in such a manner that the rail can be ridden down if the vehicle strikes the rail terminal head-on.

Because of a great concern for the behavior of compact cars on traffic barriers, Texas has modified their design further to cause the rail to break down and avoid the vaulting effect on compact vehicles. This is accomplished by using metal straps, rather than through-bolts to support the rail in the end sections.

Flared Rail. One of the least popular and least effective methods of rail treatment involves simply flaring the end to avoid impalement by a vehicle leaving the roadway. The flaring effect is only good for vehicles leaving the roadway. Those that have already left and are attempting to regain control and come back on the roadway may be lined up perfectly with the flared end. Further, the flared rail does not provide immediate ribbon strength; typically, 150 ft (46 m) of rail are required to achieve ribbon strength unless there is specific end anchorage applied.

5.3.33 Cost of Guardrail Safety End Treatment

The cost of safety end treatment of W-beam guardrail has concerned many highway engineers. Reports of excessively large costs on particular systems has limited the general application of some treatments. For this reason, an attempt was made (3, 4) to determine the experience in several states in order to provide a reasonable basis for decision making.

SAFETY END TREATMENT	AVERAGE COST
Flared and embedded rail	$300
Break-away cable terminal	$210
Turned down rail (falldown)	$200
Flared	$300

On the basis of cost, the BCT and Turned Down Rail (Falldown type) are about equal cost, while flaring increases the cost appreciably. The reader is reminded that the turned down rail (falldown type) is not currently approved for general use (5).

5.3.34 Post Spacing

Post spacing is most critical to the performance of semi-rigid traffic barriers on the bridge approach. For the W-beam rail, a normal post spacing of 6 ft- 3 in is utilized where the rail is expected to redirect the errant vehicle and guide the driver along the rail until the vehicle is brought to a stop. Where the rail attaches to the bridge, shorter post spacing is used to transition the rail behavior from a semi-rigid rail in the approach to a completely rigid rail on the bridge. Typically, posts are spaced at 3 ft- 1.5 in (0.95 m) which is 1/8 of a 25 ft section, for this effect.

5.3.35 Bridge Connection

The connection of the approach rail to the bridge is one of the more important elements. This connection must bridge the in compatibility between a semi-rigid approach rail and a rigid bridge rail. This is the point at which a colliding vehicle will pocket against the rigid end and spin helplessly out of control into the path of other vehicles,

or possibly at an angle which may cause the vehicle to go through or over the bridge rail on the opposite side of the roadway. There are several requirements of this bridge connection which include development of the tensile strength of the rail. The attachment should be at least as strong as the tensile strength of the rail on the approach. The rail connection should not be inset into the rail parapet. On the contrary, it should be mounted with a spacer that will place the approach rail out away from the concrete parapet. This helps to reduce the incompatibility of the W-section and the concrete parapet. A third requirement is that the connection should have beam strength to avoid the pocketing. Again, post spacing is used to gain this beam strength while others have used a secondary rail, such as a channel section, behind the W-beam and across the joint between the approach rail and the bridge parapet (See Reference 2 Page 42).

5.3.36 Bridge Rails

Bridge rails have several requirements including: 1) beam strength 2) redirection capability 3) height, and 4) pedestrian passage (under certain conditions). In some instances, appearance has been substituted for strength and safety design; how else can we explain the flimsy, ornamental rails that have been used in bridge design? Rails should be designed for strength to resist penetration. (Rails should stop all vehicles sometime, but not necessarily all vehicles all of the time), e.g., a rail should not be expected to stop a heavily-loaded truck at a steep angle and at high speed. On the other hand, it should be sufficient to stop the truck if it is proceeding at a slow speed and strikes the rail at a low angle.

