

HAWAII STATEWIDE UNIFORM DESIGN MANUAL FOR STREETS AND HIGHWAYS

REVISION NO. 1

Revisions to the Hawaii Statewide Uniform Design Manual for Streets and Highways, October 1980

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STATE OF HAWAII

DEPARTMENT OF TRANSPORTATION

STATEWIDE

UNIFORM DESIGN MANUAL

FOR STREETS AND HIGHWAYS

OCTOBER 1980

ISSUED BY

HIGHWAYS DIVISION

IN COOPERATION WITH

DEPARTMENTS OF PUBLIC WORKS
COUNTY OF KAUAI
CITY AND COUNTY OF HONOLULU
COUNTY OF MAUI
COUNTY OF HAWAII

DEPARTMENT OF TRANSPORTATION SERVICES
CITY AND COUNTY OF HONOLULU

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STATE OF HAWAII
DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

STATEWIDE

UNIFORM DESIGN MANUAL

FOR STREETS AND HIGHWAYS

OCTOBER 1980

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P R E F A C E

This Manual is prepared in conformance with the State of Hawaii's Rules and Regulations Governing the Design, Construction and Maintenance of Public Streets and Highways, dated September 1969.

Because of varying needs and terrain conditions, it may be necessary for individual Counties to supplement this Manual with design guides applicable to local conditions for the design of streets and highways. Such supplemental design guides should generally be within the limits contained herein.

Acknowledgment

The preparation of this Manual is the result of a team effort consisting of personnel from the following offices who contributed to the working drafts and provided excellent review comments of those drafts.

Federal Highway Administration, Hawaii Division
Department of Transportation
- Highways Division
- Motor Vehicle Safety Office

Departments of Public Works
- County of Kauai
- City and County of Honolulu
- County of Maui
- County of Hawaii

Department of Transportation Services
- City and County of Honolulu

Acknowledgment is also made to the American Association of State Highway and Transportation Officials (AASHTO), the Transportation Research Board (TRB), the California Department of Transportation (CALTRANS), Northwestern University Traffic Institute (TI), and the Federal Highway Administration (FHWA), whose materials form the basis of this Manual.

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SECTION 1.

I N T R O D U C T I O N

1-01 PURPOSE

The purpose of this Manual is to promote uniformity, clarity, and economy in the geometric design of streets and highways. The Manual provides uniform procedures and guidelines for the design of safe, efficient and attractive streets and highways. The objective of this Manual is the design of safe and attractive streets and highways which are compatible with their service characteristics, environment and cost.

The design guidelines set forth in this Manual should be used in the design of all new public streets and highways to be constructed in the State. Existing public streets and highways which do not conform to these design guides are not affected, but any replacement or upgrading made to such streets and highways, or major portions thereof, should conform, where practical, to the design guides contained herein.

This Manual is neither intended as, nor does it establish, a legal standard for the design of streets and highways.

1-02 SCOPE

Applicable to all public road facilities – whether public agency or private agency does work

For Fed-Aid projects follow manual

1-02.1 General

Strictly manual for Geometric Design (i.e. not for pvt marking, lighting etc)

This Manual includes guidelines, criteria and procedures to be used in the design of streets and highways. It is not a textbook or a substitute for engineering knowledge, experience, or judgment.

1-02.2 Criteria and Guidelines

The design criteria and guidelines given herein are generally minimum values, or they are desirable values. However, they are not inflexible; they are subject to change as conditions and experience may warrant. Variations from what is shown in this Manual will be necessary for special or unusual conditions relating to traffic, right-of-way, terrain, climate, drainage, or soils. This Manual encourages initiative and judgment in handling special situations. However, in the interest of obtaining uniformity of design, when practicable, proposed variations from this Manual should be approved by the agency having jurisdiction of the roadway under design.

In attempting to comply or abide with the design criteria and guidelines given herein, it is recognized that the agency having jurisdiction of the roadway does not have an infinite amount of money, manpower and resources. Therefore, the agency is to do as much as possible to comply with the design criteria and guidelines within the limits of its money, manpower and resources.

1-02.3 Meanings of "Shall", "Should" and "May"

To clarify the meanings intended in this Manual by the use of these words, the following definitions apply:

1. Shall

   A mandatory condition. Where certain design criterion is described in the design of the street or highway, it is mandatory that this criterion be met.

2. Should

   An advisory condition. Where the word "should" is used, it is considered to be advisable usage, recommended but not mandatory.

3. May

   A permissive condition. Design or application is optional.

1-02.4 The Use of AASHTO Standards

The American Association of State Highway and Transportation Officials has published policies on highway design. These are approved references to be used in conjunction with this Manual. AASHTO policies represent nationwide standards which do not always satisfy Hawaii conditions. When standards differ, the instructions in this Manual shall govern.

1-02.5 Form

The loose-leaf form is used for this Manual because it facilitates change and expansion. New instructions will be issued as sheets in the format of this Manual; these may consist of additional sheets or new sheets to be substituted for those superseded.

1-03 SOURCE AND METHOD

This Manual presents generally accepted criteria substantiated by the research efforts of other state agencies and various federal, educational and independent organizations. Contributions from the

following organizations, whose publications provided much of the primary reference sources, were instrumental in the development of this document.

- American Association of State Highway and Transportation Officials
- Transportation Research Board
- Federal Highway Administration
- Institute of Transportation Engineers
- California Department of Transportation
- Traffic Institute, Northwestern University

1-04 OFFICIAL NAMES AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
DOT	Department of Transportation, State of Hawaii
DTS	Department of Transportation Services, City and County of Honolulu
DPW	Department of Public Works (of the Counties)
FHWA	Federal Highway Administration, U.S. Department of Transportation
CALTRANS	California Department of Transportation

SECTION 2.

DESIGN PARAMETERS

2-01 GENERAL

Geometric design is the design of the visible dimensions of a highway with the objective of forming or shaping the facility to the characteristics and behavior of drivers, vehicles and traffic. Therefore, geometric design deals with features of location, alignment, profile, cross section, intersection and highway types.

The term highway, road or street is a general term denoting a public way for purposes of vehicular travel including the entire area within the right-of-way. For rural areas the term highway or road is used; for urban areas, highway or street is used.

Control of access is the condition where the right of owners or occupants of abutting land or other persons to access, light, air or view in connection with a highway is fully or partially controlled by public authority.

Full control of access means that the authority to control access is exercised to give preference to through traffic by providing access connections with selected public highways only by prohibiting crossings at grade or private driveway connections.

Partial control of access means that the authority to control access is exercised to give preference to a degree that, in addition to access connections with selected public highways, there may be crossings at grade and some private driveway connections.

2-02 HIGHWAY TYPES AND CLASSIFICATION

2-02.1 Highway Types

Arterial Highway

A general term denoting a highway primarily for through traffic, usually a continuous route.

Expressway

A divided arterial highway for through traffic with full or partial control of access and generally with grade separations at intersections.

Freeway

A divided, multi-lane highway with full access control and grade separations at intersections.

Major Street or Major Highway

An arterial highway with intersections at grade and direct access to abutting property, and on which geometric design and traffic control measures are used to expedite the safe movement of through traffic.

Divided Highway

A highway with separated roadbeds for traffic in opposing directions.

Collector Street or Road

A street or road that collects traffic from local streets and channels it to the arterial highway system. Conversely, a street or road that penetrates the neighborhoods, distributing trips from the arterial highway system through the area to the ultimate destination which may lie on a local street or road.

Through Street or Highway

Any street or highway protected by stop or yield signs, or other traffic control devices, from intersecting traffic.

Local Street or Road

A street or road primarily for access to residence, business, or other abutting property.

Cul-de-Sac Street

A local street open at one end only with special provisions for turning around.

Dead End Street

A local street open at one end only without special provisions for turning around.

Frontage Road

A local street or road auxiliary to and located on the side of an arterial highway for service to abutting property and adjacent areas and for control of access.

2-02.2 Highway Classification

The characteristics for classification of a facility are functionally categorized for urban areas and rural areas. The systems criteria are expressed in qualitative terms, rather than in quantitative terms. Because of varying geographic conditions (population density, spacing and size of cities and communities, and the pattern of road network), it is not feasible to define uniform statewide criteria for roads based upon population centers, on trip lengths, traffic volumes, etc.

For design purposes, the following classifications are adopted:

1. Freeway

   A divided, multi-lane highway with full access control and grade separations at intersections. This is the highest type of highway facility to be designed. Freeways may be designed for urban as well as rural areas. The Lunalilo Freeway (I-H1) is an example of a freeway in an urban area. The I-H2 Freeway is an example of a freeway in a rural area.

2. Arterial Highways

   a. Urban Areas

      Arterial highways are streets and highways identified as unusually significant to the area in which they lie in terms of the nature and composition of the traffic. The urban arterial streets and highways should serve the major centers of activity of a metropolitan area, the highest traffic volume corridors, and the longest trip desires. On Oahu, Kapiolani Boulevard, Dillingham Boulevard and Kamehameha Highway in Pearl City, would be examples of urban arterial highways. Kaahumanu Avenue and Puunene Avenue are urban arterial highways on Maui; while Kilauea Avenue and Kanoelehua Avenue are examples of urban arterial highways on Hawaii. A portion of Kuhio Highway on Kauai would be considered an urban arterial highway on Kauai.

   b. Rural Areas

      A rural arterial highway is a road connecting major population centers and, in Hawaii, may be the only highway in the area. For example, Kamehameha Highway on the North Shore of Oahu, or Kuhio Highway on Kauai, or Honoapiilani Highway on Maui, or the Volcano Road in east Hawaii are all considered rural arterial highways.

3. Collector Streets and Roads

a. Urban Areas

The collector street system provides both land access service and traffic circulation within residential neighborhoods, commercial and industrial areas. It differs from the arterial system in that facilities on the collector system may penetrate residential neighborhoods, distributing trips from the arterials through the area to the ultimate destination. Conversely, the collector street also collects traffic from local streets and channels it into the arterial system. Hawaii Kai Drive, Sand Island Road, and Komo Mai Drive on Oahu; Hardy Street on Kauai; Kamehameha Avenue on Maui; and Kekuanaoa Street on Hawaii, are examples of urban collector streets.

b. Rural Areas

The rural collector roads in the State collect traffic from local roads and provide a link from a population center to an arterial highway, or to another population center. Population centers may be as small as 2 or 3 houses within a defined area. Haleiwa Road and Hakimo Road on Oahu; Lahainaluna Road on Maui; Kawaihau Road on Kauai; and Plumeria Road and Mamane Street on Hawaii are examples of rural collector roads.

4. Local Roads

The local road or street system comprises all facilities not on one of the higher systems. It serves primarily to provide access to abutting land and access to the higher order systems. This is the lowest type facility, which includes cul-de-sac streets and/or dead end streets.

A complete discussion of concepts, criteria and procedures for functional classification is included in FHWA publication "Highway Functional Classification", dated July 1974.

2-03 DESIGN CONTROLS

2-03.1 General

The location and geometric design of highways are affected by numerous factors and controlling features. These may be considered in two, broad categories as follows:

1. Primary Controls

- Highway Systems and Classification
- Topography and Physical Features
- Traffic

2. Secondary Controls

- Design Speed
- Design Vehicle
- Capacity

2-03.2 Primary Controls

1. Highway Systems and Classification

Highways are generally designated by systems for broad planning, design and administrative purposes. Nationally, the highways are divided into systems such as the Federal-aid highway system composed of the National System of Interstate and Defense Highways, the Primary System and the Secondary System. In addition, each state and each county has its own system of streets and highways grouped into similar systems.

From a functional point of view, the streets and highways in the State are classified into four types as follows:

a. Freeway (or expressway)
b. Arterial highway or arterial road
c. Collector street or collector road
d. Local street or local road

Such classifications immediately set the stage for the general type and character of the highway. For example, a freeway indicates relatively high design speed, divided roadways, control of access, etc., whereas a local street would call for no more than two traffic lanes, driveway accesses, at-grade intersections, etc. Thus, the highway system or the particular highway classification provides a positive control for the geometric design of the highway.

2. Topography and Physical Features

The location and the geometric features of a highway are influenced to a large degree by the topography, physical features,

and land use of the area traversed. The character of the terrain has a pronounced effect upon the longitudinal features of the highway, and frequently upon the cross-sectional features as well. Geological conditions may also affect the location and the geometrics of the highway. Climatic, soil and drainage conditions may affect the profile of the road relative to existing ground.

Man-made features and land use may also have considerable effect upon the location and the design of the highway. Industrial, commercial, and residential areas will each dictate different geometric requirements.

3. Traffic

The traffic characteristics, volume, composition and speed, indicate the service for which the highway improvement is being made and directly affects the geometric features of design, such as number of traffic lanes, alignment, grades, etc.

The traffic volume affects the capacity, and thus the number of lanes required. For planning and design purposes, the demand of traffic is generally expressed in terms of the design-hourly volume (DHV), predicated on the design year.

The composition of traffic, i.e., proportion of trucks and buses, is another characteristic which affects the location and geometrics of highways. Types, sizes and load-power characteristics are some of the aspects taken into account.

The following definitions apply to traffic data elements pertinent to design:

ADT - Average daily two-way traffic volume. The applicable year (current and future) is specified.

DHV - The design-hourly volume. It is normally the estimated 30th highest hour two-way traffic volume for the design year selected.

K - Ratio of DHV to ADT, expressed as a percent.

D - The directional distribution of traffic during the design hour. It is the one-way volume in the predominant direction of travel expressed as a percentage of DHV.

T - The proportion of trucks, exclusive of light delivery trucks, expressed as a percentage of DHV.

V - The design speed in miles per hour.

2-03.3 Secondary Controls

1. Design Speed

"Design Speed" is a speed determined for design and correlation of the physical features of a highway that influence vehicle operation. It is a maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern.

The assumed design speed should be a logical one with respect to the character of terrain and the type of highway. Every effort should be made to use as high a design speed as practicable, to attain a desired degree of safety, mobility and efficiency. Once selected, all of the pertinent features of the highway should be related to it to obtain a balanced design. Some features, such as curvature, superelevation, and sight distance are directly related to, and vary appreciably with, design speed. Other features, such as widths of pavements and shoulders, and clearances to walls and rails, are not directly related to design speed, but they affect vehicle speed, and higher standards should be accorded these features for the higher design speeds. Thus, nearly all design elements of the highway are subject to increase or decrease with a change in design speed.

Since design speed is predicated on the favorable conditions of climate and little or no traffic on the highway, it is influenced principally by:

- character of the terrain;
- extent of man-made features;
- economic considerations (as related to construction and rights-of-way costs).

These three factors apply only to the selection of a specific design speed within a logical range pertinent to a particular system or classification of which the facility is a part.

When the design speed is below the posted or regulatory speed limit due to implementation of improvements, this discrepancy should be treated as a design exception and proper mitigating measures should be taken.

2. Design Vehicle

The physical characteristics of vehicles and the proportions of the various size vehicles using the highways are positive

controls in geometric design. A design vehicle is a selected motor vehicle, the weight, dimensions and operating characteristics of which are used to establish highway design controls to accommodate vehicles of a designated type. The symbols and dimensions of design vehicles are shown in Table 2-1.

Table 2-1

Design Vehicle Dimensions

Design Vehicle Type	Symbol	Height	Width	Length	Front	Rear	WB1	WB2	S	T	WB3
Passenger car	P	4.25	7	19	3	5	11				
Single unit truck	SU	13.5	8.5	30	4	6	20				
Single unit bus	BUS	13.5	8.5	40	7	8	25				
Articulated Bus	A-BUS	10.5	8.5	60	8.5	9.5	18		4a	20a	
Combination trucks											
Intermediate semitrailer	WB-40	13.5	8.5	50	4	6	13	27			
Large semitrailer	WB-50	13.5	8.5	55	3	2	20	30			
"Double Bottom" semi-trailer-full-trailer	WB-60	13.5	8.5	65	2	3	9.7	20	4b	5.4b	20.9
1983 Interstate Design Vehicle		13.5	8.5	70	4	3	20	43			
Recreation Vehicles											
Motor Home	MH		8	30	4	6	20				
Car and camper trailer	P/T		8	49	3	10	11	5	18		
Car and boat trailer	P/B		8	42	3	8	11	5	15		

a = Combined dimension 24, split is estimated.
b = Combined dimension 9.4, split is estimated.
WB1, WB2, WB3, are effective vehicle wheelbases
S is the distance from the rear effective axle to the hitch point.
T is the distance from the hitch point to the lead effective axle of the following unit.

3. Capacity and Level-of-Service Concepts

a. General

A principal objective of capacity analysis is the estimation of the maximum amount of traffic that can be accommodated by a given facility. Capacity analysis would, however, be of limited utility if this were its only focus. Traffic facilities generally operate poorly at or near capacity, and facilities are rarely designed or planned to operate in this range. Capacity analysis is also intended to estimate the maximum amount of traffic that can be accommodated by a facility while maintaining prescribed operational qualities.

Capacity analysis is, therefore, a set of procedures used to estimate the traffic-carrying ability of facilities over a range of defined operational conditions. It provides tools for the analysis and improvement of existing facilities, and for the planning and design of future facilities.

The definition of operational criteria is accomplished using levels of service. Ranges of operating conditions are defined for each type of facility, and are related to amounts of traffic that can be accommodated at each level.

The following sections present and define the two principal concepts of this manual: capacity and level of service.

b. Capacity

In general, the capacity of a facility is defined as the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions.

The time period used in most capacity analysis is 15-min, which is considered to be the shortest interval during which stable flow exists.

Capacity is defined for prevailing roadway, traffic, and control conditions, which should be reasonably uniform for any section of facility analyzed. Any change in the prevailing conditions will result in a change in the capacity of the facility. The definition of capacity assumes that good weather and pavement conditions exist.

1. Roadway conditions - Roadway conditions refer to the geometric characteristics of the street or highway, including: the type of facility and its development environment, the number of lanes (by direction), lane and shoulder widths, lateral clearances, design speed, and horizontal and vertical alignments.

2. Traffic conditions - Traffic conditions refer to the characteristics of the traffic stream using the facility. This is defined by the distribution of vehicle types in the traffic stream, the amount and distribution of traffic in available lanes of a facility, and the directional distribution of traffic.

3. Control conditions - Control conditions refer to the types and specific design of control devices and traffic regulations present on a given facility. The location, type, and timing of traffic signals are critical control conditions affecting capacity. Other important controls include STOP and YIELD signs, lane use restrictions, turn restrictions, and similar measures.

These and other factors affecting capacity are discussed in greater detail in a subsequent section of this chapter.

It is also important to note that capacity refers to a rate of vehicular or person flow during a specified period of interest, which is most often a peak 15-min. period. This recognizes the potential for substantial variations in flow during an hour, and focuses analysis on intervals of maximum flow.

c. Levels of Service

The concept of levels of service is defined as a qualitative measure describing operational conditions within a traffic stream, and their perception by motorists and/or passengers. A level-of-service definition generally describes these conditions in terms of such factors as speed and travel time, freedom to maneuver, traffic interruptions, comfort and convenience, and safety.

Six levels of service are defined for each type of facility for which analysis procedures are available. They are given letter designations, from A to F, with level-of-service A representing the best operating conditions and level-of-service F the worst.

1. Level-of-service definitions - In general, the various levels of service are defined as follows for uninterrupted flow facilities:

*Level-of-service A represents free flow. Individual users are virtually unaffected by the presence of others in the traffic stream. Freedom to select desired speeds and to maneuver within the traffic stream is extremely high. The general level of comfort and convenience provided to the motorist, passenger, or pedestrian is excellent.

*Level-of-service B is in the range of stable flow, but the presence of other users in the traffic stream begins to be noticeable. Freedom to select desired speeds is relatively unaffected, but there is a slight decline in the freedom to maneuver within the traffic stream from LOS A. The level of comfort and convenience provided is somewhat less than at LOS A, because the presence of others in the traffic stream begins to affect individual behavior.

*Level-of-service C is in the range of stable flow, but marks the beginning of the range of flow in which the operation of individual users becomes significantly affected by interactions with others in the traffic stream. The selection of speed is now affected by the presence of others, and maneuvering within the traffic stream requires substantial vigilance on the part of the user. The general level of comfort and convenience declines noticeably at this level.

*Level-of-service D represents high-density, but stable, flow. Speed and freedom to maneuver are severely restricted, and the driver or pedestrian experiences a generally poor level of comfort and convenience. Small increases in traffic flow will generally cause operational problems at this level.

*Level-of-service E represents operating conditions at or near the capacity level. All speeds are reduced to a low, but relatively uniform value. Freedom to maneuver within the traffic stream is extremely difficult, and it is generally accomplished by forcing a vehicle or pedestrian to "give way" to accommodate such maneuvers. Comfort and convenience levels are extremely poor, and driver or pedestrian frustration is generally high. Operations at this level are usually unstable, because small increases in flow or minor perturbations within the traffic stream will cause breakdowns.

*Level-of-service F is used to define forced or breakdown flow. This condition exists wherever the amount of traffic approaching a point exceeds the amount which can traverse the point. Queues form behind such locations. Operations within the queue

are characterized by stop-and-go waves, and they are extremely unstable. Vehicles may progress at reasonable speeds for several hundred feet or more, then be required to stop in a cyclic fashion. Level-of-service F is used to describe the operating conditions within the queue, as well as the point of the breakdown. It should be noted, however, that in many cases operating conditions of vehicles or pedestrians discharged from the queue may be quite good. Nevertheless, it is the point at which arrival flow exceeds discharge flow which causes the queue to form, and level-of-service F is an appropriate designation for such points.

These definitions are general and conceptual in nature, and they apply primarily to uninterrupted flow. Levels of service for interrupted flow facilities vary widely in terms of both the user's perception of service quality and the operational variables used to describe them. Each chapter of the manual contains more detailed descriptions of the levels of service as defined for each facility type.

2. Service flow rates - The service flow rate is the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions while maintaining a designated level of service. As to capacity, the service flow rate is generally taken for a 15-min time period.

The service flow rates are discrete values, while the levels of service represent a range of conditions. Because the service flow rates are defined as maximums for each level of service, they effectively define flow boundaries between the various levels of service.

3. Measures of effectiveness - For each type of facility, levels of service are defined based on one or more operational parameters which best describe operating quality for the subject facility type. While the concept of level of service attempts to address a wide range of operating conditions, limitations on data collection and availability make it impractical to treat the full range of operational parameters for every type of facility. The parameters selected to define levels of service for each facility type are called "measures of effectiveness," and represent those available measures that best describe the quality of operation on the subject facility type. Table 2-2 gives the measures of effectiveness used to define levels of service for each facility type.

Each level of service represents a range of conditions, as defined by a range in the parameter(s) given in Table 2-2. Thus, a level of service is not a discrete condition, but rather a range of conditions for which boundaries are established.

TABLE 2-2 MEASURES OF EFFECTIVENESS FOR LEVEL OF SERVICE DEFINITION

TYPE OF FACILITY	MEASURE OF EFFECTIVENESS
Freeways	
Basic freeway segments	Density (pc/mi/ln)
Weaving areas	Average travel speed (mph)
Ramp junctions	Flow rates (pcph)
Multilane Highways	Density (pc/mi/ln)
Two-Lane Highways	Percent time delay(%) Average travel speed (mph)
Signalized Intersections	Average individual stopped delay (sec/veh)
Unsignalized Intersections	Reserve capacity (pcph)
Arterials	Average travel speed (mph)
Transit	Load factor (pers/seat)
Pedestrians	Space (sq ft/ped)

Reference is made to the Highway Capacity Manual Special Report 209, 1985, for a thorough discussion on capacity and level of service concepts

2-04 GENERAL DESIGN CONTROLS FOR HIGHWAYS

2-04.1 General

The values indicated in the tables that follow are intended to be applied as general design controls for the various classifications of roadways utilized in the State. The use of more liberal values than the values set out herein is recommended where it is economically feasible. In special cases of restrictive or unusual conditions, it may not be practical to meet even these guide values. For more detailed descriptions of the various guide values indicated, reference is made to other Sections of this Manual.

2-04.2 Urban Streets and Highways

Table 2-3 shows the guide values for general design controls to be used in the design of urban streets and highways.

2-04.3 Rural Roads and Highways

Table 2-4 shows the guide values for general design controls to be used in the design of rural roads and highways.

2-04.4 Freeways

Table 2-5 shows the guide values for general design controls to be used in the design of freeways. Generally, the design controls for freeways in rural areas and freeways in urban areas do not differ significantly.

Table 2-3

General Design Controls for Urban Streets and Highways

Highway Type	Arterial	Collector	Local
Design Speed, MPH	Urban 25-45 Suburban 25-60	25-35	15-30
Level of Service	C	C	---
Design Vehicle	SU; WB-50	SU; WB-50	SU
Design-Hourly Volume, DHV	As Furnished	As Furnished	---
Number of Lanes	2 to 6	2 to 4	2
Lane Width, Feet	12 Desirable 11 Minimum	12 Desirable 10 Minimum	10-12
Median Width (If Applicable)	4' Minimum	---	---
Shoulder Width, Feet	10 Right 4 Left	Usually Curbed	Usually Curbed
Max. Superelevation, Percent	6	4	---
Curve Radius	Fig. 4-G	Fig. 4-G	Fig. 4-G
Stopping Sight Distance	Table 4-1	Table 4-1	Table 4-1
Passing Sight Distance	Table 4-1	Table 4-1	Table 4-1
Profile Grade, %	Table 9-3	4 Level 12 Rolling/Mt.	15 Max.

Table 2-4

General Design Controls for Rural Roads and Highways

Highway Type	Arterial	Collector	Local
Design Speed, MPH	Level 50-60 Rolling 45-60 Mount. 25-45	35-50	20-35
Level of Service	C	C	---
Design Vehicle	SU; WB-50	SU; WB-50	SU
Design-Hourly Volume, DHV	As Furnished	As Furnished	---
Number of Lanes	2 to 6	2 to 4	2
Lane Width, Feet	12 Desirable 11 Minimum	12 Desirable 10 Minimum	10-12
Median Width (If Applicable), Feet	22 Desirable 4 Minimum	22 Desirable 4 Minimum	---
Shoulder Width, Feet	10 Right 4 Left	10 Right 4 Left	8
Max. Superelevation, Percent	8	8	---
Curve Radius	Fig. 4-G	Fig. 4-G	Fig. 4-G
Stopping Sight Distance	Table 4-1	Table 4-1	Table 4-1
Passing Sight Distance	Table 4-1	Table 4-1	Table 4-1
Profile Grade, %	Table 9-1	Table 8-1	15 Max.

Table 2-5

General Design Controls for Freeways

Location	Urban	Rural
Design Speed, MPH	55-80	55-80
Level of Service	C	B
Design Vehicle	SU; WB-50	SU; WB-50
Design-Hourly Volume, DHV	As Furnished	As Furnished
Number of Lanes	4 to 8	4 to 6
Lane Width, Feet	12	12
Median Width (If Applicable)	22' Minimum 46' Desirable	30' Minimum 60' Desirable
Shoulder Width, Feet	10 Right 4-10 Left	10 Right 4-10 Left
Max. Superelevation, %	10	10
Curve Radius	Fig. 4-G	Fig. 4-G
Stopping Sight Distance	Table 4-1	Table 4-1
Passing Sight Distance	Table 4-1	Table 4-1
Profile Grade, %	Table 10-1	Table 10-1

SECTION 3.

DEFINITIONS AND TERMINOLOGY

3-01 GENERAL

This section includes general terminology associated with the road cross section and terms commonly used in highway design. Reference is made to "AASHTO Highway Definitions", dated 1968, and to State DOT "Hawaii Standard Specifications for Road and Bridge Construction", dated 1985.

3-02 CROSS SECTION TERMINOLOGY

The elements of the road cross section are illustrated in Figure 3-A and Figure 3-B and defined as follows:

(1) Highway, Street or Road

The area within the right-of-way lines.

(2) Highway Section

The portion of the highway included between top of slopes in cut and the toe of slopes in fill.

(3) Roadway

The portion of the highway for vehicular use included between the outside edges of the shoulders.

(4) Traveled Way

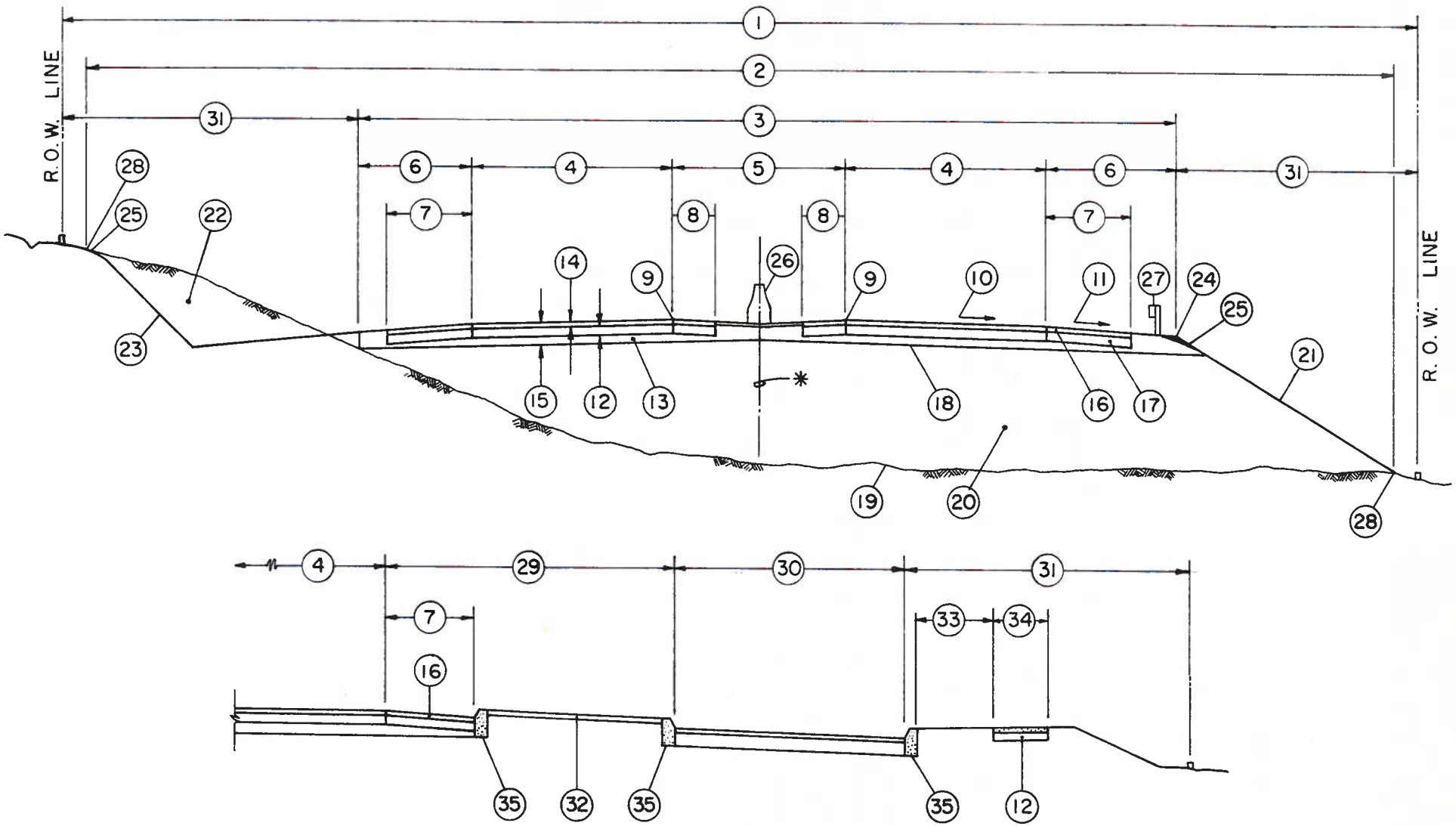
The portion of the roadway provided for the movement of vehicles, exclusive of shoulders and auxiliary lanes.

(5) Median

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

(6) Shoulder

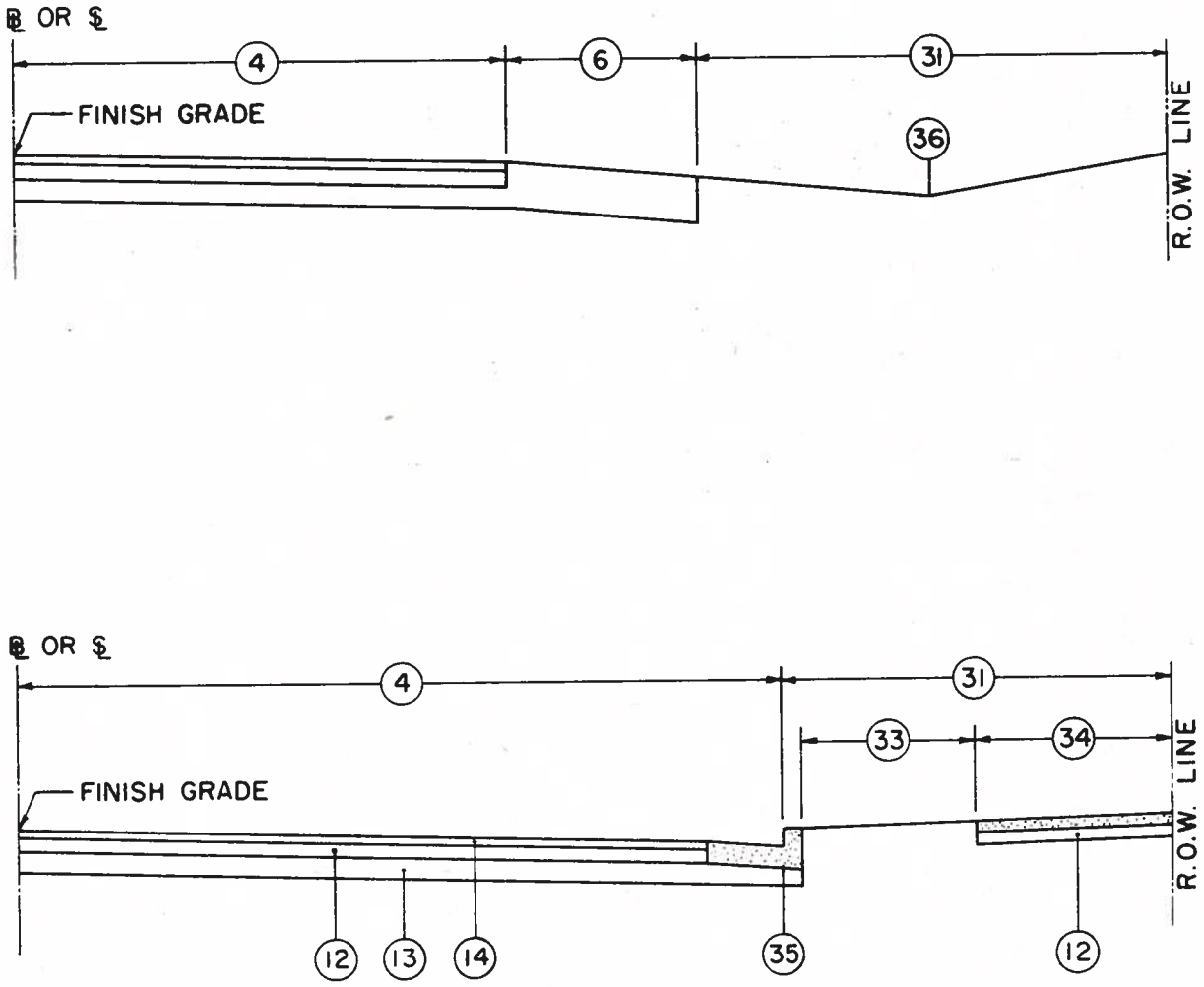
The portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles for emergency use, and for lateral support of the base and surface courses. The graded shoulder width is the distance from the edge of the traveled way to the intersection of the shoulder slope and the ditch or fill slope planes.



* BASELINE OR STATION LINE

ROAD CROSS SECTION TERMINOLOGY

FIGURE 3-A



ROAD CROSS SECTION TERMINOLOGY

FIGURE 3-B

The usable width of shoulder is the actual width that can be used when a driver makes an emergency or parking stop. Where the sideslope is 4:1 or flatter, the usable width is the same as the graded width since the usual rounding 4 to 6 ft. (1.22m to 1.83m) wide at the shoulder break will not lessen its useful width appreciably.

(7) Surfaced Right Shoulder

That portion of the right (or outside) shoulder surfaced or paved to provide all-weather load support.

(8) Surfaced Left Shoulder

The portion of the median side shoulder surfaced or paved to provide all-weather load support.

(9) Profile Grade Line

The point for control of the vertical alignment. Also, normally the point of rotation for superelevated sections.

(10) Pavement Cross Slope

See typical cross sections.

(11) Shoulder Cross Slope

See typical cross sections.

(12) Base Course

The layer or layers of specified or selected material of designed thickness placed on a subbase or a subgrade to support a surface course.

(13) Subbase Courses

The layer or layers of specified or selected material placed on a subgrade to support a base course.

(14) Surface Course

One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate.

(15) Pavement Structures

The combination of subbase, base course, and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed.

(16) Shoulder Surface Course

(17) Shoulder Base Course

(18) Subgrade

The top surface of the roadbed upon which the pavement structure and shoulders are constructed.

(19) Original (Existing) Ground

(20) Embankment (Fill)

(21) Fill Slope

(22) Cut Section

(23) Cut Slope

Also called cut face.

(24) Hinge Point

The intersection of shoulder slope planes with fill slope planes.

(25) Rounding

At the intersection of existing ground and cut slope and at intersection of shoulder slope planes with fill slope > 4:1.

(26) Median Barrier

(27) Guard Rail

(28) Catch Point

The intersection of the cut or fill slope and the original ground.

(29) Outer Separation

The portion of an arterial highway between the traveled ways of a roadway for through traffic and a frontage street or road.

(30) Frontage Road or Street

Also called marginal road or street. A local road or street auxiliary to and located on the side of an arterial highway for service to abutting property and adjacent areas and for control of access.

(31) Roadside

The area that separates the roadway from adjacent properties.

(32) Outer Separation Island

The space in the outer edge of roadway shoulder and frontage roadway shoulder and frontage road or street which may be landscaped or paved depending on width.

(33) Planting Strip

The space in the border area provided to separate the sidewalk from the vehicular travel facilities.

(34) Sidewalk

(35) Curb or Curb and Gutter

(36) Drainage Swale

Where required.

3-03 GENERAL TERMS

- Auxiliary Lane

  The portion of the roadway adjoining the traveled way for parking, speed change, or for other purposes supplementary to through traffic movement.

- Parking Lane

  An auxiliary lane primarily for the parking of vehicles.

- Speed Change Lane

  An auxiliary lane, including tapered areas, primarily for the acceleration or deceleration of vehicles entering or leaving the through traffic lanes.

- Intersection

  The general area where two or more highways join or cross, within which are included the roadway and roadside facilities for traffic movements in that area.

- Interchange

  A system of interconnecting roadways in conjunction with a grade separation or grade separations providing for the interchange of traffic between two or more roadways on different levels.

SECTION 4.

BASIC GEOMETRIC DESIGN ELEMENTS

4-01 GENERAL

Geometric highway design pertains to the visible features of the highway. It may be considered as the tailoring of the highway to the terrain, to the controls of the land usage, and to the type of traffic anticipated.

Design parameters covering highway types, design vehicles, and traffic data are included in Section 2, DESIGN PARAMETERS.

This section covers design criteria and guidelines on the geometric design elements that must be considered in the location and the design of the various types of highways. Included are criteria and guidelines on sight distances, horizontal and vertical alignment, major cross section elements, and other features common to the several types of roadways and highways.

In applying these criteria and guidelines, it is important to follow the basic principle that consistency in design standards is of major importance on any section of road. The highway should offer no surprises to the driver in terms of geometrics. Problem locations are generally at the point where minimum design standards are introduced on a section of highway where otherwise higher standards should have been applied. The ideal highway design is one with uniformly high standards applied consistently along a section of highway, particularly on major highways designed to serve large volumes of traffic at high operating speeds.

4-02 SIGHT DISTANCES Primary concern, all roads must have min. S.D.

4-02.1 General

Sight distance is the continuous length of highway ahead visible to the driver. In design, two sight distances are considered: passing sight distance and stopping sight distance. Stopping sight distance is the minimum sight distance to be provided at all points on multi-lane highways and on two-lane roads when passing sight distance is not economically obtainable. Stopping sight distance also is to be provided for all elements of interchanges and intersections at grade, including private road connections.

The following table shows the standards for passing and stopping sight distance related to design speed.

4-02.2 Passing Sight Distance

Passing sight distance is the minimum sight distance that must be available to enable the driver of one vehicle to pass another vehicle safely and comfortably, without interfering with the speed of an oncoming vehicle traveling at the design speed, should it come into view after the overtaking maneuver is started. The sight distance available for passing at any place is the longest distance at which a driver whose eyes are 3.50 feet (1.07 m) above the pavement surface can see the top of an object 4.25 feet (1.30 m) high on the road. However, this manual will adopt California's criteria and maintain the 4.0 foot (1.22 m) object height with the driver's eyes considered to be 3.50 feet (1.07 m) above the road surface in calculations for passing sight distance.

Passing sight distance is considered only on two-lane roads. At critical locations, a stretch of four-lane construction with stopping sight distance is sometimes more economical than two lanes with passing sight distance.

The curves shown on Figure 4-A and Figure 4-B give the length of vertical curve which satisfies a given sight distance when the algebraic difference in grades is known.

4-02.3 Stopping Sight Distance

The minimum stopping sight distance is the distance required by the driver of a vehicle, traveling at a given speed, to bring his vehicle to a stop after an object on the road becomes visible. Stopping sight distance is measured from the driver's eyes, which are assumed to be 3.50 feet (1.07 m) above the pavement surface, to an object 0.5 foot (0.15 m) high on the road.

Table 4-1
Minimum Passing Sight Distance for Design of Two Lane Highways

Design Speed (mph)	Assumed Speeds		Minimum Passing Sight Distance (ft)
	Passed Vehicle (mph)	Passing Vehicle (mph)	
20	20	30	800
30	26	36	1,100
40	34	44	1,500
50	41	51	1,800
60	47	57	2,100
65	50	60	2,300
70	54	64	2,500

Table 4-2
Stopping Sight Distance
(Wet Pavement)

Design Speed (mph)	Assumed Speed for Condition (mph)	Brake Reaction		Coefficient of Friction f	Braking Distance on Level[a] (ft)	Stopping Sight Distance	
		Time (sec)	Distance (ft)			Computed[a] (ft)	Rounded for Design (ft)
20	20-20	2.5	73.3- 73.3	0.40	33.3- 33.3	106.7-106.7	125-125
25	24-25	2.5	88.0- 91.7	0.38	50.5- 54.8	138.5-146.5	150-150
30	28-30	2.5	102.7-110.0	0.35	74.7- 85.7	177.3-195.7	200-200
35	32-35	2.5	117.3-128.3	0.34	100.4-120.1	217.7-248.4	225-250
40	36-40	2.5	132.0-146.7	0.32	135.0-166.7	267.0-313.3	275-325
45	40-45	2.5	146.7-165.0	0.31	172.0-217.7	318.7-382.7	325-400
50	44-50	2.5	161.3-183.3	0.30	215.1-277.8	376.4-461.1	400-475
55	48-55	2.5	176.0-201.7	0.30	256.0-336.1	432.0-537.8	450-550
60	52-60	2.5	190.7-220.0	0.29	310.8-413.8	501.5-633.8	525-650
65	55-65	2.5	201.7-238.3	0.29	347.7-485.6	549.4-724.0	550-725
70	58-70	2.5	212.7-256.7	0.28	400.5-583.3	613.1-840.0	625-850

[a]Different values for the same speed result from using unequal coefficients of friction.

The stopping sight distances in Table 4-2 should be increased by 18 percent on sustained downgrades steeper than 3 percent and longer than 1 mile.

4-02.4 Stopping Sight Distance at Grade Crests

Figure 4-C gives the length of vertical curve required to obtain stopping sight distance for a given design speed when the algebraic difference in grades is known.

4-02.5 Stopping Sight Distance at Grade Sags

From the curves in Figure 4-D, the length of vertical curve which provides headlight sight distance in grade sags for a given design speed is obtained if the algebraic difference in grade rates is known. This is the sight distance used at underpasses where the highway profile dips under the crossing facility. Where terrain features and funding permit, longer length of vertical curve should be provided.

4-02.6 Stopping Sight Distance on Horizontal Curves

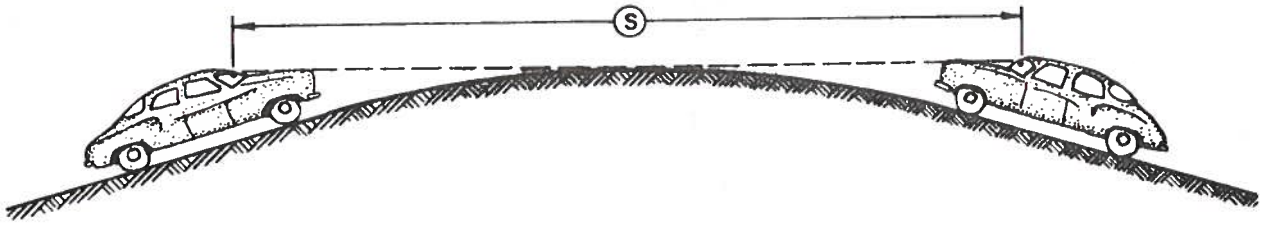
Where an object off the pavement such as a bridge pier, building, cut slope, or natural growth restricts sight distance, the minimum radius of curvature is determined by the stopping sight distance.

Stopping sight distance on horizontal curves is obtained from Figure 4-E. It is assumed that the driver's eyes are 3.50 feet (1.07 m) above the center of the inside lane (inside with respect to curve) and the object is 0.50 foot (0.15 m) high. The line of sight is assumed to intercept the view obstruction at the midpoint of the sight line and 2 feet (0.62 m) above the center of the inside lane. The clear distance (m) is measured from the center of the inside lane to the obstruction.

The general problem is to determine the clear distance from centerline of inside lane to a retaining wall, bridge pier, abutment, cut slope, or other obstruction for a given design speed. Using radius of curvature and sight distance for the design speed, Figure 4-E gives the clear distance (m) from centerline of inside lane to the obstruction.

When the design speed and the clear distance to a fixed obstruction are known, this figure also gives the required minimum radius which satisfies these conditions.