Bridge rails should have a redirection capability that will avoid snagging and spinning out. This attribute depends on a rail with a smooth face. Many bridge rails now in existence are a combination of concrete posts and concrete rails, and they do not have redirection capability. In fact, this type of design may be the cause of a high percentage of bridge accidents. To illustrate this point, reference is made to a bridge improvement project on US 90 between Houston and San Antonio. This project is reported in detail in a 1976 issue of Traffic Engineering magazine. An analysis of accident data (see Table 5.3.1) showed that approximately one-half of the accidents were due to vehicles hitting the concrete posts and spinning out into the path of on-coming traffic. This situation was corrected by cutting off the old posts and recapping them to a lower height. A continuous bridge rail was provided by extending the W-section approach rail across the bridge. This treatment has reduced the accident rate as illustrated in Table 5.3.2.

There are numerous other types of bridge rails that may be considered for improving the performance of existing bridges. These include the concrete barriers, and the newly developed thrie-beam, which is one and one-half times the width of the W-section. The thrie-beam eliminates the snagging of posts supporting the bridge rail and

TABLE 5.3.1 "BEFORE" AND "AFTER" ACCIDENT EXPERIENCE

			Accidents by Type			
Time Span	Time Period	ADT	Hit Side of Bridge	Hit Bridge End	Hit Approach Bridge Rail	Total
(Before) Jan. 69-Oct. 70	22 months	4,780	10	7	3	20
(After) Nov. 70-3/15/72	17 months	5,690	1	2	1	4

TABLE 5.3.2 "BEFORE" AND "AFTER" ACCIDENT RATE

	Rates	
	Hit Side of Bridge	Hit End of Bridge or Approach Rail
Before	1.14 acc/yr/1,000 veh	1.14 acc/yr/1,000 veh
After	0.12 acc/yr/1,000 veh	0.37 acc/yr/1,000 veh

increases the beam strength. The thrie-beam is an offshoot from an earlier application of simply overlapping two W-sections to make a stronger and wider bridge rail.

Until just recently, AASHTO standards called for rather prominent curbs on bridges. Now that we understand better the dynamics of vehicle collisions with bridge rails, we realize that the curb frequently causes the colliding vehicle to vault onto or over the rail. One method of eliminating the adverse effects of the curb is to block out the rail so that the rail may intercept the bumper of the vehicle before the curb causes the vehicle to vault. The end of the curb or walkway should be ramped in any case.

Rail Heights. There are two basic requirement relative to rail height: the rail should be high enough that the vehicle will not vault over it and low enough that the driver or passengers will be able to see over it or through it. Drivers tend to shy away from tall rails, and drivers and passengers become frustrated when they cannot see over the rail. Reconstruction processes should include consideration of reducing the height of the rail to dimensions recommended in current standards, keeping in mind that the primary function of the rail is to keep the vehicle from going over it. (See Reference 2 Page 114).

Pedestrian Accommodations. In urban areas where there is substantial pedestrian activity, sidewalks should be continued across bridges, and generally separated from the traffic way by a 6-inch (15-cm) barrier curb. This appears to be completely acceptable for normal operating conditions on arterial streets. Where pedestrian walkways must be provided on bridges parallel to high-speed traffic streams, particularly those on access-controlled facilities, there should be two bridge rails--one between the pedestrians and the traffic stream and another at the outer edge of the pedestrian walkway.

5.3.37 Traffic Control Elements

Elements that are used to control traffic and warn the driver that he is approaching a bridge include warning signs, edge stripes, delineators, hazard markers and center stripes. NARROW BRIDGE signs should be used on Class III bridges. ONE LANE BRIDGE signs should be used on Class IV bridges. No advance signs are necessary for Class I or Class II bridges (where the structure is as wide or wider than the roadway crown section).

Signs should be "gate-posted," that is, placed on both sides of the approach roadway so that drivers in the process of passing or following a large truck will have maximum opportunity to get the message; further, gate-posting increases the target value of the signs tremendously.