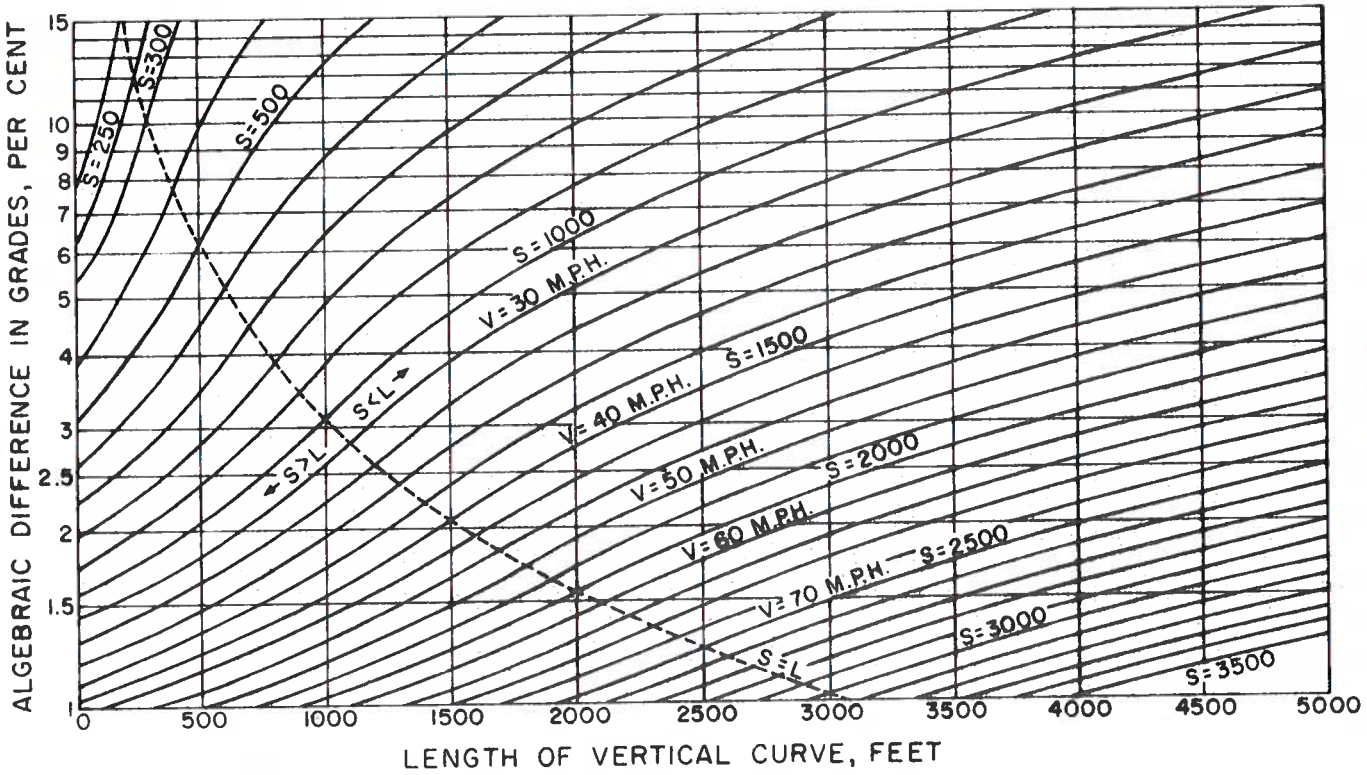
HEIGHT OF EYE 3.50 FEET
HEIGHT OF OBJECT 4.25 FEET



L = CURVE LENGTH - FT.
A = ALGEBRAIC GRADE DIFFERENCE - %
S = SIGHT DISTANCE - FT.
V = DESIGN SPEED - M.P.H. FOR "S"

WHEN S > L	WHEN S < L
$L = 2S - \frac{3100}{A}$	$L = \frac{AS^2}{3100}$

DESIGN SPEED M.P.H.	SIGHT DISTANCE FEET
20	800
30	1100
40	1500
50	1800
60	2100
70	2500



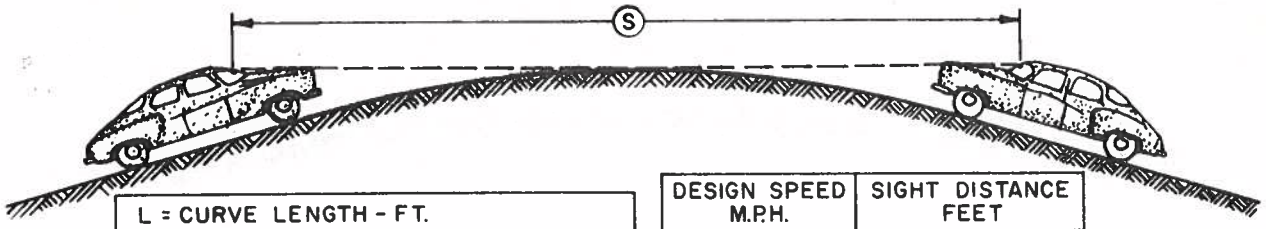
PASSING SIGHT DISTANCE ON CREST VERTICAL CURVES

FIGURE 4-A

SOURCE: CALTRANS HIGHWAY DESIGN MANUAL

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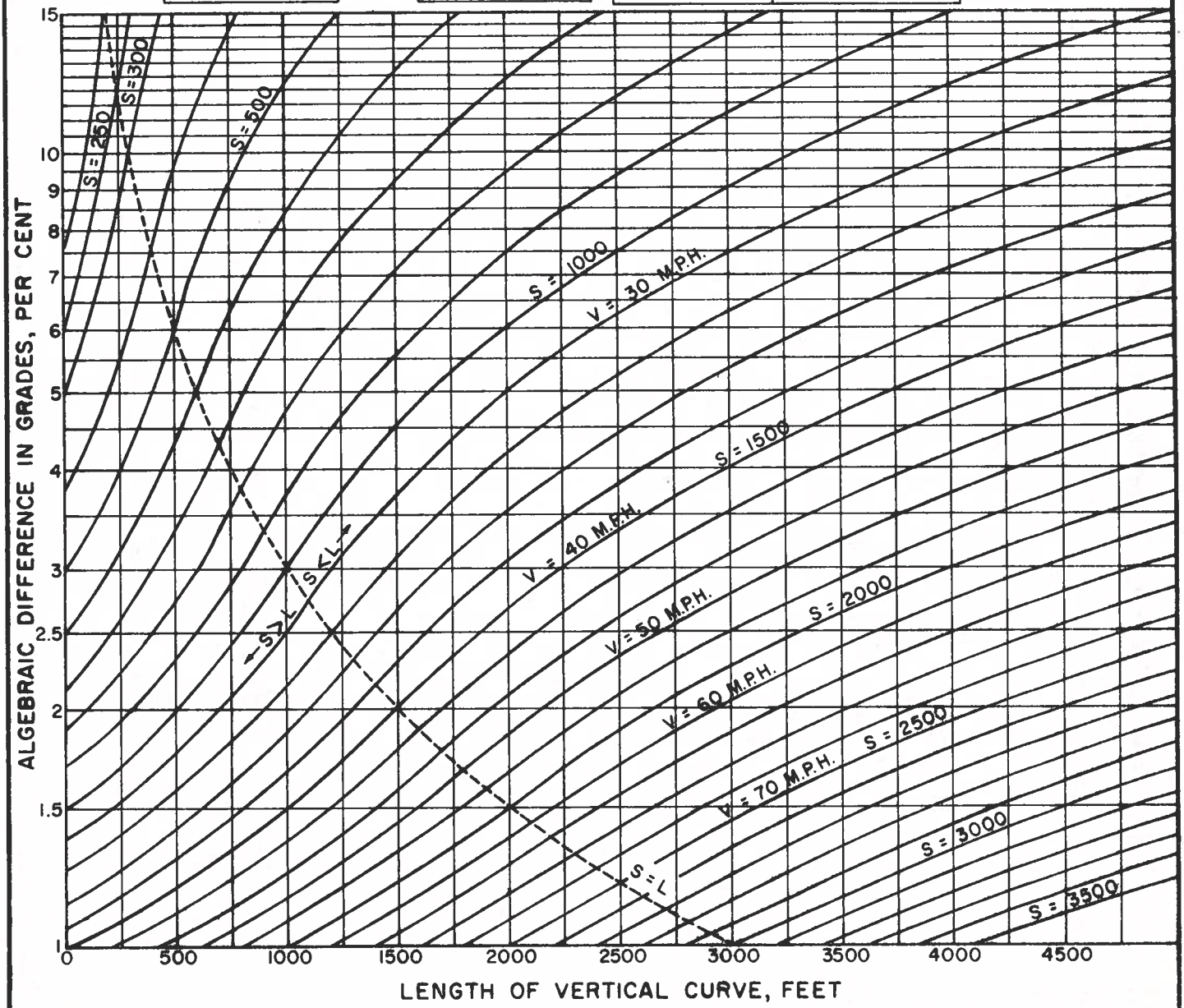
HEIGHT OF EYE 3.50 FEET
HEIGHT OF OBJECT 4.00 FEET



L = CURVE LENGTH - FT.
A = ALGEBRAIC GRADE DIFFERENCE - %
S = SIGHT DISTANCE - FT.
V = DESIGN SPEED - M.P.H. FOR "S"

WHEN S > L	WHEN S < L
$L = 2S - \frac{2997}{A}$	$L = \frac{AS^2}{2997}$

DESIGN SPEED M.P.H.	SIGHT DISTANCE FEET
20	800
30	1100
40	1500
50	1800
60	2100
70	2500

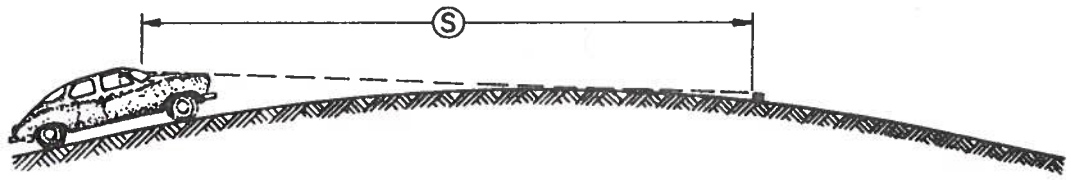


PASSING SIGHT DISTANCE ON CREST VERTICAL CURVES

FIGURE 4-B

r5/15/86

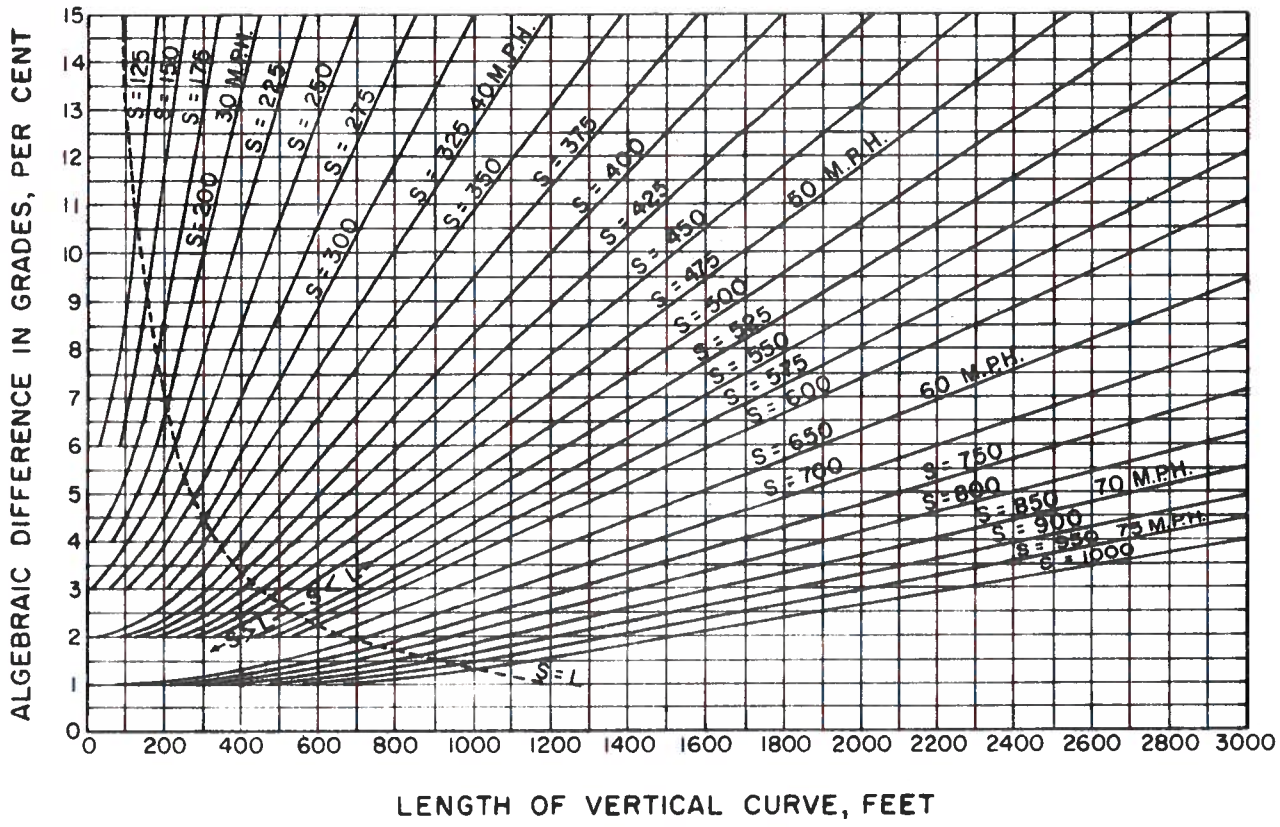
HEIGHT OF EYE 3.50 FEET
HEIGHT OF OBJECT 0.50 FEET



L = CURVE LENGTH - FT.
A = ALGEBRAIC GRADE DIFFERENCE - %
S = SIGHT DISTANCE - FT.
V = DESIGN SPEED - M.P.H. FOR "S"

WHEN S > L	WHEN S < L
$L = 2S - \frac{1329}{A}$	$L = \frac{AS^2}{1329}$

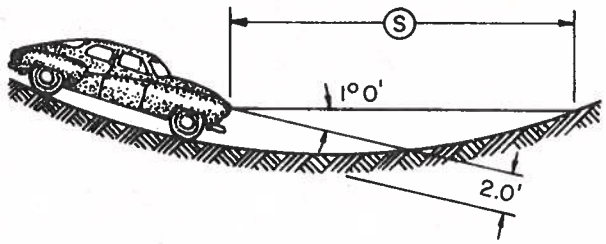
DESIGN SPEED M.P.H.	SIGHT DISTANCE
20	125 FT.
30	200 FT.
40	325 FT.
50	475 FT.
60	650 FT.
65	725 FT.
70	850 FT.



STOPPING SIGHT DISTANCE ON CREST VERTICAL CURVES

FIGURE 4-C

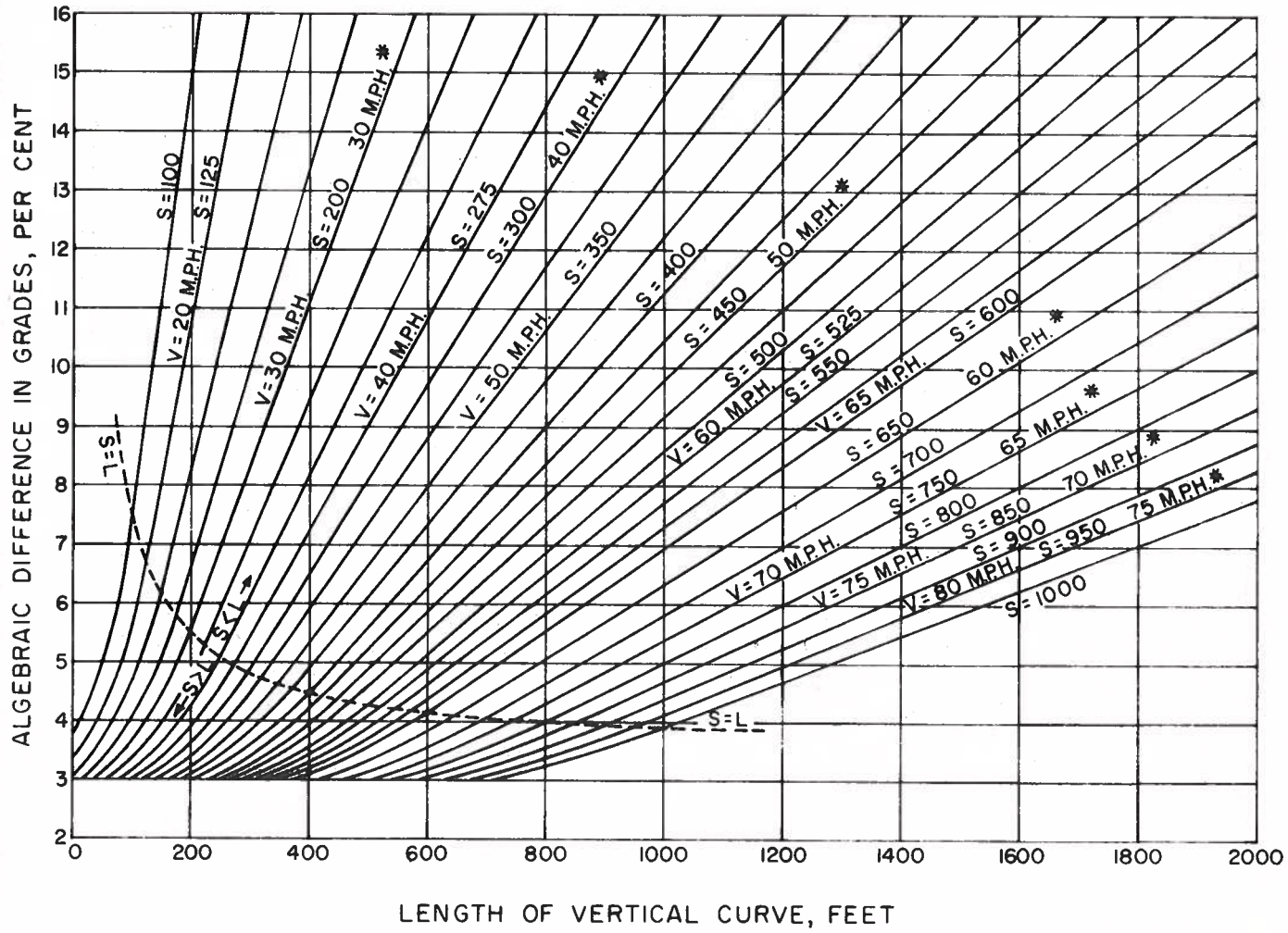
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L = CURVE LENGTH - FT.
A = ALGEBRAIC GRADE DIFFERENCE - %
S = SIGHT DISTANCE - FT.
V = DESIGN SPEED - M.P.H. FOR "S"

WHEN S > L	WHEN S < L
$L = 2S - \frac{400 + 3.5S}{A}$	$L = \frac{AS^2}{400 + 3.5S}$

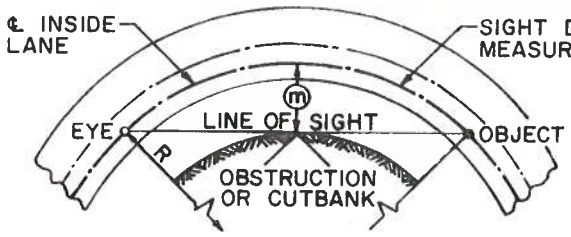
DESIGN SPEED M.P.H.	SIGHT DISTANCE
20	125 FT.
30	200 FT.
40	325 FT.
50	475 FT.
60	650 FT.
65	725 FT.
70	850 FT.



STOPPING SIGHT DISTANCE ON SAG VERTICAL CURVES

FIGURE 4-D

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HEIGHT OF EYE - 3.50 FEET
HEIGHT OF OBJECT - 0.50 FEET

LINE OF SIGHT IS 2.0 FEET ABOVE ℄ INSIDE LANE AT POINT OF OBSTRUCTION

S = SIGHT DISTANCE IN FEET
R = RADIUS OF ℄ INSIDE LANE IN FEET
m = DISTANCE FROM ℄ INSIDE LANE IN FEET
V = DESIGN SPEED FOR S IN M.P.H.

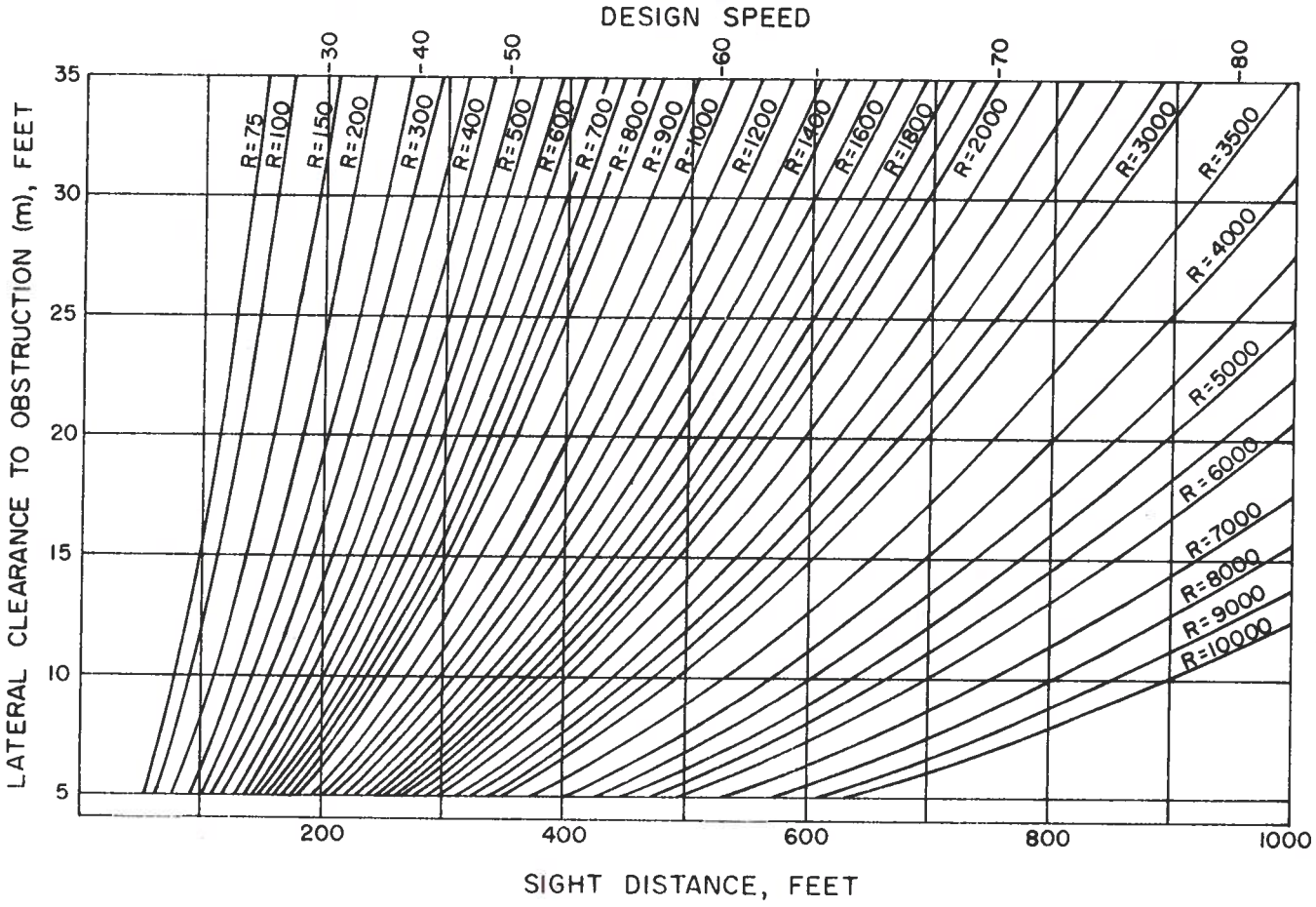
ANGLE IS EXPRESSED IN DEGREES

$$m = R\left[\text{VERS}\left(\frac{28.65S}{R}\right)\right]$$

$$S = \frac{R}{28.65}\left[\cos^{-1}\left(\frac{R-m}{R}\right)\right]$$

FORMULA APPLIES ONLY WHEN S IS EQUAL TO OR LESS THAN LENGTH OF CURVE.

DESIGN SPEED M.P.H.	SIGHT DISTANCE
20	125 FT.
30	200 FT.
40	325 FT.
50	475 FT.
60	650 FT.
65	725 FT.
70	850 FT.



STOPPING SIGHT DISTANCE ON HORIZONTAL CURVES

FIGURE 4-E

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4-03 HORIZONTAL ALIGNMENT

4-03.1 General

In the design of horizontal curves, it is necessary to establish the proper relation between design speed and curvature, and also their joint relationship with superelevation. Horizontal alignment must afford at least the minimum stopping sight distance for the design speed at all points on the roadway.

The major considerations in horizontal alignment design are: safety, grade profile, type of facility, design speed, topography and construction cost. In design, safety is always considered, either directly or indirectly. Topography controls both curve radius and design speed to a large extent. The design speed, in turn, controls sight distance, but sight distance must be considered concurrently with topography because it often demands a larger radius

than the design speed. All these factors must be balanced to produce an alignment that is safest, most economical, in harmony with the natural contour of the land and, at the same time, adequate for the design classification of the roadway or highway.

4-03.2 Superelevation

1. Basic Criteria

According to the laws of mechanics, a vehicle traveling on a curve exerts an outward force called centrifugal force.

On a superelevated highway, this force is resisted by the vehicle weight component parallel to the superelevated surface and side friction between the tires and pavement. It is impractical to balance centrifugal force by superelevation alone, because for any given curve radius, a certain superelevation rate is exactly correct for only one driving speed. At all other speeds there will be a side thrust, either outward or inward, relative to the curve center, which must be offset by side friction.

If the vehicle is not skidding, these forces are in equilibrium as represented by the following equation:

Where S = Superelevation slope in feet per foot
F = Side friction factor
R = Radius of curve in feet
V = Velocity in miles per hour

$$\text{Centrifugal Factor} = S+F = \frac{0.067V^2}{R}$$

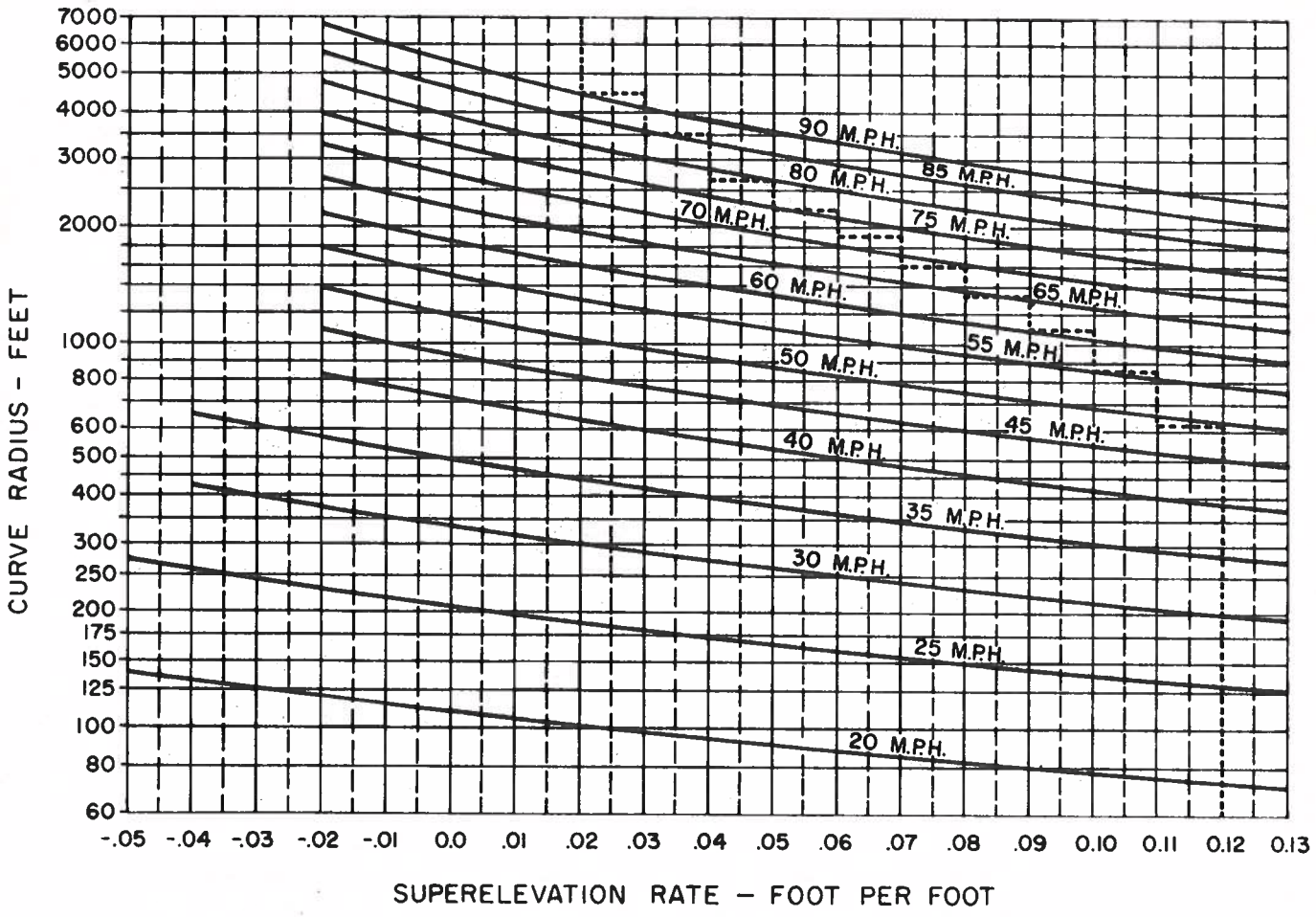
This formula is used to design a curve for comfortable operation at a particular speed.

Standard superelevation rates are designed to hold the portion of the centrifugal force that must be taken up by tire friction within allowable limits. Friction factors as related to speed are shown in Figure 4-F. These factors shall apply equally to portland cement concrete and bituminous pavement.

Comfortable speeds for various combinations of curvature and superelevation are given in Figure 4-F.

2. Guideline for Superelevation

The values of maximum superelevation rate and assumptions for maximum side friction factor in combination determine the



SPEED	*FRICTION FACTOR
20	0.17
30	0.16
40	0.15
50	0.14
60	0.12
70	0.10
80	0.08

*THESE FACTORS ARE THE VALUES WHICH PROVIDE A REASONABLE MARGIN OF SAFETY AT HIGH SPEEDS.

S = SUPERELEVATION
F = FRICTION FACTOR
V = SPEED IN MILES PER HOUR
R = RADIUS IN FEET

$$S + F = \frac{0.067V^2}{R}$$

NOTE

BROKEN LINE INDICATES STANDARD SUPERELEVATION RATE. HIGHER VALUE AT STEPS IS THE PROPER SUPERELEVATION FOR INDICATED CURVE RADIUS.

FIGURE 4-F

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maximum curvature. The maximum usable superelevation rates on streets are controlled by terrain, type of area, climatic conditions, and frequency of very slow moving vehicles that would be subject to unstable operation. The values of curve superelevation employed in road design vary from 0.02 up to 0.12 foot per foot of width. The latter figure is only used for high speed roadways. Superelevation rate higher than 0.10 should be considered as a design exception.

Values lower than usually employed in design are commonly used where pavement surfaces must be warped to meet existing street elevations and at intersections or weaving sections where vehicles follow reverse-curve paths.

In the design of horizontal curves, it is necessary to establish the proper relationships between the design speed and curvature and also their joint relationship with superelevation. The actual values used in design depend upon practical limits and factors determined over the range of variables involved.

Superelevation rates between 0.02 and 0.08 are generally acceptable. Higher rates of superelevation may be justified and must be approved by the agency having jurisdiction of the roadway under design. A superelevation rate of 0.02 foot per foot is considered to be the practical minimum for drainage.

Normal cross sections on local streets are often carried through horizontal curves because of drainage requirements, frequency of intersecting streets, etc. Each case must be decided on its own merits relative to advisory or regulatory signing. However, where design speeds are in the range of 20 to 25 MPH, an adverse or level superelevation generally can be tolerated because of actual development of side friction factors much higher than the design values.

Superelevation slopes on curves extend the full width of the shoulders, except that the shoulder slope on the low side should be not less than the minimum shoulder slope used on the tangents.

On rural two-lane roads, superelevation must remain a plane for the full width of traveled way, except on transitions.

3. Snow and Ice Conditions

In some locations, usually above 8,000 feet, where snow and ice conditions may occur, it may be necessary to reduce the

maximum superelevation rate to 0.08 foot per foot. Reduction of the standard superelevation rates on curves which require 0.08 foot per foot, or less, is not acceptable.

4. Urban Street Conditions

Lower superelevation rates may be necessary in urban areas where restricted speed zones or street intersections are controlling factors. In addition, established street grades, curbs, or drainage may prove difficult to alter. Such conditions may warrant, for example, a reduction in the superelevation rate, different rates for each half of the roadbed, or both. In warping street areas for drainage, adverse superelevation should be avoided.

5. Axis of Rotation

a. Undivided Highways

For undivided highways, the axis of rotation for superelevation is usually the centerline of the roadbed. However, in special cases where curves are preceded by long, relatively level tangents, the plane of superelevation may be rotated about the inside edge of the pavement to improve perception of the curve. In flat terrain, drainage pockets caused by superelevation may be avoided by changing the axis of rotation from the centerline to the inside edge of the pavement.

b. Ramps and Freeway to Freeway Connections

The axis of rotation may be about either edge of pavement or centerline if multi-lane. Appearance and drainage considerations should always be taken into account in selection of the axis of rotation.

c. Divided Highways

1) Freeways

Where the initial median width is 30 feet (9.15 m) or less, the axis of rotation should be at the centerline of median.

Where the initial median width is greater than 30 feet (9.15 m) and the ultimate median width is 30 feet (9.15 m) or less, the axis of rotation should be at the centerline of median, except where the

resulting initial median slope would be steeper than 10:1. In the latter case, the axis of rotation should be at the ultimate median edges of pavement.

Where the ultimate median width is greater than 30 feet, (9.15 m) the axis of rotation should be at the ultimate median edges of pavement.

To avoid sawtooth on bridges with decked medians, the axis of rotation, if not already on centerline of median, should be shifted to the centerline of median.

2) Other Divided Highways

The axis of rotation should be considered on an individual project basis and the most appropriate case for the conditions should be selected.

The selection of the axis of rotation should always be considered in conjunction with the design of the profile and superelevation transition.

6. Superelevation Transition

The superelevation transition generally consists of the superelevation runoff (length of roadway needed to accomplish the change in cross slope from a horizontal line to a fully superelevated section or vice versa), the crown runoff, and the remaining 50 feet (15.25 m) of the vertical curve as shown in Figure 4-G

A superelevation transition should be designed to satisfy the requirements of safety and comfort and be pleasing in appearance. The length of superelevation transition shall be based upon the combination of superelevation rate and width of rotated plane.

With respect to the beginning or ending of a curve, two-thirds of the superelevation runoff is on the tangent approach and one-third within the curve. This results in two-thirds of the full superelevation rate at the beginning or ending of a curve. This may be altered as required to adjust for flat spots or unsightly sags and humps when alignment is tight.

Superelevation transitions should be designed in accordance with the data shown in Figure 4-G.

FORMULA		EXPLANATION OF TERMS
2 LANE ROADS	L = 2500 S	L = LENGTH OF SUPERELEVATION RUNOFF – FT.
MULTILANE ROADS & BRANCH CONNECTIONS	L = 150 DS	S = SUPERELEVATION RATE – FT./FT.
RAMPS		
MULTILANE	L = 2500 S IF POSSIBLE	D = DISTANCE FROM AXIS OF ROTATION TO OUTSIDE EDGE OF LANES – FT.
SINGLELANE	L = 2000 S	
MINIMUM	L = 150 FT.	MAXIMUM L = 510 FT.

ADJUST COMPUTED LENGTH TO NEAREST 10 FT. LENGTH DIVISIBLE BY 3

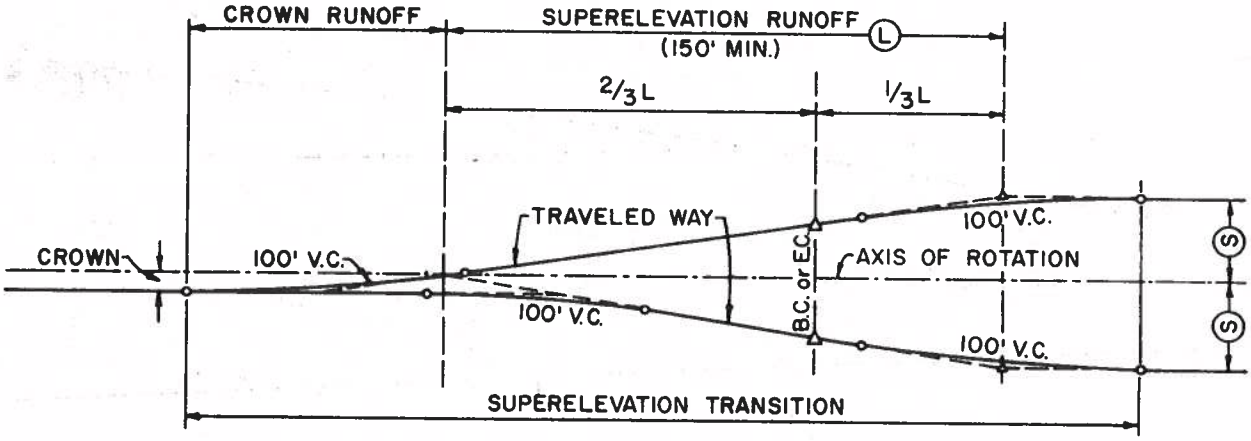


TABLE OF SUPERELEVATION RUNOFF LENGTHS

SUPERELEVATION RATE "S" FOOT/FOOT	LENGTH L – FEET								
	2-LANE & MULTI-LANE RAMPS	SINGLE LANE RAMPS	MULTILANE HIGHWAYS AND BRANCH CONNECTIONS WITH VARIOUS "D" WIDTHS						
			24 FT.	36 FT.	48 FT.	51 FT.	60 FT.	63 FT.	75 FT.
0.02	150	150	150	150	150	150	180	180	240
0.03	150	150	150	180	210	240	270	270	330
0.04	150	150	150	210	300	300	360	390	450
0.05	150	150	180	270	360	390	450	480	510
0.06	150	150	210	330	450	450	510	510	
0.07	180	150	270	390	510	510			
0.08	210	150	300	450					
0.09	240	180	330	480					
0.10	240	210	360	510					
0.11	270	210	390						
0.12	300	240	420						

FOR WIDTHS OF "D" NOT INCLUDED IN TABLE, USE FORMULA

SUPERELEVATION TRANSITION

SOURCE: CALTRANS HIGHWAY DESIGN MANUAL

FIGURE 4-G

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In special restrictive situations where the standard superelevation rate is not feasible, or a given transition length is not attainable, the highest possible rate and longest length, respectively, should be used. These situations occur on interchange ramps, frontage roads, and separate turning lanes where the length of radius, curve and adjacent tangents are short.

Superelevation transitions for shoulders cannot be indicated properly on the same superelevation transition diagram with the traveled way for the type of diagram shown in the Manual. The shoulder plane rotates about the adjacent edge of pavement as well as the rotational axis of the traveled way. Shoulder superelevation transitions should be smooth and compatible with the transition of the adjacent pavements.

After a superelevation transition is designed, profiles of edges of pavement and shoulder should be plotted and irregularities removed by introducing smooth curves. Flat areas which are undesirable from a drainage standpoint should be avoided.

Pronounced and unsightly sags may develop on the low side of the superelevation. These can be corrected by adjusting the grades on the two edges of pavement throughout the curve.

7. Superelevation of Compound Curves

Superelevation of compound curves follows the procedure shown in Figure 4-H. Where feasible, the criteria in paragraph 6. above apply.

4-03.3 Curvature

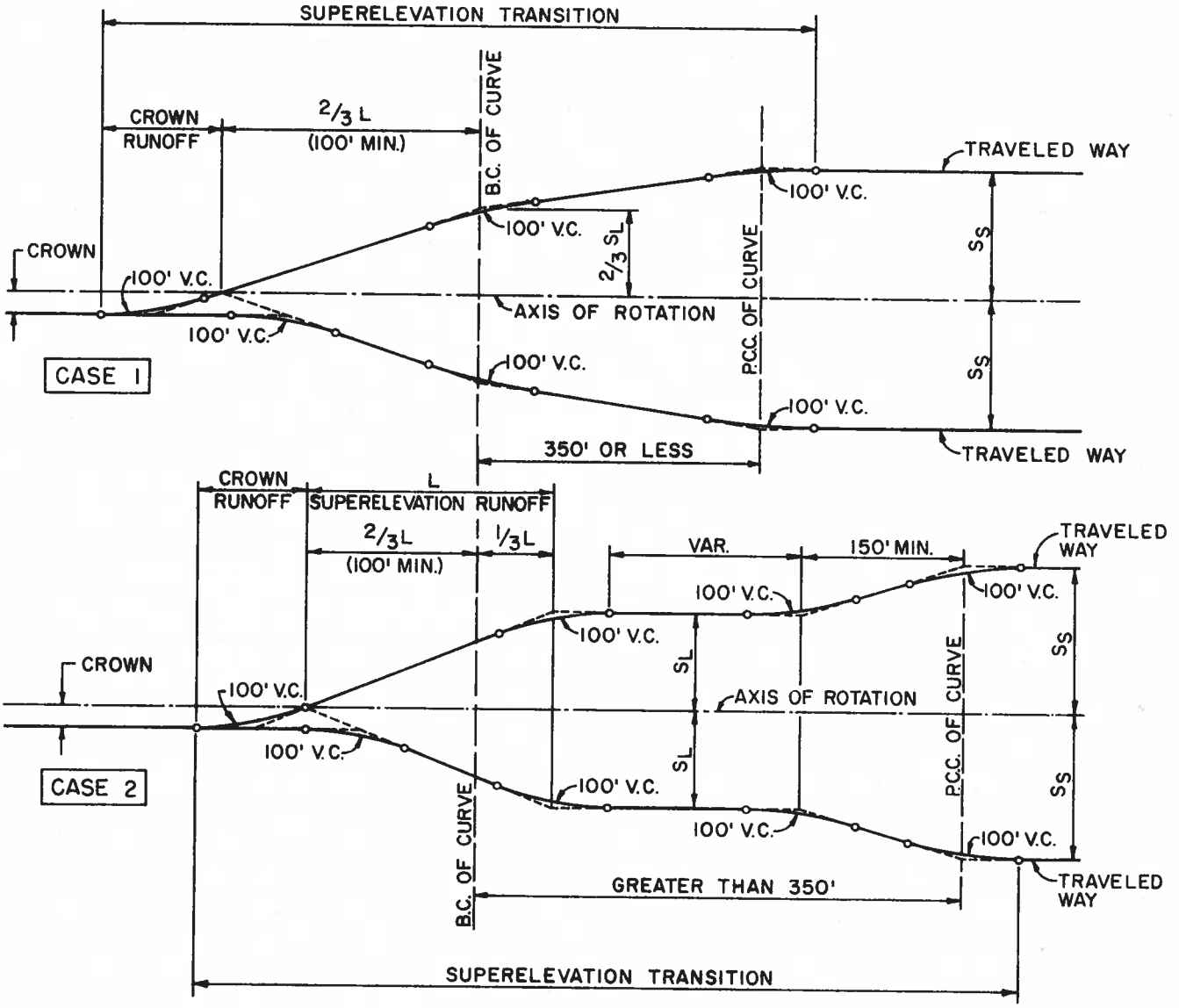
1. General

The changes in direction along a highway are basically accounted for by curves consisting of portions of a circle. Figure 4-I shows the interrelated elements of a circular curve.

2. Guidelines for Horizontal Curves

Following is a table which gives the minimum radius of curve for specific design speeds.

L = LENGTH OF SUPERELEVATION RUNOFF - FT.
S_S = SUPERELEVATION RATE FOR SMALLER RADIUS CURVE - FT./FT. OR PERCENT
S_L = SUPERELEVATION RATE FOR LARGER RADIUS CURVE - FT./FT. OR PERCENT

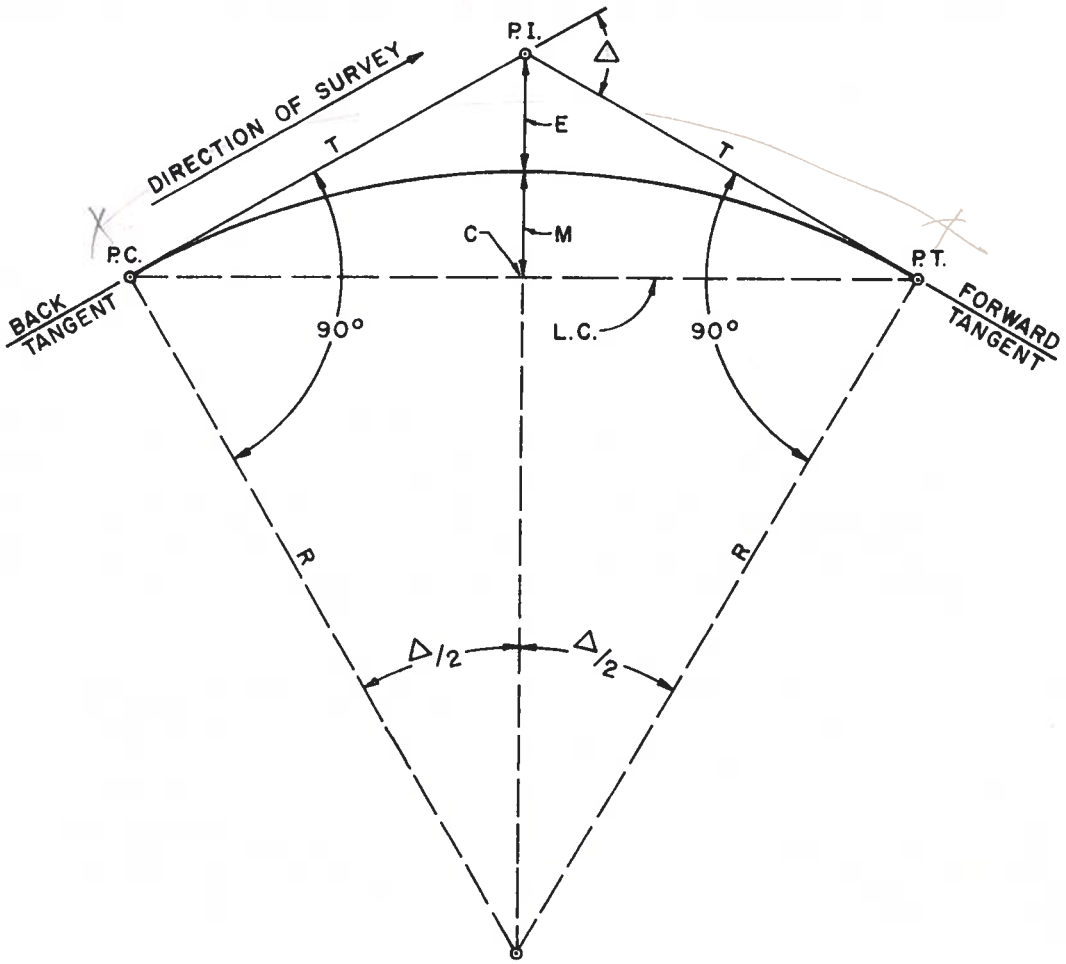


SUPERELEVATION OF COMPOUND CURVES

SOURCE: CALTRANS HIGHWAY DESIGN MANUAL

FIGURE 4-H

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GENERAL FORMULAS FOR ARC DEFINITION

$T = R \tan \frac{\Delta}{2}$

$D = \frac{5729.578}{R}$

$L.C. = 2R \sin \frac{\Delta}{2}$

$E = T \tan \frac{\Delta}{4}$

WHEN "R" IS KNOWN, $E = R \sec \frac{\Delta}{2} - R = R \operatorname{exsec} \frac{\Delta}{2}$

$M = E \cos \frac{\Delta}{2}$

WHEN "R" IS KNOWN, $M = R(1 - \cos \frac{\Delta}{2}) = R \operatorname{vers} \frac{\Delta}{2}$

LENGTH OF CURVE, $L = \frac{100\Delta}{D}$ WHEN Δ AND "D" ARE IN MINUTES

LOCATING THE P.C. AND P.T.

STA. P.C. = STA. P.I. - T

STA. P.T. = STA. P.C. + L

P.I. - POINT OF INTERSECTION

P.C. - POINT OF CURVATURE

P.T. - POINT OF TANGENCY

Δ - DEFLECTION ANGLE BETWEEN THE TANGENTS

T - TANGENT DISTANCE

E - EXTERNAL DISTANCE

R - RADIUS OF THE CIRCULAR ARC

M - MIDDLE ORDINATE

L.C. - LONG CHORD (DISTANCE BETWEEN P.C. AND P.T.)

C - MIDPOINT OF LONG CHORD

D - DEGREE OF CURVATURE

SIMPLE CURVE COMPUTATION (ARC DEFINITION)

FIGURE 4-I

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Every effort should be made to exceed the minimum values, and such minimum radii should be used only when the cost or other adverse effects of realizing a higher standard are inconsistent with the benefits.

The recommended minimum radii for freeways are 5000 feet in rural areas and 3000 feet in urban areas.

If a glare screen of any kind or a median barrier is contemplated, either initially or ultimately, adjustments may be necessary to maintain the required sight distance on curves on divided highways. In such cases a larger curve radius or a widened median may be required throughout the length of the curve. For design purposes, a planting screen is presumed to be 8 feet wide.

3. Alignment Consistency

Sudden reductions in standards introduce the element of surprise to the driver and should be avoided. Where physical restrictions on curve radius cannot be overcome and it becomes necessary to introduce curvature of lower standard than the design speed for the project, the design speed between successive curves shall change not more than 10 miles per hour. Introduction of a curve for a design speed lower than the design speed of the project shall be avoided at the end of a long tangent or at other locations where high approach speeds may be anticipated.

Table 4-3
Maximum Degree of Curve and Minimum Radius

Design Speed (mph)	Maximum e	Maximum f	Total (e + f)	Maximum Degree of Curve	Rounded Maximum Degree of Curve	Minimum Radius (ft)
20	.04	.17	.21	44.97	45.0	127
30	.04	.16	.20	19.04	19.0	302
40	.04	.15	.19	10.17	10.0	573
50	.04	.14	.18	6.17	6.0	955
60	.04	.12	.16	3.81	3.75	1,528
20	.06	.17	.23	49.25	49.25	116
30	.06	.16	.22	20.94	21.0	273
40	.06	.15	.21	11.24	11.25	509
50	.06	.14	.20	6.85	6.75	849
60	.06	.12	.18	4.28	4.25	1,348
65	.06	.11	.17	3.45	3.5	1,637
70	.06	.10	.16	2.80	2.75	2,083
20	.08	.17	.25	53.54	53.5	107
30	.08	.16	.24	22.84	22.75	252
40	.08	.15	.23	12.31	12.25	468
50	.08	.14	.22	7.54	7.5	764
60	.08	.12	.20	4.76	4.75	1,206
65	.08	.11	.19	3.85	3.75	1,528
70	.08	.10	.18	3.15	3.0	1,910
20	.10	.17	.27	57.82	58.0	99
30	.10	.16	.26	24.75	24.75	231
40	.10	.15	.25	13.38	13.25	432
50	.10	.14	.24	8.22	8.25	694
60	.10	.12	.22	5.23	5.25	1,091
65	.10	.11	.21	4.26	4.25	1,348
70	.10	.10	.20	3.50	3.5	1,637

NOTE: In recognition of safety considerations, use of e_{max} = 0.04 should be limited to urban conditions.

4. Stopping Sight Distance

Horizontal alignment should afford the stopping sight distance for the design speed shown on Figure 4-E at all points of the highway.

5. Curve Length and Central Angle

The following is applicable for freeways and rural arterial highways. The minimum curve length for central angles less than 10 degrees should be 800 feet to avoid the appearance of a kink. For central angles smaller than 30 minutes, no curve is required. A parabolic curve may be used for curve radii greater than 20,000 feet. In no event shall sight distance or other safety considerations be sacrificed to meet the above requirements.

Generally speaking, on two-lane roads a curve shall not exceed a length of one-half mile and shall be no shorter than 500 feet.

6. Passing Tangents

Passing tangents are used to provide passing opportunities on two-lane roads. One-half mile is considered an adequate length. Passing tangents should be provided as frequently as possible in keeping with the terrain.

An effort to introduce a passing tangent or to increase the length of a passing tangent is always worthy of reasonable expenditure. Nothing is gained if sweeping curves of large radii are used at the ends of a tangent if they reduce its length to less than an adequate passing tangent. It is better to use shorter radii and increase the length of intervening tangents. At the other extreme, sharp curves at ends of a passing tangent must be avoided (see paragraph 3. above).

7. Compound Curves

A compound curve should be avoided, particularly when a simple curve can be obtained at small extra cost. On a compound curve, the shorter radius shall be at least two-thirds the longer radius when the shorter radius is 1,000 feet or less.

The total arc length of a compound curve should be not less than 500 feet.

8. Reversing Curves

Reversing curves without an intervening tangent are not permitted. Severe physical restrictions may dictate the use of curves in opposite directions with a short intervening tangent. In such cases, the preferred length of tangent shall be sufficient to accommodate the superelevation transition.

9. Broken Back Curves

A broken back curve consists of two curves in the same direction joined by a short tangent. Broken back curves are unsightly and undesirable. A reasonable additional expenditure is warranted to avoid such curvature.

10. Alignment at Bridges

Superelevation transitions on bridges almost always result in an unsightly appearance of the bridge and the bridge railing. Therefore, if at all possible, horizontal curves should begin and end a sufficient distance from the bridge so that no part of the superelevation transition extends onto the bridge. Alignment and safety considerations, however, are paramount and shall not be sacrificed to meet the above criteria.

4-04 VERTICAL ALIGNMENT

4-04.1 General

The grade line is a reference line by which the elevation of the pavement and other features of the highway are established. It is controlled mainly by topography, type of highway, horizontal alignment, safety, sight distance, construction costs, cultural development, drainage and pleasing appearance. The performance of heavy vehicles on a grade must also be considered.

All portions of the grade line must meet sight distance requirements for the design speed classification of the road.

In flat terrain, the elevation of the grade line is often controlled by drainage considerations. In rolling terrain, some undulation in the grade line is often advantageous, both from the standpoint of truck operation and construction economy. But, this should be done with appearance in mind; for example, a profile on tangent alignment exhibiting a series of humps visible for some distance ahead should be avoided whenever possible. In rolling terrain, however, the profile usually is closely dependent upon physical controls.

In considering alternative grade profiles, economic comparisons should be made. For further details, see the AASHTO publication: A Policy on Geometric Design of Highways and Streets, 1984.

4-04.2 Position with Respect to Cross Section

The grade line should generally coincide with the axis of rotation for superelevation; see Section 4-03.2, paragraph 6. Its relation to the cross section should be as follows.

1. Undivided Highways

   The grade line should coincide with the highway centerline.

2. Ramps and Freeway to Freeway Connections

   The grade line may be positioned at either edge of pavement, or centerline of ramp if multi-lane.

3. Divided Highways

   The grade line may be positioned at either the centerline of the median or at the ultimate median edge of pavement. The former case is appropriate for paved medians 30 feet wide or less. The latter case is appropriate when:

   a. The median edges of pavement of the two roadways are at equal elevation.

   b. The two roadways are at different elevations. See Section 4-04.8.

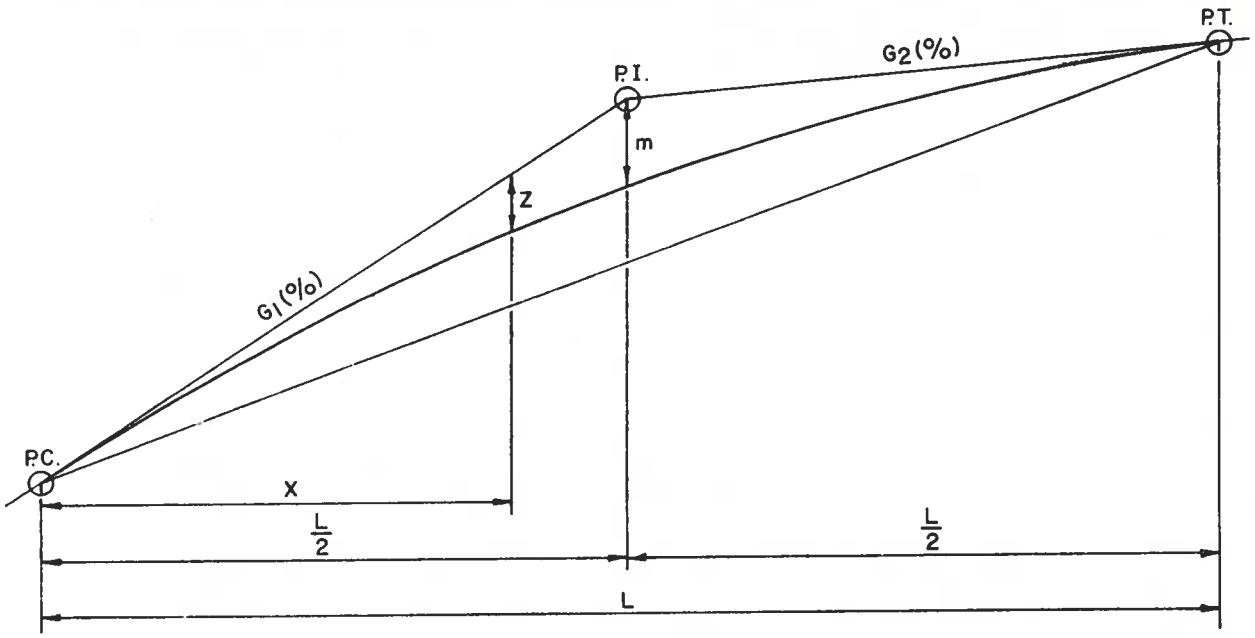
4-04.3 Standards for Grade

It is not practical to set maximum or minimum grades for all of the many types of highways constructed in widely varied terrain. In general, the maximum grade rate should be 12 percent and the minimum 0.4 percent for arterial streets and highways. The desirable maximum for freeways is 3 percent.

4-04.4 Vertical Curves

Properly designed vertical curves should provide adequate sight distance, safety, comfortable driving, good drainage, and pleasing appearance.

A parabolic vertical curve is used. Figures 4-J and 4-K give all necessary mathematical relations for computing a vertical curve,



m = MID-ORDINATE, IN FEET

Z = ANY TANGENT OFFSET, IN FEET

L = HORIZONTAL LENGTH OF VERTICAL CURVE, IN STATIONS

X = HORIZONTAL DISTANCE FROM P.C. OR P.T. TO ANY ORDINATE "Z", IN STATIONS

G_1 & G_2 = RATES OF GRADE, EXPRESSED ALGEBRAICALLY, IN PERCENT FT/FT

ALL EXPRESSIONS TO BE CALCULATED ALGEBRAICALLY

ELEV. OF P.I. = ELEV. P.C. + $G_1 \frac{L}{2}$

ELEV. OF P.T. = ELEV. P.C. + $(G_1 + G_2) \frac{L}{2}$

MID-ORDINATE DISTANCE "m" IN FEET = $\frac{(G_2 - G_1)L}{8} = \frac{1}{2}\left(\frac{\text{ELEV. OF P.C. + ELEV. OF P.T.}}{2} - \text{ELEV. P.I.}\right)$

FOR OFFSET "Z" AT DISTANCE "X" FROM P.C. OR P.T.: $Z = m\left(\frac{X}{\frac{L}{2}}\right)^2$ OR $Z = \frac{X^2(G_2 - G_1)}{2L}$

FOR SLOPE "S" OF A LINE TANGENT TO ANY POINT ON THE VERTICAL CURVE AT AN "X" DISTANCE MEASURED FROM THE P.C.:

S, IN PERCENT = $G_1 - \left[X\left(\frac{G_1 - G_2}{L}\right)\right]$

CALCULATING HIGH OR LOW POINT ON CURVE

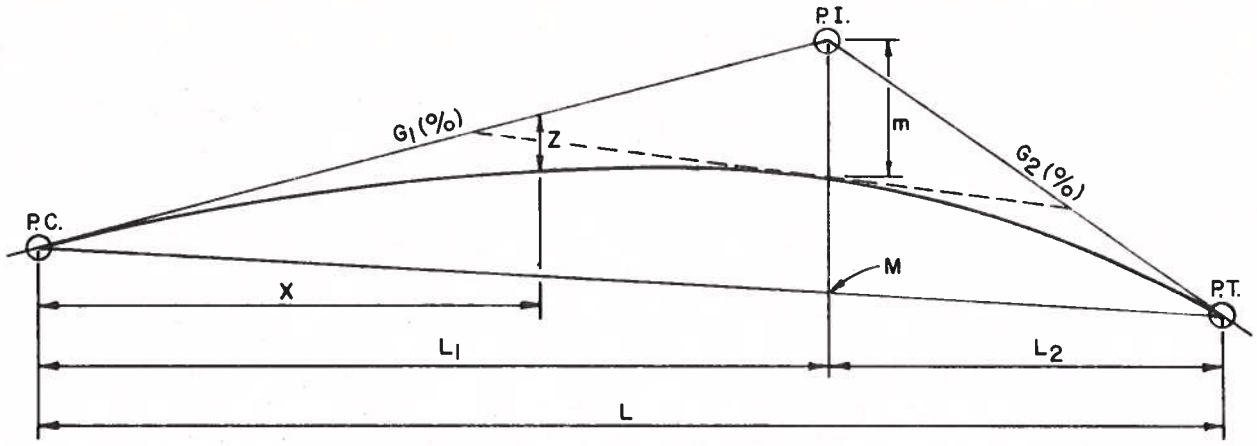
$X_T = \frac{LG_1}{G_1 - G_2}$ WHERE "X_T" EQUALS THE HORIZONTAL DISTANCE FROM THE P.C. TO THE HIGH OR LOW POINT ON THE CURVE IN STATIONS.

ELEVATION OF HIGH OR LOW POINT ON CURVE EQUALS: ELEV. P.C. - $\frac{1}{2}\frac{LG_1^2}{G_2 - G_1}$

SYMMETRICAL VERTICAL CURVE COMPUTATION

FIGURE 4-J

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M = THEORETICAL POINT ON P.I. STATION

m = OFFSET FROM THE P.I. TO THE CURVE, IN FEET

Z = ANY TANGENT OFFSET, IN FEET

L = HORIZONTAL LENGTH OF VERTICAL CURVE, IN STATIONS

L_1 = HORIZONTAL DISTANCE FROM P.C. TO P.I., IN STATIONS

L_2 = HORIZONTAL DISTANCE FROM P.T. TO P.I., IN STATIONS

X = HORIZONTAL DISTANCE FROM P.C. OR P.T. TO ANY ORDINATE "Z", IN STATIONS

G_1 & G_2 = RATES OF GRADE, EXPRESSED ALGEBRAICALLY, IN ~~PERCENT~~ FT/FT

ALL EXPRESSIONS TO BE CALCULATED ALGEBRAICALLY

ELEV. OF P.I. = ELEV. P.C. + G_1L_1

ELEV. OF P.T. = ELEV. P.C. + $G_1L_1 + G_2L_2$

ELEV. OF M = ELEV. P.C. + $\frac{L_1}{L}(G_1L_1 + G_2L_2)$

MID-ORDINATE DISTANCE "m" = $\frac{L_1L_2}{2L}(G_2 - G_1) = \frac{\text{ELEV. OF M} - \text{ELEV. OF P.I.}}{2}$

FOR OFFSET "Z" AT A DISTANCE "X" FROM P.C.: $Z = m\left(\frac{X}{L_1}\right)^2$

FOR OFFSET "Z" AT A DISTANCE "X" FROM P.T.: $Z = m\left(\frac{X}{L_2}\right)^2$

CALCULATING HIGH OR LOW POINT ON CURVE

IF HIGH OR LOW POINT OCCURS ON THE LEFT PORTION OF CURVE:

$X_T = \frac{L_1}{L_2}\left(\frac{G_1L}{G_1 - G_2}\right)$ WHERE "X_T" EQUALS THE HORIZONTAL DISTANCE FROM THE P.C. TO THE HIGH OR LOW POINT ON THE CURVE IN STATIONS

ELEV. OF THIS POINT = ELEV. P.C. $- \frac{1}{2}\frac{L_1}{L_2}\left(\frac{LG_1^2}{G_2 - G_1}\right)$

IF HIGH OR LOW POINT OCCURS ON THE RIGHT PORTION OF CURVE:

$X_T = \frac{L_2}{L_1}\left(\frac{G_2L}{G_2 - G_1}\right)$ WHERE "X_T" EQUALS THE HORIZONTAL DISTANCE FROM THE P.T. TO THE HIGH OR LOW POINT ON THE CURVE IN STATIONS

ELEV. OF THIS POINT = ELEV. P.T. $- \frac{1}{2}\frac{L_2}{L_1}\left(\frac{LG_2^2}{G_2 - G_1}\right)$

UNSYMMETRICAL VERTICAL CURVE COMPUTATION

FIGURE 4-K

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either at crests or sags. In lieu of the method shown, the rate of change of grade per station may be used in computations.

As flat vertical curves may develop poor drainage at the level section, this difficulty may be overcome by a slight gutter grade adjustment or by shortening the vertical curve.

On two-lane roads, extremely long crest vertical curves over one-half mile should be avoided, since many drivers refuse to pass on such curves, despite adequate sight distance. It is sometimes more economical to use four-lane construction than to obtain passing sight distance by the use of a long vertical curve.

Broken back vertical curves consist of two vertical curves in the same direction, separated by a short grade tangent. A profile with such curvature normally should be avoided.

4-04.5 Heavy Grades

Except in level terrain, often it is not economically feasible to design a grade profile that will allow uniform operating speeds for all classes of vehicles. Sometimes, a long sustained gradient is unavoidable.

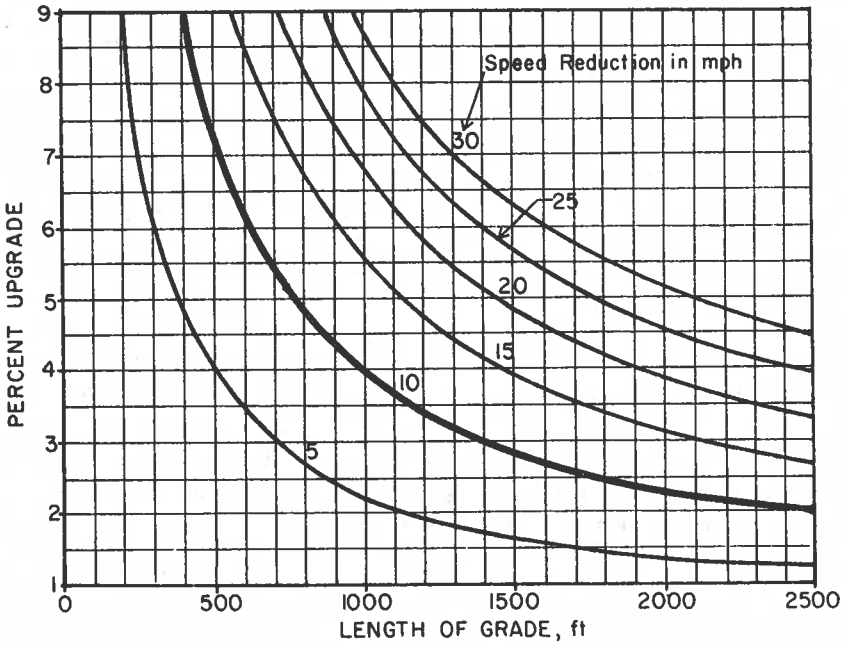
From a truck operation standpoint, a grade profile with sections of maximum gradient broken by lengths of flatter grade is preferable to a long sustained grade only slightly below the maximum allowable. It is considered good practice to use the steeper rates at the bottom of the grade, thus developing slack for lighter gradient at the top or elsewhere on the grade. Reference is made to Section 14, OTHER DESIGN CONSIDERATIONS, for design guides in climbing lanes.

4-04.6 Rolling Profile

Economy of construction without appreciable loss in operating efficiency often can be achieved by a rolling grade line, rather than a straight grade line; consideration should be given to appearance, however.

Figure 4-L gives the critical length of grade tangent related to percent upgrade and speed reduction. For ordinary design purposes, the 15 mile per hour speed reduction curve should be used. For further details on the use of this chart, see the AASHTO publication: A Policy on Geometric Design of Highways and Streets, 1984.

ASSUMED TYPICAL HEAVY TRUCK OF 300 POUNDS PER HORSEPOWER ENTERING SPEED = 55 MPH.



CRITICAL LENGTH OF GRADE FOR DESIGN

FIGURE 4-L

4-04.7 Coordination with Horizontal Alignment

A proper balance between curvature and grades should be sought. When possible, vertical curves should be superimposed on horizontal curves. This reduces the number of sight distance restrictions on the project, makes changes in profile less apparent, particularly in rolling terrain, and results in a pleasing appearance. For safety reasons, the horizontal curve should lead the vertical curve. On the other hand, where the change in horizontal alignment at a grade summit is slight, it safely may be concealed by making the vertical curve overlap the horizontal curve.

When vertical and horizontal curves are thus superimposed, the superelevation may cause distortion in the outer pavement edges. Profiles should be plotted and smooth curves introduced to remove any irregularities.

A sharp horizontal curve should not be introduced at or near a pronounced summit or grade sag. This presents a distorted appearance and is particularly hazardous at night.

Horizontal and vertical curvature at intersections should be as flat as physical conditions permit.

On long open curves, a uniform grade line should be used, as a rolling profile makes for a poor appearance.

Some undesirable and desirable designs of horizontal and vertical alignment are shown in Figures 4-M and 4-N.

4-04.8 Separate Grade Lines

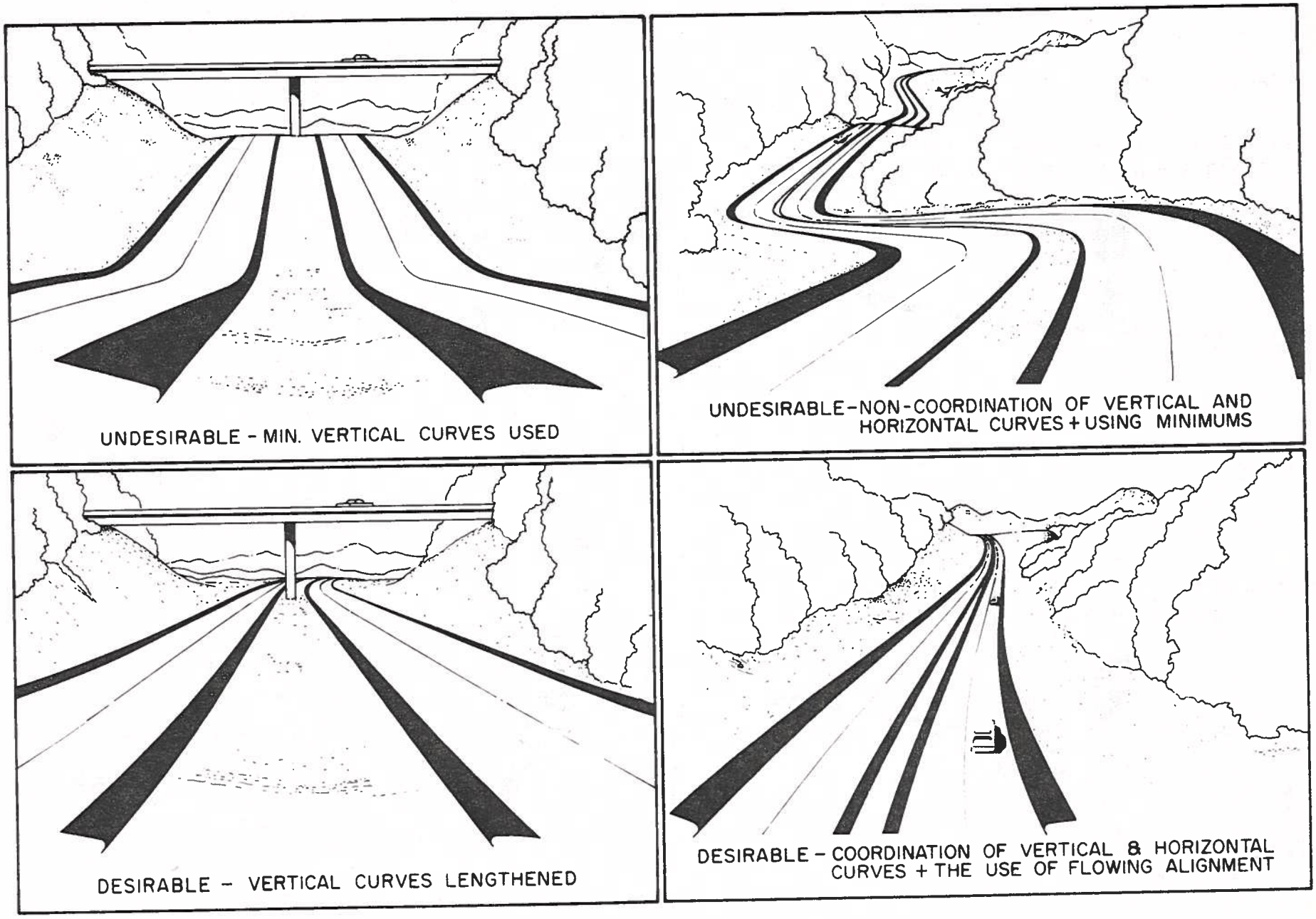
Separate or independent grade lines are appropriate in some cases for freeways and divided arterial highways.

They are not normally considered appropriate where medians are less than 60 feet wide. Exceptions to this may be minor differences between opposing grade lines in special situations.

In addition, any appreciable grade differential between roadbeds should be avoided in the vicinity of at-grade intersections. For traffic entering from the crossroad, confusion and wrong-way movements could result if the pavement of the far roadway is obscured due to excessive differential.

4-05 PAVEMENT TRANSITION

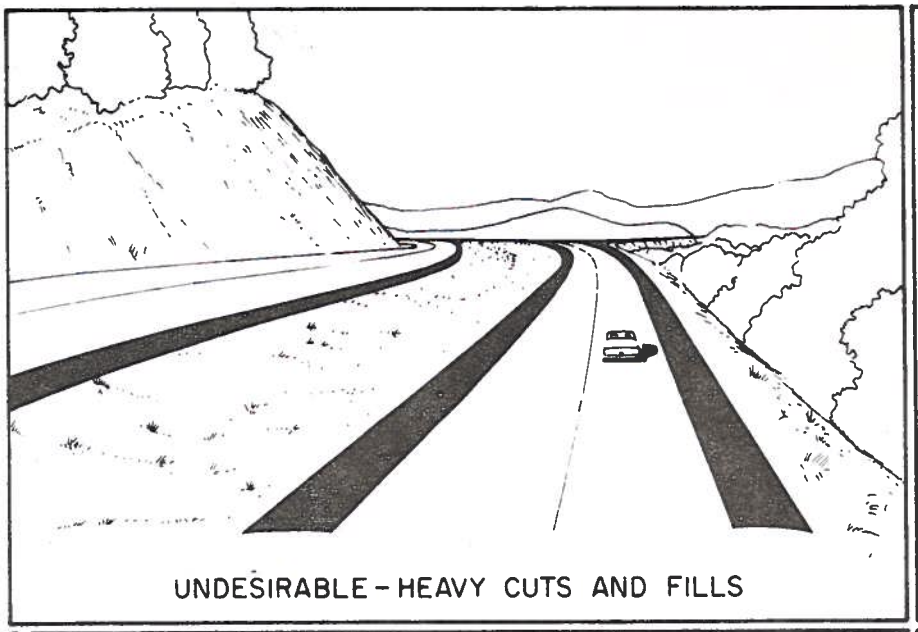
Design standards of the various features of the transition between roadways of different widths should be consistent with the design



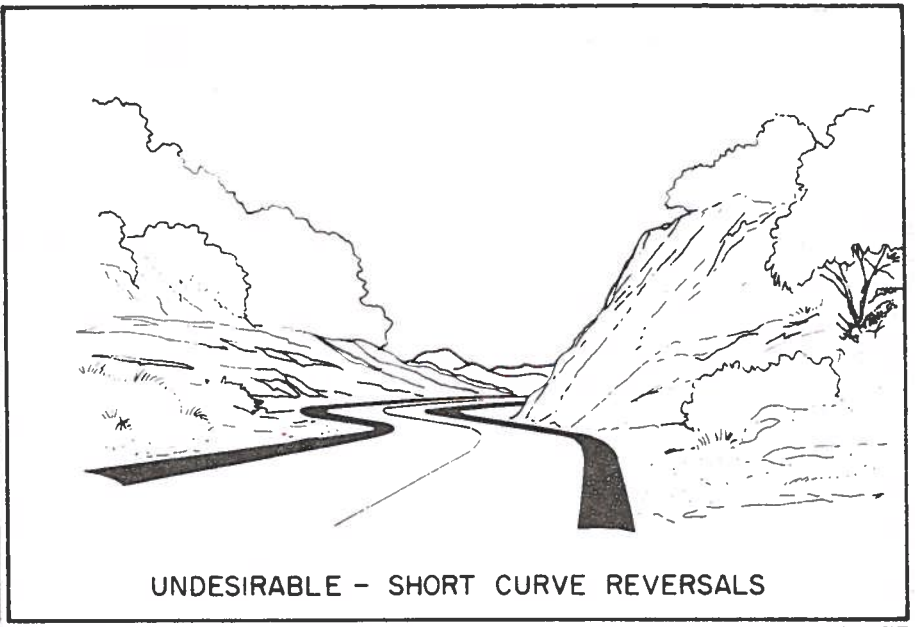
COORDINATION OF HORIZONTAL AND VERTICAL ALIGNMENT

FIGURE 4-M

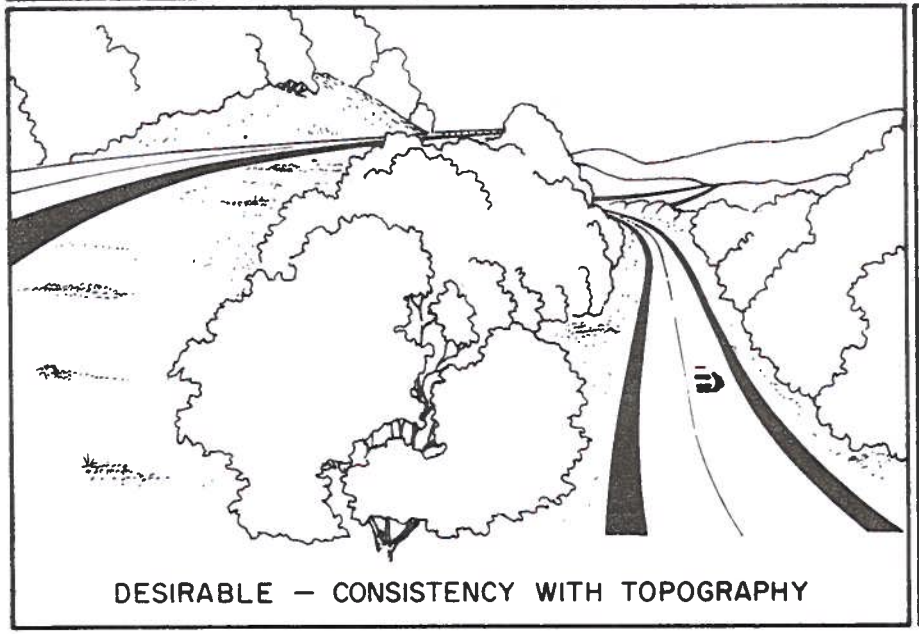
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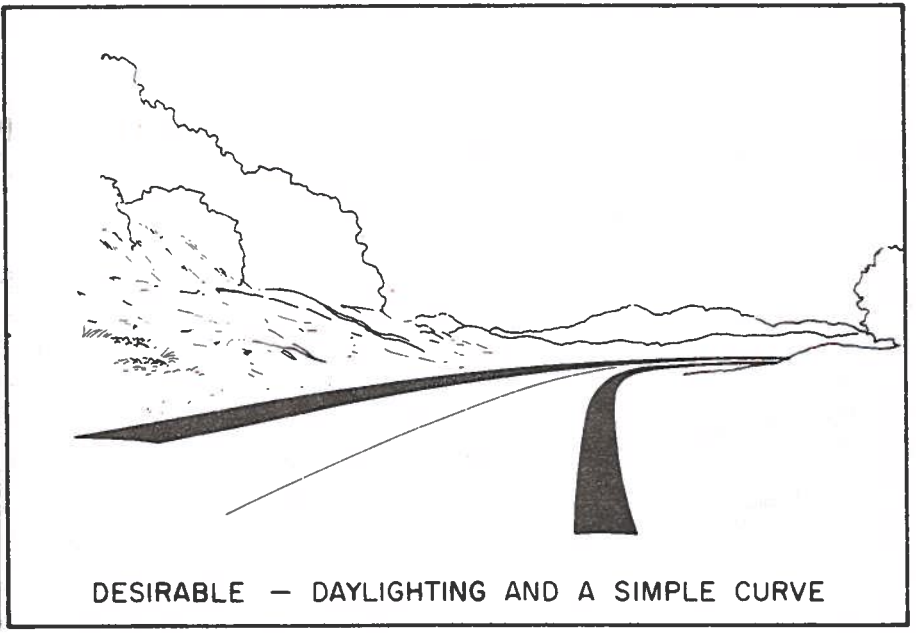
UNDESIRABLE - HEAVY CUTS AND FILLS



UNDESIRABLE - SHORT CURVE REVERSALS



DESIRABLE — CONSISTENCY WITH TOPOGRAPHY



DESIRABLE — DAYLIGHTING AND A SIMPLE CURVE

HORIZONTAL ALIGNMENT

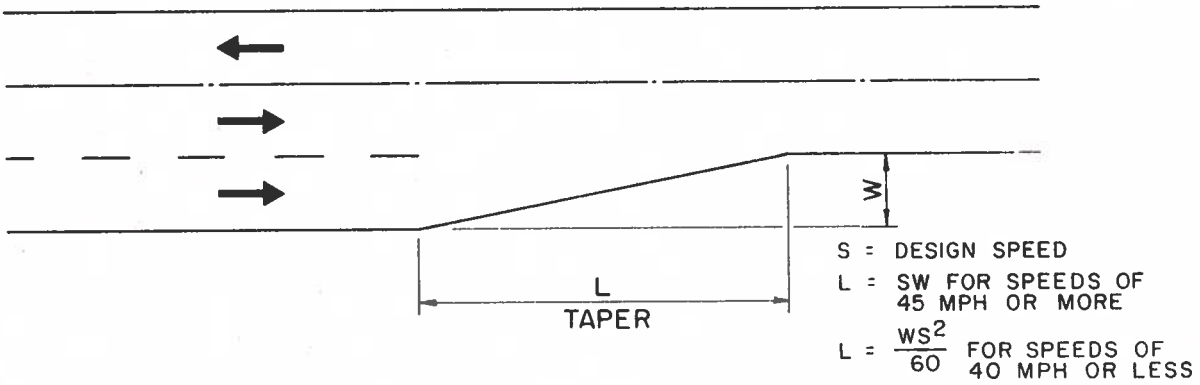
FIGURE 4-N

standards of the superior roadway. The transition should be made on a tangent section whenever possible and should avoid locations with horizontal and vertical sight distance restrictions. Whenever feasible, the entire transition should be visible to the driver of a vehicle approaching the narrower section.

The design should be such that at-grade intersections within the transition area are avoided.

Figure 4-O shows the minimum required taper length based upon the design speed of the roadway. In all cases, a taper length longer than the minimum should be provided where possible.

In general, when a lane is dropped by tapering, the transition should be on the right so that traffic merges to the left.



PAVEMENT TRANSITION

FIGURE 4-O

SECTION 5.

MAJOR CROSS SECTION ELEMENTS

5-01 GENERAL

The major cross section elements considered in the design of streets and highways include the pavement surface type, cross slope, lane widths, shoulders, curbs, driveways and sidewalks. Section 12, ROADSIDE DESIGNS, covers the design considerations outside of the edge of shoulder, such as side slopes and clear areas.

5-02 PAVEMENT

5-02.1 Surface Type

The type of pavement is determined by the volume and composition of traffic, the soil conditions, the availability of materials, the experience of contractors, the initial cost, and the extent and cost of maintenance, all of which affect the relationship of cost to traffic service.

Generally, all roadways in the State are surfaced with bituminous asphalt concrete or portland cement concrete. These types of surfacing provide good riding qualities, retain the cross section, and adequately support the expected volume and weights of vehicles without failure due to fatigue. Some local roads in rural areas of the State may be simply of graded earth or surface-treated earth, or of stabilized materials such as gravel or coral.

Important characteristics in relation to geometric design are the ability of a surface to retain its shape and dimensions, the ability to drain, and the effect on driver behavior.

5-02.2 Cross Slope

The cross slope of the pavement is the slope of the pavement surface measured transverse to the centerline of the highway. The normal cross slope of the roadway is sometimes called the crown or the high point. Two-lane and wider undivided pavements on tangents or on flat curves have a crown or high point in the middle of the highway and slope downward toward both edges.

The normal cross slope of all paved roadways should be 2 percent. The cross slope shall be uniform across the pavement section, from the high point to the edge of pavement.

On divided highways, each one-way pavement may be crowned separately, as on two-lane highways, or it may have a unidirectional slope across the entire width of pavement, which is almost always downward to the outer edge.

A cross section with each roadway crowned separately, as shown in Figure 5-A, has an advantage in rapidly draining the pavement during rainstorms. In addition, the difference between high and low points in the cross section is kept to a minimum. Disadvantages are that more inlets and subsurface drainage lines are required, and treatment of at-grade intersections is more difficult because of the several high and low points on the cross section. Use of such sections preferably should be limited to regions of high rainfall. Sections having no curbs and a wide depressed median are particularly well suited for high rainfall conditions.

Roadways that slope in only one direction, as shown in Figure 5-B, are more comfortable to drivers because vehicles tend to be pulled in the same direction when changing lanes. Roadways having a unidirectional slope may drain away from or toward the median. Drainage away from the median may effect a savings in drainage structures and simplify treatment of intersecting streets. Advantages of drainage toward the median are an economical drainage system, in that all surface runoff is collected into a single conduit, and the outer lanes, which are used by most traffic, are freer of surface water. A major disadvantage of this section is that all the pavement drainage must pass over the inner, higher speed lanes. Where curbed medians exist, drainage is concentrated next to and on the higher speed lanes. This concentration results in the annoying and hazardous splashing on the windshields of opposing traffic when the median is narrow.

The rate of cross slope on curves as well as on tangent alignment is an important element in cross section design. Pavement superelevation on curves is determined by the speed-curvature relationships given in Section 4, BASIC GEOMETRIC DESIGN ELEMENTS. Cross slopes up to 2 percent are barely perceptible as far as effect on vehicle steering is concerned, but cross slopes steeper than 2 percent are noticeable. The latter rate requires a conscious effort in steering and would increase the susceptibility to lateral skidding when vehicles brake on wet pavements, and even on dry pavements when stops are made under emergency conditions.

Low type surfaces, such as graded earth or stabilized gravel, may require steeper cross slopes on tangents to prevent absorption of water into the surface.



FIGURE 5-A EACH PAVEMENT SLOPES TWO WAYS

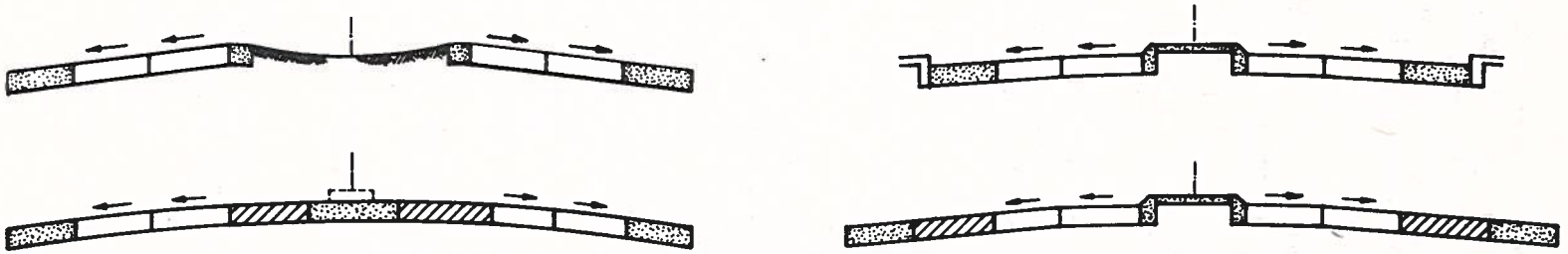


FIGURE 5-B EACH PAVEMENT SLOPES ONE WAY

5-03 LANE WIDTHS

Lane widths have a great influence on the safety and comfort of driving. On high-type highways, the predominant lane width is 12 feet (3.66 m). The extra cost of 12 foot (3.66 m) over 10 foot (3.05 m) lanes is offset to some extent by a reduction in cost of shoulder maintenance and reduction in surface maintenance due to lessened wheel concentrations at the pavement edges.

Although lane widths of 12 feet (3.66 m) are desirable, there are circumstances that necessitate the use of lanes less than 12 feet (3.66 m) wide. In urban areas, the use of 11 foot (3.35 m) lanes is acceptable, and even 10 foot (3.05 m) and 9 foot (2.75 m) wide lanes may be acceptable where right-of-way and existing development become stringent controls.

Auxiliary lanes at intersections are often provided to facilitate traffic movements. Such lanes should be not less than 10 feet (3.05 m) wide, and desirably at least equal in width to the through lanes.

5-04 SHOULDERS

5-04.1 General

A shoulder is the portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles, for emergency use, and for lateral support of subbase, base and surface courses. The shoulder may vary in width from 2 feet (0.62 m) on low volume roads to 12 feet (3.66 m) on highways with high truck volumes; it should be completely paved for a minimum of 2 feet (0.62 m) from the edge of traveled way, and desirably for the entire width of the shoulder. State DOT uses min. 2' paved shldr.

Some of the more important advantages of providing shoulders are:

1. Space for the motorist to pull completely off the roadway for emergencies, or to review a map.
2. An escape hatch to avoid potential accidents or reduce their severity.
3. Contributes to driver comfort by creating a sense of openness; improves highway capacity.
4. Sight distance can be improved in cut sections.
5. Enhances lateral clearance for the placement of signs or guardrails.

6. Space for pedestrian or bicycle usage.

7. Eliminates pavement edge dropoff when shoulder is paved or stabilized.

5-04.2 Width of Shoulders

Desirably, a vehicle stopped on the shoulder should clear the pavement edge by at least 1 foot (0.31 m), preferably by 2 feet (0.62 m). This preference has led to the adoption of 10 feet (3.05 m) as the normal usable shoulder width that should be provided along high-type facilities. In difficult terrain and on low-volume highways, usable shoulders of this width may not be feasible. A minimum usable shoulder width of 2 feet (0.62 m) should be considered for the lowest type highway, and a 6 or 8 foot (1.83 or 2.44 m) width would be preferable. Heavily traveled and high-speed highways, and those carrying large numbers of trucks, should have usable shoulders at least 10 feet (3.05 m) and preferably 12 feet (3.66 m) wide. Shoulder widths for specific classes of highways are enumerated as parts of the total cross sections discussed in following Sections.

Where guardrails, walls, or other vertical elements are used, the graded shoulder should be wide enough that these vertical elements can be offset a minimum of 2 feet from the outer edge of the usable shoulder.

Although it is desirable that a shoulder be wide enough for a vehicle to be driven completely off the traveled way, narrower shoulders are better than none at all. Partial shoulders are sometimes used where full shoulders are unduly costly, as on long-span bridges or in mountainous terrain. Regardless of the width, a shoulder should be continuous where feasible.

Shoulders on structures should have the same width as the usable shoulders on the approach roadways, both right and left. This design is essential on freeways, and is desirable on all arterials where shoulders are provided. As previously discussed, the narrowing or loss of shoulders, especially on structures, may cause serious operating and safety problems. Long-span, high-cost structures usually warrant detailed special studies to determine feasible dimensions. Wherever practicable, full shoulders should be included, but as has been indicated, for some cases, it may be judged proper to use only partial-width shoulders.

5-04.3 Cross Sections

Shoulders are important links in the lateral drainage systems. Shoulders should be flush with the roadway surface and abut the

edge of the travel lane. On divided highways with a depressed median, all shoulders should be sloped to drain away from the traveled way. With a raised narrow median, the median shoulders may slope in the same direction as the traveled way. All shoulders should be pitched sufficiently to rapidly drain surface water, but not to the extent that vehicular use would be hazardous. Because the type of shoulder construction has a bearing on the cross slope, the two should be determined jointly. Bituminous and concrete-surfaced shoulders should be sloped from 2 to 6 percent; gravel or crushed rock shoulders from 4 to 6 percent; and turf shoulders from 6 to 8 percent. Where curbs are used on the outside of shoulders, the minimum cross slope should not be less than 4 percent to minimize ponding on the roadway.

Rigid adherence to the slope rates outlined presents a minor traffic operation problem when applied without regard to the cross section of the paved surface. On tangent or on long radius curved alignments with a normal crown, the maximum shoulder slope should produce an algebraic difference in the pavement and shoulder grades of no more than 7 percent at the edge of the paved surface.

Shoulder slopes that drain away from the paved surface on the outside of well-superelevated sections should be designed to avoid too great a cross slope break. The algebraic difference produced by the pavement superelevation and the shoulder slope should not exceed 7 percent. Where possible, the shoulder slope on the high side of the superelevation should be sloped upward at about the same rate as the pavement superelevation. A "high-side of super" ditch should be provided at the outside edge of the shoulder for drainage purposes as necessary.

The shoulder on the inside of the curve or the low side of super should be sloped at the same rate as that of the superelevated pavement once the superelevated rate equals the normal slope of the shoulder.

5-04.4 Intermittent Shoulders or Turnouts

It will not always be economically feasible to provide desirably wide shoulders continuously along the highway through high cut areas or along steep mountainsides. In such cases, consideration should be given to the use of intermittent sections of shoulders or turnouts that can be placed at favorable locations along the highway. Where intermittent shoulders or turnouts are provided, transition sections of ample length should be provided to encourage usage and to permit safe entry and exit.

5-05 CURBS

5-05.1 General

The type and location of curbs appreciably affect driver behavior and, in turn, the safety and utility of a highway. Curbs often serve two or more of the following purposes: drainage control, pavement edge delineation, aesthetics, delineation of pedestrian walkways, and assistance in orderly roadside development. To be considered a curb, some raised aspect or vertical element is required.

Curbs are used extensively on all types of urban highways. On rural highways, caution should be exercised in the use of curbs. In the interest of safety, curbs should be omitted on rural highways when the same objective can be attained by other acceptable means.

5-05.2 Types of Curbs

The two general classes of curbs are barrier curbs and mountable curbs and each has numerous types and detailed designs. Each may be designed as a separate unit, or integrally with the pavement. Barrier and mountable curbs may be designed with a gutter to form a combination curb and gutter section.

Barrier curbs are relatively high and steep faced, designed to inhibit or at least discourage vehicles from leaving the roadway. As shown in Figure 5-C, they have a 6 inch (152.4 mm) face. A sloping face is desirable, but generally the face batter is about 1 inch per 6 inches (25.4 mm per 152.4 mm) of height. The upper corner is rounded on a radius of 1 inch (25.4 mm).

Curbs should not be used on freeways and are considered undesirable on other high-speed arterials. Generally, barrier curbs should not be used where design speeds exceed 40 mph. When accidentally struck at high speeds, it is difficult for the operator to retain control of the vehicle. In addition, most barrier curbs are not adequate to prevent a vehicle from leaving the roadway. Where positive action is required, such as along narrow medians or adjacent to bridge substructures, suitable median barrier or guardrail should be provided.

Generally, barrier curbs should not be provided inside the faces of bridge parapets. A preferred, and more widely used, method is to design the parapet in the shape of a slope-faced barrier such as the New Jersey-type barrier. A barrier curb is sometimes used on bridges on urban streets, usually with a curb height somewhat higher than the normal roadway curb.



BARRIER CURB WITH GUTTER MOUNTABLE CURB

FIGURE 5-C

TYPICAL HIGHWAY CURBS

Barrier curbs and safety walks may be desirable along the faces of long walls and tunnels, particularly if full shoulders are not provided. The curb tends to prevent vehicles from scraping the wall, and the safety walk reduces the danger to persons walking from disabled vehicles.

Mountable curbs are designed so that vehicles can cross them readily when required. As shown in Figure 5-C, they are low with flat sloping faces. Mountable curbs can be used at median edges to outline channelizing islands in intersection areas, or sometimes at the outer edge of the shoulder. Curbs should not be placed adjacent to high-speed lanes unless they serve a definite purpose. The mountable section in Figure 5-C may be used for a median curb. Where curbs are used for drainage purposes in conjunction with guardrails, they should be installed directly under or behind the barrier. Improperly placed curbs may cause errant vehicles to vault a barrier, or to strike it so that the vehicle overturns. Section 13, TRAFFIC BARRIERS, discusses curb placement in conjunction with traffic barriers.

Shoulder curbs are mountable curbs placed at the outer edge of the shoulder to control drainage, improve delineation, and reduce erosion. They may be part of the longitudinal drainage system being combined with a gutter section. The mountable section is applicable for this use. If the surfaced shoulders are not wide enough for a vehicle to park, the shoulder curb must appear to be easily mountable to encourage motorists to park clear of the traveled way.

Gutter sections may be provided on the traveled way side of a barrier curb to form the principal drainage system for the roadway. Inlets should be provided in the gutter or the curb, or both. Generally, it is not practical to design gutter sections to contain all of the runoff, even from frequent rains, and some overflow onto the traveled surface can be expected. The spread of water on the traveled way is kept within tolerable limits by the proper sizing and spacing of inlets. Grate inlets and depressions or curb-opening inlets should not be placed in the traveled lane because of their adverse effect on drivers and bicycle riders who veer away from them. Warping of the gutter for curb-opening inlets should be limited to the portion within 2 to 3 feet (0.62 to 0.92 m) of the curb to minimize adverse driving effects.

5-05.3 Placement of Curbs

Barrier curbs should not be used on freeways or other high-speed arterials, but if provided in special cases, the curb should not be closer than the outer edge of shoulder. Barrier curbs introduced intermittently along streets should be offset 2 feet (0.62 m) from the edge of pavement; where continuous, as along a median, barrier curbs should be offset at least 1 foot (0.31 m), and preferably 2 feet (0.62 m).

5-06 WHEELCHAIR RAMPS

Where the highway design requires accommodations for pedestrian traffic, consideration must be given to the requirements of physically handicapped persons whose means of mobility is dependent upon wheelchairs. Wheelchair ramps shall be located at all marked crosswalk locations where steep-faced curbs are installed.

Design details of curb-cut and wheelchair ramps will vary in relation to the following factors:

1. Sidewalk width;
2. Sidewalk location with respect to the back face of curb;
3. Height and width of curb cross section;
4. Design turning radius and length of curve along the curb face;
5. Angle of street intersections;
6. Planned or existing location of sign and signal control devices;
7. Drainage inlets and public surface utilities;
8. Possible sight obstructions;
9. Street width.

These factors necessitate the establishment of basic ramp types and differing design applications predicated on acceptable design standards of each agency and the geometric characteristics of each intersection.

The location of the curb-cut opening and ramp must be carefully coordinated with respect to the pedestrian crosswalk lines. Where possible, the ramp opening (at the fully depressed curb) should be situated within the parallel boundaries of the crosswalk markings.

Ramps for the handicapped are not limited to intersections, and marked crosswalks and ramps should also be provided at other appropriate or designated points of pedestrian concentration, such as loading islands, midblock pedestrian crossings, and locations where pedestrians could not otherwise recognize the proper place to cross the highway or street.

5-07 SIDEWALKS

Sidewalks are accepted as integral parts of the urban street system. However, in many rural areas, the need is great for sidewalks along high-speed highways, such as at schools, local businesses, and commercial developments that result in pedestrian concentrations near or along the highway.

Shoulders on rural highways may obviate the need for sidewalks if they are of the type that encourage pedestrian use in all weather. Sidewalks in urban areas typically separate the roadway from the homes or businesses along the streets or highways. Sidewalk widths in residential areas may vary from 4 to 8 feet (1.22 to 2.44 m). A planting strip is usually provided between the curb and the sidewalk for street lights, fire hydrants, street hardware and aesthetic planting. In commercial areas, the sidewalk usually abuts the curb to provide a large sidewalk area.

5-08 BUS TURNOUTS

5-08.1 General

Bus travel is an increasingly important mode of mass transportation. Bus turnouts serve to remove the bus from the traffic lanes. The location and design of turnouts should provide ready access in the safest and most efficient manner possible.

5-08.2 Expressways

The basic requirement for a bus turnout, when exclusive bus roadways are not provided, is that the deceleration, standing and acceleration of buses be effected on pavement areas clear of and

separated from the through-traffic lanes. Other elements in the design of bus turnouts include passenger platforms, ramps, stairs, railings, signs and markings. Speed change lanes should be long enough to enable the bus to leave and enter the through-traffic lanes at approximately the average running speed of the highway without undue inertial discomfort or sidesway to passengers. The length of acceleration lanes from bus turnouts should be well above the normal minimum values, as the buses start from a standing position and the loaded bus has a lower acceleration capability than passenger cars. Normal-length deceleration lanes are suitable. The bus standing area and speed change lanes, which include the shoulders, should be 20 feet (6.10 m) wide to permit passing a stalled bus.

The dividing area between the outer edge of expressway shoulder and the edge of bus turnout lane should be as wide as feasible, preferably 20 feet (6.10 m) or more. However, in extreme cases, this width could be reduced to a minimum of 4 feet (1.22 m). A barrier is usually needed in the dividing area, and fencing is desirable to keep pedestrians from entering the expressway. Pedestrian loading platforms should not be less than 5 feet (1.53 m) wide and preferably 6 to 10 feet (1.83 to 3.05 m) wide.

5-08.3 Arterials

The interference between buses and other traffic can be considerably reduced by providing turnouts clear of the lanes for through traffic.