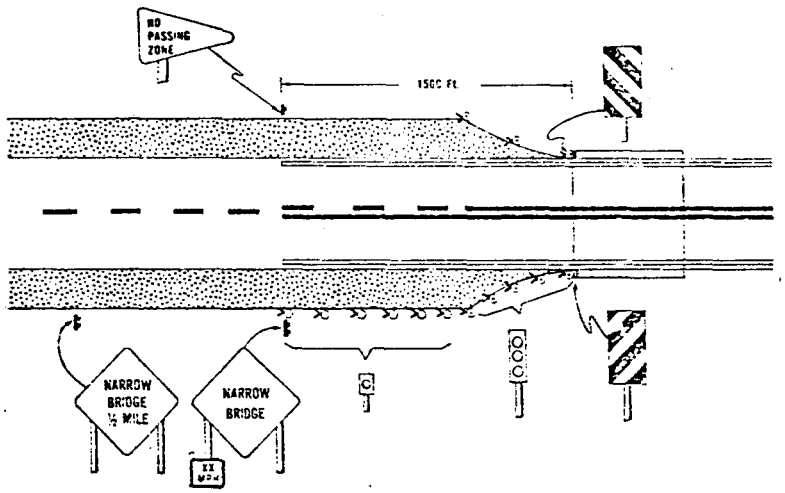
The placement of signs is a critical factor, and it is not adequately covered in the Manual on Uniform Traffic Control Devices. Signs should be placed well in advance of the bridge, and it is suggested that the signs be placed 10 times the 85 percentile speed in advance of the bridge end.

Edge stripes may be used to increase the target value of the narrow bridge. Where edge stripes are already in place, these should be increased in width to 6 or 8 inches (15 to 20 cm) beginning at the NARROW BRIDGE sign and continuing to the bridge. Continuing the edge stripe across the bridge is desirable, especially where the bridge is wider than the roadway. Where the bridge is essentially the same width as the roadway, dirt and debris at the edge of the bridge tend to obliterate the edge stripe. Furthermore, the driver needs a closer reference point than the edge stripe when he gets into the close confinement of the bridge. The better treatment is to switch from edgeline to bridge rail delineation at the bridge. It is important, however, that we provide some delineation across the bridge.

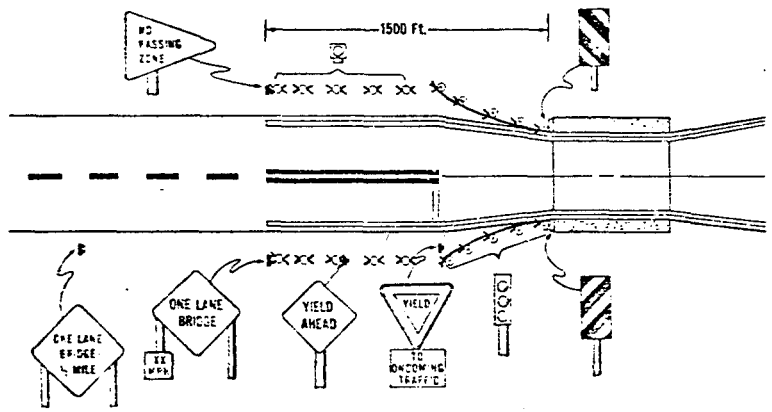
Where paved shoulders are provided and the bridge is narrower than the crown width, a tapered edge line should be placed beginning at the outside edge of the shoulder at the NARROW BRIDGE and tapered into the throat of the bridge. Then, the area between the tapered edge line and approach rail should be marked diagonally with heavy white reflectorized lines in such a manner that lines funnel the drivers onto the bridge (See Figure 5.3.2). This outer edge line may be supplemented further with reflectorized pavement markers. Roadside delineators should be used to delineate the traffic barrier on the bridge approach. These delineators should be placed immediately behind the rail at intervals of 25 ft (7.6 m), desirably. These delineators should be up above the rail so they will avoid most of the splash pattern of the traffic stream.

Hazard markers should be used to mark the end of the bridge for Class II, Class III, Class IV bridges. Hazard markers should be of the types recommended in the Manual on Uniform Traffic Control Devices, and they should be applied to the bridge-end or in direct alignment with the bridge rail. Hazard markers should be aligned with the closest obstruction such as curbs where curbs are provided on the bridge.

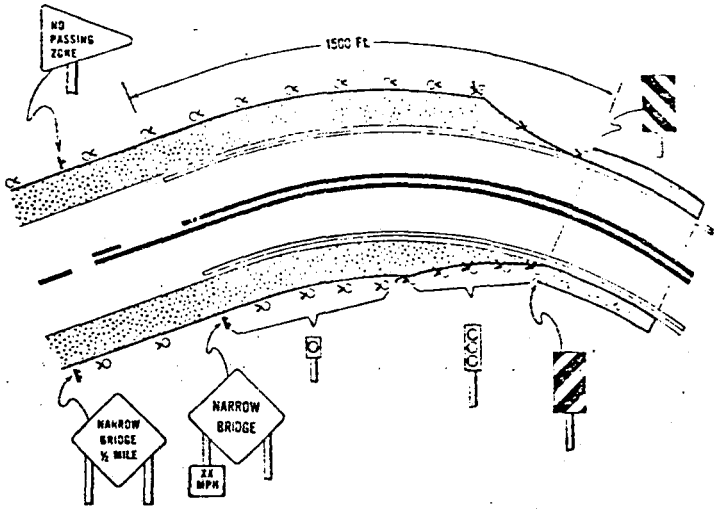
Of all the elements of narrow bridge treatments, delineation is perhaps the most important. Delineation can be used to draw the attention of the driver to the bridge and to identify the location of obstructions and the limits of the traveled



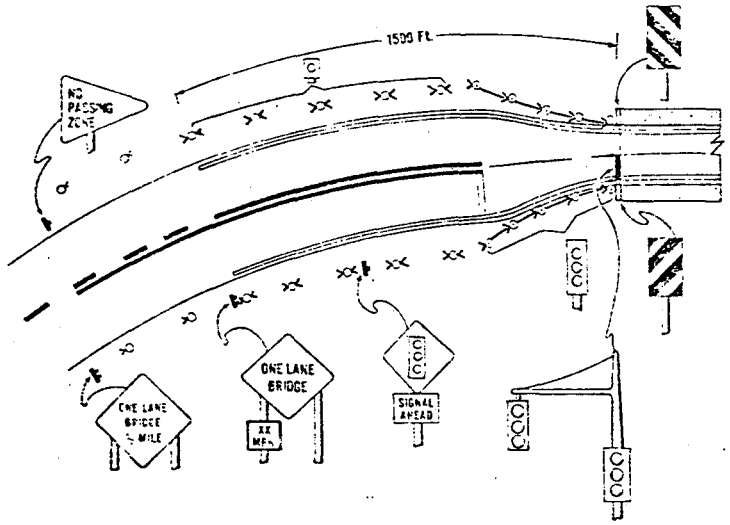
A. Class III Bridge



B. Class IV Bridge, One Lane



C. Class III, On Curve



D. Class IV Bridge, On Curve

Figure 5.3.2 Suggested Narrow Bridge Treatments

way. If this is done properly, then, hopefully there will be less need for the interception and redirection devices.

Centerline Applications. Generally, passing is prohibited on Class II, III and IV bridges. Passing is restricted by using a double solid yellow line beginning 100 to 500 ft (30 to 150 m) in advance of the bridge.

Markings for no-passing on bridges may be effectively supplemented with the use of yellow raised reflectorized pavement markers. These markers provide good visibility even in wet weather conditions and produce an audible sensation to warn a driver that he may be straying into the opposing traffic lane.

It is important that a psychological balance be achieved in the delineation of narrow bridges. Strong delineation along the approach rail and bridge rail may force the driver to the left unless the centerline is equally well delineated. Then the objective is to develop a visual channel with two delineation lines and force the driver between them.

5.3.4 SUMMARY

It is obvious at this point in time that there are many obsolete bridges throughout the country. Many of these bridges need to be replaced, whereas others need improvement. There should be a proper balance between replacement and improvement programs, and those bridges that are scheduled for replacement should be improved to every extent practicable in the interim period. We cannot wait, lest we may be faced with the catastrophic results of the spectacular bridge accident in New Mexico a few years ago. A review of the conditions surrounding that crash indicate that perhaps even a few of the points recommended in this section would have been sufficient to prevent that crash. Therefore, it is incumbent upon us to make the best use of the bridge structures we have until something better can be provided.