It is somewhat rare that sufficient right-of-way is available on the lower type arterial streets to permit turnouts in the border area, but advantage should be taken of every opportunity to do so.

To be fully effective, bus turnouts should incorporate (1) a deceleration lane or taper to permit easy entrance to the loading area, (2) a standing space sufficiently long to accommodate the maximum number of vehicles expected to occupy the space at one time, and (3) a merging lane to enable easy re-entry into the through-traffic lanes.

The deceleration lane should be tapered at an angle flat enough to encourage the bus operator to pull completely clear of the through lane. Usually, it is not feasible to provide a length sufficient to permit deceleration from highway speeds clear of the traveled way. A taper of about 5:1, longitudinal to transverse, is a desirable minimum. When the bus stop is on the far side of an intersection, the intersection area may be used as the entry area to the stop.

The loading area should provide about 50 feet (15.25 m) of length for each bus. The width should be at least 10 feet (3.05 m), and preferably 12 feet (3.66 m).

The merging or re-entry taper may be somewhat more abrupt than the deceleration taper but, preferably, should not be sharper than 3:1. Where the turnout is on the near side of an intersection, the width of cross street is usually great enough to provide the necessary merging space.

The total length of turnout for a two-bus loading area of minimum design should be about 180 feet (54.90 m) for a midblock location; 150 feet (45.75 m) for a near-side location; and 130 feet (39.65 m) for a far-side location. These dimensions are based on a loading area width of 10 feet (3.05 m). They should be increased by 10 to 15 feet (3.05 to 4.57 m) for a width of 12 feet (3.66 m). Greater lengths of bus turnouts expedite bus maneuvers, encourage full compliance on the part of bus drivers, and lessen interference with through traffic.

5-09 DRIVEWAYS

5-09.1 General

Driveway terminals are, in effect, low volume intersections. The number of driveways and the location of driveways have a definite effect on highway capacity, primarily of arterial highways.

Generally, driveway regulations are effected through the issuance of permits by the agency having jurisdiction of the roadway under design. Such regulations control right-of-way encroachment, driveway location, driveway design, sight distance, setback, signing, etc.

5-09.2 Design Requirements

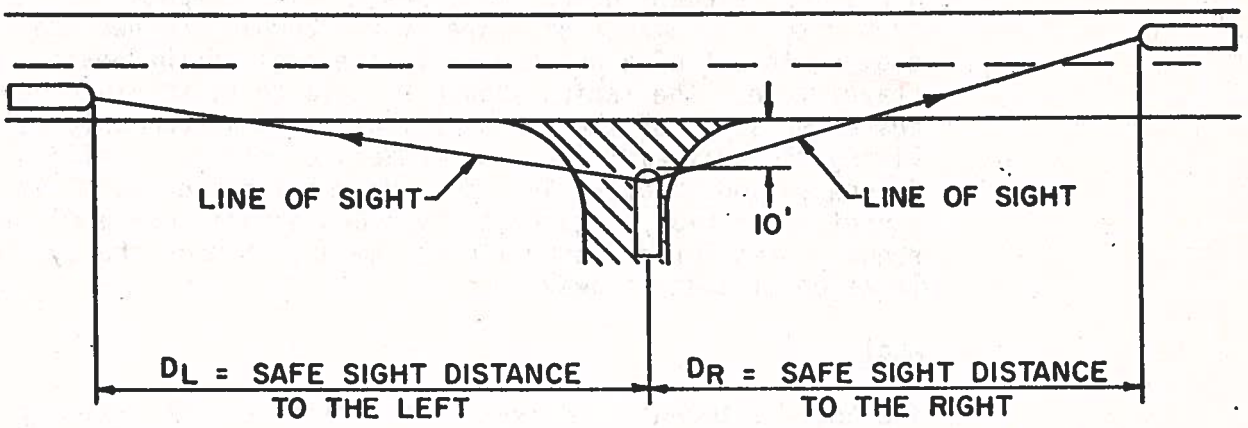
1. Sight Distance

   The driveway must be visible for safe entrance and exit. Sight distance requirements for driveway egress and left-turn ingress are provided in Figure 5-D.

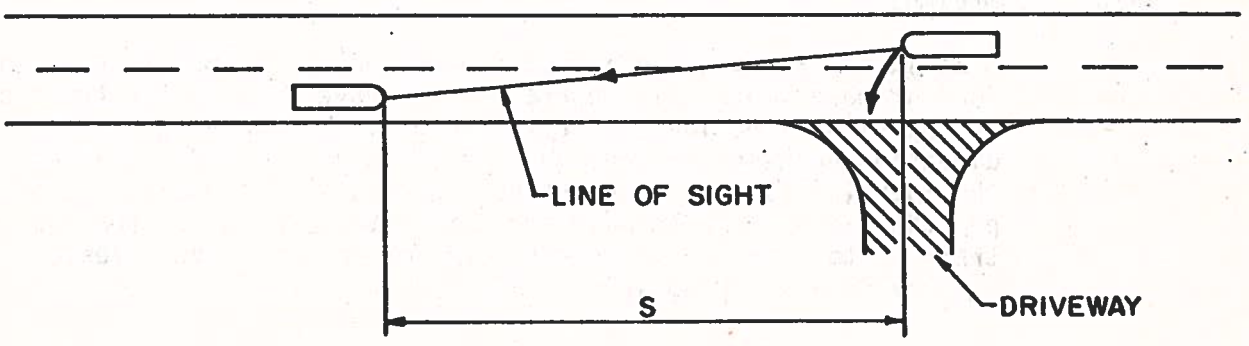
2. Location

   It is important to locate driveways away from major intersections. This constraint is as much for the ability to enter and leave the property, as for the benefit of intersection

	D = DISTANCE ALONG MAJOR ROAD FROM DRIVEWAY TO ALLOW VEHICLE TO ENTER SAFELY (FEET). SEE ACCOMPANYING ILLUSTRATION															
	30 MPH				40 MPH				50 MPH				60 MPH			
	2 LANE		4 OR 6 LANE		2 LANE		4 OR 6 LANE		2 LANE		4 OR 6 LANE		2 LANE		4 OR 6 LANE	
	D_L	D_R	D_L	D_R	D_L	D_R	D_L	D_R	D_L	D_R	D_L	D_R	D_L	D_R	D_L	D_R
PASSENGER CARS	350	260	220	260	530	440	380	440	740	700	620	700	950	1050	950	1050
TRUCKS	500	400	400	400	850	850	850	850	1600	1600	1600	1600	2500	2500	2500	2500



	S = DISTANCE ALONG MAJOR ROUTE FOR VEHICLE TO SAFELY TURN LEFT ONTO DRIVEWAY (FEET). SEE ILLUSTRATION BELOW											
	30 MPH			40 MPH			50 MPH			60 MPH		
	2 LANE	4 LANE	6 LANE	2 LANE	4 LANE	6 LANE	2 LANE	4 LANE	6 LANE	2 LANE	4 LANE	6 LANE
PASSENGER CARS	230	250	270	370	390	420	520	550	580	700	740	780
TRUCKS	400	440	480	570	620	670	810	880	950	1000	1100	1200



SIGHT DISTANCES AT ENTRANCES

FIGURE 5-D

Northwestern criteria

safety and operations. Each agency has minimum distance requirements between the edge of the driveway and the end of curb return radius at a corner. This distance is 15 feet (4.57 m) for driveways on State highways. The spacing and number of driveways serving a single piece of property is also an agency permit requirement.

3. Radii and Width

A critical element of driveway design is the radius of the curb return or amount of flare of the curbing connecting the edge or throat of a driveway with the edge of the nearest travel lane. The radius should be related to the turning characteristics of the types of vehicles most commonly utilizing the driveway; i.e., passenger vehicle for residential driveways and SU or WB-40s for commercial driveways along the street or highway. Again, the various agencies have minimum requirements for driveway widths, dependent upon the type and location of the driveway.

4. Angles

The angle between the driveway centerline and the roadway edge should be based primarily on safety requirements. The speed at which a vehicle can enter or leave a public roadway is affected by the angle of approach or departure. Where there are physical constraints to providing the appropriate angle to the driveway connection, acceleration and deceleration lanes may be considered. The choice between 90 degree or angled driveways may also be dictated by direction of travel, as in the case of one-way streets or divided roadways.

5. Other Design Factors

Other design elements to be considered in driveway design include on-site circulation for proper driveway operation, driveway grade and roadway drainage.

5-10 MEDIANS

A median is a highly desirable element on all arterials carrying four or more lanes. It separates the traveled ways for traffic in opposing directions. The median width is expressed as the dimension between the through-lane edges and includes the left shoulders, if any. The principal functions of a median are to provide the desired freedom from the interference of opposing traffic, to provide a recovery area for out-of-control vehicles, to provide a stopping area in case of emergencies, to provide

for speed changes and storage of left-turning and U-turning vehicles, to minimize headlight glare, and to provide width for future lanes. Another benefit of a median in an urban area is that it may add an open green space. For maximum efficiency, a median should be highly visible both night and day and in definite contrast to the through traffic lanes. A median may vary in scope from a simple traffic stripe to an expansive area of varying width between two independently designed roadways. Medians may be depressed, raised, or flush with the pavement surface.

Medians should be as wide as feasible, but of a dimension in balance with other components of the cross section. The general range of median widths is from a minimum of 4 feet (1.22 m) to a desirable dimension of 80 feet (24.40 m) or more on freeways in rural areas.

In general, the median should be as wide as can be used advantageously. As far as the safety and convenience of motor vehicle operation are concerned, the farther the pavements are apart, the better. However, economic factors limit the width of median that can be provided. Cost of construction and maintenance increases generally with an increase in the width of roadbed, but the additional cost may not be appreciable compared with the cost of the highway as a whole, and may be justified in view of the benefits derived. A distinct advantage of the wider medians on roadways other than freeways is to provide adequate shelter for vehicles crossing at intersections with public roads and at crossovers serving commercial and private drives. However, wide medians are a disadvantage when the intersection is signalized. The increased time for vehicles to cross the median may lead to inefficient signal operation.

If the right-of-way is restricted, the median should not be widened beyond a desirable minimum at the expense of narrowed roadside areas. A reasonable roadside width is required to adequately serve as a buffer between the private development along the road and the traveled way, particularly where zoning is limited or nonexistent. Space must be provided in the roadside areas for sidewalks, highway signs, utility lines, parking, drainage channels and structures, and for proper slopes and any retained native growth. Narrowing these areas may tend to develop hazards and hindrances similar to those that the median is designed to avoid.

Raised medians have application on arterial streets where it is desirable to regulate left-turn movements. They are also frequently used where the median is to be planted, particularly where the width is relatively narrow. It must be pointed out, however, that planting in narrow medians creates hazardous conditions for maintenance operations.

Flush medians are used to some extent on all types of urban arterials. When used on freeways, a median barrier may be required. The median should be slightly crowned or depressed for drainage.

Additional discussion on median openings and intersections is included in Section 6, AT-GRADE INTERSECTIONS.

SECTION 6.

A T - G R A D E I N T E R S E C T I O N S

6-01 GENERAL

An at-grade intersection is the general area where 2 or more highways join or cross at the same level, within which are included the roadway and roadside facilities for traffic movements in that area. Each highway radiating from an intersection and forming part of it is an intersection leg. The common intersection of 2 highways crossing each other has 4 legs.

An intersection is an important part of a highway because, to a great extent, the efficiency, safety, speed, cost of operation, and capacity depend on its design. Along most urban streets, the capacity, travel time and delay characteristics are controlled principally by the number and type of at-grade intersections. In addition, a large part of the accident and safety aspect of the street is attributable to intersection operation.

In varying degrees, 4 principal factors determine the character of an intersection. These factors are traffic, physical, economic and human.

Traffic factors to be considered include: possible and practical capacities, turning movements, size and operating characteristics of vehicles, control of movements at points of intersection, vehicle speeds, pedestrian movements, transit operations, and accident experience.

Physical factors which control intersection design and application of channelization are topography, abutting land use, geometric features of the intersecting roadways, traffic control devices, and safety features.

Economic factors, which are important and often controlling, include the cost of improvements and the economic effect on abutting businesses where channelization restricts or prohibits certain vehicular movements within the intersection area.

Human factors such as driving habits, ability of drivers to make decisions, effect of surprise, decision and reaction times, and natural paths of movements must be considered.

An intersection may be extremely simple or highly developed, depending upon the proper evaluation of the foregoing factors.

6-02 GENERAL DESIGN CONSIDERATIONS

6-02.1 Capacity Analysis

Capacity analysis is one of the most important considerations in the design of intersections. This is especially true in the design of at-grade intersections on urban streets and highways. Optimum capacities can be obtained when intersections include auxiliary lanes, proper use of channelization, and traffic control devices. Reference is made to the 1985 Highway Capacity Manual, Special Report 209.

6-02.2 Spacing

The spacing of intersections on major arterials is important to the capacity and safety of the roadway. In urban areas, the capacity of the arterial is determined by the capacity of the signalized intersections along the roadway. Ideally, intersections should be located no closer than 1200 feet (366 m) apart.

On collector streets and roads, the spacing of intersections will vary to accommodate the subdivision street patterns.

6-02.3 Alignment and Profile

Intersections are points of conflict between vehicles, pedestrians and facilities and, hence, are potentially hazardous. The alignment and grade of the intersecting roads should permit drivers to discern and perform readily the maneuvers necessary to pass through the intersection safely and with minimum interference between vehicles. To these ends, the alignment should be as straight as possible and gradient as low as practical. The sight distance should be equal to or greater than the minimum values for the specific intersection conditions. Sight distance is discussed later in this section.

1. Alignment

   Regardless of the type of intersection, intersecting highways should meet at or nearly at right angles. Roads intersecting at acute angles require extensive turning roadway areas. Intersections on sharp curves should be avoided wherever possible because the superelevation and widening of pavements on curves complicate the intersection design.

2. Profile

Combinations of grade lines that make vehicle control difficult should be avoided. Substantial grade changes should be avoided at intersections, although it is not always feasible to do so. Adequate sight distance should be provided along both highways and across corners, even where one or both intersecting highways are on vertical curves.

The gradients of intersecting highways should be as flat as practical on those sections that are to be used for storage space for stopped vehicles. A minimum storage space for 2 vehicles, approximately 40 feet (12 m), should be provided for minor streets where stop sign control is employed and the approach gradient is up towards the intersection. Such slopes shall be desirably less than one percent and no more than 3 percent.

for intersections where one leg comes to intersection at an upgrade where sight dist. is restricted

The profile grade lines and cross sections on the intersection legs should be adjusted for a distance back from the intersection proper to provide a smooth junction and proper drainage. Normally, the grade line of the major highway should be carried through the intersection, and that of the cross road should be adjusted to it. This design requires transition of the crown of the minor highway to an inclined cross section at its junction with the major highway. For simple unchannelized intersections involving low posted speed and stop control signs, it may be desirable to warp the crowns of both roads into a plane at the intersection, the particular plane depending on direction of drainage and other conditions. Changes from one cross slope to another should be gradual. Intersections with a minor road crossing a multi-lane divided highway with a narrow median and superelevated curve should be avoided whenever possible because of the difficulty in adjusting grades to provide a suitable crossing. Grade lines of separate turning roadways should be designed to fit the cross slopes and longitudinal grades of the intersection legs.

As a rule, the alignment and grades are subject to greater restriction at or near intersecting roads than on the open road. Their combination at or near the intersection must produce traffic lanes that are clearly visible to the vehicle operators at all times and definitely understandable for any desired direction of travel, free from sudden appearance of potential hazards, and consistent with the portions of the highway just traveled.

The combination of vertical and horizontal curvature should also be considered. A sharp horizontal curve following a crest vertical curve is very undesirable in the intersection area.

6-02.4 Cross Section

The cross section of the pavement surface within an intersection should be reviewed on a case-by-case basis. The development of the centerline profiles and edge of pavement profiles should flow smoothly through the intersection. A 2 percent minimum cross slope should be provided for drainage in warping the pavement area within the intersection.

6-03 SIGHT DISTANCE

The operator of a vehicle approaching an at-grade intersection should have an unobstructed view of the whole intersection and a sufficient length of the intersecting highway to permit control of the vehicle to avoid collisions. When traffic at the intersection is controlled by signals or signs, the unobstructed view may be limited to the area of control. The minimum sight distance considered safe under various assumptions of physical conditions and driver behavior is directly related to vehicle speeds and to the resultant distances traversed during perception, reaction time, and braking.

Uncontrolled intersections (no stop or signal control at the intersection) are not considered standard in the State and, therefore, will not be discussed in this Manual.

6-03.1 Stop Control on Cross Street

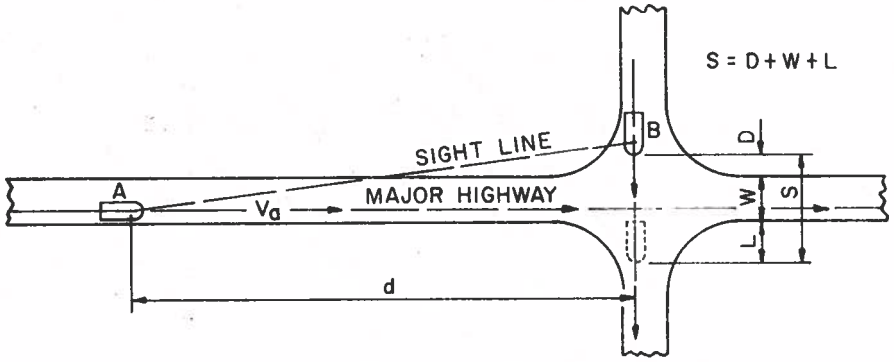
At an intersection where traffic is controlled by stop signs on the minor road, for safety reasons, the driver of a stopped vehicle must see enough of the major highway to be able to cross before a vehicle on it reaches the intersection, even though this vehicle comes into view just as the stopped vehicle starts to cross. The length of the major highway open to view must be greater than the product of its design speed and the time necessary for the stopped vehicle to start and cross the road. Referring to Figure 6-A, the required sight distance along the major highway can be expressed as follows:

$$d = 1.47V(J + t_a)$$

where d = minimum or desirable sight distance along the major highway from the intersection, ft
V = design speed on the major highway, mph

J = sum of the perception time and the time required to actuate the clutch or actuate an automatic shift, sec.

t_a = time required to accelerate and traverse the distance S to clear the major highway pavement, sec.



D = DISTANCE FROM NEAR EDGE OF PAVEMENT TO THE FRONT OF A STOPPED VEHICLE (ASSUME THE STOP BAR)

W = PAVEMENT WIDTH

L = VEHICLE LENGTH

STOP CONTROL ON MINOR ROAD

FIGURE 6-A

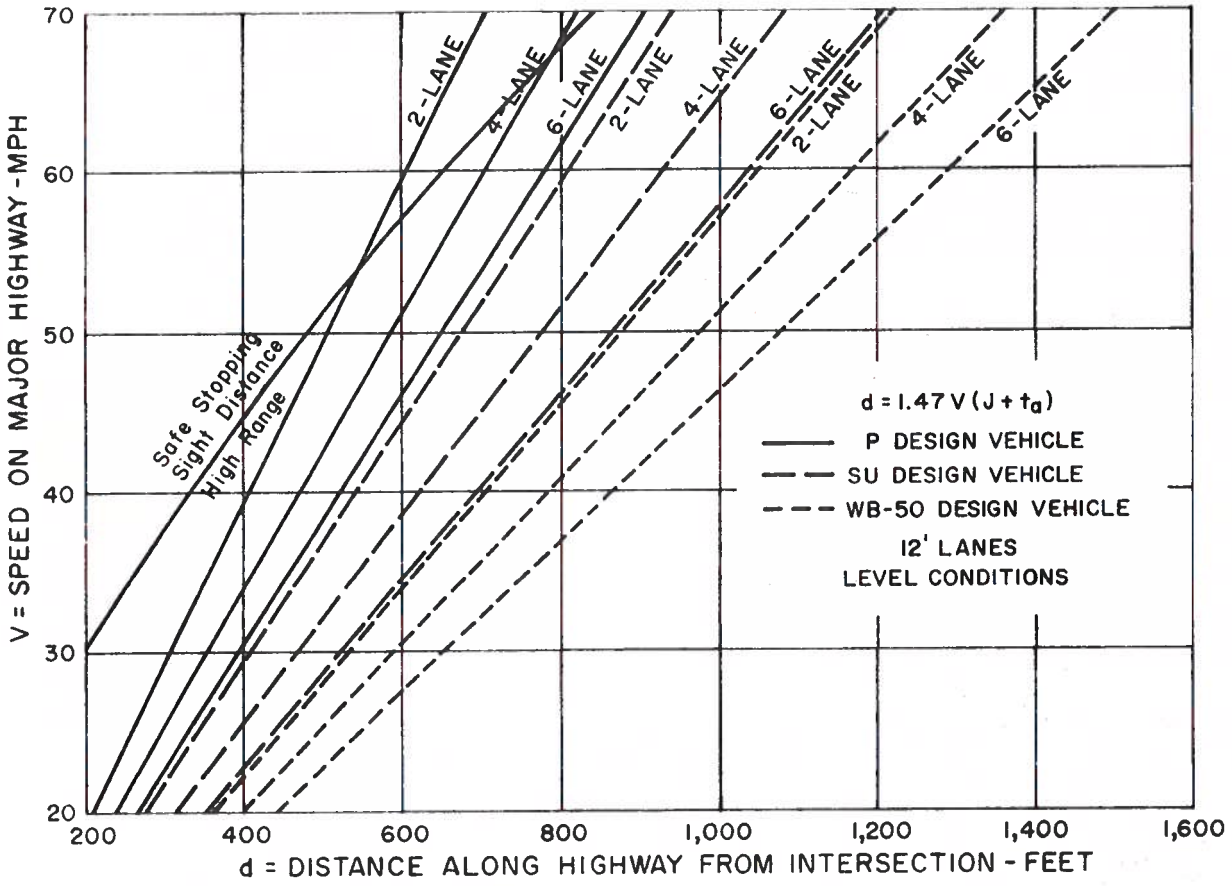
Figure 6-B shows the minimum sight distance d necessary for safe crossing from a stopped position under the following conditions: (1) lane widths are 12 feet; (2) intersecting roadways are nearly 90 degrees; (3) cross road gradient is level.

Where the median width on a divided highway is equal to or greater than the vehicle lengths, the crossing can be accomplished in 2 steps. The vehicle crosses the first pavement, stops within the protected area, and proceeds when a safe gap in traffic occurs to cross the second pavement. However, when the median width is less than that of a vehicle, the crossing must be made in one step and the median width must be included as part of the 'W' value in Figure 6-A.

Figure 6-C provides data on acceleration from stop for level terrain, which is the t_a value in the above equation.

6-03.2 Sight Distance at Driveways

Sight distance for egress and ingress at driveway connections are covered in Section 5, CROSS SECTION ELEMENTS.

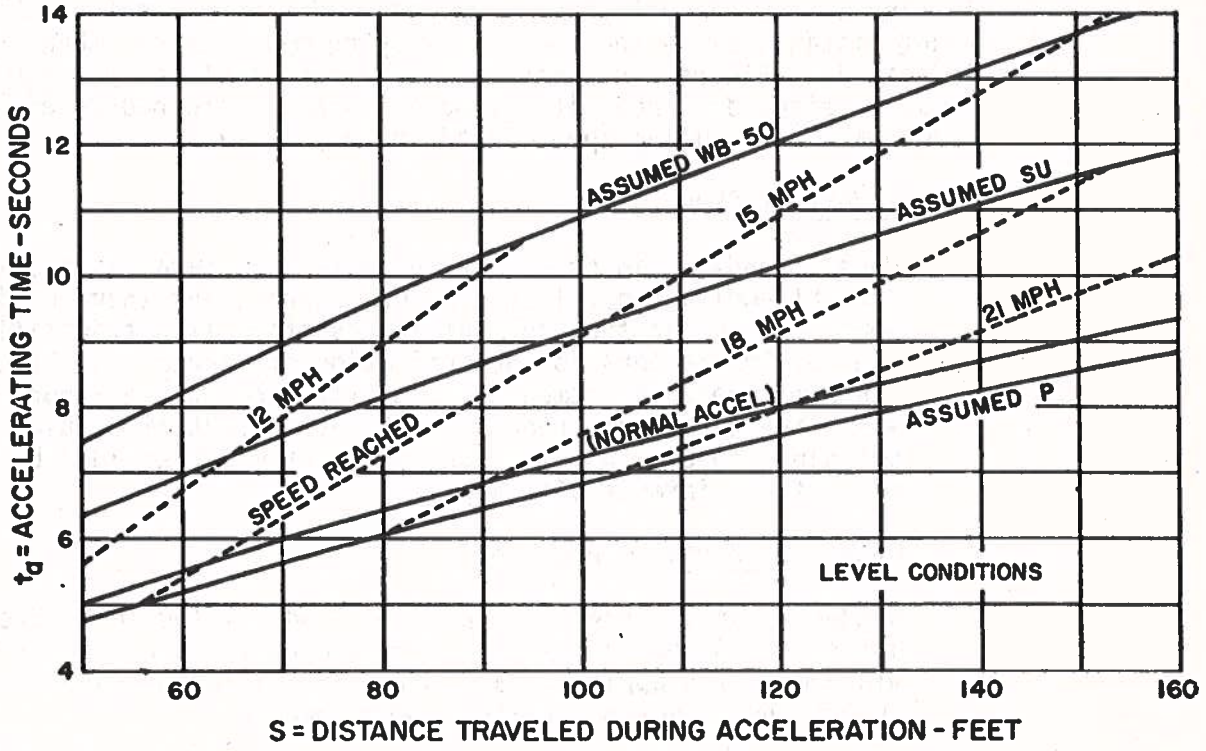


NOTE: J = 2 SECONDS

REQUIRED SIGHT DISTANCE ALONG MAJOR ROADWAY AT INTERSECTION

FIGURE 6-B

r5/15/86



ACCELERATION FROM STOP

FIGURE 6-C

6-03.3 Sight Distance at Signalized Intersections

Intersections controlled by traffic signals presumably do not require sight distance between intersecting traffic flows because the flows move at separate times. However, drivers should be provided with some view of the intersecting approaches in case a crossing vehicle violates the signal indication. In addition, sight distance requirements for vehicles permitted to turn right on red signal indications must be considered. Line-of-sight should consider the effect of parked vehicles.

6-03.4 Effect of Skew and Grades on Sight Distance

In many cases, the effect of skew and grades requires that the sight distance along the major highway be much greater than that indicated in Figure 6-B. Where it is not feasible to provide the required sight distance, it may be necessary to introduce effective means of reducing speed on the major highway.

1. Effect of Skew

When 2 roadways intersect at an angle considerably less than a right angle, e.g., less than 60 degrees, and when realignment to increase the angle of intersection is not justified, some of the factors for determination of corner sight distance may need adjustment. Primarily, the 'W' in Figure 6-A will have to be computed to represent the actual path of the crossing vehicle, which is the pavement width divided by the sine of the intersecting angle.

2. Effect of Vertical Profile

Grades on an intersection leg should be limited to 3 percent unless the sight distances are considerably in excess of the minimums for stopping on a level grade, in which case the grades should not be greater than 6 percent.

The time required to cross the major highway is materially affected by the approach grade of the minor street. Normally, the grade across an intersection is so small that it need not be considered, but when curvature on the major highway requires the use of superelevation, the grade across it may be significant, in which case the sight distance along the major highway should be increased. The approximate ratios of the accelerating time on the grade to that on the level, for the likely range of crossing distances are given in Table 6-1.

Table 6-1

Effect of Gradient on
Accelerating Time at Intersection

Ratio, Accelerating Time on Grade to Accelerating Time on Level Section					
Design Vehicle	Crossroad Grade (percent)				
	-4	-2	0	+2	+4
P	0.7	0.9	1.0	1.1	1.3
SU	0.8	0.9	1.0	1.1	1.3
WB-50	0.8	0.9	1.0	1.2	1.7

6-04 TURNING MOVEMENTS

6-04.1 General

One of the primary concerns of intersection design is to provide adequately for left and right turning movements. The pavement and roadway widths of turning roadways at intersections are governed by the volumes of turning traffic and the types of vehicles to be accommodated.

6-04.2 Design Vehicles

The overall dimensions of the design vehicles considered in geometric design are shown in Table 2-1 of Section 2, DESIGN PARAMETERS. The minimum turning radius of these design vehicles is shown in Table 6-2.

The minimum turning paths of the design vehicles are illustrated in Figures 6-D through 6-I. These figures should be used as guides in determining the turning radii at intersections and the widths of turning roadways. The principal dimensions affecting design are the minimum turning radius and those affecting the path of the inner rear tire, tread width and wheel base.

Vehicle turning path templates are included at the back of this Manual.

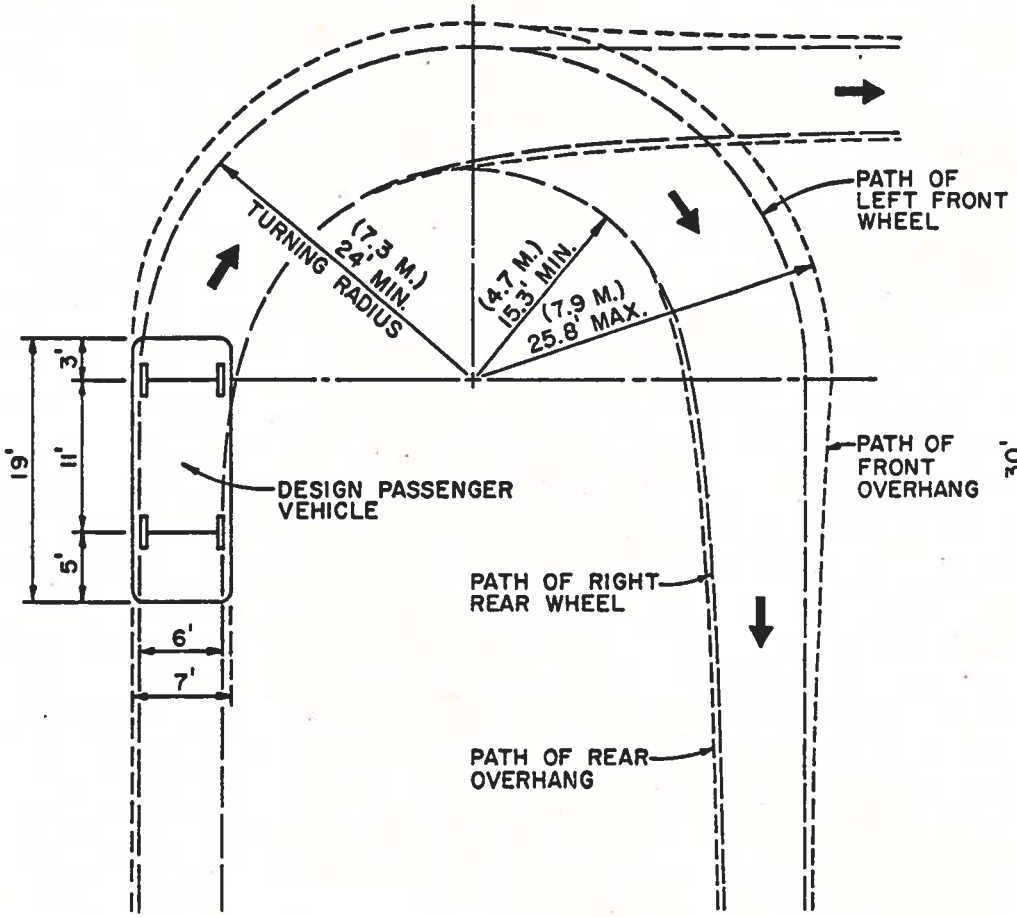


FIGURE 6-D

MINIMUM TURNING PATH FOR
P DESIGN VEHICLE

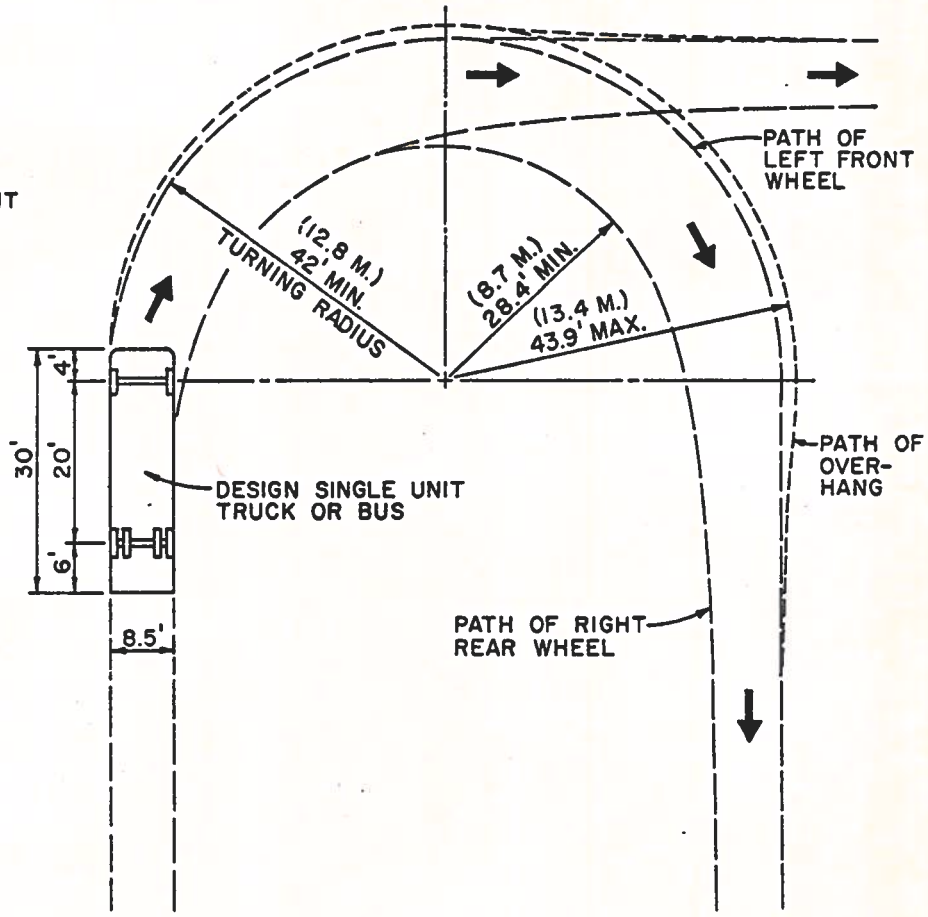


FIGURE 6-E

MINIMUM TURNING PATH FOR
SU DESIGN VEHICLE

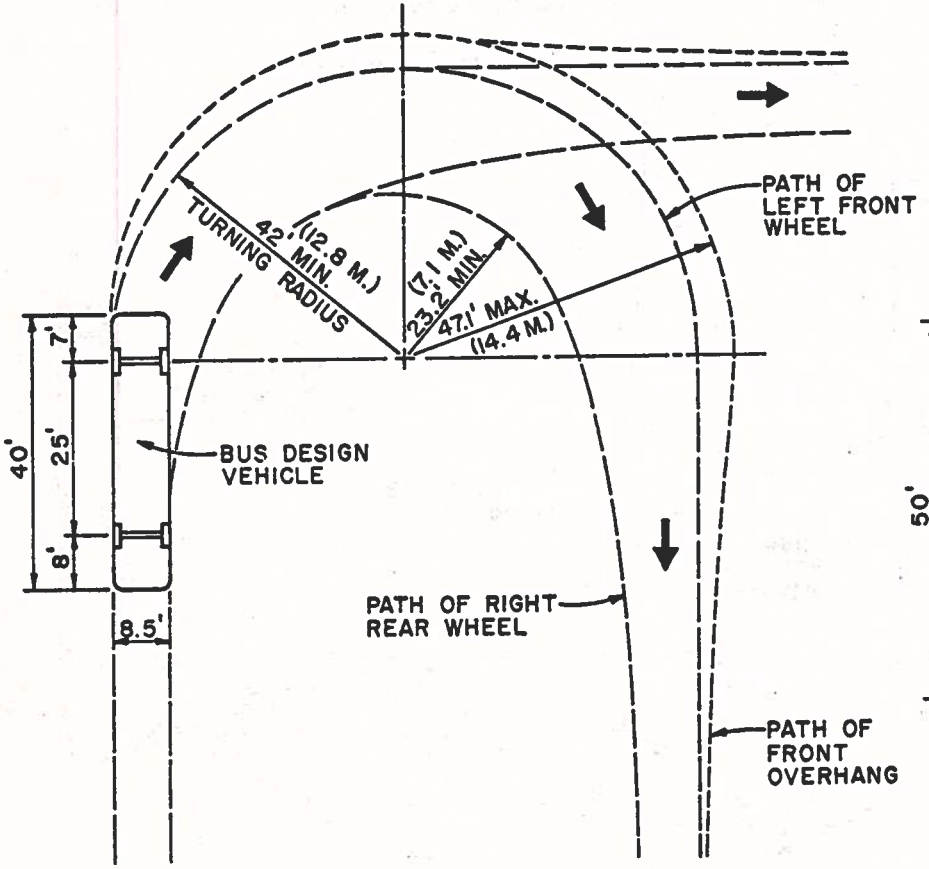


FIGURE 6-F

MINIMUM TURNING PATH FOR BUS DESIGN VEHICLE

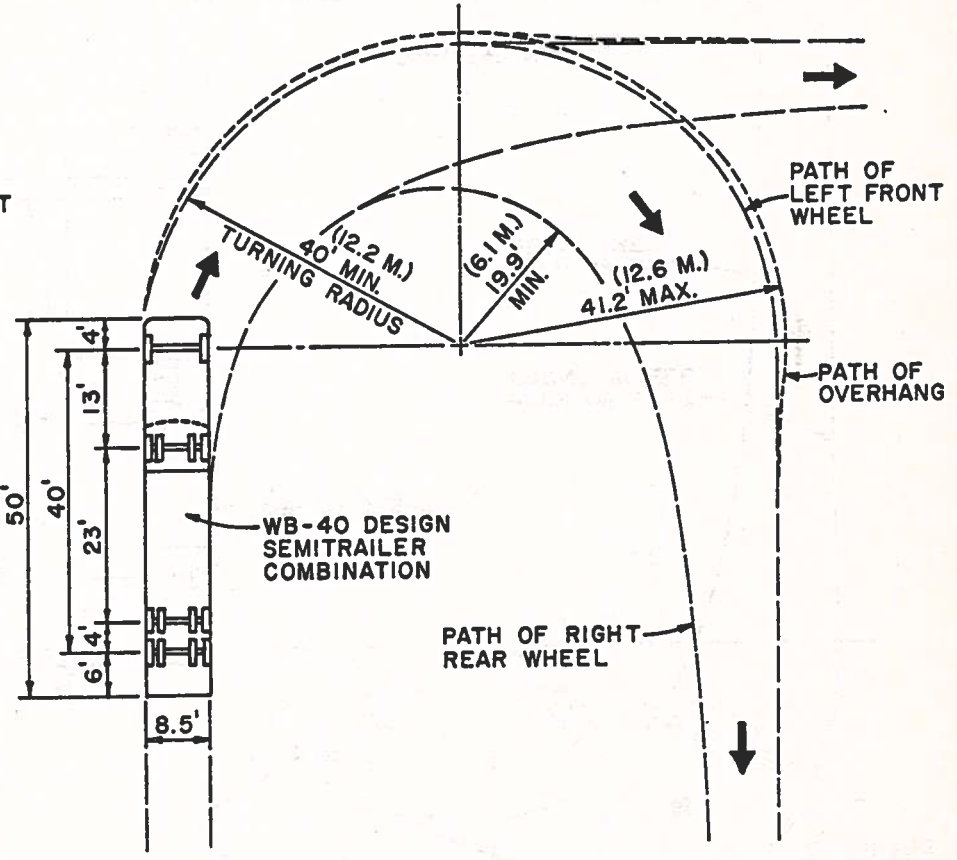


FIGURE 6-G

MINIMUM TURNING PATH FOR WB-40 DESIGN VEHICLE

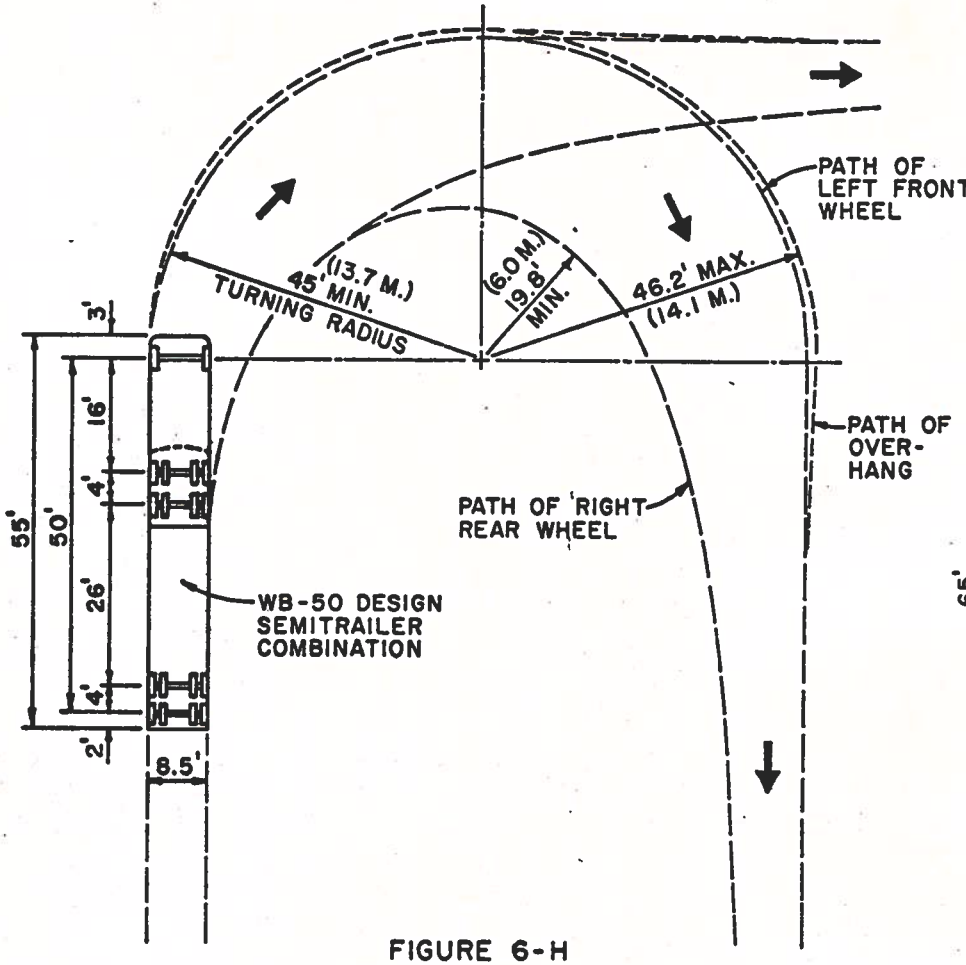


FIGURE 6-H

MINIMUM TURNING PATH FOR WB-50 DESIGN VEHICLE

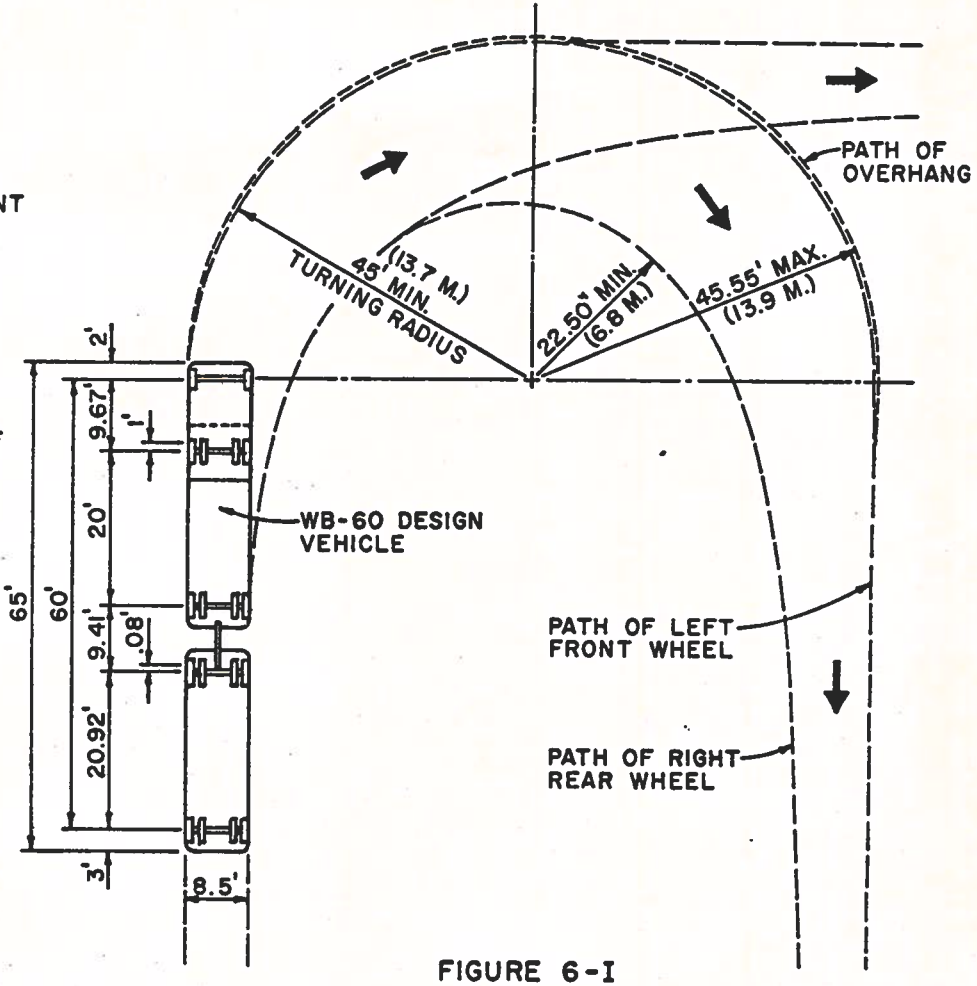


FIGURE 6-I

MINIMUM TURNING PATH FOR WB-60 DESIGN VEHICLE

Table 6-2

Minimum Turning Radius of Design Vehicles

Design Vehicle Type	Symbol	Minimum Turning Radius		Minimum Inside Radius	
		Feet	Meters	Feet	Meters
Passenger Car	P	24	7.3	15.3	4.66
Single Unit Truck	SU	42	12.8	28.4	8.66
Single Unit Bus	BUS	42	12.8	23.2	7.12
Semitrailer-Intermediate	WB-40	40	12.2	19.9	6.07
Semitrailer-Large	WB-50	45	13.7	19.8	6.04
Semitrailer-Full Trailer	WB-60	45	13.7	22.5	6.90

6-04.3 Minimum Edge of Pavement Design for Turns

Where it is necessary to provide for turning vehicles within minimum space and slow speeds (less than 10 mph), as at unchannelized intersections, the minimum turning paths of the design vehicles apply.

For most simple intersections with angle of turn of 90 degrees or less, a single circular arc joining the tangent edges of pavement provides an adequate design. However, where provision must be made for the larger truck units, or when the angle of turn exceeds 90 degrees, a single circular arc would require a much larger radius, and a 3-centered compound curve should be used.

Recommended minimum edge of pavement designs for turns at intersections without corner islands are given in Table 6-3. These are based on the characteristics of the various design vehicles, low turning speeds (less than 10 mph), and no encroachment of adjacent lanes. The application of 3-centered compound curve is illustrated in Figure 6-J.

Table 6-3 covers angles of turn from 30 degrees through 180 degrees (U-Turn). It should be noted that angles of turn in excess of 120 degrees are considered substandard and are to be avoided whenever possible through realignment of the intersection legs. For 3-centered compound curves, values are given only of the preferred symmetric design. Values for asymmetric design for 3-centered compound curves may be found in AASHTO rural and urban design policies.