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TOPIC 6 SESSION 1

LEGAL RESPONSIBILITY OF PUBLIC OFFICIALS FOR HIGHWAY SAFETY

Objectives:

1. The participant should be cognizant of the background and development of tort liability as it relates to the modern setting, and

2. To be able to establish an engineering management program that will reduce the likelihood of losses due to tort liability on the part of the individual and the employing agency.

6.1.1 INTRODUCTION

What is the legal responsibility of public officials for highway safety? This is a very complex question, and the answer today may be somewhat different than it is tomorrow. Laws relative to tort liability at both the federal and state levels have been changed in recent years, and these changes, in turn, have altered materially the posture of the public official in the legal responsibility for highway safety. To better understand our position, perhaps we should approach the situation from the viewpoint of the average citizen.

You all have heard the expression, "You can't fight city hall . . ." Well, let me tell you something; the average citizen not only can fight city hall, but recent court decisions have held that, under many circumstances, he can win the fight. He can win because, more and more, the courts of the United States are saying, in effect, that the government is responsible for the torts committed by its agents, officers, and employees when they are acting within the scope of governmental authority. I repeat, the government is liable for the torts of its employees.

That brings up the next question, and that is, what is a *tort*. A tort is a private or civil wrong; it is a wrong, independent of any contract, which gives rise to a civil law suit for money damages. The most common tort is negligence, and negligence is the failure to use reasonable care in one's actions. Reasonable care, on the other hand, is that care that is expected to be exercised by a prudent person acting under the same or similar circumstances.

Negligent acts can take many forms, but the ones that we are primarily concerned with are those relating to highway safety. Generally, someone is injured in an automobile accident allegedly due to someone else's negligence. It may be the other driver, or it may be the government agent responsible for the highway facility, who allegedly committed the tort of negligence. At any rate, the injured party wants to be compensated financially for the personal injury and property damages that he has suffered. How does he accomplish this particular goal? Well, he has the right to sue--but who does he sue? He may, of course, sue the person he feels is responsible for his injury. However, winning the suit is only half the battle - he must collect before he actually accomplishes his goal. Being awarded a judgment against a defendant does not insure compensation - unless the defendant is solvent, is insured or is an agent for a firm or governmental body that is jointly responsible.

Since government employees are not known for their great personal riches, their employer, the government, is the plaintiff's target for just compensation. As everybody knows, the government appears to be made out of money. So, if the plaintiff can just sue the government, the major part of his battle has been won. The trouble was, however, that, up until modern times, within the past 20 to 40 years, an injured plaintiff has not been able to sue the government even though he was injured by the negligent acts of agents, officers, or employees of that governmental unit. Why could he not sue the government? The reason was due to the theory or concept that is called "sovereign or governmental immunity," exemplified by the cliché, "The King Can Do No Wrong."

6.1.2 THE IMMUNITY HISTORY

How did this concept of sovereign or governmental immunity come into existence? Well, one theory has it that it started at the Hill at Senlac, located near the city of Hastings, England, in 1066. William the Conqueror, who was oftentimes called Crazy Willie or Willie the Bastard by his troops, defeated the Anglo-Saxon King at the Battle of Hastings and proclaimed himself, William, as King of England; and he brought with him a peculiar institution called feudalism. The interesting thing about feudalism is that under this particular institution the king owned all the land and, in order to raise money for himself, he leased this land to tenants who, in turn, paid him rent. The tenant, in order to pay his rent, subleased a part of his land to other tenants, who, in turn, in order to pay their rent, subleased

a part of their land to other tenants. This concept of subleasing the land to tenants was the main idea behind feudalism.

Feudalism worked fine for everyone except the "little person" down on the bottom of the ladder, meaning the so-called share cropper. All he did was work all day and pay all of the products of his labor to his landlord as rent. Now these poor devils had practically no rights. In England at that time, there was no constitution; there were no statutes. The only law they had was called executive decree; the king simply decided what the law was. Now the poor people objected to this form of law, and through a period of time they were able to convince the king that they were entitled to some type of justice and some type of forum through which they could attempt to obtain justice. This led to the creation of courts by the king. These were called common law courts, and naturally the king said, "Since these courts have been created by me expressly for the people, there are going to be certain ground rules that I am going to insist upon. The first rule is that I am never going to be a defendant in my own court, meaning you cannot sue me. The King Can Do No Wrong."