Table 6-3

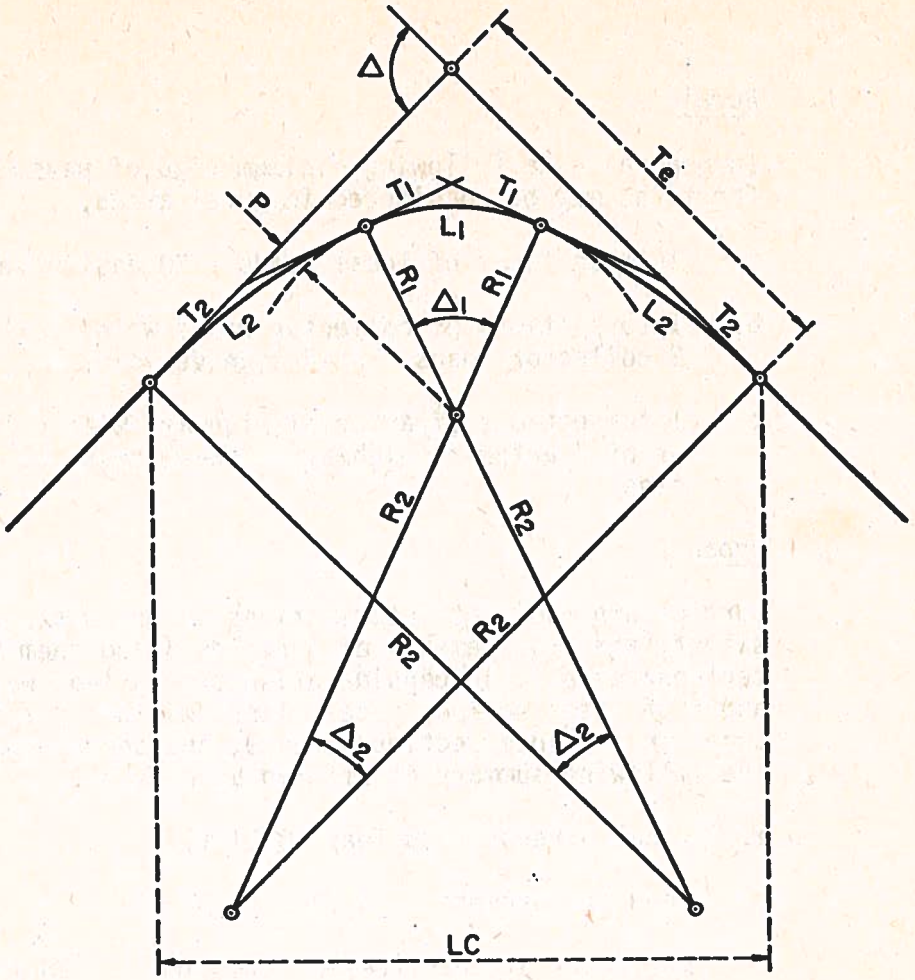
Minimum Edge of Pavement Designs for Turns at Intersections* (Without Corner Island, Speeds Less Than 10 MPH)

Design Vehicle	Angle of Turn Degrees	Simple Curve Radius (Feet)	3-Centered Compound Curve Radii (Feet)	3-Centered Compound Curve Offset (Feet)	Angle of Turn Degrees	3-Centered Compound Curve Radii (Feet)	3-Centered Compound Curve Offset (Feet)
P	30	60	---	---	105	100-20-100	2.5
SU		100	---	---		100-35-100	3.0
BUS		140	---	---		100-35-100	4.0
WB-40		150	---	---		100-35-100	5.0
WB-50		200	---	---		180-45-180	8.0
P	45	50	---	---	120	100-20-100	2.0
SU		75	---	---		100-30-100	3.0
BUS		80	---	---		100-30-100	4.0
WB-40		120	---	---		120-30-120	6.0
WB-50		170	200-100-200	3.0		180-40-180	8.5
P	60	40	---	---	135**	100-20-100	1.5
SU		60	---	---		100-30-100	4.0
BUS		70	---	---		100-30-100	5.0
WB-40		90	---	---		120-30-120	6.5
WB-50		---	200- 75-200	5.5		160-35-160	7.0
P	75	35	100- 25-100	2.0	150**	75-18-75	2.0
SU		55	120- 45-120	2.0		100-30-100	4.0
BUS		80	120- 45-120	3.0		100-30-100	5.0
WB-40		85	120- 45-120	5.0		100-30-100	6.0
WB-50		---	150- 50-150	6.0		160-35-160	7.0
P	90	30	100- 20-100	2.5	180	50-15- 50	0.5
SU		50	120- 40-120	2.0	U-Turn	100-30-100	1.5
BUS		85	120- 40-120	4.0		130-25-130	8.0
WB-40		---	120- 40-120	5.0		100-20-100	9.5
WB-50		---	180- 60-180	6.0		130-25-130	9.5

*With no encroachment on adjacent lanes.
**Angle of turn substandard.

Note: Metric Conversion Unit is 1 ft. - 0.3 m.

Straight tapered wedges having a taper of about 15:1 may be substituted for longer radius portions of the 3-centered compound curves.



BASIC FORMULAS

$$\Delta_1 = \Delta - 2\Delta_2 \qquad \Delta_2 = \text{vers}^{-1}\frac{P}{R_2 - R_1}$$

OR, IF ARC LENGTHS OF THE 3 CURVES ARE EQUAL ($L_1 = L_2$), THEN

$$\Delta_2 = \frac{\Delta R_1}{2R_1 + R_2} \text{ AND } \Delta_1 = \frac{\Delta R_2}{2R_1 + R_2}$$

$$P = (R_2 - R_1)\,\text{vers}\,\Delta_2$$

$$T_e = \left[R_2 \sin\frac{\Delta}{2} - (R_2 - R_1)\sin\frac{\Delta_1}{2}\right]\sec\frac{\Delta}{2}$$

$$LC = 2\,T_e \cos\frac{\Delta}{2} = 2\left[R_2 \sin\frac{\Delta}{2} - (R_2 - R_1)\sin\frac{\Delta_1}{2}\right]$$

$$L_1 = \frac{\Delta_1 R_1}{57.2958} \qquad L_2 = \frac{\Delta_2 R_2}{57.2958}$$

$$T_1 = R_1 \tan\frac{\Delta_1}{2} \qquad T_2 = R_2 \tan\frac{\Delta_2}{2}$$

PROPERTIES OF
3-CENTERED CURVES

FIGURE 6-J

1. Rural

In general, the following minimum edge of pavement designs for turns may be considered in rural areas.

a. Intersections of local roads - SU design vehicle.

b. Intersections of collector roads with local roads or of 2 collector roads - SU design vehicle.

c. Intersections of arterial highways with collector roads or of 2 arterial highways - WB-40 or WB-50 design vehicles.

2. Urban

Curb return radii at intersections on urban streets should satisfy the requirements of vehicles using them to the extent practicable 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.

a. Local Streets - 28 feet (8.54 m).

b. Dead End Streets - 23 feet (7.02 m).

c. Collector Streets (residential area) - 28 feet (8.54 m).

d. Collector Streets (industrial area) - 38 feet (11.59 m).

e. Arterial Streets - 38 feet (11.59 m).

Where streets of different width intersect each other, the curb return radii of the lower facility should be utilized.

6-04.4 Lane Widths

Widths of roadways for minimum turns may vary from 14 to 38 feet (4.27 to 11.59 m), depending upon design vehicle and the angle of turn. Minimum widths of turning paths measured from curb to curb are given in Table 6-4 for different vehicles and angle of turn.

Table 6-4

Width of Lane (Feet)

Angle of Turn (Degrees)	Design Vehicle		
	P	SU	WB50
90	14	18	20
105	15	22	30
120	16	24	34
135	16	26	35
150	16	30	38

6-05 CHANNELIZATION

6-05.1 General

Where the inner edges of pavement for right turns at intersections are designed to accommodate semitrailer combinations, or where the design permits passenger vehicles to turn at speeds of 15 mph or more, the pavement area at the intersection may become excessively large for proper control of traffic. To avoid this condition a corner island, curbed or painted, should be provided to form a separate turning roadway (a connecting roadway for traffic turning between two intersection legs).

At-grade intersections having large paved areas, such as those with large corner radii and those at oblique angle crossings, permit and encourage hazardous, uncontrolled vehicle movements, require long pedestrian crossings, and have unused pavement areas. Even at a simple intersection, appreciable areas may exist on which some vehicles can wander from natural and expected paths. Conflicts may be reduced in extent and intensity by layout design to include islands. An at-grade intersection in which traffic is directed into definite paths by islands is termed a channelized intersection.

Channelization of at-grade intersections is the separation or regulation of conflicting traffic movements into definite paths of travel by use of pavement markings, raised islands, or other suitable means to facilitate the safe and orderly movement of both vehicles and pedestrians.

6-05.2 Islands

An island is a defined area between traffic lanes for control of vehicle movements. Islands also provide an area for pedestrian refuge and traffic control devices. Within an intersection, a

median or an outer separation is considered an island. This definition makes evident that an island is no single physical type -- it may range from an area delineated by barrier curbs to a pavement area marked by paint.

Islands generally are included in intersection design (channelization) for one or more of the following purposes:

1. Separation of conflicts;

2. Control of angle of conflict;

3. Reduction in excessive pavement areas;

4. Regulation of traffic and indication of proper use of intersection;

5. Arrangements to favor a predominant turning movement;

6. Protection of pedestrians;

7. Protection and storage of turning and crossing vehicles;

8. Location of traffic control devices.

Islands generally are either elongated or triangular in shape, and are situated in areas normally unused as vehicle paths. The dimensions depend on the particular intersection design. Islands should be located and designed to offer little hazard to vehicles, be relatively inexpensive to build and maintain, and occupy a minimum of roadway space but yet be commanding enough that motorists will not drive over them. Island details depend on particular conditions and should be designed to conform to the general principles that follow.

Curbed islands are sometimes difficult to see at night because of the glare from oncoming headlights or from distant luminaires or roadside businesses. Accordingly, where curbed islands are used, the intersection should have fixed-source lighting.

When various intersections are involved in a given project and the warrants are sufficiently similar, a common geometric design for each intersection should be used. This design approach will enhance driver expectancy. The designer should also refer to Part V of the Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD) for design guidance.

Painted, flush medians and islands may be preferred to the curbed type under certain conditions including the following: in lightly developed areas; at intersections where approach speeds are relatively high; where there is little pedestrian traffic; where fixed-source lighting is not provided; and where signals, signs, or lighting standards are not needed on the median or island.

Islands may be grouped into 3 major functional classes: (1) channelizing islands designed to control and direct traffic movement, usually turning, (2) divisional islands designed to divide opposing or same-direction traffic streams, usually through movements, and (3) refuge islands to provide refuge for pedestrians. Most islands combine 2 or all of these functions.

1. Size

Island sizes and shapes vary materially from one intersection to another. Islands should be large enough to command attention. The smallest curbed island that normally should be considered is one that has an area of approximately 50 square feet (15.25 m^2) for urban streets, and 75 square feet (22.8 m^2) for rural intersections. However, 100 square feet (30.5 m^2) is the desirable size for islands used in both urban and rural areas.

2. Delineation

Islands may be delineated or outlined by a variety of treatments, depending upon their size, location and function. Three common ways of delineating an island are: (1) raised islands outlined by curbs; (2) islands delineated by pavement markings placed upon all-paved areas; and (3) non-paved areas formed by pavement edges, possibly supplemented by delineators on posts.

Raised islands with curbs are generally used in the urban areas. Delineation by pavement markers on a paved surface may be used in urban areas where speeds are low and space is limited, and in rural areas where approach speeds are high. Delineation by non-paved areas as in 3 above is generally used in rural areas. In this case, landscaping in the form of ground cover or grass may be appropriate.

3. Approach-End Treatment

The approach end of a curbed island should be conspicuous to approaching drivers and should be definitely clear of vehicle paths, physically and visually, so that drivers will not veer from the island.

The nose offset should be 4 to 6 feet (1.22 to 1.83 m) from the normal edge of through pavement and 2 to 3 feet (0.62 to 0.92 m) from the pavement edge of the turning roadway. Figure 6-K shows the recommended design details of curbed triangular islands under conditions of no shoulder on the approach roadways.

On highways with auxiliary lanes or shoulders, the corner islands should be offset the full auxiliary lane or shoulder width on both the main highway and the cross road.

4. Divisional Islands

The most common type of elongated divisional island is the median island, for which a design guide is illustrated on Figure 6-L.

6-05.3 Turning Roadways with Corner Island

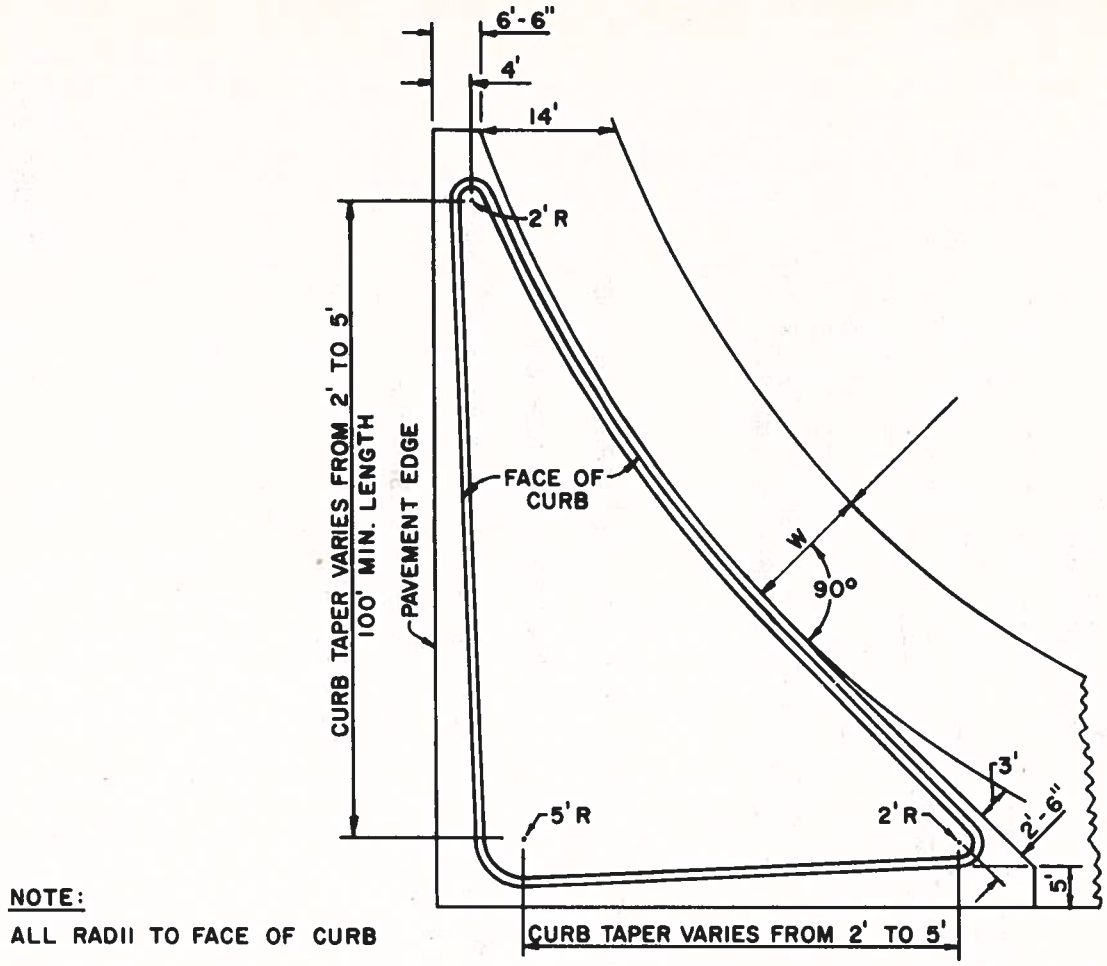
A turning roadway is a connecting roadway between two intersection legs. Turning roadways vary in form from a simple right-turn lane with a small channelizing island, to a sweeping ramp at a directional interchange.

Turning roadways at intersections are used where it is necessary to accommodate large trucks or buses without encroaching on adjacent lanes, to permit passenger vehicles to turn at high speeds of 15 mph or more, at skewed intersections, and at important intersections or ramp terminals. Turning roadways facilitate traffic control through separation of the various movements, shorten travel paths, reduce pavement areas within the intersection, and provide islands which serve as pedestrian refuge areas.

The principal control for the design of turning roadways is the alignment of the inner edge of pavement and the width of the roadway channel to accommodate the design vehicle. Minimum design values for turning roadways with corner islands are given in Table 6-5.

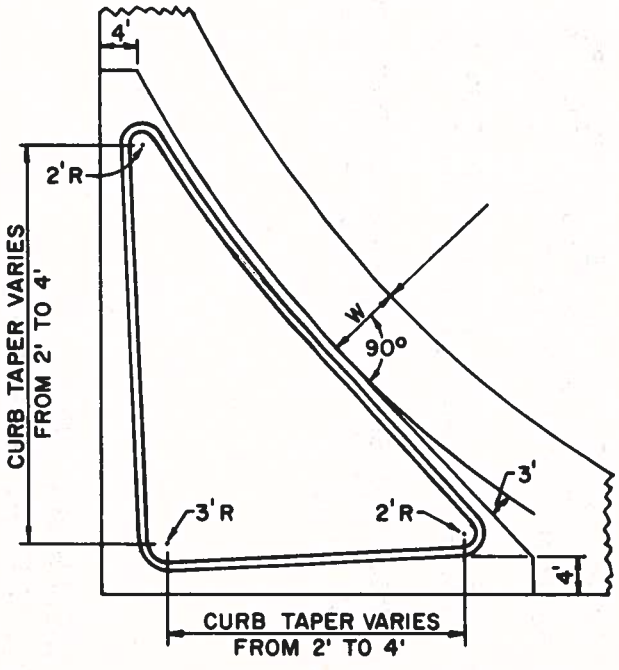
6-05.4 Speed-Curvature Relations on Turning Roadways

In intersection design, the feasible curves often are of limited radii and length. However, drivers anticipate sharp curves and accept operations with higher side friction and, therefore, shorter minimum radii curves on flatter superelevation rates can be used in intersection design as compared to open highway designs.

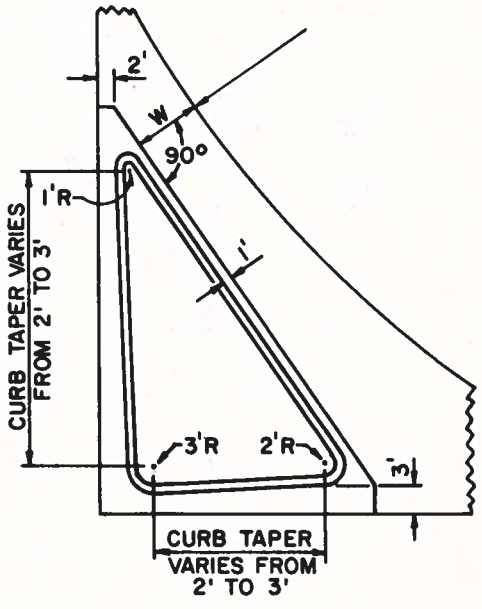


NOTE:
ALL RADII TO FACE OF CURB

LARGE ISLAND



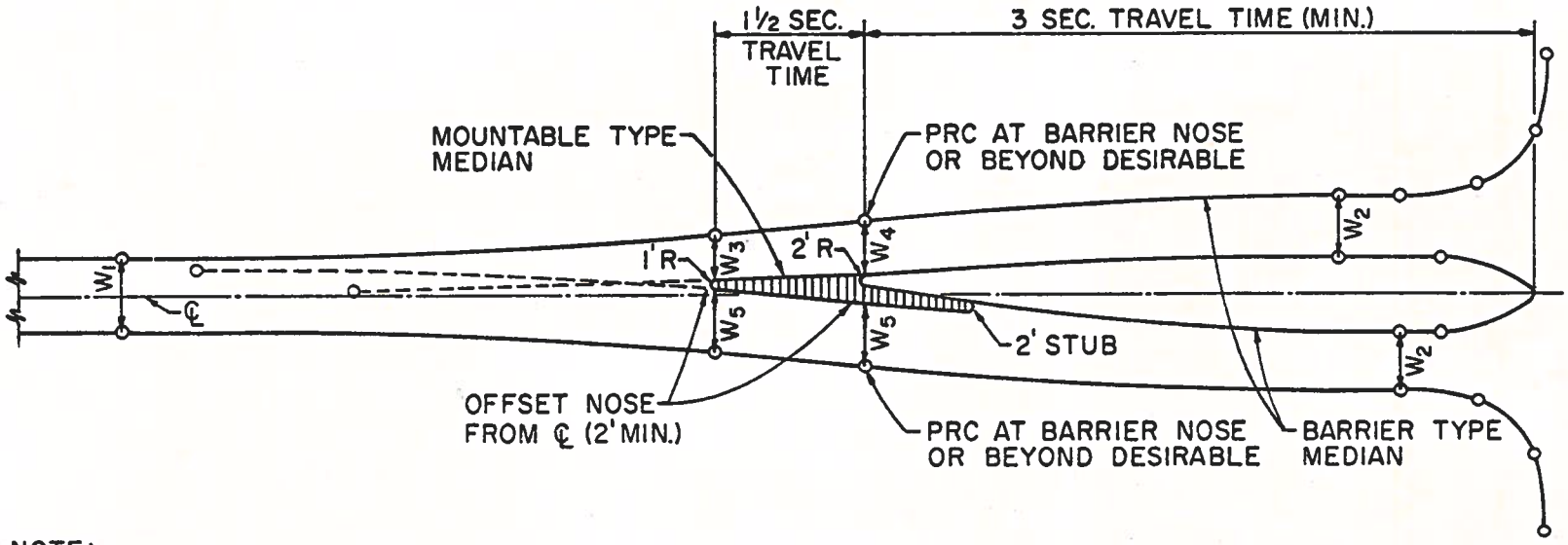
INTERMEDIATE ISLAND



SMALL ISLAND

DETAIL OF ISLANDS

FIGURE 6-K



NOTE:

1. WIDTH OF ISLAND CONTROLLED BY NOSE PLACEMENT AS DETERMINED BY CONTROL RADII AND ANGLE OF INTERSECTION OR ADDED LEFT TURN LANE.
2. VERTICAL AND HORIZONTAL ALIGNMENT ON APPROACH TO PROVIDE MINIMUM STOPPING SIGHT DISTANCE.
3. CONTROL DIMENSION, W, SHOULD BE FROM EDGE TO EDGE OF PAVEMENT AND SHOULD NOT INCLUDE GUTTER FLAG.

W_1 = UNDIVIDED APPROACH WIDTH

W_2 = DIVIDED APPROACH WIDTH

$W_3 = \frac{W_1}{2}$ OR 14' WHICHEVER IS LARGER

$W_4 = \frac{W_3 + W_2}{2}$ DESIRABLE

$W_5 = W_2 + 1'$

DESIGN CRITERIA FOR CHANNELIZATION APPROACHES

FIGURE 6-L

Table 6-5

Minimum Designs for Turning Roadways
(With Islands)

Angle of Turn	*Design Classification	3-Centered Compound Curve		Width of **Lane	Approximate Island Size
		Radii	Offset		
Degrees		Feet	Feet	Feet	Square Feet
75	A	150-75-150	3.5	14	60
	B	150-75-150	5.0	18	50
	C	180-90-180	3.5	20	50
90	A	150-50-150	3.0	14	50
	B	150-50-150	5.0	18	80
	C	180-65-180	6.0	20	125
105	A	120-40-120	2.0	15	70
	B	100-35-100	5.0	22	50
	C	180-45-180	8.0	30	60
120	A	100-30-100	2.5	16	120
	B	100-30-100	5.0	24	90
	C	180-40-180	8.5	34	220
135***	A	100-30-100	2.5	16	460
	B	100-30-100	5.0	26	370
	C	160-35-160	9.0	35	640
150***	A	100-30-100	2.5	16	1400
	B	100-30-100	6.0	30	1170
	C	160-35-160	7.1	38	1720

*A -- Primarily passenger vehicles; permits occasional design single-unit truck to turn with restricted clearances.

B -- Provides adequately for SU; permits occasional WB-50 to turn with slight encroachment on adjacent traffic lanes.

C -- Provides fully for WB-50.

Note: Asymmetric 3-centered compound curves and straight tapers with a simple curve can also be used without significantly altering the width of pavement or corner island size.

**Edge to edge desirable, face to face minimum.

***Substandard Angle of Turn.

Table 6-6 shows minimum edge of pavement radii for different design speeds and superelevation rates.

Table 6-6

Minimum Radii for Pavement Edge Based on Design Speed

Design turning speed, mph	15	20	25	30	35	40
Minimum Radius - Feet (Meters)	50 (15)	90 (27)	150 (45)	230 (70)	310 (100)	430 (150)
Minimum Superelevation (e)	.02	.03	.04	.06	0.08	0.08
Desirable Superelevation (e)	.05	.05	.07	.08	0.10	0.10

The principles of superelevation runoff, discussed in Section 4, BASIC GEOMETRIC DESIGN ELEMENTS, apply generally to superelevation on intersection curves. Primarily, the control of the rate of change of cross slope is that of riding comfort and appearance.

6-05.5 Speed-Change Lanes

A speed-change lane is an auxiliary lane, including tapered areas, primarily for the acceleration or deceleration of vehicles entering or leaving the through-traffic lanes. The terms "speed-change lane", "deceleration lane", or "acceleration lane", as used herein, apply broadly to the added pavement joining the traveled way of the highway or street with that of the turning roadway and do not necessarily imply a definite lane of uniform width. A speed-change lane should be of sufficient width and length to enable a driver to maneuver a vehicle onto it properly and once onto it, to make the necessary change between the speed of operation on the highway or street and the lower speed on the turning roadway.

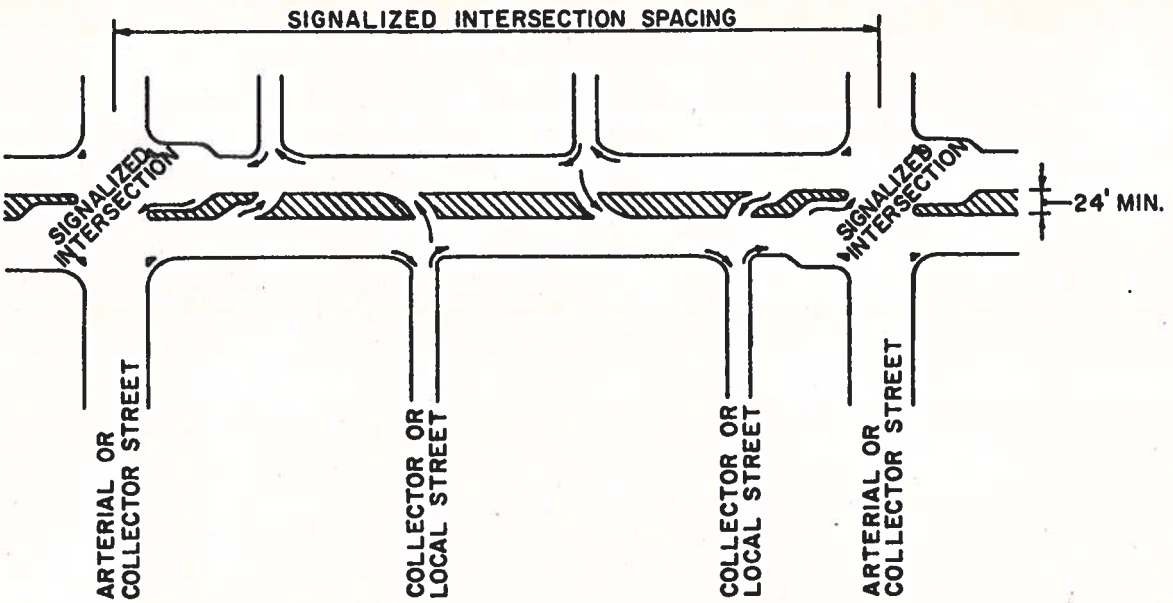
Deceleration lanes are always advantageous, particularly on high-speed roads, because the driver of a vehicle leaving the highway has no choice but to slow down on the through-traffic lane if a

deceleration lane were not provided. On the other hand, acceleration lanes are not always necessary at stop controlled intersections where entering drivers can wait for an opportunity to merge without disrupting through traffic. Acceleration lanes are advantageous on roads without stop control and on all high-volume roads even with stop control where openings between vehicles in the peak-hour traffic streams are infrequent and short. (See Section 11, INTERCHANGES, for design guidelines relative to lengths of deceleration and auxiliary lanes.)

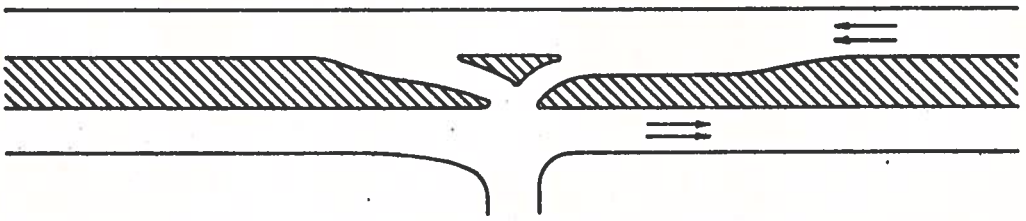
6-05.6 Median Openings

Median openings in divided roadways are provided to permit intended movements only. Figure 6-M shows application of median openings to control the various types of movements along a divided roadway.

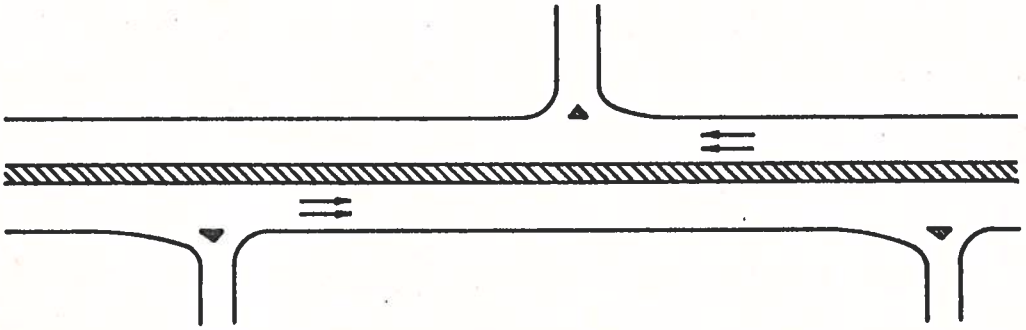
Median openings without left turn lanes generally have application only at intersections with very minor cross roads. A simple median opening of minimum design is normally sufficient on divided arterial or collector roads on which design speeds are less than 45 mph, traffic volumes moderate, and left turn movements less than 35 vehicles per hour.



SEPARATION OF LEFT TURNS TO AND FROM A DIVIDED ROADWAY



LEFT TURNS TO AND FROM THE ARTERIAL AT THE SAME POINT

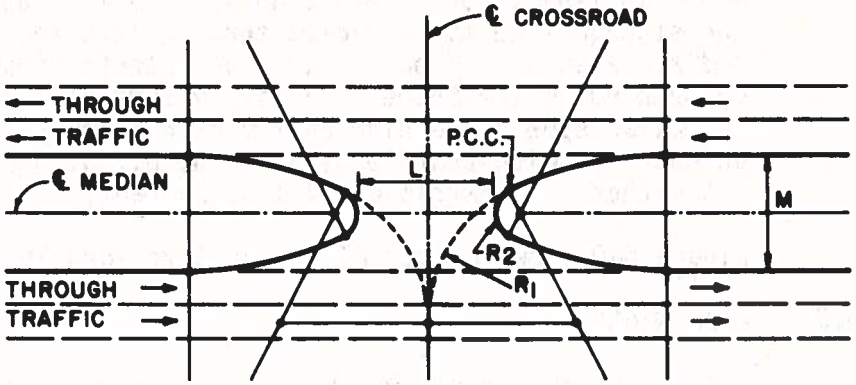


"T" INTERSECTIONS AT INTERMEDIATE UNSIGNALIZED LOCATIONS ON ARTERIAL

APPLICATION OF MEDIAN AND MEDIAN OPENINGS TO CONTROL ACCESS POINTS

FIGURE 6-M

Figure 6-N shows a simple median opening.



M – MEDIAN WIDTH. (DESIRABLY 24 FEET (7.3 M.) OR GREATER FOR UNSIGNALIZED INTERSECTION; 14 FEET (4.3 M.) FOR SIGNALIZED INTERSECTION.)

R_1 – CONTROL RADIUS

R_2 – $\frac{1}{5}$ M BUT NOT LESS THAN 2 FEET (0.62 M.)

L – MEDIAN OPENING, MINIMUM 40 FEET (12 M.)

MEDIAN OPENING FIGURE 6-N

The length (L) of the median opening should equal the full roadway of the cross road, shoulder to shoulder. The control radius (R_1) is determined by the design vehicle as follows:

P and SU	- 40 feet (12 m).
SU, BUS, WB-40	- 50 feet (15 m).
WB-50	- 75 feet (25 m).

On urban divided roadways, median openings for U-turns should not be provided. U-turn movements may be permitted at signalized intersections where there is sufficient pavement width to accommodate the movement. Provisions for U-turns should be made on rural divided roadways where intersections are spaced in excess of one-half mile apart. Median widths in such cases should be at least 30 feet (9.1 m) to provide adequate protection for the vehicle executing the U-turn movement.

6-06 MEDIAN LEFT-TURN LANE

6-06.1 General

A median lane is provided at an intersection as a deceleration and storage lane for vehicles turning left to leave the highway. Median lanes should be provided at intersections and other median openings where there is a high volume of left turns, or where vehicular speeds are high on the main roadway. Median lanes may be operated with traffic signal control, with stop signs, or without either, as traffic conditions warrant.

Figure 6-0 shows a typical median left turn lane.

6-06.2 Lane Width

Left turn lane should be 10 feet (3.0 m) wide and desirably 12 feet (3.6 m) wide. The lane width may be measured from the curb face to the channelizing lane line.

Median widths of 20 to 25 feet (6.1 to 7.6 m) or more are desirable at intersections with single left-turning lane, but widths of 16 to 18 feet (4.9 to 5.5 m) are acceptable.

6-06.3 Length

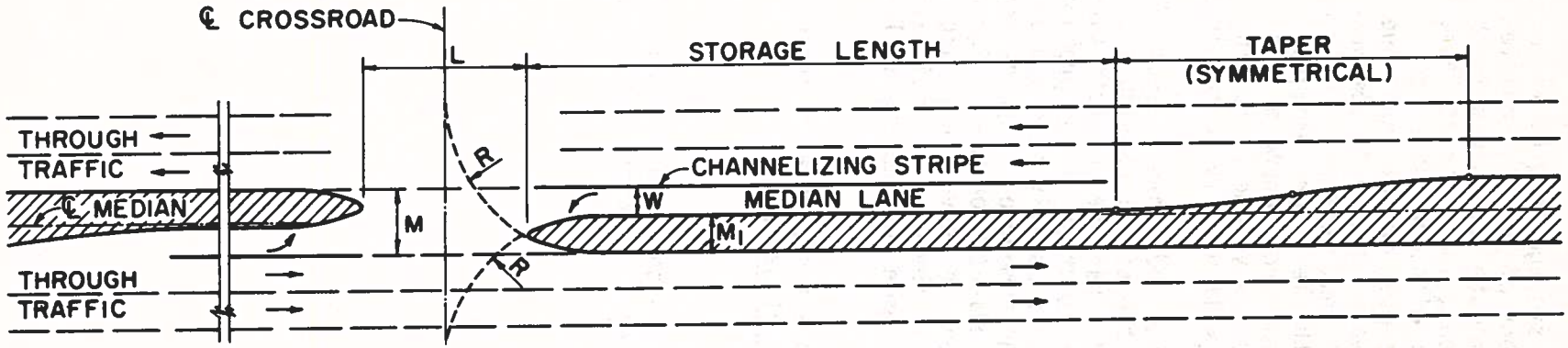
Desirably, the total length of the left turn lane is the sum of deceleration length, storage length, and entering taper.

1. Deceleration

   Determination of deceleration distances for high-speed facilities is covered in Section 11, INTERCHANGES. For normal urban design, common practice is to accept a moderate amount of deceleration within the through lanes and to consider the taper as a part of the deceleration length. Therefore, for purposes of this Manual, the deceleration length is ignored and only taper and storage lengths are considered.

2. Storage Length

   The median left turn lane should be sufficiently long to store the number of vehicles likely to accumulate during a critical period. The storage length should be liberal to avoid the possibility of left-turning vehicles stopping in the through lanes. The storage length should be sufficiently long so that the entrance to the auxiliary lane is not blocked by vehicles standing in the through lanes waiting for a signal change or for a gap in the opposing traffic flow.



NOTES:

1. MEDIAN WIDTH (M)
2. MINIMUM NARROWED END (M_1) – 4 FT.
3. MEDIAN LANE WIDTH (W) – 10 FT. TO 12 FT. (3.6 M.) DESIRABLE
4. TAPER LENGTH
5. RADII OF DESIGN VEHICLE (R)

TYPICAL MEDIAN LANE DESIGN

FIGURE 6-0

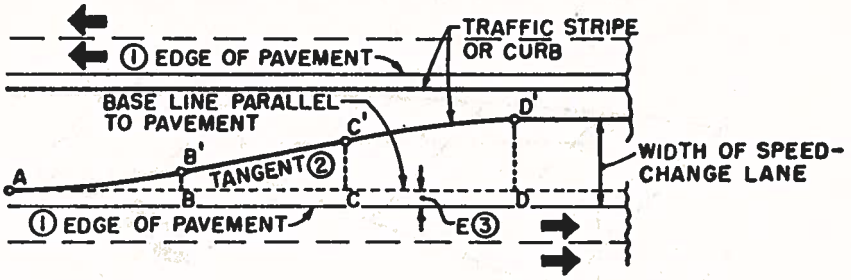
At unsignalized intersections the storage length, exclusive of taper, may be based on the number of turning vehicles likely to arrive in an average 2 minute period within the peak hour. As a minimum requirement, space for at least 2 passenger cars should be provided; with over 10 percent truck traffic, provision should be made for at least one car and one truck. The 2 minute waiting time is somewhat arbitrary, and some other interval that depends largely on the opportunities for completing the left-turn maneuver might be used. These intervals, in turn, depend on the volume of opposing traffic. Where the volume of turning traffic is high, a traffic signal will usually be required.

At signalized intersections the required storage length depends on the signal cycle length, the signal phasing arrangement, and the rate of arrivals and departures of left-turning vehicles. The storage length is a function of the probability of occurrence of events and should usually be based on one- and one-half to 2 times the average number of vehicles that would store per cycle, which is predicated on the design volume. This length will be sufficient to serve heavy surges that occur from time to time. As in the case of unsignalized intersections, provision should be made for storing at least 2 vehicles.

Traffic signal design fundamentals are discussed further in the Manual on Uniform Traffic Control Devices.

3. Taper

Figure 6-P shows the taper length and shape for median speed change lane where the median is curbed or design speeds exceed 50 mph.



AD = LENGTH OF TAPER
AB = BC = CD = 1/3 AD
AB' & C'D' ARE PARABOLIC CURVES

DATA FOR POINT B'
DATA FOR POINT C'

LENGTH OF TAPER-FEET			OFFSET DISTANCE		
60	90	120	DD' = 10'	DD' = 11'	DD' = 12'
DISTANCE FROM POINT "A"					
—	—	—	0	0	0
5	7.5	10	0.16	0.17	0.19
10	15.	20	0.62	0.69	0.75
15	22.5	30	1.41	1.55	1.69
B' 20	30.	40	2.50	2.75	3.00 B'
30	45.	60	5.00	5.50	6.00
C' 40	60.	80	7.50	8.25	9.00 C'
45	67.5	90	8.59	9.45	10.31
50	75.	100	9.38	10.31	11.25
55	82.5	110	9.84	10.83	11.81
60	90.	120	10.00	11.00	12.00

NOTES

① THE TABLE GIVES OFFSETS FROM A BASE LINE PARALLEL TO THE EDGE OF PAVEMENT AT INTERVALS MEASURED FROM POINT "A". ADD "E" FOR MEASUREMENTS FROM EDGE OF PAVEMENT.

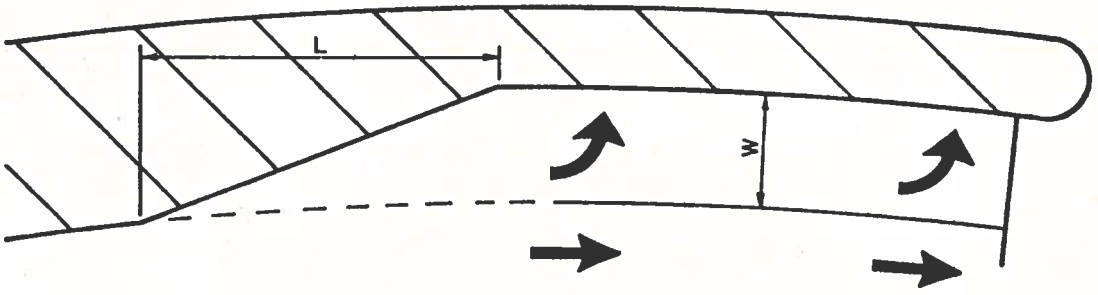
② WHERE EDGE OF PAVEMENT IS A CURVE, NEITHER BASE LINE NOR TAPER BETWEEN B & C WILL BE A TANGENT. USE PROPORTIONAL OFFSETS FROM B TO C.

③ THE ADOPTED OFFSET "E" IS USUALLY 2' ALONG OUTSIDE EDGE OF PAVEMENT BUT MAY VARY IN SOME CASES.

TAPER FOR MEDIAN SPEED-CHANGE LANES

FIGURE 6-P
(applicable where median is curbed)

Figure 6-Q shows the taper shape for a left-turn storage lane where delineation is achieved by striping and design speed is 50 mph or less.



L = 8 : 1 FOR DESIGN SPEEDS UP TO 30 MPH

L = 15 : 1 FOR DESIGN SPEEDS 35 MPH TO 50 MPH

(applicable where median is paved)

LEFT-TURN STORAGE LANE USING PAINTED DELINEATION

FIGURE 6-Q

6-07 TRAFFIC CONTROL DEVICES

Traffic control devices are necessary for regulating, warning and guiding traffic, and are a primary determinant in the safe and efficient operation of the intersection. It is essential that intersection design be accomplished simultaneously with the development of traffic control plans to insure that sufficient space is provided for proper installation of traffic control devices.

An intersection requiring traffic signal control is best designed by considering jointly the geometric design, capacity analysis, design-hour volumes, and physical controls. Details on the design and location of most forms of traffic control devices, including the general warrants, are given in Part IV of the Manual on Uniform Traffic Control Devices.

6-08 OTHER CONSIDERATIONS

6-08.1 Parking Restrictions at Intersections

Vehicular parking should not be permitted within the immediate limits of at-grade intersections.

6-08.2 Lighting at Intersections

Lighting affects the safety of highway and street intersections and the ease and comfort of traffic operations. In urban and suburban areas where there are concentrations of pedestrians and roadside and intersectional interferences, fixed-source lighting tends to reduce accidents. Whether or not rural at-grade intersections should be lighted depends on the planned geometrics and the turning traffic volumes involved. Intersections that generally do not require channelization are seldom lighted. However, for the benefit of non-local highway users, lighting at rural intersections is desirable to aid the driver in ascertaining sign messages during non-daylight periods.

Intersections with channelization, particularly multiple-road geometrics, should include lighting. Large channelized intersections especially need illumination because of the higher range of turning radii that are not within the lateral range of vehicular headlight beams. Vehicles approaching the intersection also must reduce speed. The indication of this need should be definite and visible at a distance from the intersection that may be beyond the range of headlights. Illumination of at-grade intersections with fixed-source lighting accomplishes this need.

The planned location of intersection luminaire supports should be designed to present the least possible hazard to out-of-control vehicles. The breakaway support base should not be used within the limits of an at-grade intersection, particularly in densely developed areas with adjacent sidewalks. When struck, these light standards could be hazardous to pedestrians and compound damage to adjacent property and other moving or non-moving vehicles. Because of the lower operating speeds within an intersection, there is much less chance of fatalities to vehicle occupants striking a fixed light standard.

SECTION 7.

L O C A L R O A D S A N D S T R E E T S

7-01 GENERAL

Local streets or roads are generally two-lane roadways providing access to abutting land from higher classification roadways. The cul-de-sac street and dead end road represent the lowest type of local street or road.

County agencies differ on minimum design criteria for the lower types of local street or road, e.g., the roadway widths for cul-de-sac streets and turnaround areas vary among the various Counties. Therefore, the designer should consult the appropriate County ordinance and design criteria.

The use of more liberal values of design dimensions than the guidelines described herein is recommended where economically feasible. In all cases, every effort should be made to attain the best possible alignment, grade, sight distance and proper drainage that are consistent with the terrain, present and future development, and available funds.

Safety is an important factor in all roadway improvements. Especially on rural roads, where traffic volumes are low, drivers tend to move at higher speeds and, therefore, as much clear roadside as possible should be provided. The judicious use of flatter slopes, guardrails, and warning devices helps to achieve roadside safety. Proper placement of utility facilities also assists in achieving safer roadsides.

7-02 LOCAL RURAL ROADS

7-02.1 General

The local rural road may serve a cluster of several homes in an isolated farming locality, or a subdivision in a rural town or village. Generally, the local roads are paved, two-lane facilities with graded shoulders. In some areas of the State, local rural roads may be paved for only one lane of traffic, about 10 feet (3.05 m), although the right-of-way can easily accommodate 2 full paved lanes and some shoulder area.

7-02.2 Design Parameters

1. Design Traffic Volume

Generally, traffic volume is not a design criterion for rural local roads.

2. Design Speed

Geometric design features should be consistent with a design speed appropriate for the conditions. Low design speeds are generally applicable to roads in mountainous terrain, or in built-up areas such as a subdivision. High design speeds are generally applicable to roads on flat, open terrain, away from roadside development.

Design speeds on rural local roads could vary from 35 mph in flat, open areas to 15 mph in winding, mountainous terrain. Desirable design speed for local rural roads is 30 mph.

3. Level of Service

This criterion is not considered in the design of local rural roads.

4. Design Vehicles

The minimum turning radius should be based upon a single unit truck (SU) or school bus.

7-02.3 Geometric Design Elements

The detail requirements for geometric design of roadways are covered in Section 4, BASIC GEOMETRIC DESIGN ELEMENTS, and should be consulted for appropriate guidelines in the design of local rural roads.

1. Sight Distance

Minimum stopping and passing sight distances are a direct function of design speed. Criteria applicable to local rural roads are covered in Section 4. Passing sight distance should be provided where feasible on local roads more than 2 miles in length.

2. Alignment

For local roads of one mile or more, the following is applicable. Alignment between control points should be designed to

as high a standard as possible consistent with the environmental impacts, topography, terrain, design speed, and the amount of reasonably obtainable right-of-way. Sudden changes between curves of widely different radii, or between long tangents and sharp curves should be avoided. The design should include passing opportunities. For local roads in built-up areas, the alignment should follow the terrain as close as possible.

3. Grades

Grades may vary from 0.4 percent in flat areas to 15 percent in mountainous terrain. Special terrain conditions may warrant steeper grades. Individual Counties may permit variances for these special conditions.

4. Superelevation

Superelevation commensurate with the design speed and curvature should be provided. Superelevation runoff should be adjusted to provide smooth riding qualities, good drainage, and good appearance. In built-up areas, normal roadway crown may be adequate because of the low design speeds.

7-02.4 Cross Section Elements

Detail requirements of cross section elements are discussed in Section 5, CROSS SECTION ELEMENTS, and should be referred to for appropriate guidelines in the design of rural local roads.

1. Pavement Surface

Pavement surface should be either of bituminous asphalt concrete, or portland cement concrete. However, where the roadway grade line is in excess of 12 percent, portland cement concrete surface should be used.

2. Cross Slope

Pavement cross slope shall be a minimu of 2 percent to provide for proper drainage of the pavement surface.

3. Lane Width and Shoulder Area

Lanes should be a minimum of 10 feet (3.05 m) wide, and desirably 12 feet (3.66 m) wide. A cleared shoulder area of at least 4 feet (1.22 m) should be provided in open areas.

4. Right-of-Way

The procurement of rights-of-way to widths that accommodate the construction, adequate drainage, and proper maintenance of the highway is a very important part of the overall design. Wide rights-of-way permit the construction of gentle side slopes, resulting in more safety for the motorist, and allowing easier and more economical maintenance. The procurement of sufficient right-of-way at the time of the initial improvement permits the future widening of the roadway and the widening and strengthening of the pavement at reasonable costs as traffic demand increases. The designer should refer to the appropriate County ordinance and design criteria for the local road right-of-way requirements.

7-02.5 Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the current AASHTO Standard Specifications for Highway Bridges.

All approach roadway widths should be carried across the bridge structure.

7-02.6 Clearance to Obstruction

1. Vertical Clearance

A minimum of 14.5 feet (4.4 m) vertical clearance over the entire roadway shall be provided. This provides a 3 to 6 inch (1.18 to 2.36 cm) allowance for future resurfacing of the road.

2. Horizontal Clearance

A clear roadside recovery area, appropriately graded and having gentle slopes and rounded cross-sectional design, should be considered. Exception may be made (1) in cut sections where fixed objects are sufficiently far up the cut slope that there is little likelihood of their being struck, and (2) where guardrail protection is being provided for other purposes. The recovery area should be clear of all unyielding objects such as trees or unyielding sign supports, utility poles, light poles, and any other fixed objects that might severely damage an out-of-control vehicle.

To the extent feasible, where another highway structure passes over, the structure should be designed so that the pier or abutment supports have lateral clearance as great as the clear roadside area on the approach roadway.

Where it is not feasible to carry the full width approach roadway across a bridge, an appropriately transitioned guardrail should be securely anchored to the bridge rail.

At selected locations, such as the outside of a sharp curve, a broader recovery area with greater horizontal clearances should be provided to any roadside obstruction.

7-02.7 Intersection Design

Refer to Section 6, AT-GRADE INTERSECTIONS, for appropriate intersection design guidelines.

7-03 LOCAL URBAN STREETS

7-03.1 General

The design features of local urban streets are governed by practical limitations to a greater extent than those of similar roads in rural areas. The two dominant design controls are (1) the type and extent of urban development with its limitations on rights-of-way, and (2) zoning or regulatory restrictions. Some streets primarily are land service streets in residential areas. In such cases, the overriding consideration is to foster a safe and pleasant environment. The convenience of the motorist is a secondary consideration. Other streets are land service only in part, and features of traffic service may be predominant.

On streets serving industrial or commercial areas, the vehicle dimensions, traffic volumes, and vehicle loads differ greatly from those on residential streets, and different dimensional and structural design values are appropriate. Here, safety and service to traffic usually are the major design controls. Where a particular design feature varies depending on the area served, viz., residential, commercial, or industrial, different guide values are enumerated for each condition. The designer should be apprised of County ordinances and regulations that affect certain design features.

The design parameters used in the design of urban local roads are discussed in detail in Section 2, DESIGN PARAMETERS.

7-03.2 Design Parameters

1. Design Traffic Volume

Traffic volume is not usually a major criterion in determining the geometric values to be used in designing residential streets. Traditionally, these streets are designed to a

standard two-lane cross section, but may be a four-lane cross section in certain urban areas, as governed by traffic volume, administrative policy, or other community considerations.