This was the beginning of the concept that we call sovereign or governmental immunity. This doctrine crossed the Atlantic Ocean to America and became well established as the law in the United States. It was two-pronged: (1) You could not sue the government unless the government gave you permission to do so, and (2) Even if you could sue the government, the government was not responsible for the acts of its employees. These two rules pretty much prevented anybody from ever being successful in suing the government or ever being able to collect anything if they were able to sue. It is plain to see that if a person is going to be injured, the best way to be injured is by a guy who's driving a $25,000 automobile rather than a fellow who is wearing a government uniform.

6.1.3 WHAT ARE THE IMMUNITIES?

These are freedoms from all tort liability as a "favored defendant." Historically, there have been three - family, charitable and governmental. We are concerned only with the last.

After many years, the courts of the United States came to the conclusion that it was unfair to injured plaintiffs that they should have to suffer the entire consequences of the loss if they happened to be injured by the acts of governmental agents. Therefore, the court system actually was the first agency that began to eat away at the concept of governmental immunity. The courts began to rule that the government, when it is acting in a proprietary manner, meaning to make a profit, should be responsible for the acts of its employees; but when it was acting in a governmental function, meaning to perform those particular duties that are required by law, then it should retain immunity from tort claims.

This worked fine as long as the difference between a proprietary function and a governmental function was perfectly clear; however, it soon became evident from the decisions of the various courts throughout the United States that it was extremely difficult in many instances to distinguish between a proprietary action and governmental function. Courts held that the construction and maintenance of a sanitary sewer were governmental functions, but, on the other hand, that the construction and maintenance of a storm sewer were proprietary functions. Some courts held that the operation of a municipal airport was a governmental function, but that the operation of a port authority was a proprietary function. In fact, one court even held that the operation of a miniature train in a city park was a governmental function, and that the actions of the employees therein were not attributable to their employer, the city. It became evident that the legislative bodies of governments would have to step in and try to do something to correct these obvious deficiences.

The first act was established in 1946 by the Congress with the passing of the Federal Tort Claims Act which, among other provisions, held that the federal government would be liable for the negligent acts of its employees while acting under the scope of governmental authority in certain limited areas. Many states have followed the federal guidelines and have passed similar types of tort claims acts. Still other states, even though they have not gone as far as to pass a tort claims act, have passed specific statutes that make the state and local government liable for certain types of negligent acts of their employees, agents, and officers.

At this point, it is obvious that negligence is the key to tort liability. To better understand the position of the public official and the government body in tort liability, let's take a look at the classic negligence case. The plaintiff, if he is to win a judgment, must prove that:

(1) The defendant had a duty to use reasonable care towards the plaintiff

(2) He breached that duty, meaning that the defendant committed an act of negligence

(3) The negligence committed by the defendant was the proximate cause of the plaintiff's injury

(4) The plaintiff was not guilty of contributory negligence which caused the injury, and

(5) He incurred resulting damages

It is not our intent to instruct you in how to win a case against the plaintiff. First, that is the duty of your attorney; second, in reality, the plaintiff is due compensation for bona fide damages. On the other hand, the public official needs to be able to understand those situations in which the tort claims may be real or justified and where they are not. For this reason, we should look at the defense aspects of the tort liability case.

The best defense against tort liability is not necessarily an air-tight case against the plaintiff. It is, in fact, the avoidance of negligence by exercising sound judgment and due care - a prevent defense. We will address this point in greater detail later.

The second major point in the classic negligence case pertains to whether or not negligence on the part of the public official was the proximate cause of the injury to the plaintiff. What is meant by proximate cause? Proximate cause is defined as, "that which, in a natural and continuous sequence unbroken by any independent intervening cause, produces the injury and without which the result could not have occurred." A better name than proximate cause might be the direct cause.