For streets serving industrial or commercial areas, however, traffic volume is a major factor. The design-hourly volume (DHV) projected to some future design year should be the basis of design. It usually is difficult and costly to modify the geometric design of an existing street unless provision is made at the time of initial construction. The design traffic should be that estimated for at least 10 years, and preferably 20 years, from the date of completion of construction.

2. Design Speed

For consistency in design elements, design speeds ranging from 20 to 30 mph may be used, depending on available right-of-way, terrain, adjacent development, and other area controls. On dead-end and cul-de-sac streets, a design speed of 15 mph may be used.

In the typical street grid, the closely spaced intersections usually limit vehicular speeds, making the effect of design speed of little significance. Design speeds exceeding 30 mph in residential areas may require longer sight distances and increased curve radii, which would be contrary to the basic function of a local street.

3. Level of Service

This criterion is not considered in the design of local rural roads.

4. Design Vehicles

In residential and commercial areas, the design vehicle should be the single unit truck (SU) and bus.

In industrial areas, the design vehicle should be the semi-trailer combination WB-50.

7-03.3 Geometric Design Elements

The detail requirements for geometric design of roadways are covered in Section 4, and should be consulted for appropriate guidelines in the design of urban local streets.

1. Sight Distance

The minimum stopping sight distance is a direct function of design speed. Criteria applicable to urban local streets are covered in Section 4. Passing sight distance is seldom applicable on local streets.

2. Alignment

Alignment in residential areas should fit closely with the existing topography to minimize the need for cuts or fills without sacrificing safety. There is an advantage in residential areas in purposely arranging the alignment to discourage through traffic. The alignment design should be such that the safety of the facility is not reduced.

Street alignment in commercial and industrial areas should be commensurate with the topography, but should be as direct as possible. Street curves should be designed with as large a radius curve as feasible, the minimum radius being 125 feet (38.12 m) for speed of 20 mph with zero superelevation.

3. Grades

The grades for local residential streets should be as flat as is consistent with the terrain. Grades should not exceed 15 percent in mountainous areas and no more than 4 percent in level areas. Special terrain conditions may warrant a variance from the maximum grade of 15 percent.

In commercial and industrial areas, the grades should not exceed 8 percent, and desirably it should not exceed 5 percent.

Grades shall not be less than 0.4 percent without special consideration for the handling of onsite drainage.

4. Superelevation

Superelevation is advantageous for street traffic operation, but in built-up areas, the combination of wide pavement areas, proximity of adjacent development, control of cross slope, profile for drainage, frequency of cross streets, and other urban features combine to make the use of superelevation impractical. Usually, superelevation is not provided on local streets in residential and commercial areas; it should be considered on local streets in industrial areas to facilitate safe operation.

7-03.4 Cross Section Elements

Detail requirements of cross section elements are discussed in Section 5, and should be referred to for appropriate guidelines in the design of urban local roads.

1. Pavement Surface

Pavement surface should be either of bituminous asphalt concrete or portland cement concrete. However, where the roadway grade line is in excess of 12 percent, portland cement concrete surface should be used.

2. Cross Slope

Pavement cross slope shall be a minimum of 2 percent to provide for proper drainage of the pavement surface.

3. Width of Roadway

Street lanes for moving traffic should be at least 10 feet (3.05 m) wide. Where feasible, they should be 11 feet (3.35 m) wide, and in industrial areas they should be 12 feet (3.66 m) wide. Where available or attainable width of right-of-way imposes severe limitations, 9 foot (2.75 m) lanes can be used in residential areas, as can 11 foot (3.35 m) lanes in industrial areas. Added turning lanes, where used at intersections, should be at least 9 feet (2.75 m) wide, and desirably 10 to 12 feet (3.05 to 3.66 m) wide, depending on the percentages of trucks.

Where needed and where severe limitations exist in residential areas, a parallel parking lane at least 7 feet (2.14 m) wide should be provided on one or both sides of the street, as the conditions of lot size and intensity of development may require. In commercial and industrial areas, parking lanes should be at least 9 feet (2.75 m) wide; usually they are provided on both sides of the street.

Parking lane width determinations in commercial and industrial areas should include consideration for use of the parking lane for moving traffic during the peak hour that may occur where industries have high employment concentrations.

Where curb and gutter sections are used, the gutter width may be included as a part of the parking lane width.

The minimum clear width for all new bridges on streets with curbed approaches should be the same as the curb-to-curb width of the approaches. For streets with shoulders and no curbs, the clear roadway width preferably should be the same as the approach roadway width. Sidewalks on the approaches should be carried across the structures.

4. Curbs

Streets normally are designed with curbs to allow greater use of available width and for control of drainage, protection of pedestrians, and delineation.

5. Right-of-Way

The right-of-way width should be sufficient to accommodate the ultimate planned roadway including median (if used), shoulder (if used), landscaping strip, sidewalks, utility strips in the border areas, and necessary outer slopes unless provided for by easement. The designer should refer to appropriate County ordinances and design criteria for local street right-of-way requirements.

7-03.5 Cul-De-Sac Streets and Dead End Streets

A local street open at one end only should have a special turning area at the closed end. This turning area desirably should be circular and have a radius appropriate to the types of vehicles expected. Each County agency has a design criteria for a cul-de-sac turnaround area, or an "L" or "T" type turnaround area.

A dead end street generally has a maximum length of 200 to 300 feet (60 to 90 m) for special design criteria. Streets longer than this length(s) must be designed as through local streets. The designer should refer to the appropriate County ordinance and regulations when designing cul-de-sac or dead end streets.

7-03.6 Sidewalks

Sidewalks used for pedestrian access to schools, parks, shopping areas, and transit stops, and placed along all streets in commercial areas, should be provided along both sides of the street. In residential areas, sidewalks should be provided on at least one side of all streets. The sidewalks should be located as far as practicable from the traffic lanes and usually close to the right-of-way lines.

Clear sidewalk width should be 4 feet (1.22 m) minimum; widths of 8 feet (2.44 m) or greater may be needed in commercial areas. If roadside appurtenances are situated on the sidewalk adjacent to the curb, additional width is required to secure the clear width.

Wheelchair ramps shall be provided at crosswalks to accommodate physically handicapped persons. Wheelchair ramps are discussed in Section 5.

7-03.7 Driveways

A driveway is an access constructed within the public way, connecting the public roadway with adjacent property and intended to be used in such a way that the access into the adjacent property will be complete and will not cause the blocking of any sidewalk border area or street roadway.

Some of the principles of intersection design apply directly to driveways. One important feature is the elimination of large graded or paved areas adjacent to the traveled way, which allow drivers to enter or leave the street randomly.

Driveway design criteria are discussed in Section 5.

7-03.8 Clearance to Obstructions

1. Vertical Clearance

A minimum vertical clearance of 14.5 feet (4.42 m) shall be provided over the entire roadway. This provides a 3 to 6 inch (1.18 to 2.36 cm) allowance for future resurfacing of the road.

2. Horizontal Clearance

Generally, on all streets, a clearance of at least 2.0 feet (0.62 m) should be provided between the curb face or shoulder edge and obstructions such as utility poles, lighting poles, and fire hydrants. In areas of dense pedestrian traffic, the construction of barrier-type curbing and adjacent street fixtures and planting of trees aid in protecting the high-volume pedestrian traffic from vehicles that go out of control.

Trees may be minimal hazards along local streets where speeds are low and barrier curbs exist. Generally, prudence should be exercised and careful analysis should weigh the benefits against the possibly adverse effects that tree removal could have on the roadside environment.

Guardrail is not used extensively on local streets, except where there is a hazard to motorists and pedestrians, such as along sections with steep side slopes, and at the approaches to overcrossing structures.

7-03.9 Intersection Design

Adequate sight distance shall be provided at all local street intersections. Section 6 discusses in detail the design of at-grade intersections.

7-03.10 Other Design Considerations

1. Drainage

Drainage is an important consideration in an urban area because of high runoff coefficients and the potential hazard to property and life. Surface flow from adjacent tributary areas is intercepted by the street system. There, it is collected and retained within the roadway by curbs and gutters that convey the flow to the main drainage system. Where drains are available under or near the roadway, the flow is removed at frequent intervals from the street cross section via curb-opening basins into underground drainage lines.

2. Street Lighting

Good visibility under day or night conditions is one of the fundamental requirements enabling motorists to move on roadways in a safe and coordinated manner. Properly designed and maintained street lighting will produce quick, comfortable, and accurate visibility at night, which will facilitate and encourage both vehicular and pedestrian traffic. Thus, where adequate illumination is provided, efficient night use can be made of the large investment in roadway construction and motor vehicles. Determinations on lighting should be coordinated with crime prevention and other community needs.

Because of the high cost of energy, the designer should consider the use of energy efficient systems, such as high pressure sodium light.

Suggested references for street light design are the IES' "American Standards Practice for Roadway Lighting" and AASHTO's "An Informational Guide for Roadway Lighting".

3. Landscaping

Landscaping should be provided for aesthetic and erosion control purposes in keeping with the character of the street and

its environment. Landscaping should be arranged to permit sufficiently wide, clear and safe pedestrian walkways. Combinations of turf, shrubs and trees are desirable in border areas along the roadway. However, care should be exercised to insure that guidelines for sight distances and clearance to obstructions are observed, especially at intersections.

4. Bikeway

See Section 16 for bikeway considerations.

SECTION 8.

COLLECTOR ROADS AND STREETS

8-01 GENERAL

The collector road or street links neighborhoods or areas of homogeneous land use with arterial streets or highways. Collector streets serve traffic movements between arterial and local streets and serve through traffic within local areas. They may be multi-laned roadways or two-lane roadways and, because they serve neighborhoods, they have numerous driveway accesses along the routes and have at-grade intersections.

The use of more liberal values of design dimensions than the guidelines described herein is recommended where economically feasible. In all cases, every effort should be made to attain the best possible alignment, grade, sight distance, and proper drainage that are consistent with the terrain, present and proposed development, and available funds.

Safety is an important factor in all roadway improvements. On low volume streets in urban areas, it may not be possible to obtain the desired obstacle-free roadside. However, as much clear roadside as practicable should be provided. The judicious use of flatter slopes, guardrail, and warning signs help to achieve roadside safety. Proper placement of utility facilities also assists in achieving safer roadsides.

8-02 RURAL COLLECTOR ROADS

8-02.1 General

The majority of rural collector roads are two-lane highways. These roadways should be designed to the highest standards attainable and compatible with traffic and topography. Motorists tend to drive at their desired speeds in the rural areas because of low traffic volumes and the general feeling of openness; therefore the design standards should be the highest possible.

Generally, the design criteria to be considered have been indicated in Section 2, DESIGN PARAMETERS.

8-02.2 Design Parameters

1. Design Traffic Volumes

The collector road should be designed for specific traffic volumes determined by accepted procedures. The average daily traffic (ADT), either current or projected, is the primary basis for design.

2. Design Speed

Geometric design features should be consistent with a design speed appropriate for the conditions. Low design speeds are generally applicable to highways with winding alignment in rolling or mountainous terrain, or where environmental conditions dictate, such as in a built-up area. High design speeds are generally applicable to highways in level terrain and open areas.

Design speeds on rural collector roads should range between 35-50 mph on level terrain and 25-40 mph in rolling or mountainous terrain.

3. Level of Service

Collector roads should be designed to operate at Level of Service 'C'.

4. Design Vehicles

Collector roads should be designed to accommodate buses and semi-trailer combinations (WB-50).

8-02.3 Geometric Design Elements

The detail requirements for geometric design of roadways are covered in Section 4, BASIC GEOMETRIC DESIGN ELEMENTS, and should be consulted for appropriate guidelines in the design of collector roads.

1. Sight Distance

Minimum stopping and passing sight distances are a direct function of design speed. Criteria applicable to collector roads are covered in Section 4.

2. Horizontal Alignment

Horizontal and vertical alignment must complement each other and be considered in combination. Topography, traffic volume and composition, and right-of-way constraints are controlling factors. Sudden changes in horizontal alignment should be avoided. Frequent opportunities for passing (on two-lane roads) should be provided.

3. Superelevation

Superelevation compatible with the design speed and curvature should be provided. Design criteria are presented in Section 4. Generally, superelevation on rural collector roads should not exceed 8 percent. The axis of rotation will generally be the centerline of the roadway. Superelevation runoff should be adjusted to provide smooth riding qualities, good drainage, and good appearance.

4. Vertical Alignment

Vertical alignment must be designed in conjunction with the horizontal alignment. Maximum grades as related to speed and terrain conditions are suggested in Table 8-1. Minimum profile grade shall be not less than 0.4 percent for drainage purposes.

Table 8-1

Maximum Grades

Terrain	Design Speed (mph)				
	20	30	40	50	60
Level	7	7	7	6	5
Rolling	10	9	8	7	
Mountainous	12	10	10		

Vertical curves create a gradual change between breaks in grade rates. Properly designed vertical curves should provide adequate sight distance, safety, good drainage, and pleasing appearance.

8-02.4 Cross Section Elements

Detail requirements of cross section elements are discussed in Section 5, CROSS SECTION ELEMENTS, and should be referred to for appropriate guidelines in the design of rural collector roads.

1. Pavement Surface

Pavement surface should be either of bituminous asphalt concrete or portland cement concrete placed over base course and/or subbase course as required.

2. Cross Slope

Pavement cross slope shall be a minimum of 2 percent to provide for proper drainage of the pavement surface.

3. Lane Width, Shoulder and Roadway

Basic traffic lane width should be 12 feet (3.66 m) for both two-lane and multi-lane roads. In specific cases, traffic lanes less than 12 feet wide may be used.

Graded shoulder width is measured from the edge of the traveled way to the hinge point (see Section 3, DEFINITIONS AND TERMINOLOGY). Shoulder width should be 8 feet (2.44 m) wide. On divided roadways, the left shoulder should be at least 2 feet (0.62 m) wide. Cross slope on shoulders should conform to the guidelines of Section 5. All shoulders (right and left of traffic) should be paved for a minimum of 2 feet (0.62 m) from the edge of the traveled way.

Where guardrail is proposed, the shoulder width should be increased by 2 feet (0.62 m) where possible.

For side slopes and recovery areas, refer to Section 12, ROADSIDE DESIGN.

4. Rights-of-Way

The procurement of rights-of-way to widths that accommodate the construction, adequate drainage, and proper maintenance of a highway is a most important part of the overall design. Wide rights-of-way permit the construction of gentle slopes, resulting in greater safety for the motorist and providing for easier and more economical maintenance. The procurement of sufficient rights-of-way at the time of the initial improvement permits the widening of the roadway and the widening and strengthening of the pavement at reasonable cost as

traffic demand increases. The designer should refer to the appropriate County ordinance and design criteria for collector road right-of-way requirements.

In developed areas, it may be more feasible to limit the right-of-way width to a practical minimum. In every case, however, the right-of-way should be not less than that required for all elements of the design cross section and appropriate border areas.

8-02.5 Structures

The design of bridges, culverts, walls, tunnels and other structures should be in accordance with current AASHTO Standard Specifications for Highway Bridges.

The shoulder width of the approach roadway should be carried across all structures.

8-02.6 Clearances to Obstructions

1. Vertical Clearance

A minimum of 16.5 feet (5.03 m) vertical clearance should be provided for the entire roadway width for bridge structures over a collector road. This includes 0.5 foot (0.15 m) for future resurfacing of the road surface.

2. Horizontal Clearance

Where design speeds are in excess of 45 mph, a clear roadside recovery area is desirable, preferably 20 to 30 feet (6.10 to 9.15 m) or more from the edge of the shoulder. Side slopes from the hinge point should be graded at 4:1 or flatter slope. Structures and fixed objects such as bridge piers and sign supports should be set back to provide a clear area as great as the clear area on the approach roadway. When this is not possible, the fixed objects should be shielded by traffic barriers, or breakaway supports should be utilized, as appropriate.

Roadside design is discussed in detail in Section 12, ROADSIDE DESIGN.

8-02.7 Intersection Design

Intersections should be designed to provide adequate approach sight distance. Designs should avoid steep profile grades, or intersections located on short-crest vertical curves, just beyond a short-crest vertical curve, or on a sharp horizontal curve. Where necessary, cut slopes should be flattened, and horizontal or vertical curves should be lengthened to provide the required sight distance. Section 6, AT-GRADE INTERSECTIONS, contains a detailed discussion of at-grade intersection design.

8-03 URBAN COLLECTOR STREETS

8-03.1 General

The function of urban collector streets is equally divided between mobility and access. Almost all collector streets permit access to abutting properties, except where access rights have been acquired. Collector streets may be multi-laned roadways, or two-lane roadways which provide access and circulation within residential, commercial and industrial areas.

Collector streets also serve pedestrian traffic, and very often accommodate public utility facilities within their rights-of-way.

The general design criteria for collector streets are covered in Section 2.

8-03.2 Design Parameters

1. Design Traffic Volumes

   Traffic volume is the major criterion in determining the geometric values to be used in designing collector streets. Specifically, the Design-Hourly Volume (DHV) projected to the (future) design year should be the basis of design. The number of lanes to be provided should be determined by highway capacity analysis for the design traffic volume.

2. Design Speed

   Design speed is a factor in the design of collector streets. In the typical urban street grid, the closely spaced intersections usually limit vehicular speeds, thus making the effect of design speed of little significance. Nevertheless, the longer sight distances and curve radii, commensurate with design speeds higher than the actual speeds attainable, result in safer streets and should be used to the extent practicable and feasible.

Design speeds on urban collector streets should range between 25 and 35 mph, depending upon the terrain and adjacent land use.

3. Level of Service

Urban collector streets should be designed to operate at Level of Service 'C'.

4. Design Vehicles

The design vehicle used for the design of urban collector streets is dependent on the service area (land use) of the collector street. Generally, the following guide is applicable:

- Commercial area - SU and BUS
- Industrial area - WB-50
- Residential - SU and BUS.

8-03.3 Geometric Design Elements

The detail requirements for geometric design of roadways are covered in Section 4, and should be consulted for appropriate guidelines in the design of collector streets.

1. Sight Distance

Minimum stopping sight distance is a direct function of the design speed. Criteria applicable to collector street design are covered in Section 4. Design for passing sight distance seldom is applicable on collector streets.

2. Horizontal Alignment

The horizontal alignment should fit the existing topography as closely as possible. However, caution should be exercised in the design so as not to compromise the safety of the facility. In residential areas, there are advantages in following the topography closely, so as to discourage speeding and through traffic.

Minimum curve radii in relation to design speed and superelevation are given in Section 4.

3. Superelevation

Superelevation may, in specific locations, be advantageous for collector street traffic operation, but in built-up areas, the

combination of wide pavement areas, proximity of adjacent development, control of cross slope, profile for drainage, frequency of cross streets, and other urban features combine to make its use impractical. When used, superelevation on collector streets should not normally exceed 4 percent. The absence of superelevation on urban collectors for low speeds is not detrimental to the user. Generally, some warping or partial removal of the tangent pavement crown facilitates the user's operation.

4. Vertical Alignment

Grades should be 12 percent or less in mountainous terrain, and no more than 4 percent in level terrain. The minimum should be not less than 0.4 percent for drainage purposes.

Design guidelines for vertical curves are given in Section 4.

8-03.4 Cross Section Elements

Detail requirements of cross section elements are discussed in Section 5, and should be referred to for appropriate guidelines in the design of urban collector streets.

1. Pavement Surface

Pavement surface should be either of bituminous asphalt concrete or portland cement concrete placed over base course and/or subbase course as required.

2. Cross Slope

Pavement cross slope shall be a minimum of 2 percent to provide for proper drainage of the pavement surface.

3. Lanes

The collector street width should be planned as the sum of the ultimate lanes for moving traffic and parking, including median where appropriate.

The basic traffic lane width is desirably 12 feet (3.66 m). In industrial areas, the lane width should be 12 feet (3.66 m). In business and residential areas, minimum lane width of 10 feet (3.05 m) may be acceptable. Added turning lanes at intersections should be at least 9 feet (2.75 m) wide, and desirably 12 feet (3.66 m), depending on the percentage of trucks.

Parallel parking lanes should be at least 8 feet (2.44 m) wide, preferably 10 feet (3.05 m) wide in industrial and commercial areas. Parking lane width determination should include consideration for likely ultimate use as a traffic lane, either during peak hours or continuously. Generally, the width of the gutter should be in addition to the lane width required.

In addition, consideration of the right curb lane for bike route usage should also be given. Section 16 covers bikeway considerations.

4. Median

Collector streets designed for 4 or more lanes should include width for appropriate median treatment whenever necessary and practical. There are substantial advantages for each increment in median width, and medians should be as wide as practical for the specific conditions. For the general types of street median treatment, the following widths should be provided: (1) paint-striped separation, 2 to 4 feet (0.62 to 1.22 m) wide; (2) narrow raised or curbed sections, 2 to 6 feet (0.62 to 1.83 m) wide; (3) paint-striped or curbed sections, 10 to 14 feet (3.05 to 4.27 m) wide providing space for separate left-turn lanes; and (4) curbed sections, 20 to 40 feet (6.10 to 12.20 m) wide providing space for shielding protection of a vehicle crossing at an intersection and space for parkway landscape treatment.

On collector streets with raised median, openings should be provided only at street intersections and at reasonably spaced driveways serving major traffic generators such as industrial plants and shopping centers. Whenever practical, median openings should be designed to include left-turn lanes.

Openings should be situated only where there is adequate sight distance. The shape and length of the median openings depend on the width of the median and the vehicle types that must be accommodated. The minimum length of median openings should be that of the projected roadway width of intersecting cross street or driveway. Desirably, the length of median openings should be great enough to provide for a 50 foot (15.25 m) turning control radius for left-turning vehicles between the inner edge of the lane adjacent to the median and the centerline of the intersection roadway. Section 6 contains detail discussion on median openings and left-turn lanes.

On many urban collectors, it may be impractical to place a raised median. A continuous left-turn lane, flush with the

adjoining traffic lanes and painted, is an acceptable approach.

Section 5 contains further discussion on medians.

5. Curbs and Gutters

Streets normally are designed with curbs for high utilization of available width, control of drainage, protection of pedestrians, and delineation. The curb on the right of traffic should be about 6 to 8 inches (15 to 20 cm) high, usually with appropriate batter. On lightly traveled residential streets with grades less than 2 percent, a rolled curb that does not require modification at driveway entrances may be used.

On divided streets, the type of median curbs should be determined in conjunction with the median width and the type of turning movement control to be effected.

In addition, surface runoff is gathered along the curbs and carried in gutters to catch basins located at appropriate locations.

6. Sidewalks

Sidewalks should be provided along both sides of collector streets that are used for pedestrian access to schools, parks, shopping areas, and transit stops and along all collectors in commercial areas. In residential areas, a sidewalk should be provided on at least one side of all collector streets. The sidewalk should be situated as far as practical from the traffic lanes, usually close to the right-of-way line.

Clear sidewalk width should be 4 feet (1.22 m) minimum; widths of 8 feet (2.44 m) or greater may be needed in commercial areas. If roadside appurtenances are situated on the sidewalk adjacent to the curb, additional width is required to secure the clear width.

Curb-cuts shall be provided at crosswalks to accommodate the physically handicapped person. Section 5 covers the details for wheelchair ramp.

7. Rights-of-Way

The right-of-way width should be sufficient to accommodate the ultimate planned roadway, including median, shoulder (if any), grass strip, sidewalks, public utility facilities and roadside design (if any).

The area between the curb face and the right-of-way line should be wide enough to serve several purposes, including provisions for buffer space between pedestrian and vehicular traffic, sidewalk space, and an area for both underground and above-ground utilities such as traffic signals, parking meters, and fire hydrants.

The designer should refer to the appropriate County ordinance and design criteria for collector street right-of-way requirements.

8-03.5 Clearance to Obstructions

The minimum clear width for all new bridges on collector streets with curbed approaches should be the same as the curb-to-curb width of the approaches. For collector streets with shoulders and no curbs, the full width of approach roadways preferably should be extended across bridges. Sidewalks on the approaches should be extended across all new structures.

Generally, on all collector streets with shoulders, a clear area of at least 6 feet (1.83 m) plus the shoulder width should be provided. If curbed with no shoulders, an area of 6 feet (1.83 m) beyond the face of the curb is desirable. Where a continuous parking lane is provided, no clear area is required, but a minimum 2 foot (0.62 m) setback to obstructions is desirable to avoid interference with opening car doors. Preferably, these obstructions are located at or near the right-of-way line and outside of the sidewalks. In areas of dense pedestrian traffic, the construction of barrier-type curbing between the traveled way and adjacent street fixtures and trees would aid in protecting the high volume of pedestrian traffic from an out-of-control vehicle.

Other off-roadway obstacles such as trees that may seriously damage out-of-control vehicles should be removed from the roadside wherever feasible. However, the potential benefits from the removal of trees should be weighed against the adverse effects that their removal may have on the roadside environment, and they should be removed only when necessary for reasons of safety. Depending on the conditions, only those fixed objects in very vulnerable locations should be considered for removal.

Traffic barriers are not used extensively on collector streets, except where there is a hazard to motorists and pedestrians, such as along sections with steep side slopes and at the approaches to overcrossing structures.

8-03.6 Intersection Design

The pattern of traffic movements at the intersections, and the volume of traffic on each approach, including pedestrian traffic, during one or more peak periods of the day are indicative of the type of traffic control devices necessary, the widths of pavements required, including auxiliary lanes and, where applicable, the degree of channelization needed to expedite the movement of all traffic. The arrangement of islands and the shape and length of auxiliary pavements may differ, depending on whether or not traffic signal control is necessary. The composition and character of traffic is a design control; movements involving large trucks necessitate larger intersection areas and flatter approach grades than those required at intersections where traffic predominantly consists of passenger cars. Bus stops located near an intersection may further modify the arrangement. Approach speeds of traffic also have a bearing on the geometric design, as well as on control devices and markings.

The number and locations of approach roadways and their angles of intersection are major controls for the intersection geometric pattern, the location of islands, and the types of control devices. At-grade intersections preferably should be limited to no more than four approach legs.

The more important design considerations for at-grade intersections fall into two major categories: first, the geometric design including a capacity analysis; and second, the location and type of traffic control devices. For the most part, these considerations are applicable to both new and existing intersections, although on existing intersections in built-up areas, heavy development usually makes extensive design changes impractical.

Section 6 contains the discussion on the design of at-grade intersections.

8-03.7 Landscaping

Landscaping should be provided in keeping with the character of the street and its environment for aesthetic and erosion control purposes. Landscape designs should be arranged to permit a sufficiently wide, clear, and safe pedestrian walkway. Combinations of turf, shrubs, and trees should be considered in continuous border areas along the roadway. However, care should be exercised to insure that sight distances and clearance to obstruction guidelines are observed, especially at intersections.

In addition, the designer should consider the maintenance of the landscape plantings, i.e., provide for ease of maintenance and for the safety of the maintenance personnel, as well as the safety of the traveling public.

8-03.8 Street Lighting

Properly designed street lighting should be provided on all collector streets for safety and efficient use of the roadway facilities. The designer should refer to the IES' "American Standards Practice for Roadway Lighting" and AASHTO's "An Informational Guide for Roadway Lighting". Special consideration should be given to the use of energy efficient systems, such as high pressure sodium lights.

SECTION 9.

ARTERIAL HIGHWAYS AND STREETS

9-01 GENERAL

Arterial roadways provide the high-speed, high volume network for long distance travel between major points. In Hawaii, roadways providing the only link between population centers are considered as arterial roadways, as are roadways in the urban areas which have high volumes that serve major centers of activity. Some examples of arterial roadways are given in Section 2, DESIGN PARAMETERS.

The arterial roadways in the State vary from two-lane roadways to multi-lane divided highways. Generally, the arterial roadways in the State do not have restricted access control and, therefore, must include driveway access and at-grade intersections in their design. Because arterial roadways are designed for high volume or high speeds, they must be designed to the highest practical standards attainable.

This section treats rural and urban arterials separately, although it is understood that in special cases, some of the design features of one may be needed by the other. It is important to be familiar with the aspects of both types because, to form a continuous system, these types must be skillfully blended as the roadway flows from rural to urban sections and vice versa.

A controlled access arterial highway is considered as a freeway and its design requirements are covered in Section 10, FREEWAYS.

9-02 RURAL ARTERIALS

9-02.1 General

The major part of the rural highway system in Hawaii consists of two-lane highways and four-lanehighways. These roadways are adequate where traffic volumes are light and long sight distances are generally available.

9-02.2 Design Parameters

The design parameters are covered in Section 2.

1. Design Traffic Volume

Based upon traffic counts, or projected traffic demand for new facilities, the Average Daily Traffic (ADT) can be established for the present year and for the design year, normally 20 years in the future. These ADT values can then be translated to the Design-Hourly Volume (DHV) to be used to determine the service design of the arterial highway.

For low volume rural arterials, the ADT is usually sufficient for design since neither capacity nor intersection operations govern the overall operation. These roadways are of minimum service design and normally provide free flow under all conditions.

2. Design Speed

Geometric design features should be consistent with a design speed appropriate for the conditions. Low design speeds are usually applicable to highways in mountainous terrain. High design speeds are applicable to highways in rolling or level terrain in open areas.

The following design speeds are suggested for the various terrain conditions:

Level, Open - 50-60 mph
Rolling, Open - 45-60 mph
Mountainous - 25-45 mph

3. Level of Service

Arterial highways in rural areas should be designed to operate at Level of Service 'C'. In built up areas Level 'C' might be hard to obtain

4. Design Vehicles

Arterial highways should be designed to accommodate a semi-trailer combination (WB-50).

9-02.3 Geometric Design Elements

The detail requirements for geometric design of roadways are covered in Section 4, BASIC GEOMETRIC DESIGN ELEMENTS, and should be consulted for appropriate guidelines in the design of arterial highways.

1. Sight Distance

Minimum stopping and passing sight distances are a direct function of design speed. Criteria applicable to arterial highways are covered in Section 4.

In the design of two-lane rural highways, provisions for safe passing at frequent intervals should be considered. Provisions for passing are discussed in detail later in this section.

2. Alignment

A smooth flowing alignment is desirable on a rural arterial. Changes in alignment, both horizontal and vertical, should be so gradual that the driver will not be surprised by any of the changes. Special consideration should be given to providing adequate sight distance at locations such as at high-volume intersections and at transitions of roadway widths or numbers of lanes. Adequate alignment is usually more safe and effective than control by signing or pavement marking and should be achieved whenever feasible.

The length and steepness of grades directly affect the operational characteristics of an arterial. Table 9-1 suggests the maximum grade related to speed and terrain.

Table 9-1

Relation of Maximum Grades to Design Speed

Terrain	Design Speed, mph				
	25	30	40	50	60
Level	7	6	5	4	3
Rolling	7	7	6	5	4
Mountainous	10	8	7	7	6

Grades less than the maximum are always desirable, as there seldom are advantages in using the maximum grade values.

3. Superelevation

Superelevation compatible with the design speed and curvature should be provided. Design criteria are presented in Section 4. Superelevation should not exceed 8 percent.

9-02.4 Cross Section Elements

Detail requirements of cross section elements are covered in Section 5, CROSS SECTION ELEMENTS, and should be referred to for appropriate guidelines in the design of rural arterial highways.

1. Pavement Surface

Pavement surface should be either of bituminous asphalt concrete or portland cement concrete.

2. Cross Slope

Pavement cross slope shall be a minimum of 2 percent to provide for proper drainage of the pavement surface. Two-lane highways are usually crowned at the centerline to drain away from the centerline, except where superelevation is required.

Divided, multi-lane highways may have cross slopes as indicated in Section 5.

3. Lane Width and Shoulder

Basic traffic lane width should be 12 feet (3.66 m) for all rural arterials. Under special conditions, 11 foot (3.35 m) lanes may be considered.

Shoulders should be provided on rural arterials for safety reasons and others as discussed in Section 5. Shoulder widths should be related to traffic demand. Table 9-2 shows the recommended range of shoulder width for two-lane arterials. On multi-lane highways, the shoulder width should be 10 feet (3.05 m).

Table 9-2

Widths of Shoulders for Two-Lane Arterials

Design Volume		Shoulder Width (Ft.)
Current ADT	DHV	
250-400		4 to 8
400-450	100-200	6 to 10
---	200-400	8 to 10
---	Over 400	10

The lower value of the shoulder width range recommended should be used on roadways in the lower value of the volume range and in special cases where terrain or design speeds are low, or both. The wider shoulder in each volume group is the desirable width where speeds and volumes are high.

Shoulders should normally be usable in all weather conditions. Accordingly, shoulders should be paved or should be of stabilized material. The shoulder should be paved for a minimum of 2 feet (0.62 m) from the edge of the traveled way. Shoulder widths should be uniform for relatively long stretches of roadway.

Guardrails or retaining walls should be placed at least 2 feet (0.62 m) from the edge of the shoulder.

4. Right-of-Way

The procurement of right-of-way to widths that accommodate the construction, adequate drainage, and proper maintenance of a highway is a most important part of the overall design. Where additional lanes may be required in the future, the initial right-of-way should be adequate to provide the wider roadway section. The designer is referred to the appropriate County ordinance and design criteria for rural highway right-of-way requirements.

9-02.5 Structures

The design of bridges, culverts, walls, tunnels and other structures should be in accordance with current AASHTO Standard Specifications for Highway Bridges.

The full width of the approach roadways should be carried across all bridges.

9-02.6 Clearances to Obstructions

1. Vertical Clearance

A minimum of 16.5 feet (5.03 m) vertical clearance should be provided for the entire roadway. This includes 0.5 foot (0.15 m) for future resurfacing.

2. Horizontal Clearance

Where design speeds are from 45 to 60 mph, a clear roadside area of 30 feet (10 m) or more is desirable. Side slopes should be graded at 4:1 or flatter. Structures and fixed

objects such as bridge piers, sign supports and drainage structures should be set back to provide a clear area as great as the clear area on the approach roadway. When this is not possible, guardrail protection should be provided for fixed objects, or breakaway fixtures should be utilized.

Roadside design guidelines are discussed in Section 12, ROADSIDE DESIGN.

9-02.7 Provisions for Passing

In the design of two-lane rural arterials, the horizontal and vertical alignment should be designed to provide as much of the highway length as feasible with sight distance for safe passing. Minimum values of passing sight distances for different design speeds are provided in Section 4.

There may be restrictive cases where passing sight distance is economically difficult to justify. In those instances, passing sections should at least provide the desired level of service. Where achievement of adequate passing sight distance is not practical, truck climbing lanes or perhaps passing lanes should be considered. Although climbing lanes are normally provided to prevent unreasonable reductions in operating speeds, they also provide opportunities for passing in areas that normally would not permit passing sections. Climbing lanes are usually provided to the right of the normal traffic lane and should be 12 feet (3.66 m) wide, with a somewhat reduced value of allowed shoulder width. Because of the lower traffic speeds, a minimum 4 foot (1.22 m) shoulder is normally considered adequate.

Passing lanes should normally only be considered in areas where the roadway can be expanded to four lanes. The introduction of short four-lane sections requires that consideration be given to adequate transitions, and particularly to marking and signing. Climbing lanes are preferred over passing lanes because they provide a normal type of treatment without the risk of the motorist thinking that he is entering a four-lane roadway of any length. Adequately designed and well marked climbing lanes will usually be used by the slow moving vehicles, both trucks and automobiles, and will allow passing by drivers who prefer to move at normal speeds.

The design elements and requirements for the use of climbing lanes are covered in Section 14, OTHER DESIGN CONSIDERATIONS.

9-02.8 Multi-Lane Undivided Arterial Highway

A multi-lane undivided arterial is the narrowest arterial on which each traffic lane is intended to be used by traffic in one direction, and passing is accomplished on lanes not subject to use by opposing traffic. The ability to pass without traveling in the lane of opposing traffic results in a freer and smoother operation and a large increase in arterial capacity over that of two-lane arterials. Because of the generally higher volumes, drivers on multi-lane arterials are confronted with added traffic frictions, not only from opposing traffic and from the roadsides, but also from traffic in the same direction. The accident rate for all reported accidents on multi-lane undivided arterials, as an average for all volumes, is higher than that on two-lane arterials.

The elements of design discussed earlier in this section are applicable generally to multi-lane undivided arterials, except that passing sight distance is not essential. The minimum required sight distance at all points is the stopping sight distance, because passing is effected without the necessity of using a lane for opposing traffic. Longer than minimum required sight distance is desirable, as it is on any type of arterial.

Adequate shoulders, which encourage all drivers to use them in emergencies, are essential on multi-lane undivided arterials. Vehicles stopping on through traffic lanes are very hazardous, probably more so than on a divided arterial, because following vehicles that maneuver to pass encourage and sometimes force vehicles behind them and in the inner lane to edge over the centerline.

Generally, multi-lane arterials in rural areas should be designed as divided roadways because of the high speed and volume of traffic anticipated. Undivided arterials of four or more lanes are more applicable in urban and suburban areas where there is a concentration of development along the highway frontage.

9-02.9 Divided Arterial Highway

1. General

A divided arterial is one with separated roadways for traffic in opposite directions. It may be situated on a single roadbed, or the two roadways may be widely separated. The width of median may vary, and is governed largely by the type of area, character of terrain, intersection treatment, and economics. An arterial is not normally considered to be divided

unless two full lanes are provided in each direction of travel, and the median is four feet (1.22 m) or more wide, and constructed or marked in a manner to preclude its use by moving vehicles except in emergencies or for left turns.

A four-lane rural facility should have adequate median width to provide for protected left turns. Vehicles making a left turn should not be required to stop in the passing lane of a roadway designed for high volumes and speeds.

Geometric design elements and cross section elements discussed earlier in this section are applicable generally to divided arterials. The minimum required sight distance at all points is the stopping sight distance. Design values should exceed the guide values given whenever feasible.

2. Horizontal and Vertical Alignment

A divided arterial generally is designed for high-volume and high-speed operation, for which a flowing type of alignment should be provided.

Because a divided arterial consists of 2 separated roadways, there may be instances where median widths and roadway elevations can be varied for special considerations. Special topographic or intersection requirements may necessitate such treatments for economic or operational reasons. Precaution should be taken that such variations do not adversely affect operations. Such problems as sharp reverse curves, headlight glare, and grades of intersection crossings must always be considered.

Maximum grades should be between 3 and 7 percent, depending on the design speed and the type of terrain being traversed by the arterial highway.

Because stopping distance is the governing sight distance control on divided roadway designs, there may be a tendency to use a roller coaster profile, especially in rolling terrain, which usually results in a displeasing roadway appearance. Therefore, the relation of horizontal and vertical alignment should be studied to obtain a suitable combination. To avoid undesirable appearance, profile designs should be checked in long continuous plottings, wherein the foreshortened aspect can be simulated.

3. Superelevation

Superelevation requirements are discussed in Section 4. Axis of rotation for multi-laned arterials should follow the guidelines for freeways.

4. Lane Widths, Shoulders

Pavements on divided arterials should be designed with 12 foot (3.66 m) wide lanes.

The width of the usable outside shoulder should be 10 feet (3.05 m) wide. The left shoulder along the median should be at least 4 feet (1.22 m) wide.

The normal roadway section, including full shoulders, should be carried across all structures.

5. Medians

Median widths as wide as possible should be used. Although medians as narrow as 4 to 6 feet (1.22 to 1.83 m) wide may be required under very restricted conditions, a median width of 22 feet (6.71 m) or wider should be provided to protect left turning vehicles at intersections.

There are many advantages to wide medians, and a width of 60 feet (18.30 m) or more should be provided wherever feasible. A wide median allows the use of independent roadway profiles. Reduced frequency of crossover accidents, reduction of headlight glare, and greater flexibility of traffic control systems for intersections are safety features associated with a wide median.

Median widths from 22 to 60 feet (6.71 to 18.30 m) should be carefully considered from an operational standpoint at intersections. These widths do not always provide median storage space for longer vehicles crossing the median; however, those in the wider ranges often invite the driver to attempt the crossing of the roadways independently. If these operating conditions control, the designer must consider the mainline operating benefits of a wide median as compared with intersection control.

In areas where median barriers may be required, the lateral clearance from the normal edge of pavement to the base of the barrier should desirably be not less than 4 feet (1.22 m).

6. Sections with Widely Separated Roadways

Occasionally, it is practical to widely separate the one-way roadways of a divided arterial. This situation may follow where an existing two-lane arterial proves inadequate and is

improved to a four-lane section, but on which direct widening is not feasible because of topography or adjacent development. In such case, the old roadway is not disturbed, but is converted to one-way operation upon construction of another, completely separate, one-way roadway.

Widely separated one-way roadways may be particularly fitting to certain topographic conditions. In valleys where drainage may make the location difficult, individual roadways may be situated on each side of the valley. Along ridges or where there is a continual change in ground cross slopes, the separate roadways may be better fitted to the terrain than an arterial on a single roadbed. Such arrangements simplify location problems because only half of the total roadway width is considered at a time. With reduced roadway prisms, construction scars are kept to a minimum and more of the natural growth is retained, particularly that between the separate roadways. In areas where right-of-way is not a problem, designs of this character often result in lower construction costs.

Crossings at grade with the one-way pavements are greatly simplified in design and operation. Accident potential is generally reduced and the capacity of intersections is increased. Moreover, operation on widely separated roadways provides the maximum in driver comfort. Strain is lessened by largely eliminating the view and influence of opposing traffic. Substantial reduction or elimination of headlight glare at night is especially helpful in easing driver tension and in minimizing hazards resulting therefrom.

9-02.10 Intersections

The liberal use of high-type intersections, and even interchanges, is highly desirable on arterial highways. Adequate turning widths with acceleration and deceleration tapers should be considered.

Rural intersections controlled by traffic signals are not desirable. Drivers generally do not anticipate signals in rural areas that have high operating speeds. On high speed arterials, channelized intersections with raised traffic islands also present hazards to the unsuspecting driver.

Design of at-grade intersections is covered in Section 6, AT-GRADE INTERSECTIONS.

9-03 URBAN ARTERIALS

9-03.1 General

Urban arterials carry large volumes of traffic within and through urban areas. They vary in cross section from two-lane streets to multi-lane, divided streets. Many of the arterial streets are one-way streets.

The urban arterial streets not only serve major centers of activity in the metropolitan area, but also serve the abutting properties. Therefore, the design of urban arterials must include design considerations for driveway accesses and at-grade intersections.

Major improvement of existing arterials can be extremely costly, particularly where additional rights-of-way must be acquired through intensively developed areas. Accordingly, it is frequently necessary to use dimensional values that are somewhat less than desirable and are below those used where wide rights-of-way are available or can be acquired economically. When restricted conditions are encountered, consideration should be given to providing above desirable values for such items that tend to offset those created by the restriction. An example would be to restrict turns through an area having below normal lane widths.

9-03.2 Design Parameters

Generally, the design criteria to be considered have been indicated in Section 2.

1. Design Traffic Volumes

   Average Daily Traffic (ADT), projected to a future design year, and the resulting Design-Hourly Volume (DHV) are used to establish the basis of design for urban arterial streets.

2. Design Speed

   Design speeds for arterial streets range from 25 to 45 mph in built-up areas of the city, and from 25 to 60 mph in the suburban areas of the metropolitan area.

3. Level of Service

   Urban streets should be designed desirably to operate at Level of Service 'C'. Intersection capacity analysis is one of the

more important considerations in the design of urban arterials because of the generally close spacing of at-grade intersections.

4. Design Vehicles

The design vehicles for use on urban arterials are dependent upon land use of the area served by the arterial. The following guideline is suggested in the selection of design vehicles for use in the design of urban arterial streets:

a. Arterials serving primarily residential and commercial areas of the city - Single Unit Truck (SU) or Bus.

b. Arterials serving industrial areas - Semi-Trailer Combination (WB-50).

9-03.3 Geometric Design Elements

The detail requirements for geometric design of roadways are covered in Section 4, and should be consulted for appropriate guidelines in the design of urban arterial streets.

1. Sight Distance

The provision of adequate sight distance is important in urban arterial design. Adequate stopping sight distance must be provided at all points on the arterial streets.

Passing sight distance is usually not a criterion in urban street systems.

2. Horizontal and Vertical Alignment

Ideally, urban arterials should be designed in accordance with the design speed selected, but because of economic and land use constraints, this is generally not possible. Therefore, it is desirable that the most liberal design possible, using curve radii greater than suggested in the guidelines and flatter grades, be used.

On routes where there may be a high percentage of trucks, grades flatter than those in rural areas should be considered to avoid undesirable reduction in speed.

Suggested maximum grades for urban arterials are given in Table 9-3. To provide for drainage runoff, the minimum longitudinal slope should not be less than 0.4 percent.

Table 9-3

Relation of Maximum Grades to Design Speed

Terrain	Design Speed, MPH				
	25	30	40	50	60
Level	8	8	7	6	5
Rolling	10	9	8	7	7
Mountainous	12	12	11	9	8

3. Superelevation

Superelevation commensurate with the design speed and curvature should be provided. Superelevation runoff should be adjusted to provide smooth riding qualities, good drainage, and good appearance. In built-up areas, normal roadway crown may be adequate because of low design speeds.

9-03.4 Cross Section Elements

Detail requirements of cross section elements are discussed in Section 5 and should be referred to for appropriate guidelines in the design of urban arterial streets.

1. Pavement Surface

Pavement surface should be of bituminous asphalt concrete or of portland cement concrete.

2. Cross Slope

Pavement cross slope shall be a minimum of 2 percent to provide for adequate drainage of the pavement surface. Because of close intersection spacing in urban areas, pavement cross slope may be required to vary in order to meet cross street pavement grades.

3. Lane Widths

Lane widths of 12 feet (3.66 m) are desirable on arterials having high speed and free flow conditions.

Right-of-way and economic constraints may dictate narrower lanes. Arterial through lanes should be a minimum of 11 feet (3.35 m) wide and left turn lanes should be a minimum of 10 feet (3.05 m) wide. Parking lanes should be at least 10 feet (3.05 m) wide, to permit their use as traffic lanes during peak periods.

Lanes adjacent to curbs, both left and right of traffic, should be 12 feet (3.66 m) wide.

Special lane widths for bike routes should also be considered when establishing lane widths for urban arterials, although bike lanes should be placed on other streets, where possible, because of the high volume and speeds expected on arterial streets.

4. Medians

Medians are a desirable feature on arterial streets, and should be provided where space permits. Medians are discussed in Section 5.

Where right-of-way is limited, it is frequently necessary to decide how best to allocate the available space between border areas, traffic lanes, and medians. On the less important arterials, the decision is often resolved in favor of no median at all. A median only 4 feet (1.22 m) wide is better than none; however, each additional foot provides an added increment of safety and improved operation. At intersections where left turns are made, a left-turn lane is always desirable from a capacity and safety standpoint. Left turn lanes are covered in Section 6.

Where intersections are widely spaced, e.g., 0.5 mile or more, the width of median may be varied by using a narrow width between intersections where necessary for economy and gradually widening on the approach to the intersection to accommodate the left-turn lane. This solution is hardly practical, however, and should never be used where intersections are closely spaced because the curved alignment of the lane lines results in excessive lane changes. It is far more desirable that the median be of uniform width.

Where an arterial must pass through a developed area having numerous street and driveway intersections, and where it is impractical to limit left turns, the continuous left-turn median lane is often desirable. Because all left turns are

thus protected, the interference to through traffic lanes is minimized. Successful operation of a continuous left turn lane requires adequate lane marking.

A raised median with barrier type curbing is commonly used on arterial streets. This type is desirable where it is necessary to prevent midblock turns. On streets serving low speed traffic, this type will prevent most cross-median accidents. Raised medians also provide a refuge for pedestrians and a good location for signs, signals, and other appurtenances.

On high speed arterials, raised median with barrier curbs is not desirable. A flush median with median barrier should be considered. Median barriers are covered in Section 13, TRAFFIC BARRIERS. Median barriers permit positive separation of traffic and discourage indiscriminate pedestrian crossing.

5. Shoulders

Shoulders are desirable on any highway, and urban arterial streets are no exception. They contribute to safety by affording maneuver room and providing space for immobilized vehicles. They offer a measure of safety to the occasional pedestrian in sparsely developed areas where sidewalks are not appropriate, and they serve as speed-change lanes for vehicles turning into driveways.

However, in the highly developed areas of the urban area, shoulder usage is usually restricted by limited right-of-way and the need to use all available space for traffic lanes.

6. Curbs and Sidewalks

Curbs are usually used on arterial streets in the urban areas. They serve to delineate the edge of pavement and protect pedestrian and roadside appurtenances from traffic. In commercial areas, barrier curbs are usually used. Curbs are discussed in Section 5.

Sidewalks should be provided on both sides of all arterial streets in the urban areas. A further discussion on pedestrian facilities is included in this section.

7. Right-of-Way

The width of right-of-way for the complete development of an arterial street is influenced by traffic requirements,

topography, land use, cost, intersection design, and extent of ultimate expansion. The required width of right-of-way is the summation of the various cross-section elements such as pavements, median, auxiliary lanes, shoulders, sidewalks and, where required, frontage roads, outer separations, side slopes, and retaining walls. The width of right-of-way should be based on the preferable dimensions of each element to the extent feasible. In built-up districts, however, it often is necessary to consider less desirable dimensions. In such a case, the designer is confronted with the problem of providing an overall cross section that will give maximum service within a limited width of right-of-way. Right-of-way widths in urban areas are governed primarily by economic considerations, physical obstructions, or environmental reasons. Along any arterial route, conditions of development and terrain vary and, accordingly, the availability of right-of-way varies. For this reason, the right-of-way on a given facility should not be a fixed width predetermined on the basis of the most critical point in its location. Instead, every opportunity should be taken to provide a desirable width along most, if not all, of the facility. The designer should refer to the appropriate County ordinance and design criteria for arterial street right-of-way requirements.

9-03.5 Clearance to Obstructions

Clear roadside design is recommended for urban arterials whenever feasible. On curbed street sections, this design is often impossible, particularly in restricted areas. In those areas, a clearance from curb face to object of 2 feet (0.62 m) should be considered the minimum. A 3 foot (0.92 m) clearance should desirably be provided, particularly near turning radii at intersections and driveways. This desirable offset provides the clearance required for overhang of trucks from striking the roadside object. When pedestrians are not a factor, obstructions should be well set back, protected, or provided with breakaway features. Protective curbs or guardrail may be considered in special cases.

Roadway width for bridges should carry the curb to curb width of the approach roadway across the bridge. Arterial highways with shoulders should have the full shoulder width carried across the bridge structure.

9-03.6 Pedestrian Considerations

Arterial streets must accommodate both vehicles and pedestrians, and the design should include sidewalks, crosswalks, and sometimes grade separations for pedestrians. Pedestrian facilities and control measures will vary, depending largely on the volume

of pedestrian traffic, the volume of vehicular traffic to be crossed, the number of lanes to be crossed, and the number of vehicles turning at intersections.

On many sections of arterial streets that traverse relatively undeveloped areas, the number of pedestrians will be so small that sidewalks are not needed immediately. Because these areas will usually be developed in the future, the design should allow for the ultimate installation of sidewalks. However, as a general practice, sidewalks should be constructed initially along all arterial streets that are not provided with shoulders, even though pedestrian traffic may be light.

The major pedestrian-vehicular conflict usually occurs at intersections. At intersections with minor cross streets where turning movements are light, pedestrian facilities are usually limited to crosswalk markings. Features that help the pedestrian include fixed-source lighting, refuge islands, barriers and signals.

On the more important arterials with heavy traffic volumes, the interference between pedestrians and vehicles at at-grade intersections sometimes presents serious problems. The problem is especially acute where the arterial traverses a business district and there are intersections with important cross streets. In extreme cases, grade separations for pedestrians provide the only satisfactory solution. Although separations for pedestrians are justified in some instances, at-grade crosswalks will remain the predominant form of crossing. Minimum conflict and hazard result if the crosswalks are properly placed, designed, maintained, and operated.

Indiscriminate closing of pedestrian crossings will lead to illegal crossings that are more hazardous. Therefore, proper and reasonable design for pedestrians is important.

The pedestrian walk signal is especially desirable on wide arterials. On exceptionally wide arterial streets, pedestrian signals should be mounted in the median as well as on the far side of the intersection.

The use of crosswalks at typical curbed-street intersections is difficult for people in wheelchairs. Curb-cut ramps of sufficient width must be provided in curbed areas that have sidewalks. The subject of wheelchair ramps is covered in Section 5.

9-03.7 Intersection Design

The design and operation of intersections has a significant effect on the operational quality of an arterial. Sight distance, capacity, grades, and provision for turning movements all affect intersection operation. Turning movements on principal arterials should not encroach on adjacent lanes, or if necessary, its effects should be considered. Section 6 discusses intersection development in detail. It is recommended that each individual intersection be carefully evaluated in the early design phases.

9-03.8 Street Lighting

Adequate lighting is very important to safe operation of an urban arterial. The higher volumes and speeds require the driver to make correct decisions with adequate time to make the proper maneuvers without creating undue conflict in the traffic lanes. Sudden braking and swerving are minimized where lighting is adequate. The visibility of signing and pavement marking also helps to smooth traffic flow. A safely designed, adequate lighting system is more important to optimum operation of an urban arterial than any other city street. The lighting should be continuous and an energy-saving type. Lighting in an urban area is also often a matter of civic pride and is a deterrent to crime. The AASHTO Informational Guide for Roadway Lighting and the IES' "American Standards Practice for Roadway Lighting", are recommended as a source of lighting information.

SECTION 10.

F R E E W A Y S

(This Section follows California Design)

10-01 GENERAL

The highest type of arterial highway is the freeway. The freeway is a divided arterial highway for through traffic with full access control and grade separations at intersections.

Freeways are selected for principal arterial corridors that are intended to provide for the safe and efficient movement of high volumes of traffic at relatively high speeds.

Essential freeway elements include medians, grade separations at cross streets, ramp connections for entrance to and exit from the freeway for interchange of traffic and, in some cases, frontage roads.

Freeways may be classified as Interstate Freeways or simply as freeways. In addition, freeways may be considered as urban freeways or rural freeways. Both urban and rural freeways are similar in concept. Generally, the rural freeway would utilize more liberal design standards in alignment and cross sections because of the greater availability of rights-of-way. In addition, design speeds are higher on rural freeways. However, because of the high cost of rights-of-way in Hawaii, alignment and cross sectional elements must necessarily be conservative; and, therefore, there is very little significant difference in the design of urban and rural freeways.

Sections 2, 3, 4 and 5 discussed in detail those design elements, controls and criteria applicable to all classes of highways. This section will identify and discuss those design features and elements unique to freeways.

10-02 GENERAL DESIGN CONSIDERATIONS

10-02.1 Design Parameters

1. Design Traffic Volumes

Based upon traffic counts or projected traffic demand for new facilities, the Average Daily Traffic (ADT) can be established for the present year and for the design year, usually 20

years in the future. These ADT values can then be transformed into Design-Hourly Volume (DHV) to be used to determine the service design of the freeway.

2. Design Speed

The design speed should reflect the desired safe operating speed during non-peak hours. Generally, the following design speed ranges are recommended:

Urban areas	- 50-55 mph
Suburban areas	- 55-65 mph
Rural	- 60-80 mph

In mountainous terrain, the design speed should be 45 to 55 mph, which is consistent with driver expectancy.

3. Level of Service

Under non-peak hour traffic conditions, the freeway should be designed to operate at Level of Service 'C' for urban freeways and at Level of Service 'B' for rural freeways.

During peak periods of traffic, the urban freeway should operate at no less than Level of Service 'D'.

The Highway Capacity Manual presents a thorough discussion of the level of service concept.

4. Design Vehicles

Freeways should be designed to accommodate all classes of vehicles including the semi-trailer combination (WB-50).

10-02.2 Geometric Design Elements

The detail requirements for geometric design elements of roadways are covered in Section 4, BASIC GEOMETRIC DESIGN ELEMENTS, and should be consulted for appropriate guidelines in the design of freeways. Design values should exceed the guide values whenever feasible.

1. Sight Distance

Minimum required stopping sight distance shall be provided continuously along the freeway.

2. Horizontal Alignment

The recommended desirable radii for freeways are 3,000 feet (915 m) for urban areas and 5,000 feet (1525 m) for rural areas.

If glare screen or landscape planting is proposed in the median, adjustment may be necessary to maintain the required sight distance on curves. In such cases, a larger radius or a widened median may be required throughout the length of the curve.

3. Grades

Grades on freeways should be as flat as possible with due consideration for drainage. Where sustained upgrades are required, the need for climbing lanes should be investigated (see Section 14, OTHER DESIGN CONSIDERATIONS). Table 10-1 shows recommended maximum grades as a function of design speed and type of terrain.

Table 10-1

Maximum Grades for Urban and Rural Freeways

Terrain	Design Speed (mph)		
	50	60	70
	Grades (Percent)		
Level	4	3	3
Rolling	5	4	4
Mountainous	7	6	5

4. Superelevation

Superelevation rates should range from 0.02 ft./ft. to 0.08 ft./ft. Figure 4-G may be used to establish superelevation rates on freeways.

10-02.3 Cross Section Elements

Detail requirements of cross section elements for roadways are discussed in Section 5, CROSS SECTION ELEMENTS, and should be referred to for appropriate guidelines in the design of freeways.

1. Pavement Surface

Pavement surface for freeways should be of either bituminous asphalt concrete or portland cement concrete.

2. Cross Slope

The cross slope on a tangent section should not exceed 2 percent and the cross slope should be uniform across the pavement

surface with the high point at the edge nearest the median. Section 5 discusses alternative ways of crowning divided, multi-laned highways.

For elevated freeways on viaducts, the cross slope should be uniform across the full width of the roadway.

Cross slope on ramps may be crowned in the center for 2 lane ramps or uniformly across the entire width of the roadway, depending on conditions at the ramp connection at the freeway and at the local street intersection.

3. Lane Width and Shoulders

Freeway through lanes should be 12 feet (3.66 m) in width. Auxiliary lanes and separate roadways should also have 12 foot (3.66 m) wide lanes. Continuously paved shoulders on both the right and left sides of the traveled way should be provided. Table 10-2 provides recommended paved shoulder widths for freeway facilities.

Table 10-2

Paved Shoulder Widths for Freeway Facilities

Type of Construction	Paved Shoulder Width (Feet)	
	Left of Traffic	Right of Traffic
Four and Six Lanes	6	10
Eight Lanes	10	10
Separate Roadways	6	10
Auxiliary Lanes	-	10
Single-Lane Ramps	4	8

Cross slope for shoulder sections is discussed in Section 5.

4. Medians

A wide separation between opposing travel lanes is safe and more comfortable for the motorists. Extra median width can also be used for mass transit or for additional lanes if more capacity is needed in the future.

a. Urban Freeways

The minimum median width for a 4-lane freeway is about 10 feet (3.05 m), which provides for two 4 foot (1.22 m) shoulders and a 2 foot (0.62 m) median barrier. The desired median width is 22 feet (6.71 m) for all urban freeways. The median shoulder should be completely paved and sloped in the plane of the pavement surface.

Minimum sight distance requirements along the median lane should be checked for relatively short radius curves.

Median crossovers for emergency or maintenance purposes are not warranted because of the close spacing of interchanges on urban freeways.

b. Rural Freeways

The median widths on rural freeways vary depending on the terrain and economics. The minimum width of the median should be 30 feet (9.15 m), with a 4 or 5 foot (1.22 or 1.52 m) paved shoulder and an unsurfaced area sloping gently (6:1 or flatter) from the adjoining shoulders to form a shallow swale in the center. This area should be cleared of all physical obstructions to provide a maneuvering area for out-of-control vehicles.

Where the terrain is extremely rolling, separate roadway profiles may be required and the median separation may be left in its natural state, providing a clear zone is provided for maneuvering area for out-of-control vehicles.

Emergency and maintenance crossovers on rural freeways are normally located where interchange spacing exceeds 5 miles, resulting in extreme adverse travel for emergency vehicles. Maintenance or emergency crossovers should not be located closer than 1,500 feet (458 m) to the end of a speed-change taper of a ramp or to any structure. Crossovers should be located only where above minimum stopping sight distance is provided and preferably should not be located within curves requiring superelevation.

The width of the crossover should be sufficient to provide safe turning movements and should have a surface capable of supporting the maintenance equipment used on it. The crossover should be depressed below shoulder

level to be inconspicuous to traffic and should have 10:1 or flatter side slopes to minimize its effect as an obstacle to uncontrolled vehicles. Crossovers should not be placed in restricted-width medians unless the median width is sufficient to accommodate the vehicle length, say 25 feet (7.62 m) or more. Where median barriers are employed, each end of the barrier at the opening should have a crashworthy terminal.

5. Freeway Cross Section

The general cross-sectional elements of a freeway are shown on Figure 10-A. Freeways may be designed as two separate roadways where appropriate to fit the terrain. Economy, aesthetics and safety are factors to be considered in this determination.

10-03 FREEWAY CONFIGURATION

10-03.1 General

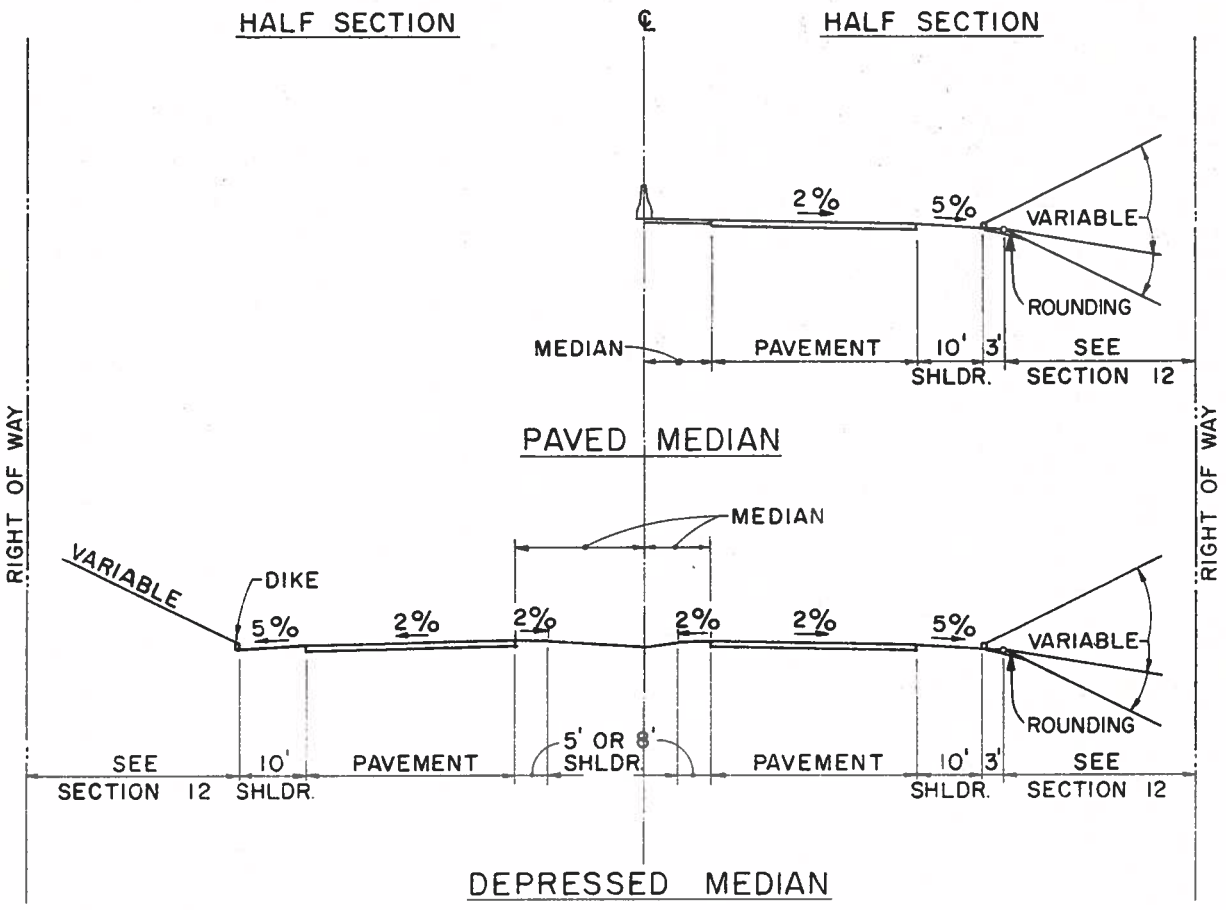
Freeways may be either depressed, elevated, ground level, or combinations of these, as is appropriate for conditions.

10-03.2 Depressed Freeway

This configuration is commonly called a freeway in "cut". The adjacent local streets are above the elevation of the freeway through lanes and important surface streets cross over the freeway. Interchange with the surface streets is usually accomplished by diamond interchanges where no frontage road exists. Higher type interchange is provided at intersections with major arterials.

Depending upon the availability of right-of-way, the side slopes of the freeway may be the same as any cut slope, or in restricted areas, retaining walls may be required, or combinations of retaining walls and cut slopes. Soil characteristics will determine the steepness of the cut slope.

Sight distance should be checked when designing slopes and retaining walls. On curved alignment, the slopes, walls and other side obstructions should be sufficiently removed from the pavement edge to provide the required stopping sight distance for vehicles in the right lane.



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CROSS SECTION ELEMENTS FOR FREEWAYS

FIGURE 10-A

10-03.3 Elevated Freeway

1. General

An elevated freeway may be on either a viaduct or an embankment. Continuous elevation of the freeway may be appropriate in level terrain where restricted right-of-way, high water table, extensive underground utilities, close pattern of streets that must be retained, or other circumstances, make depressed construction undesirable and perhaps uneconomical.

Several types of structures are used for viaducts carrying elevated freeways. The design of these viaducts is influenced by traffic requirements, right-of-way, topography, foundation conditions, character of urban development, requirements for interchange, availability of materials, and economic considerations. Viaducts are perhaps the most difficult of all freeway types to fit harmoniously into the environment.

The supporting columns for viaducts are located to provide reasonable clearance on each side and to leave much of the ground-level area free for other use. This design has advantages in that (1) practically all cross streets can be left open with little or no added expense, (2) existing utilities that cross the freeway right-of-way are little disturbed, and (3) surface traffic on cross streets usually can be maintained during construction with few, if any, detours. Conversely, the disadvantages of the design are the high costs of maintaining the structure and its closed drainage system, difficulty in obtaining a pleasant appearance, and possible policing of undeveloped space beneath the structure.

An elevated highway on an earth embankment must be of sufficient height to permit intersecting surface streets to pass under it. Freeways on embankments are feasible in suburban areas where crossing streets are widely spaced and where wide right-of-way and fill material are available. Usually, an embankment section occurs on a combination-type freeway in rolling terrain where excavated material from depressed portions is used for embankment. Where necessary, the fill may be confined by partial- or full-height walls on one or both sides. The sloped areas are available for planting to improve the appearance of the freeway.

2. Medians

Where a wide median can be attained, say 40 feet (12.20 m) or more, and the freeway is on a viaduct, 2 separate one-way

structures should be provided to permit more light and air to reach the area below. Where the median is approximately 40 feet (12.20 m) or less, with an 18 to 30 foot (5.49 to 9.15 m) opening, it is desirable from a safety standpoint to deck the median so that it is flush with the pavement, and to provide a median barrier.

3. Ramps

The design of ramps and connections for all types of freeways is covered in Section 11, INTERCHANGES, but some details and controls peculiar to elevated sections are discussed herein.

Freeways on viaducts are generally located in densely developed areas where property values are high and space is limited. However, the various forms of ramp connections, such as loops, diagonal ramps, and semidirect connections, are as adaptable to elevated freeways as to depressed or other freeway types.

Despite the high cost of elevated freeways, the lengths of speed-change lanes should not be reduced. The length of acceleration and deceleration lanes should conform to the requirements discussed in Section 11. Long acceleration lanes are especially needed because a ramp leading to an elevated structure is usually on a relatively steep upgrade.

Gore areas at exits from an elevated structure have a high accident potential. There appears to be no practical way to design the parapet and rail at these gores that are not hazardous if struck by an out-of-control vehicle. Therefore, the design should provide as much space in the gore area as practical, not only for recovery, but also to permit the installation of a crash cushion device.

4. Clearance to Building Line

The minimum lateral clearance between a freeway viaduct and adjacent buildings may be a significant cross sectional element. Major factors in determining the lateral clearance, where buildings are close to the roadway, are (1) working space for maintenance and repairs of structure or buildings, (2) protective space against possible fire damage, and (3) space for ladders and other firefighting equipment to reach upper floors of buildings from the street. Building offsets should be sufficient to insure adequate sight distance to signs where the alignment is curvilinear. All of these space requirements are reasonably well satisfied by a clearance of 15 to 20 feet (4.57 to 6.10 m).

10-03.4 Ground Level Freeway

Ground level freeway is often used in suburban and rural areas where cross streets are widely spaced. A major feature is that of profile change on each cross road so that it passes over or under the freeway. Substantial lengths of ground-level freeways are generally not feasible in heavily developed areas because the profiles of cross roads cannot be altered without severe impact on the community.

In residential areas, the outer separations and border areas should be widened to insulate the freeway from the surrounding area. Landscaping and earth berms are desirable in the border area.

10-04 STRUCTURE CLEARANCE

10-04.1 Horizontal Clearances

The horizontal clearance to bridge piers, abutments and retaining walls for freeways should be determined upon the basis of engineering judgment with the objective of eliminating fixed objects from near the edge of shoulder wherever economically feasible. A horizontal clearance of 30 feet (9.15 m) or more from the edge of the traveled way is desirable. Lesser clearances may be used where span length, median width or other controls make the desired clearance unreasonable.

10-04.2 Vertical Clearances

The minimum vertical clearance for grade separation structures over the freeway shall be 16-1/2 feet (5.03 m) over the entire width of traveled way and shoulders on the through facility, speed change lanes, ramps and collector roads on all parts of the freeway system.

The vertical clearance for pedestrian separation structures over the freeway shall be 2 feet (0.62 m) greater than the vertical clearance provided for other grade separation structures on the portion of the freeway involved. If the pedestrian separation structure is protected by a grade separation structure, a clearance greater than that of the grade separation structure need not be provided.

Sign structures should have a vertical clearance of 18 feet (5.49 m).

10-05 OTHER DESIGN CONSIDERATIONS

10-05.1 Traffic Barriers

1. General

Traffic barriers are classified into 2 categories, longitudinal barriers and impact attenuation devices or crash cushions. Detailed discussions on barriers are covered in Section 13, TRAFFIC BARRIERS.

2. Median Barriers

a. Warrants (See Section 13)

In general, a median barrier is optional for freeways with an ADT of less than 20,000 VPD and with a median width of 30 feet (9.15 m) or greater.

Barriers are not required for freeways with median widths greater than 50 feet (15.25 m). However, sections of freeway with high crossover accident experience warrant use of a median barrier.

b. Type

In Hawaii, the median barrier used on freeways shall be State Standard Guardrail Type 4 (rigid barrier).

3. Roadside Barriers

Roadside barriers are longitudinal systems used to shield vehicles from hazards in the roadside. Roadside barriers are commonly called guardrails and are located at the edge of high embankments, or to shield hazards, such as sign posts, bridge piers and the like, to reduce the severity of damage and/or injury to errant vehicles and occupants.

Refer to Section 12, ROADSIDE DESIGN and Section 13 for design guides relative to roadside barriers.

4. Crash Cushions

Crash cushions should be placed at all rigid objects such as bridge piers, sign posts, exit gores at bridge rail end, and the like. See Section 13 for design guidelines.

10-05.2 Curbs and Bituminous Dikes

Curbs and dikes should generally not be used in freeway design.

Bituminous dikes should only be used to confine drainage where it is necessary to protect side slopes susceptible to erosion.

10-05.3 Roadside Design

1. General

Roadside design is of major importance in freeway safety and should be considered as an integral part of the overall design, rather than a byproduct of highway safety programs. The cleared roadside should ideally be designed to enable an errant vehicle to recover and return to the roadway or, at the very least, reduce the severity of the accident.

2. Side Slopes

a. Side Slope Standards

Side slope design should follow the recommendation of the project soils engineer's report. In general, side slopes should be no steeper than 2:1 for cut and fill slopes.

Slopes flatter than the steepest allowable should be used, if economically feasible. In general, flatter slopes are safer, more resistant to erosion, and more pleasing to the eye.

For aesthetics, where normal slopes catch in a distance less than 18 feet (5.49 m) from the hinge point, a uniform catch point at least 18 feet (5.49 m) from the edge of shoulder should be used.

b. Clearance from Slope to Right-of-Way Lines

The minimum clearance from the right-of-way line to catch point of a cut or fill slope should be 10 feet (3.05 m). When feasible, at least 15 feet (4.57 m) should be provided.

Following are minimum clearances recommended for cuts higher than 30 feet (9.15 m):

(1) 20 feet (6.10 m) for cuts from 30 feet (9.15 m) to 50 feet (15.25 m) high.

(2) 25 feet (7.62 m) for cuts from 50 feet (15.25 m) to 75 feet (22.85 m) high.

(3) 1/3 the cut height for cuts above 75 feet (22.85 m), but not to exceed a width of 50 feet (15.25 m).

c. Contour Grading and Slope Rounding

Pleasing aesthetic roadside effects can be developed with easy flowing contours. Contour grading is an important factor in roadside preparation, erosion control, and maintenance of planting.

Contour grading plans should be prepared to facilitate anticipated roadside treatment. These plans should show flattening of slopes where irregular widths of right-of-way permit.

The tops and ends of all cut slopes should be rounded where the material is other than solid rock. A layer of earth overlying a rock cut also should be rounded.

d. Access Control Fencing

Fences are provided along the freeway rights-of-way to control access. In areas where existing fences or walls along the right-of-way provide a satisfactory barrier, fencing may be omitted.

Generally, chain link fencing is installed in urban areas and wire fence with metal post is installed in rural areas. The designer should consider the potential for trespassing in determining the appropriateness of chain link fencing in rural areas.

All access control fencing shall be installed within the State right-of-way.

SECTION 11.

I N T E R C H A N G E S

(follows California criteria)

11-01 GENERAL

The capacity of arterial highways, particularly in urban areas, to handle high volumes of traffic safely and efficiently depends, to a considerable extent, upon their ability to accommodate crossing and turning movements at intersecting highways. The greatest efficiency, safety and capacity are attained when the intersecting through traffic lanes are grade separated.

A traffic interchange is a combination of ramps and grade separations at the junction of two or more highways for the purpose of reducing or eliminating traffic conflicts. Safety and traffic capacity are increased by the provision of a traffic interchange. Crossing conflicts are eliminated by grade separations. Turning conflicts are either eliminated or minimized, depending upon the type of interchange design.

11-02 WARRANTS FOR INTERCHANGES

11-02.1 Freeways

Interchanges are provided on freeways at all intersections where access is to be permitted. Other intersecting roads or streets are either grade separated, terminated, or rerouted.

11-02.2 Other Highways

On highways with only partial control or no control of access, definite warrants cannot be specified as they may differ at each location. The following factors should be considered in analyzing a particular situation:

1. Elimination of Congestion

   Insufficient capacity at the intersection of heavily traveled highways results in intolerable delays and congestion in one or all approaches. The inability to provide the essential capacity with an intersection at grade provides the warrant for an interchange.

2. Elimination of Hazard

Some intersections at grade have a high accident rate even though serving light traffic volumes. Other, more heavily traveled intersections, have a history of serious accidents. If the hazards at such intersections cannot be eliminated by more inexpensive methods, providing an interchange facility may be warranted.

3. Site Topography

At some sites, the topographic conditions may be such that the provision of an interchange facility may entail no more cost than an at-grade intersection.

4. Traffic Volume

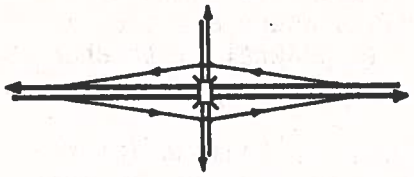
For a new intersection under design, an interchange would be warranted where a capacity analysis indicates that an at-grade design cannot be provided to satisfactorily serve, without undue delay and congestion, the traffic volumes and turning movements expected.

11-03 INTERCHANGE TYPES

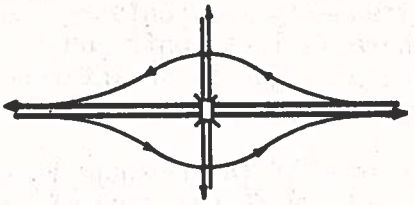
11-03.1 General

The selection of an interchange type and its design are influenced by many factors, including the following: the speed, volume and composition of traffic to be served, the number of intersecting legs, the standards and arrangement of the local street system including traffic control devices, topography, right-of-way controls, local planning, proximity of adjacent interchanges, community impact consideration and cost. Even though interchanges are, of necessity, designed to fit specific conditions and controls, it is desirable that the pattern of interchange ramps along a freeway follow some degree of consistency. It is frequently desirable to rearrange portions of the local street system in connection with freeway construction in order to effect the most desirable overall plan of traffic service and community development.

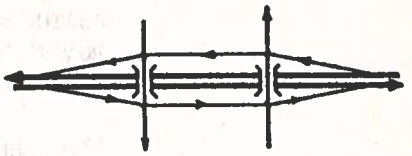
Interchange types are characterized by the basic shapes of ramps: namely; diamond, loop, directional or variations of these types. Many interchange designs are combinations of these basic types. Schematic interchange patterns are illustrated in Figure 11-A.



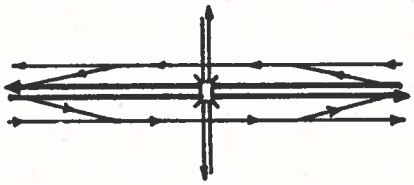
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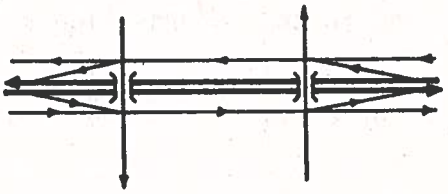
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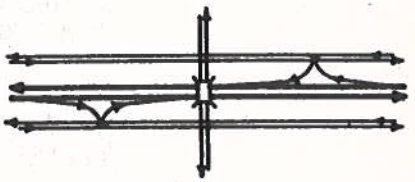
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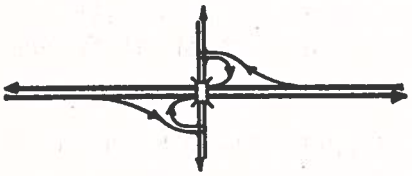
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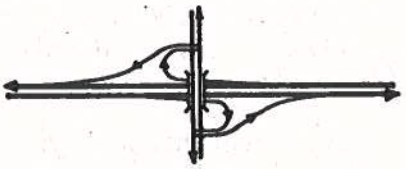
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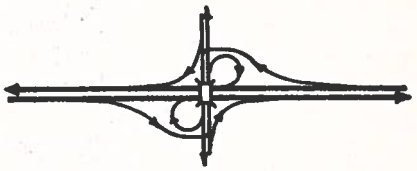
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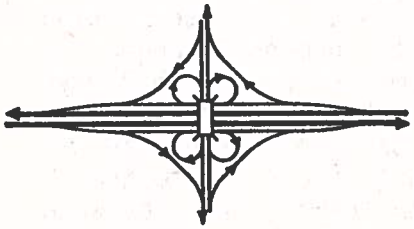
TYPE L-7



TYPE L-8



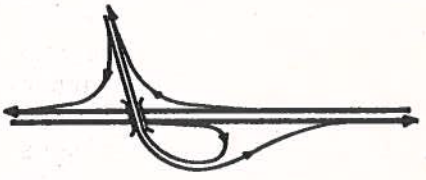
TYPE L-9



TYPE L-10



TYPE L-11



TYPE L-12

SAMPLES OF LOCAL STREET INTERCHANGES

FIGURE II-A

11-03.2 Local Street Interchanges

The use of isolated ramps or partial interchanges should be avoided because wrong-way movements are more prevalent at isolated off-ramps and there is less confusion to motorists where all traffic movements are provided at an interchange. In general, interchanges with all ramps connecting with a single cross street are preferred.

The simplest form of interchange is the diamond. Diamond interchanges provide a high standard of ramp alignment, direct turning maneuvers at the crossroad, and usually minimum construction costs. The diamond type is adaptable to a wide range of traffic volumes. The capacity is limited by the capacity of the intersection of the ramps at the cross road. This capacity may be increased by widening the ramps to 2 or 3 lanes at the cross road and by widening the cross road in the intersection area.

The compact diamond (Type L-1) is most adaptable where the freeway is depressed or elevated and the cross street retains a straight profile.

The spread diamond (Type L-2) is adaptable where the grade of the cross street is changed to pass over or under the freeway. The ramp terminals are spread in order to achieve maximum sight distance and minimum intersection cross slope, commensurate with construction and right-of-way costs, travel distance and general appearance. A spread diamond has the advantage of low profile ramps and the flexibility of permitting the construction of future loop ramps if required.

The split diamond interchange (Type L-3) is appropriate when interchanging with a couplet of one way streets.

Types L-4, L-5 and L-6 are interchange systems used where the freeway alignment is placed between parallel streets. Types L-4 and L-5 are used where the parallel streets will operate with one-way traffic. In Type L-4, slip ramps merge with the frontage street, and in Type L-5, the ramps terminate at the intersection of the frontage road with the cross street, forming 5-legged intersections. In Type L-6, the freeway ramps connect with 2 way parallel streets. This type should rarely be used because it includes short radius hook off ramps. The parallel streets in the Types L-4, L-5 and L-6 situation are usually too close to the freeway to permit ramp intersections on the cross street between the parallel frontage streets.

The simplest cloverleaf interchange is the two quadrant cloverleaf, Type L-7 or Type L-8, or a combination where the 2 loops

are on the same side of the cross street. Two quadrant cloverleaf interchanges should be used only in connection with controls which preclude the use of diamond ramps in all 4 quadrants. These controls include right-of-way controls, a railroad track paralleling the cross street, and a short weaving distance to the next interchange.

The Type L-9 interchange provides loop on ramps in addition to the 4 diamond type ramps. This interchange is suitable for large volume turning movements. Left turn movements from the cross road are eliminated, thereby permitting 2 phase operation at the ramp intersections when signalized. Because of this feature, the Type L-9 interchange usually has capacity to handle the volume of interchange traffic which can be accommodated on the cross road.

The 4 quadrant cloverleaf interchange (Type L-10) has free flow characteristics for all movements. It has the disadvantage of a higher cost than a diamond or partial cloverleaf design and a relatively short weaving section between the loop ramps. Collector distributor roads should be incorporated in the design of 4 quadrant cloverleaf interchanges to separate the weaving conflicts from the through freeway traffic.

A trumpet design, Type L-11 or Type L-12, may be used when a cross road terminates at a freeway. This design shall not be used if future extension of the cross road is probable.

11-03.3 Freeway to Freeway Interchanges

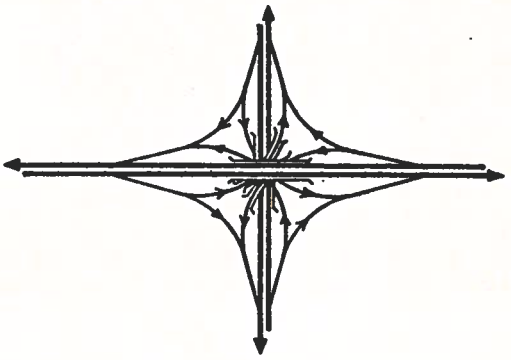
1. General

The function of the freeway to freeway interchange is to link freeway segments together so as to provide the optimum highway system. Parameters such as cost, environment, community values, mobility, and safety can sometimes be in direct conflict. The designer must meet the challenges involved in resolving these issues. A comprehensive evaluation of alternative interchange plans is necessary to achieve the most cost effective solution.

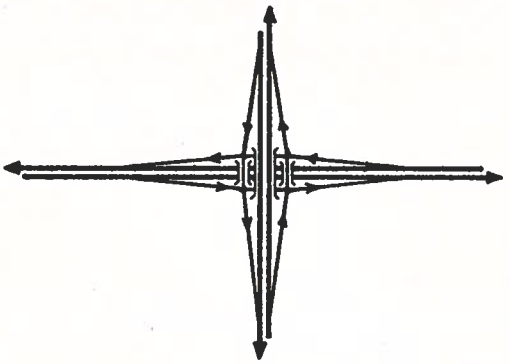
2. Types

Several freeway to freeway interchange design configurations are shown on Figure 11-B. Many combinations and variations may be formed from these basic interchange types.

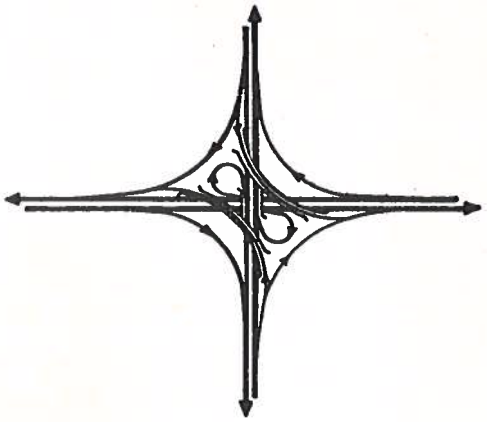
The 3 level diamond, Type F-2, is a possible design solution where turning movements are accommodated through intersections



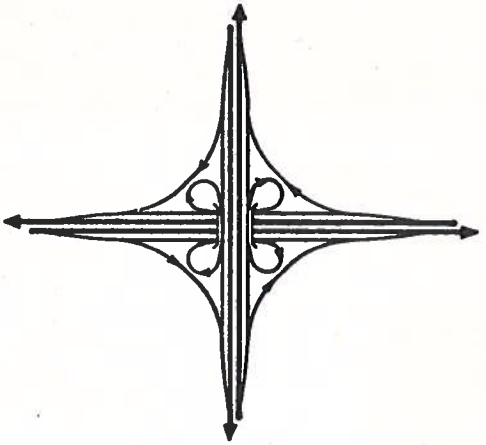
TYPE F-1



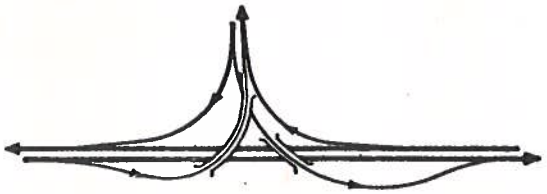
TYPE F-2



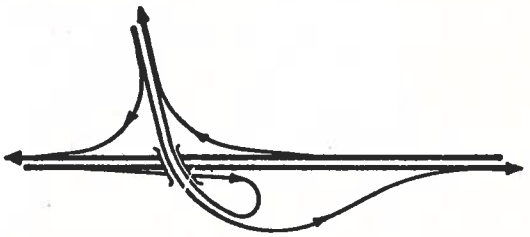
TYPE F-3



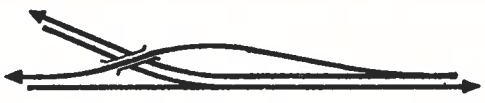
TYPE F-4



TYPE F-5



TYPE F-6



TYPE F-7



TYPE F-8

SAMPLES OF FREEWAY TO FREEWAY INTERCHANGES

FIGURE II-B

separated from the through freeway. While in a literal sense the 3 level diamond is not truly a freeway to freeway interchange, it does exemplify one means of reducing costs and right-of-way requirements at the intersection of 2 freeways. When appropriate, local service, one way street couplets can be accommodated in this design.

The 4 quadrant cloverleaf with collector roads, Type F-4, is ordinarily the most economical freeway to freeway interchange solution when all turning movements are provided. The 4 quadrant cloverleaf is generally applicable in situations where turning volumes are low enough to be accommodated in the short weaving sections.

Direct connections are appropriate in lieu of loops when required by traffic demands or other specific site conditions. The 2 loop, 2 direct connection interchange, Type F-3, is often an appropriate solution. The weaving conflicts which ordinarily constitute the most restrictive traffic constraint are eliminated, yet cost and right-of-way requirements may be kept within reasonable bounds. The Type F-1 interchange, with all direct connections, provides the maximum in mobility and safety. However, the high costs associated with this design require that the benefits be fully substantiated.

Types F-5, F-6, F-7 and F-8 are examples of interchange designs where one freeway terminates at the junction with another freeway. In general, the standard of alignment provided on the left or median lane connection from the terminating freeway should equal or approach as near as possible that of the terminating freeway. Terminating the median lane on a loop should be avoided. It is preferable that both the sign route and the major traffic volume be to the left at the branch connection diverge. The choice between Types F-7 and F-8 should include considerations of traffic volumes, route continuity, and map relatability. When these considerations are in conflict, the choice is made on the basis of judgment of their relative merits.

3. Design Considerations

a. Cost

The differential cost between interchange types is often significant. A cost effective approach will tend to assure that an interchange is neither over nor underdesigned. Decisions as to the relative values of the previously mentioned parameters must be consistent with decisions reached on adjacent main line freeways.

b. System Balance

The freeway to freeway interchange is a critical link in the total freeway system. The level of traffic service provided will have impact upon the mobility and overall effectiveness of the entire roadway system. For instance, traffic patterns will adjust to avoid repetitive bottlenecks, and to the greatest degree possible, to temporary closures, accidents, etc. The freeway to freeway interchange should provide flexibility to respond to these needs so as to maximize the cost effectiveness of the total system.

c. Elimination of Connections

Freeway to freeway interchanges need not include all possible turning movements. Connections serving minor traffic volumes or significantly out of direction traffic movements should be omitted unless it can be demonstrated that traffic service and other benefits justify the costs. Considerations include:

(1) Traffic Volumes--Turning traffic volumes may be nominal or a small percentage of the total interchanging traffic.

(2) Circuity--Connections may only serve significantly out of direction traffic movements.

(3) Use of Local Streets--Low turning volumes may be accommodated reasonably well by way of local interchanges and the local street system. There may be both traffic operational advantages and economic savings from utilizing and improving this local system in lieu of providing the freeway to freeway connections.

(4) Staging--Staging possibilities should be thoroughly assessed.

(5) Effect on Other Traffic Movements--Provision of minor movements may be detrimental to traffic operation on major branch connections and the main line freeways.

(6) Costs--All construction and right-of-way savings and costs attributable to the elimination of turning movements should be considered. This includes

possible additional local interchange and street costs, as well as reductions in the freeway to freeway interchange costs.

(7) Signing--Freeway to freeway traffic may be signed via the local street system. Routes shall be sufficiently direct and well oriented to insure that the unfamiliar driver can follow them.

d. Local Traffic Service

In metropolitan areas, a freeway to freeway interchange is usually superimposed over an existing street system. Local and through traffic requirements are often in conflict. Combinations of local and freeway to freeway interchanges can result in designs that are both inordinately costly and so complex that the important design concepts of simplicity and consistency are compromised. Therefore, alternate plans separating local and freeway to freeway interchanges should be fully explored. Less than desirable local interchange spacing may result; however, this may be compensated for by upgrading the adjacent local interchanges and street system.

Local traffic service interchanges should not be provided in conjunction with a freeway to freeway interchange unless it can be demonstrated that traffic operational benefits justify the costs.

e. Alignment

It is not considered practical to establish fixed freeway to freeway interchange alignment standards. An interchange must be designed to fit into its environment. Alignment is often controlled by external factors such as terrain, buildings, street patterns, route adoptions, and community value considerations. Normally, loops have radii in the range of 150 feet to 200 feet (45.8 m to 61 m) and direct connections have radii in the range of 600 feet to 1,000 feet (183 m to 305 m). Larger radii may be proper in situations where the skew or other site conditions will result in minimal increased costs. Direct connection radii of at least 850 feet (259.2 m) are desirable from a traffic operational standpoint. Provision of ample sight distance to curves and points of traffic divergence is as important from a safety standpoint as are high alignment standards.

11-04 INTERCHANGE DESIGN ELEMENTS

11-04.1 General

Geometric design for all interchange roadways should follow the design guides as covered in Section 4, BASIC GEOMETRIC DESIGN ELEMENTS. Ramp terminals at surface streets should be designed in accordance with guidelines provided in Section 6, AT-GRADE INTERSECTIONS, except as noted in this section. Additional design guides peculiar to interchanges will be discussed in this section.

11-04.2 Sight Distance

Sight distance along the through roadways and all ramps should be at least equal to the minimum safe stopping sight distance for the applicable design speed. In addition, the minimum stopping sight distance to an exit or branch connection on a freeway should be 1000 feet (305 m), and the minimum stopping sight distance to a secondary exit on a collector distributor road should be 600 feet (183 m), both measured to center of the ramp lane right of the exit nose.

11-04.3 Alignment, Profile and Cross Section

Traffic passing through an interchange should be provided the same degree of utility and safety as on the approaching highways. The standards for design speed, alignment, profile and cross section for the main lanes through the interchange area should be as high as on the approach legs. Desirably, the alignment and profile of the through highways at an interchange should be relatively flat with high visibility. The full roadway cross section should be continued through the interchange area and adequate clearances provided at structures.

The grade lines for overcrossing and undercrossing roadways should, in general, be determined by the anticipated design speed on the cross road.

11-04.4 Ramps

1. General

The term ramp denotes a turning roadway at an interchange for travel between intersection legs. The components of a ramp are a terminal at each end and a connecting road, usually with some curvature, and on a grade. The ramp between freeways is called a branch connection. Ramps are one way roadways.

2. Design Speed

It is not practical to provide design speeds on ramps that are comparable to those on the through roadways. Generally, ramp design speeds range from 25 mph to 40 mph. Branch connection design speeds range from 40 mph to 60 mph. The sharpest, or the controlling curve on the ramp usually determines the design speed of the ramp.

3. Grades

Ramp grades are usually controlled by the requirements of sight distance. Grade at exits should be so designed that at least 50 feet (15.25 m) of ramp pavement beyond the exit nose is visible from the beginning of the exit gore. Crest vertical curves located just beyond the exit nose should be designed with a minimum 50 mph stopping sight distance. Beyond this point, design speed as low as 30 mph should be considered only in the most restrictive situations. On-ramp profiles should approximately parallel the profile of the freeway for at least 100 feet (30.5 m) prior to the inlet nose to provide intervisibility in merging situations.

The grades on the central portion of the ramp, i.e., other than at the freeway or at the local street intersection, should be as follows for on ramps with ascending grades:

- design speed 35 to 40 mph - 4 to 6 percent
- design speed 25 to 30 mph - 5 to 7 percent
- design speed 15 to 25 mph - 6 to 8 percent

Generally for on ramps on a downgrade, the grades may be up to 2 percent higher. The important criteria is to provide a long enough vertical curve so the on ramp profile and the freeway profile are approximately parallel for at least 100 feet (30.5 m) prior to the inlet nose.

Grade lines are also controlled by structure clearances.

4. Location of Ramp Intersection on Cross Road

Factors which influence the location of ramp intersections on the cross road include sight distance, construction and right-of-way costs, circuity of travel for left turn movements, cross road gradient at ramp intersections, storage requirements for left turn movements off the cross road, and the proximity of other local road intersections.

For left turn maneuvers from an off ramp at an unsignalized intersection, the length of cross road open to view should be greater than the product of the prevailing speed of vehicles on the cross road, and the time required for a stopped vehicle on the ramp to safely execute a left turn maneuver. This time is estimated to be 7-1/2 seconds.

Horizontal restrictions to sight distance may be caused by bridge railing, bridge piers, or grading slopes, and is measured between the center of the outside lane approaching the ramp and the eye of the driver of the ramp vehicle, assumed 8 feet back of the stop bar at the cross road. Figure 11-C illustrates the determination of ramp setback from an overcrossing structure on the basis of sight distance controlled by the bridge rail. The same relationship exists for sight distance controlled by bridge piers or grading slopes.

Where design controls prevent locating the ramp terminal a sufficient distance from the structure to achieve the required sight distance, the sight distance should be obtained by flaring the end of the overcrossing structures or setting back the piers or end slopes of an undercrossing structure.

It is desirable to provide at least 300 feet (91.5 m) distance between ramp intersections and local road intersections. Double or closely spaced intersections are difficult to properly signalize, sign and delineate.

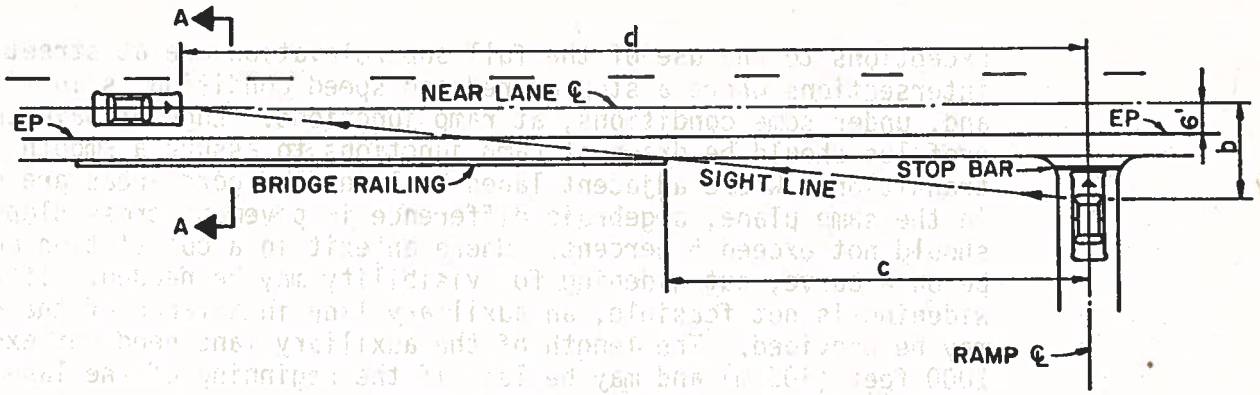
Sharp curves at an off ramp terminal (at the intersection with the local street) should be avoided, even if such an intent is to provide an acceleration lane for merging into the local street traffic. It is often better to provide a near 90 degree intersection with stop sign control.

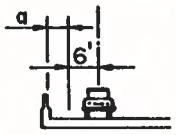
Slip ramps from the freeway to the local street should also be discouraged because of limited sight distance usually encountered at the merge with the local street traffic.

11-05 SUPERELEVATION FOR RAMPS

The factors controlling superelevation rates for main line conditions discussed in Section 4 apply also to ramps. Standard superelevation should be provided on ramp curves. Ramp alignment which precludes the attainment of standard superelevation within a reasonable transition distance should be avoided. In special restrictive situations where full superelevation with a given ramp alignment is not feasible within the transition lengths given in Figure 4-H, the highest possible rate should be used.