The classic negligence case implies that the plaintiff must not be guilty of contributory negligence. Under the old classic concept of negligence suits this was true. Many states now have passed statutes dealing with comparative negligence which says that, even though a plaintiff is negligent, he still will be able to recover if the defendant was more negligent than the plaintiff. Let me explain it this way: If the defendant is found to be 100% negligent, and the plaintiff is found to be 40% negligent, then under the comparative negligence doctrine, the plaintiff would receive only 60% of what he would have received had he not been negligent at all.

There have been lawsuits in the United States in which plaintiffs have recovered very large judgments, and a close examination of the opinions as written in those cases leads one to believe that there was not enough negligence on the part of the defendant to justify this type of verdict. In other words, it appears that the jury returned a verdict for the plaintiff because the plaintiff was, in fact, injured and because the defendant could afford to pay. Based on those particular cases, it appears that the court system feels that it is too harsh to make an injured plaintiff suffer the entire burden of his misfortune himself if the burden of his loss can be spread among many such as the government. Regardless of the legal basis of a case, where a jury is involved, it sometimes appears that the plaintiff has to prove only the following points:

(1) The plaintiff was injured,

(2) The defendant can afford to pay, and

(3) There is some causal connection between the plaintiff and the defendant.

It can be seen readily through this latter concept that if all governmental units accept full responsibility for all negligent acts of their employees the cost of providing governmental services could escalate beyond any reasonable governmental financial capability.

For this reason and without exception, the federal government and every state government that has passed any type of legislation making governmental agencies liable in tort suits, have included in this legislation various exceptions to liability.

The ultimate goal, under the law, is not to escape liability, but to be protected from liability while using your best judgment and knowledge in building the best and safest highways and streets.

6.1.4 LIABILITY OF STATE HIGHWAY DEPARTMENTS AND LOCAL GOVERNMENTS

Now we do not have to be kicked in the head by a mule in order to see that the future is going to expand both governmental and personal liability for negligent actions in a majority of areas. With regard to the tort liability of state highway departments and local governments for design, construction, and maintenance negligence, the question is often decided on the basis of whether or not the activity or decision involved is classified as a discretionary function and therefore exempt from liability. The great majority of case law at this time agrees that highway design is discretionary because it involves planning and policy decisions. Design functions are quasi-legislative, and many experts feel that these functions must be protected from judicial interference. However, courts will not invoke design immunity in instances in which the design is unreasonable, careless, inherently dangerous, or where changed conditions dictate remedial action. It is highly unlikely that negligence in the construction of highways and roads will receive governmental immunity. This is especially true if there is negligence in carrying out the design or if new features are built into the plan. The area that is least likely to receive the protective umbrella of tort immunity is maintenance which fails to employ a reasonable degree of care. Maintenance of highways falls into the operational area as contrasted to the discretionary function of planning and design. Because of this fact, courts generally will find governmental liability in cases of apparent negligence in road maintenance.

Today, because many courts and legislatures have snipped the membrane of governmental immunity, the doctrine of *respondeat superior* allows the injured to look to the government as well as the engineer, employee or administrator for judgments in tort claims. In the vast majority of jurisdictions, even under the new concepts of government responsibility, the official employee is still personnally liable for his negligent acts. However, under some tort claims acts and other similar state laws, a judgment or settlement against the employer constitutes a complete bar to any action against the employee whose act or omission gave rise to the claim. The future, therefore, seems to hold situations in which injured plaintiffs will have a better chance of compensation. The cost of doing business as a unit of government will tend to increase, and the governmental employee may well see his own personal liability become more remote due to the responsibility of his employer.

Now what can you as a government official or employee do to best avoid liability for yourself and your employer? You can:

- Fulfill general duties, such as planning, design, construction, and maintenance with reasonable or even extreme care according to the most reliable, effective and proven methods that are available.

- Create or establish a system of regular inspection of physical premises such as roadways, signs, and signals with an emphasis on age, condition, function, and reliability. Try to anticipate changing conditions. Have a definite chain of command for reporting present and potential defects as well as changed conditions. Fix a definite responsibility for rapid remedial action in response to those reports. The keeping of an inventory of hazardous locations is not self-incriminating according to recent court rulings.

- Abide by established policies, guidelines and manual specifications. In fact, as a general rule, design should be predicated on criteria well above established minimum standards. Minimum standards should be used only when absolutely necessary.

- Stress the importance of reasonable care by all parties in areas of maintenance and construction.

- Prepare standards of performance in the areas of design, construction, maintenance, and regulation and then follow or exceed those standards in every instance.

- Beware of false economy.

The foolish cutting of necessary expenditures in order to appear to the taxpayer fiscally sound leads inevitably to careless and negligent work.

Our transportation system must offer the safest means of travel possible even to the unthinking motorist. The failure to appropriate and expend the funds necessary to insure the creation of this condition eventually will lead to a greater burden upon the the taxpayer as a direct result of ever-increasing money judgments awarded by courts and juries to victims of a system that has become unsafe for lack of proper funding. The only way to avoid any possible tort liability of governments, their agents and employees, is to avoid all accidents and resulting personal injury and property damage. Complete avoidance is, of course, impossible, but substantial reduction of accidents is not only possible but highly probable if every government official and every employee, agent or professional consultant, does their job as if their life and their property depended upon it which, in fact, may well be the case.

The public is entitled to be safe in the knowledge that public highways, roads and streets will be designed, constructed, and maintained in the best manner possible, and that the watchwords will be "public safety" and not "false economy." "Penny wise and dollar foolish" is not the solution to our common problems.

6.1.5 GUIDELINES AND PROCEDURES FOR AVOIDING TORT CLAIMS

6.1.51 Handling Complaints and Reports

The most basic feature of the tort claims cases is negligence on the part of the agency employee. Characteristically, these actions claim failure to respond to complaints or failure to respond in a reasonable period of time. Thus, the approaches to minimizing claims involve rapid and orderly response to complaints along with maintenance of adequate records to document the actions taken.

Possibly the most critical requirement in the handling of complaints is to designate one person or one office to receive all complaints or notices of defective traffic control devices. This person should:

- Record the date and time of complaint.

- Record the name, address and telephone number of individual.

- Record the location and nature of the complaint.

- Decide the nature of action to be taken.

- Instruct maintenance personnel to take appropriate action in case of malfunctions or loss of traffic control devices, pavement holes and similar potentially critical problems.

- Ask for police support until repairs can be completed, etc.
- Maintain a record system by location (i.e., intersection or roadway segment.)
- Review periodically these files and designate locations to be critically reviewed.

6.1.52 Inventory of Traffic Control Devices

Routine inspection of all traffic control devices, by day and at night, on a regular basis is a fundamental step in loss prevention. The period between inspections will vary, but a good guideline for most agencies is a six-month review. Traffic control in construction and maintenance areas should be reviewed at the close of each work day. Additionally, all agency employees should be trained to look for and report any defective devices. This is particularly important for police, solid waste collection personnel, utility workers, and other agency personnel who routinely work on the street system. Emphasis should be placed on identification and reporting of defective or damaged devices. Each agency vehicle should contain a reminder card or display of the appropriate telephone number or office to notify when a defective device is identified.

6.1.53 Design and Operational Reviews

Based on complaint history, sites should be selected periodically for critical review. Both the basic design and the traffic control elements should be reviewed in the field. If possible, interviews with persons who have filed complaints should be conducted. Often rather minor improvements can result in substantial improvement in the safety record of a location.

6.1.54 Tort Liability Insurance

In spite of all reasonable care on the part of agency personnel, some incidents are certain to occur. Claims resulting from these agencies can place a substantial burden on local governmental funds. This means that most agencies must maintain some form of insurance. Larger agencies will, in all likelihood, be self-insured. Smaller units of government will most certainly be required to purchase liability insurance.

6.1.55 Summary of Tort Loss Reduction Guidelines

- Provide a central focal point for all complaints dealing with highway or street design or traffic control.
- Develop an organized program to respond to complaints and to evaluate points on the system which have an unusually high complaint rate.
- Inventory traffic control devices on a regular basis.
- Conduct a design and operational review of locations with unusually high complaint rate.
- Develop a feedback system from agency employees.
- Provide liability insurance against claims

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