UNSIGNALIZED AND BASED ON 7-1/2 SECOND
HORIZONTAL SIGHT DISTANCE CRITERIA





SECTION A-A

$$c = d\left[\frac{b-(a+6')}{b}\right]$$

V MPH	d (FT.)
30	330
40	440
50	550
60	660

a = DISTANCE FROM EDGE OF PAVEMENT TO BRIDGE RAILING

b = DISTANCE FROM CENTER OF NEAR LANE TO EYE OF RAMP VEHICLE RAMP DRIVER'S EYE IS ASSUMED 8' BEHIND STOP BAR. STOP BAR IS LOCATED A MINIMUM OF 6' FROM TRAVELED WAY OR AT OUTSIDE EDGE OF SHOULDER, IF GREATER. WHERE A CROSSWALK IS PROVIDED, IT IS ASSUMED THAT VEHICLE WILL BE STOPPED AT EDGE OF CROSSWALK.

c = RAMP SET BACK FROM END OF BRIDGE RAILING

d = SIGHT DISTANCE ALONG HIGHWAY FROM INTERSECTION (d = 1.467V x 7-1/2 SEC.)

v = ANTICIPATED PREVAILING SPEED ON CROSSROAD

a (FT.)	b (FT.)	RAMP SET-BACK = C (FT.)			
		30 MPH	40 MPH	50 MPH	60 MPH
4	21	173	230	288	346
7*	24	151	202	252	302
8	23	129	172	215	258
13*	30	121	161	202	242

*INCLUDES SIDEWALK

NOTE:

TABULAR VALUES BASED ON THE FOLLOWING WIDTHS:
SIDEWALK = 5'
CROSSWALK = 6'
STOP BAR AND CROSSWALK LINES = 12"

MINIMUM SET-BACK FOR DIAGONAL RAMP INTERSECTIONS

FIGURE II-C

Exceptions to the use of the full superelevation are at street intersections where a stop or reduced speed condition is in effect and, under some conditions, at ramp junctions. Edge of pavement profiles should be drawn at ramp junctions to assure a smooth transition. Where adjacent lanes or lanes and gore areas are not in the same plane, algebraic difference in pavement cross slope should not exceed 5 percent. Where an exit in a cut section must be on a curve, cut widening for visibility may be needed. If this widening is not feasible, an auxiliary lane in advance of the exit may be provided. The length of the auxiliary lane need not exceed 1000 feet (305 m) and may be less if the beginning of the lane occurs on a tangent section of the freeway.

11-06 FREEWAY ENTRANCES AND EXITS

11-06.1 Basic Policy

All freeway entrances and exits shall connect to the right of through traffic.

11-06.2 Standard Designs

Figure 11-D illustrates the standard entrance and exit designs to be used on all freeways.

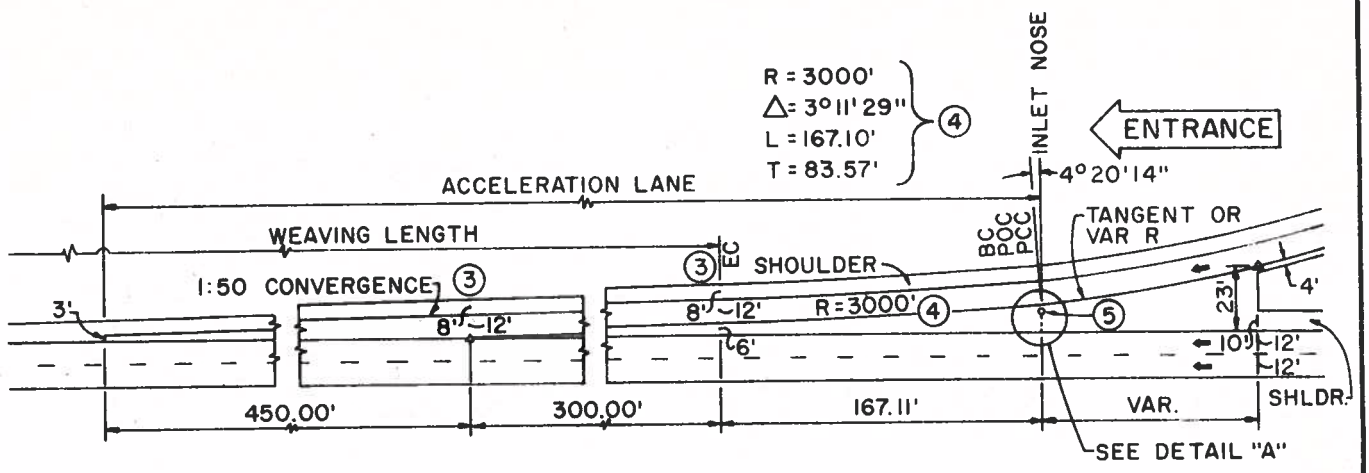
11-06.3 Location on a Curve

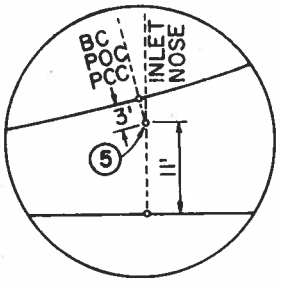
Freeway entrances and exits should be located on tangent sections wherever possible in order to provide maximum sight distance and optimum traffic operation. Where curve location is necessary, the ramp tapers should also be curved. At freeway exits, the radius of the taper should be about the same radius as the freeway edge of pavement in order to develop the same degree of divergence as the standard design. At freeway entrances on a curve, the ramp alignment will vary with the radius of curvature of the freeway.

11-07 RAMP TYPES

11-07.1 Single-Lane Ramps

Single-lane (local) ramps on tangent alignment and on curves with radii equal to or greater than 200 feet (61 m) shall consist of a 12 foot (3.66 m) width of traveled way, bordered by an 8 foot (2.40 m) paved shoulder on the right and a 4 foot (1.22 m) paved shoulder on the left. On curves less than 200 foot (61 m) radius, the lane width given in Table 11-1 should be used. When both left and right turns are permitted at the intersections with the local



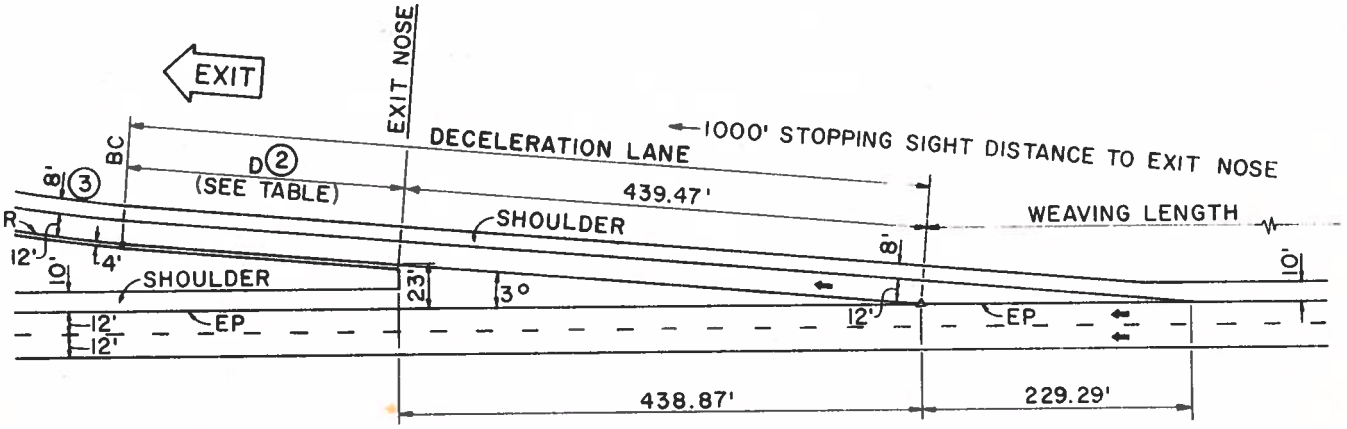


DETAIL "A"

R (FEET)	MIN. D (FEET)
LESS THAN 300	300
300 - 500	200
500 - 1000	150
1000 OR MORE	0

NOTES

1. MINIMUM LENGTH BETWEEN EXIT NOSE AND END OF RAMP IS 450' FOR FULL STOP AT END OF RAMP.
2. "D" DISTANCE MAY BE ELIMINATED IN CETAIN CASES SUCH AS A RAMP FROM A COLLECTOR ROAD.
3. SINGLE LANE FREEWAY TO FREEWAY CONNECTION THE RIGHT PAVED SHOULDER SHALL BE 10 FEET.
4. WHEN FREEWAY IS NOT ON TANGENT ALIGNMENT, SELECT RADIUS TO APPROXIMATE SAME DEGREE OF CONVERGENCE.
5. LOCATE AS IF IT WERE TO BE CENTER OF A 1' CURB NOSE.



FREEWAY ENTRANCES AND EXITS

FIGURE II-D

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road, the ramp may be widened to permit these turns to be made simultaneously. Exit ramps in urban areas may require 2 or 3 lanes at the intersection with the cross road to provide additional storage and capacity. If the length of single-lane ramp exceeds 1000 feet (305 m), consideration may be given to providing a section of 2 lane width on the ramp to permit passing maneuvers. An illustration of a single-lane exit ramp widening to a 2 lane ramp is shown on Figure 11-E. The decision as to the use of Alternate A or Alternate B is generally based on providing the additional lane for the minor movement.

Table 11-1

Ramp Lane Widths

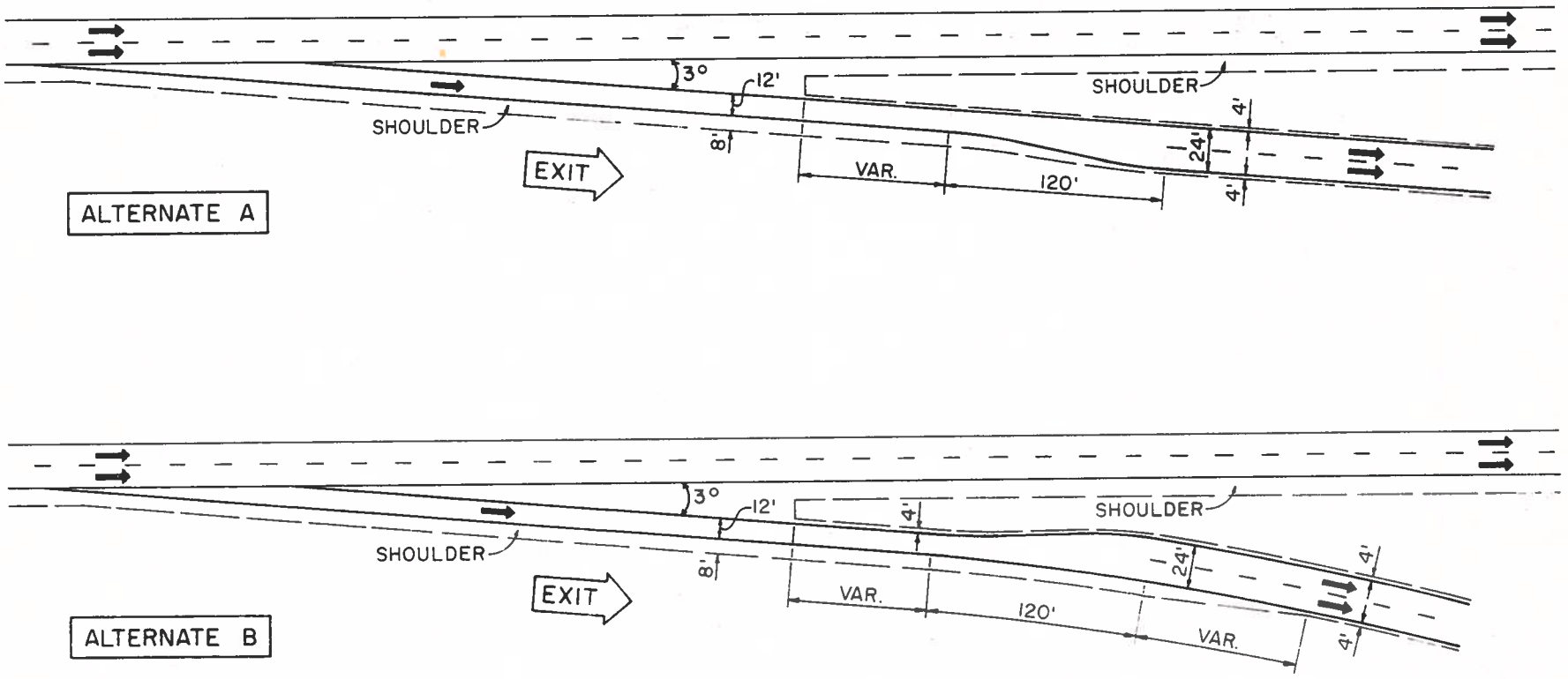
Radius of Outer Pavement Edge (Feet)	Lane Width Feet
75	18
100	15
130	14
150	13
180	13
200 and up	12

11-07.2 Two-Lane Exit Ramps

In general, where design year estimated values exceed 1500 passenger equivalent vehicles per hour, a 2 lane width of ramp should be provided initially. Provision may be made for possible widening to 3 lanes at the cross road intersection. Figure 11-F illustrates the standard design for a 2 lane exit. An auxiliary lane approximately 1300 feet (397 m) long should be provided in advance of a 2 lane exit. For volumes less than 1500, but more than 900, a one lane width exit ramp should be provided with provision for adding an auxiliary lane and an additional lane on the ramp.

11-07.3 Two-Lane Entrance Ramps

A standard 2 lane entrance ramp is illustrated in Figure 11-F. This design should be used where the estimated design year volume exceeds 1500 passenger equivalent vehicles per hour. With a 2



TRANSITION TO 2-LANE EXIT RAMP

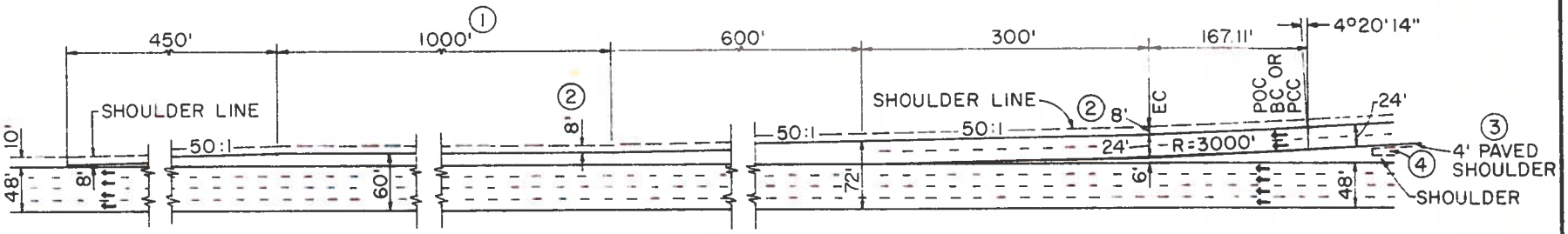
FIGURE II-E

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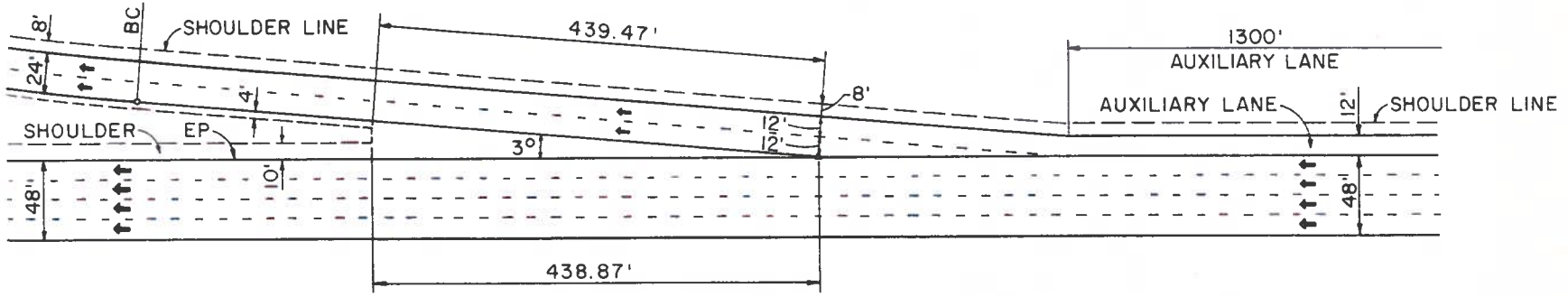
NOTES:

1. 2500' FOR BRANCH CONNECTIONS
2. 10' FOR BRANCH CONNECTIONS
3. 5' FOR BRANCH CONNECTIONS
4. B-4 CURB OPTIONAL

R = 3000'
Δ = 3°11'29"
L = 167.10'
T = 83.57'



TWO-LANE ENTRANCE FOR RAMPS OR BRANCH CONNECTIONS



STANDARD 2-LANE RAMP EXIT CONNECTION (CASE I SHOWN)

CONNECTIONS USING STANDARD ENTRANCE AND EXIT

FIGURE 11-F

lane entrance, a 1000 foot (305 m) acceleration lane parallel to the freeway shall be provided beyond the merge of one lane of the inlet. For design year volumes between 900 and 1500 vehicles per hour, a one lane entrance should be provided with provision for ultimate widening to a 2 lane entrance, including the addition of the parallel acceleration lane.

11-07.4 Distance Between Successive Exits

The minimum distance between successive exit ramps shall be 1000 feet (305 m) on the freeway and 600 feet (183 m) on collector roads.

11-07.5 Curbs

Curbs should not be used on ramps except in the following locations:

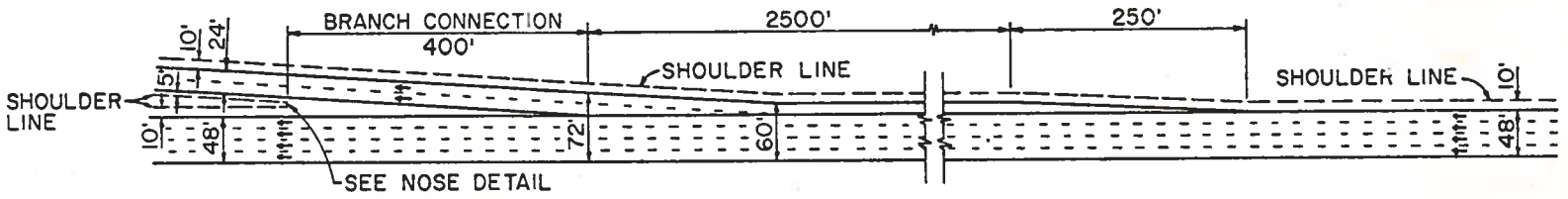
1. Curbs may be used where necessary at the ramp connection with the local street for the protection of pedestrians, for channelization and to provide continuity of construction at the local facility.

2. Curbs may be used where necessary to control drainage.

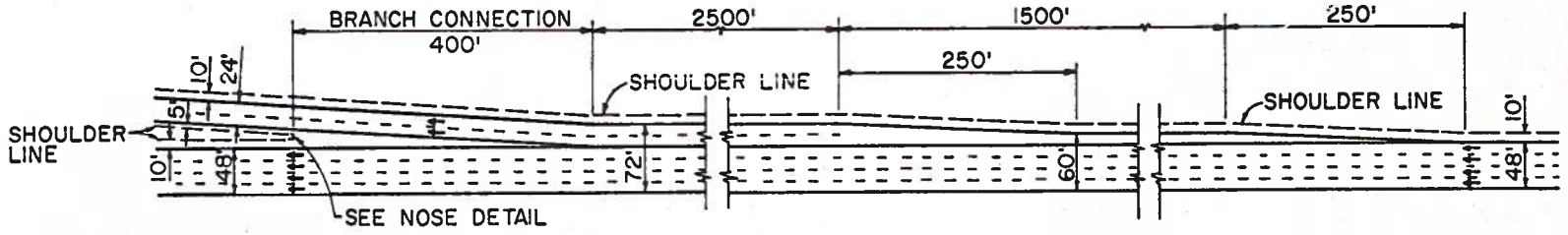
11-08 FREEWAY-TO-FREEWAY CONNECTIONS

Freeway-to-freeway connections may be single lane or multi-lane. A multi-lane connection should be provided when the design year volume exceeds 1500 passenger equivalent vehicles per hour. Where design year volume is between 900 and 1500 vehicles per hour, initial construction shall provide one lane width connection with the capability of ultimately adding an additional lane if required. The width of shoulders on a single lane connection shall be 5 feet (1.53 m) on the left and 10 feet (3.05 m) on the right. A single lane connection in excess of 1500 feet (458 m) in length shall be widened to 2 lanes with 5 foot (1.53 m) shoulders.

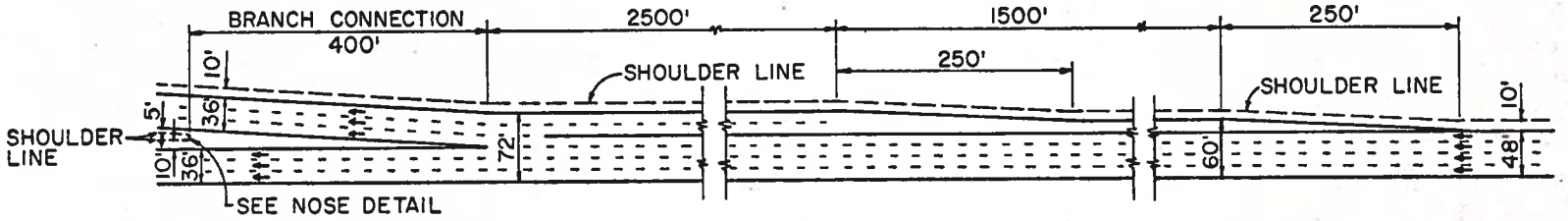
A branch connection is defined as a multi-lane connection between 2 freeways. Branch connections should be designed as shown in Figure 11-G for diverging connections and as shown in Figure 11-F for merging connections. At a branch merge, a 2500 foot (763 m) length of auxiliary lane shall be provided beyond the merge of one lane of the inlet, except where it does not appear that capacity on the freeway will be reached in the foreseeable future. In this case, the length of auxiliary lane should be 1000 feet (305 m). For diverging connections where less than capacity conditions beyond the design year are anticipated, the length of auxiliary lane in advance of the exit should be 1300 feet (397 m).

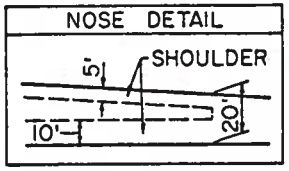


CASE 1 LESS THAN 35% TURNING TRAFFIC



CASE 2 35% TO 50% TURNING TRAFFIC





CASE 3 MORE THAN 50% TURNING TRAFFIC

(PROVIDE MAIN LINE STRUCTURE CLEARANCE BEYOND RAMP NOSE TO ACCOMMODATE FUTURE THROUGH LANE.)

NOTE: TURNING VOLUMES EXPRESSED AS A PERCENT OF TOTAL APPROACH VOLUME.

DIVERGING BRANCH CONNECTIONS

FIGURE 11-G

11-09 AUXILIARY LANES

In order to insure satisfactory operating conditions, auxiliary lanes may be added to the basic width of traveled way.

Where an entrance ramp of one interchange is closely followed by an exit ramp of another interchange, the acceleration and deceleration lanes may be joined with an auxiliary lane. This should be the general practice where the weaving distance, measured as shown in Figure 11-D, is less than 2000 feet (610 m). Where interchanges are more widely spaced and ramp volumes are high, the need for an auxiliary lane between the interchanges should be determined by an across-freeway-lane volume check. This check should include consideration of freeway grade and volume of trucks.

Auxiliary lanes may be used for the orientation of traffic at 2 lane ramps or branch connections as illustrated on Figure 11-F and Figure 11-G. The length and number of auxiliary lanes in advance of 2 lane exits are based on percentages of turning traffic.

11-10 LANE REDUCTION

Lane reduction below the basic number of lanes is not permissible through a local interchange. It will be permitted at branch connections where more than 50 percent of the traffic is turning, as illustrated in Figure 11-G, Case 3. Structure clearance beyond the branch connection should be adequate to accommodate future construction of the dropped lane if required.

Where the reduction in traffic volumes is sufficient to warrant a decrease in the basic number of lanes, a preferred location for the lane drop is beyond the influence of an interchange and preferably at least one half mile from the nearest exit or inlet. It is desirable to locate lane drops on tangent alignment with a straight or sag profile so that there will be maximum visibility to the pavement markings in the merge area.

11-11 ROUTE CONTINUITY

Route continuity refers to the provision of a directional path along and throughout the length of a designated route. The designation pertains to a route number or a name of a major highway.

Ideally, the driver continuing on the designated route should travel smoothly and naturally in his lane without being confronted with points of decision. This means the chosen through lane(s) should neither terminate nor exit. It is desirable, therefore, that each exit from the designated route or entrance to the designated route be on the right, i.e., vehicular operation on the through route occurs on the left of all other traffic.

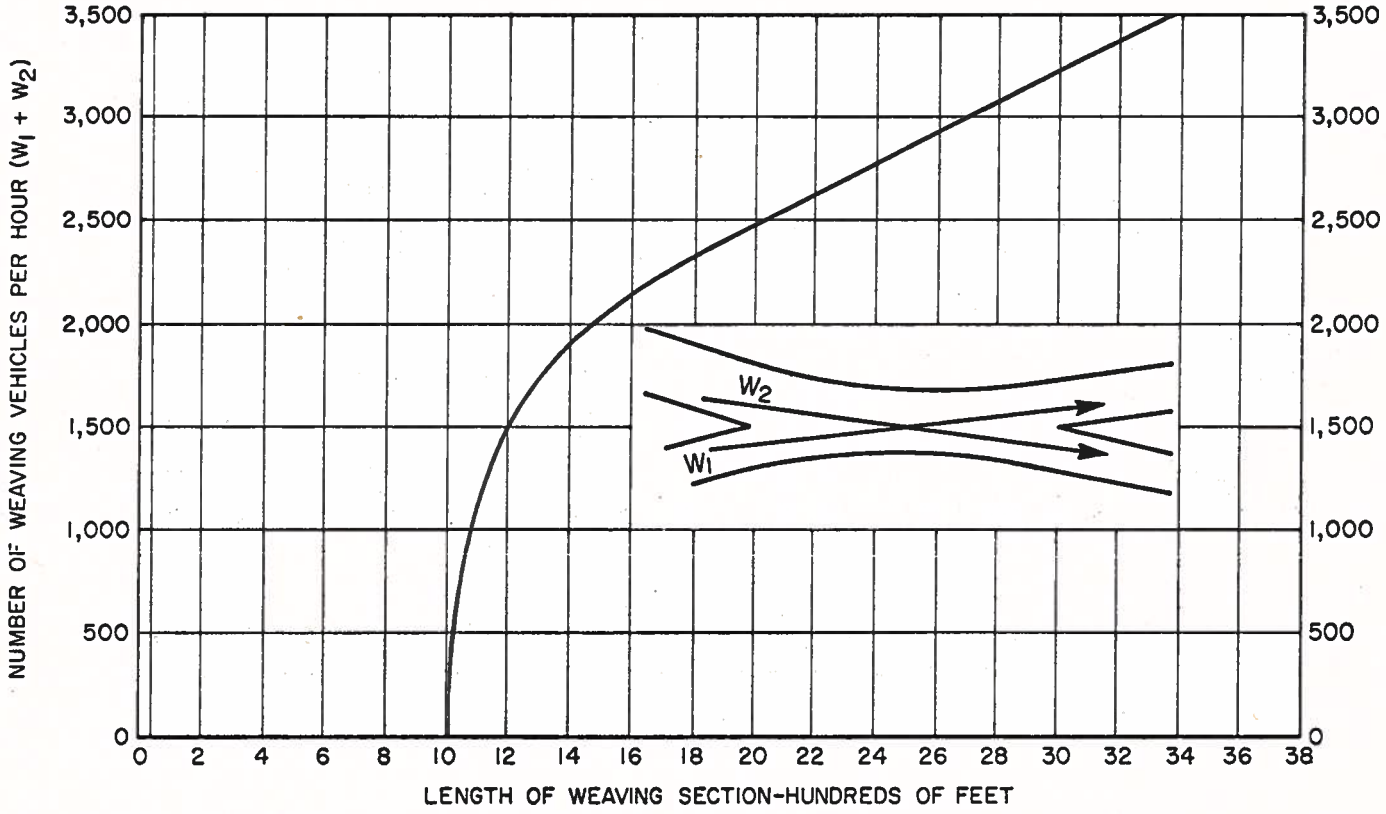
11-12 WEAVING SECTIONS

A weaving section is a length of one way roadway, accommodating merging and diverging maneuvers. A weaving section may be an inherent part of a specific interchange, or it may be formed by successive interchanges closely spaced.

A single weaving section involves a single inlet followed by a single exit. A multiple weaving section is characterized by more than one point of entry followed by one or more points of exit. The curve in Figure 11-H may be used for freeway designs involving a single weaving section. Where the total number of weaving vehicles with respect to length of weaving section are to the right of the curve, the weaving maneuver will be acceptable. However, every effort should be made to provide a minimum 1600 feet (488 m) weaving distance, measured as shown in Figure 11-D. Also, where additional lanes must be crossed, an added 1000 feet (305 m) should be provided for each lane crossed. For other conditions, refer to the Highway Capacity Manual for capacity analysis of weaving sections.

11-13 ACCESS CONTROL

Access rights should be acquired along interchange ramps to their junction with the nearest existing public road. At such junctions, access control should extend at least to the end of the ramp taper, or at least 50 feet (15.2 m) beyond the end of the curb return or ramp radius, as the case may be.



DESIGN CURVE FOR FREEWAY
WEAVING SECTIONS

FIGURE II-H

SECTION 12.

ROADSIDE DESIGN

12-01 GENERAL

The roadside, or area immediately adjacent to the traveled way, plays an important part in the safe operation of the highway. Modern highways are designed in such a manner that it is not difficult for a capable driver to operate his vehicle safely within the roadway, but statistics show that a certain percentage of drivers are likely to leave the roadway unintentionally.

In recent years, an extensive effort has been made to improve highway safety. To accomplish this, a major emphasis has been placed on elimination of hazardous roadside conditions and on improving traffic barriers to shield those hazards that cannot be eliminated.

While consideration should always be given to providing an unobstructed roadside for all highway designs, special emphasis and direction are given in this section for roadside designs where the highway design speeds are in excess of 45 mph. This would generally be the case for highways in rural areas of the State. Design speeds of urban roadways are usually 35 mph or less.

12-02 CRITERIA AND PROCEDURES

Less than 45mph, look at roadside clearance & use judgement

The clear roadside should be a part of the design of the highway where highway design speeds are 45 mph or greater. This section suggests criteria and procedures for treatment of the roadside to improve highway safety.

For added safety, it is desirable to provide an unencumbered roadside recovery area that is as wide as practical on the specific highway section. Research has indicated that the width of the clear area should desirably be 30 feet (9.15 m) or more from the edge of the traveled way on high speed highways.

The clear recovery area may be variable in width, or of fairly consistent width, but whatever the width, it should be free of obstacles such as unyielding landscape, hazardous drainage facilities, unyielding highway hardware such as sign and luminaire supports, utility poles, and steep slopes. Good engineering judgment should be used in establishing the width of the clear recovery area. Section 13, TRAFFIC BARRIERS, includes design guides that assist in determining the clear recovery area.

Treatment of obstacles should be considered in the following priority:

1. Eliminate the hazard. Every effort should be made to clear the roadside of all obstructions and obstacles and to develop the terrain of the roadside in such a manner that it can be safely traversed by an out-of-control vehicle for a distance sufficient to permit the vehicle to be brought under control or to a safe stop.

2. Relocate the hazard to a location where it is less likely to be struck.

3. Use breakaway devices to reduce the hazard.

4. Select a cost-effective device for the re-direction (longitudinal barrier) or the reduction of accident severity (crash cushion).

The design guides for the need and placement of traffic barriers and crash cushions are covered in Section 13. The objective in placing protective barriers is to lessen the hazard to traffic and not to protect any part of the roadway. As protective barriers may be a hazard themselves, the primary endeavor should be to preclude or minimize the need for a protective barrier through design of the geometric features of the roadside.

12-03 ROADSIDE GEOMETRICS

12-03.1 General

Roadside safety is dependent upon the dynamic reaction of an errant vehicle and the surface of the roadside. Consideration must be given to the speed of the vehicle leaving the roadway and its runout path.

Therefore, geometric design of the roadside is concerned with the terrain or contour of the land. In relatively flat areas, gentle slopes in the vicinity of the road are easily provided. In hilly and mountainous areas, flat slopes become more difficult and costly to construct. In addition to the shaping of the roadside, it is necessary also to design appurtenances adjacent to the roadway in such a way that they "blend" into the terrain.

Roadside conditions also vary between highways in rural areas and highways in urban areas. Urban highways introduce differences in the application of safety criteria. Some of these differences include the presence in urban areas of more pedestrians and bicyclists, utility and traffic signal appurtenances, generally lower

operating speeds, greater need and use of curbs, more frequent access to abutting properties, more restrictive rights-of-way, presence of mass transit vehicles, and greater extent and influence of people-oriented environmental considerations.

Large numbers of pedestrians, for example, preclude the use of breakaway luminaire supports where they could fall upon pedestrians. Even in the case of urban freeways, economic and/or environmental constraints may require modification or adjustment of safety criteria. An example of this situation is that of restricted right-of-way which may preclude the use of an adequate clear roadside recovery area.

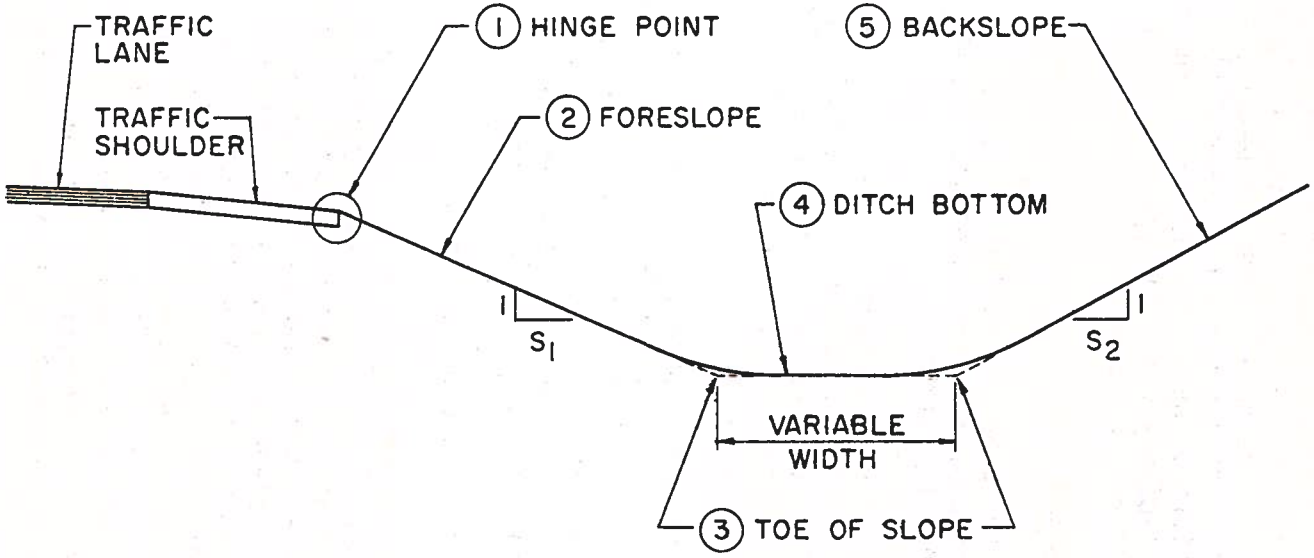
12-03.2 Roadside Slopes

Side slopes should be designed to insure the stability of the roadway and to provide a reasonable opportunity for recovery for an out-of-control vehicle.

Three regions of the roadside are important when evaluating the safety aspects: the top of the slope (hinge point), the foreslope, and the toe of the slope (intersection of the foreslope with level ground or with a backslope, forming a ditch). Figure 12-A illustrates these 3 regions.

The hinge point contributes to loss of steering control because the vehicle tends to become airborne in crossing this point. The foreslope region is important in the design of high slopes where a driver could attempt a recovery maneuver or reduce speed before impacting the ditch area. In many situations, the toe of the slope is within the clear zone and the probability of reaching the ditch is high, in which case safe transition between fore- and back-slopes should be provided.

Rounding at the hinge point, though not necessary from a vehicle rollover standpoint, can significantly reduce its hazard potential. Rounded slopes reduce the chances of an errant vehicle becoming airborne, thereby reducing the hazard of encroachment and affording the driver more control over the vehicle. Section 13 includes a discussion on shoulder rounding parameters. Foreslopes of 6:1 or flatter are recommended where feasible. Foreslopes steeper than 4:1 are not desirable because their use severely limits the choice of backslopes. Slopes 3:1 or steeper are recommended only where site conditions do not permit use of flatter slopes. When slopes steeper than 3:1 must be used, consideration should be given to the use of a roadside barrier.



DESIGNATION OF ROADSIDE REGIONS

FIGURE 12-A

Another safety factor of significance for intersecting roadways is the angle of break between a side slope and a transverse slope. Field observations indicate that more consideration should be given in roadway design to carrying the desirable flat side slopes through intersections, driveway approaches, median crossovers, and cut sections. Providing a flatter slope between the shoulder edge and the ditch bottom, locating the ditch a little farther from the roadway, or even enclosing short sections of drainage facilities, will enhance the safety of the roadside, often at a small increase in cost.

Earth cut and fill slopes should be flattened and liberally rounded as fitting with the topography and consistent with the overall type of highway. Effective erosion control, low-cost maintenance, and adequate drainage of the subgrade largely are dependent upon proper shaping of the side slopes. Slope and soil data are used in combination to approximate the stability of the slopes and the erosion hazard potential. Overall economy depends not only on the element of first cost, but also on the cost of maintenance, of which slope stability is a factor. In addition to these reasons for flat and rounded slopes on any highway, the proximity of any highway to the development and people of the community call for additional attention to naturalized slopes and overall appearance factors.

On freeways and other arterials with reasonably wide roadsides, side slopes on embankments and in cuts should be designed to provide a reasonable opportunity for recovery of an out-of-control vehicle. Where the roadside at the point of departure is reasonably flat, smooth, and clear of fixed objects, many potential crashes can be averted. A rate of slope of 6:1 or flatter on embankments can be negotiated by a vehicle with a good chance of recovery and should, therefore, be provided where feasible. For moderate heights with good roundings, steeper slopes up to about 3:1 can also be traversable. On intermediate-height fills, the cost of a continuous flat slope may be prohibitive, but it may be feasible to provide a recovery area with a reasonably flat and rounded area adjacent to the roadway. The recovery area should extend well beyond the edge of the shoulder as specific conditions may permit.

Consistent with traffic demand, local roads and arterial streets with wide borders should also be designed with a similar clear roadside recovery area approach. However, because of generally lower speeds and narrower side clearances along streets, the clear roadside area concept, at best, can only be partially used. This is also true for widening and other reconstruction within available limited right-of-way.

Desirably, slope combinations would be selected so 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 the cost effectiveness of such design may dictate the use of slope combinations steeper than desirable. If constraints other than safety dictate that the necessary recovery distance cannot be provided, the installation of a roadside barrier should be considered.

Where height and slope of roadway embankments are such that the severity of potential accidents will be reduced by the placing of a roadside barrier, the cross section should be designed to allow adequate slope rounding and to support the barrier.

Rock outcroppings have sometimes been left in place during construction of new highways for reasons of economy or aesthetics. These are lethal if struck by an automobile, and any such obstacle should not be tolerated within the clear roadside recovery area where removal is feasible. If they cannot be removed, they should be made inaccessible by the installation of a roadside barrier of appropriate design.

12-03.3 Drainage Channels

The primary purpose for construction of roadside channels is to control surface drainage, and the most economical method usually entails formation of open-channel ditches by cutting into the natural roadside terrain to produce a hydraulic channel. From a hydraulic efficiency standpoint, the most desirable channel contains steep sides. Limitations on slope stability usually require somewhat flatter slopes, and construction and maintenance factors have imposed still further restrictions on the degree of slope steepness that is practical alongside a highway. The off-setting factor of right-of-way costs must also be considered when selecting combinations of slopes to be used. The effect of slope combinations and safety during traversal by an errant vehicle is also an important consideration in designing the roadside.

In general, the severity of traversal of roadside channels less than about 4 to 8 feet (1.22 to 2.44 m) wide is essentially the same for comparable slope combinations regardless of channel shape. Slope combinations forming these narrow channels can be selected to produce cross sections that can be safely traversed by an unrestrained occupant, the range of slopes being within the steepness usually found along highways.

Foreslopes steeper than 4:1 (horizontal to vertical) are not desirable for several reasons. Their use severely limits the range of backslopes. The flatter the foreslope, the greater the choices of combinations of fore- and back-slope that permit safe traversal. The flatter foreslope also provides greater recovery distance for an errant vehicle.

The broad, flat, rounded drainage channel also provides a sense of openness that relaxes driver tension. With a channel side slope of 4:1 or flatter and a 10 foot (3.05 m) shoulder, the whole roadside channel is visible to the driver. This lessens the feeling of restriction and adds measurably to the driver's willingness to use the shoulder in an emergency.

12-03.4 Roadside Obstacles

Culvert headwalls, utility poles, large trees, drainage inlets, sign supports, luminaire supports, guardrails, curbs, bridge piers and similar features should be placed an adequate distance from the main lanes of travel or shielded from errant vehicles.

Drainage items such as headwalls, dikes, and steep-sided ditches are hazards which should be moved away from the roadway to outside the clear recovery area. If removal is not possible, effort should be made to lessen the hazard of their being struck by an errant vehicle; for example, enclosing an open channel section, or by use of a flush grated inlet in the place of a raised side opening inlet structure.

Highway hardware, such as sign supports and luminaire supports, should be offset as far away as possible from the traveled way, or utilize breakaway supports.

Utility poles should also be set back as far as possible from the traveled way. Utility poles which occupy the road right-of-way should be placed as far as practical behind the curb, or even beyond the sidewalk area.

While trees add to the beautification of the highway, trees with large diameter trunks become potential hazards when they grow close to the traveled way. Trees should be planted beyond the clear area for the section of road, and low shrubs and ground cover utilized to beautify the clear area.

12-03.5 Medians

Although a median is generally regarded as a substantial safety feature, accidents which do occur when a vehicle crosses a median often have catastrophic results.

The most obvious and effective preventive is to separate the two roadways by medians wide enough to permit all out-of-control vehicles to recover. Widely separated roadways also reduce headlight glare, allow flatter cross slopes, and permit grades and horizontal alignment which better fit existing topography.

However, economic constraints will, in many cases, dictate the use of narrower medians within the design guides discussed in Section 6, AT-GRADE INTERSECTIONS, and in Section 5, CROSS SECTION ELEMENTS.

In any case, the median area should also be cleared of all roadside obstructions and obstacles. Side slopes should be 6:1 or flatter, desirably 10:1. On narrow medians, a median barrier should be considered with flush paved shoulders.

12-03.6 Roadside Appurtenances

1. Signs and Sign Supports

While traffic control devices are very necessary for the safe operation of a highway, their design and placement must be on safety as well as on communication.

Experience over the last several years has shown that on high-speed rural highways with wide and clear areas, guide signs can be offset considerable distances from the roadway and still be effective. Care must be taken to obtain a satisfactory angle for the sign face in relation to approaching traffic and to provide favorable viewing conditions.

When signs must be located in proximity to the edge of the traveled way or shoulder, breakaway supports for large, ground-mounted signs and yielding or breakaway supports for small signs should be used. Supports for overhead signs must be shielded by protective barriers. Concrete foundations for sign supports should be constructed and maintained flush with the ground level so that they do not themselves become unyielding obstacles.

On freeways, overhead signs can be mounted on overpass bridge structures, thus eliminating a potential ground hazard. Signs and luminaires can also be cantilevered from retaining walls where feasible.

2. Traffic Signal Equipment

The importance of the traffic signal as a traffic control device is such that signal display effectiveness should not

be compromised by location of the supports. As a general rule, in urban areas where sidewalks, curbed sections and approach speeds of 35 miles per hour or less are involved, it is not practical to locate signal supports very far behind the curb or to protect them with guardrail. A minimum clearance of 2 feet (0.62 m) should be maintained between the face of the curb and any part of the signal equipment. Where curbs do not exist or are mountable, a minimum clearance of 10 feet (3.05 m) between the pavement edge and any part of the signal equipment, or a minimum of 2 feet (0.62 m) between the shoulder edge and any part of the signal equipment should be maintained. In urban or suburban areas with medium approach speeds (35 to 50 miles per hour), no sidewalks, negligible pedestrian activity, and curbed or uncurbed roadways, all traffic signal supports should be placed a minimum of 2 feet (0.62 m) outside the shoulder or a minimum of 10 feet (3.05 m) beyond the edge of the traveled way. Where this is not practical or feasible, properly installed guardrails should be utilized.

Similar requirements for clearances apply at intersections of uncurbed rural highways. In these locations, there is also the additional factor that there is a need to attain clear recovery areas. When a fallen traffic signal in the traveled roadway is judged to be a lesser hazard than a non-yielding roadside obstruction, traffic signal pedestals or poles of yielding, breakaway or frangible design may be used as supports for non-overhead signals. Overhead signal supports, either mast arm or span wire, should not be breakaway or frangible.

SECTION 13.

T R A F F I C B A R R I E R S

13-01 GENERAL

As a major part of the effort to make the roadside safer for the motorist, emphasis has been placed on the elimination of hazardous roadside conditions and on the improvement of traffic barriers to shield those hazards that cannot be eliminated.

This section presents design guidelines for highway traffic barriers, including conditions which warrant barrier protection, the type of barriers available and, to some extent, how the barrier should be installed. The guidelines presented in this section are generally applicable to highways with design speeds of 45 mph or greater.

Extensive reference is made to "Guide for Selecting, Locating and Designing Traffic Barriers", AASHTO 1977, and National Cooperative Highway Research Program (NCHRP) Report 118, "Location, Selection and Maintenance of Highway Traffic Barriers", published in 1971.

13-01.1 Definitions

Area of Concern--An object or roadside condition that warrants shielding by a traffic barrier.

Barrier Warrant--A criterion that identifies an area of concern which should be shielded by a traffic barrier. The criterion may be a function of relative safety, economics, etc., or a combination of factors.

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

Clear Zone--That roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. Establishment of a minimum width clear zone implies that rigid objects and certain other hazards with clearances less than the minimum width should be removed, relocated to an inaccessible position or outside the minimum clear zone, remodeled to make safely traversable or breakaway, or shielded.

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

Crash Cushion--A traffic barrier used to safely shield fixed objects or other hazards from approximately head-on impacts by errant vehicles. Examples are sand-filled plastic barrels, water-filled tubes, vermiculite concrete cartridges, and steel drums. Also called impact attenuator.

Crashworthy Barrier--One that can be impacted by a vehicle at or below the anticipated operating speed of the roadway with low probability of serious injury to the vehicle's occupants.

Experimental Barrier--One that has performed satisfactorily in full-scale crash tests and promises satisfactory in-service performance.

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

Length of Need--Total length of a longitudinal barrier, measured with respect to centerline of roadway needed to shield an area of concern.

Longitudinal Barrier--A barrier whose primary functions are to prevent penetration and to safely redirect an errant vehicle away from a roadside or median hazard. The 3 types of longitudinal barriers are guardrail, median barriers, and bridge rails.

Median Barrier--A longitudinal barrier used to prevent an errant vehicle from crossing the portion of a divided highway separating the traveled ways for traffic in opposite directions.

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

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

Research and Development Barrier--One that is in the development stage and has had insufficient full-scale tests and in-service performance to be classified otherwise.

Roadside Barrier--A longitudinal barrier used to shield hazards located within an established minimum width clear zone. It may also be used to shield hazards in extensive areas between the roadways of a divided highway. It may occasionally be used to protect pedestrians or "bystanders" from vehicular traffic.

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

Shy Distance--Distance from the edge of the traveled way beyond which a roadside object will not be perceived as an immediate hazard by the typical driver, to the extent that he will change his vehicle's placement or speed.

Traffic Barrier--A device used to shield a hazard that is located on the roadside or in the median, or a device used to prevent crossover median accidents. As defined herein, there are 4 classes of traffic barriers; namely, roadside barriers, median barriers, bridge rails, and crash cushions.

13-01.2 Warrants

1. General Approach

Traffic barrier warrants are decision criteria that identify sites along highways needing traffic barrier installations. These warrants are delineated in terms of geometry and location of roadside features; for the special case of median barriers, traffic volume is also a use decision factor.

The purpose of traffic barriers is to reduce accident fatalities and injuries by decreasing severity of crashes. Crash cushion barriers are designed for locations, such as off ramp gores and bridge piers in the median, for which the primary problem is a head-on collision; generally, a crash cushion will decrease the severity of such direct-on impact. In contrast, the longitudinal barrier affords only a relative degree of protection to vehicle occupants, as a collision with this type of barrier can result in a severe accident; hence, longitudinal barriers are warranted only at highway locations where the severity of a collision with the roadside feature would be greater than that with the traffic barrier.

Installation of traffic barriers may increase the frequency of accidents by presenting a larger "target" located closer to the roadway than the hazard being shielded. For this reason, traffic barrier installations should be kept to a minimum. Highway designers should strive to eliminate all traffic barriers; and where traffic barrier requirements are indicated by warrants, the roadway should be examined to determine the feasibility of adjusting site features so that the barrier will not be required (e.g., flattening an embankment slope, removing a tree, or eliminating a drainage headwall).

2. Determination of Need

Highway features that may warrant traffic barrier installations are delineated in Table 13-1, together with indications as to whether the barrier system candidate may be a longitudinal or a crash cushion design, or both.

13-02 ROADSIDE BARRIERS

13-02.1 General

A roadside barrier is a longitudinal system used to shield vehicles from hazards in the roadside. It may also be used to shield hazards other than opposing traffic in extensive areas between divided highways. It may occasionally be used to protect pedestrians and "bystanders" from vehicular traffic.

Highway hazards that may warrant shielding by a roadside barrier are classified into embankments and roadside obstacles.

13-02.2 Warrants

1. Embankments

Height and slope of the embankment are the basic factors in determining barrier need for a fill section. Warranting criteria for fill sections are shown in Figure 13-A. Embankments with slopes and height combinations below the curve do not warrant protection. However, obstacles on the slope may warrant protection. Embankments with slope and height combinations above the curve warrant protection.

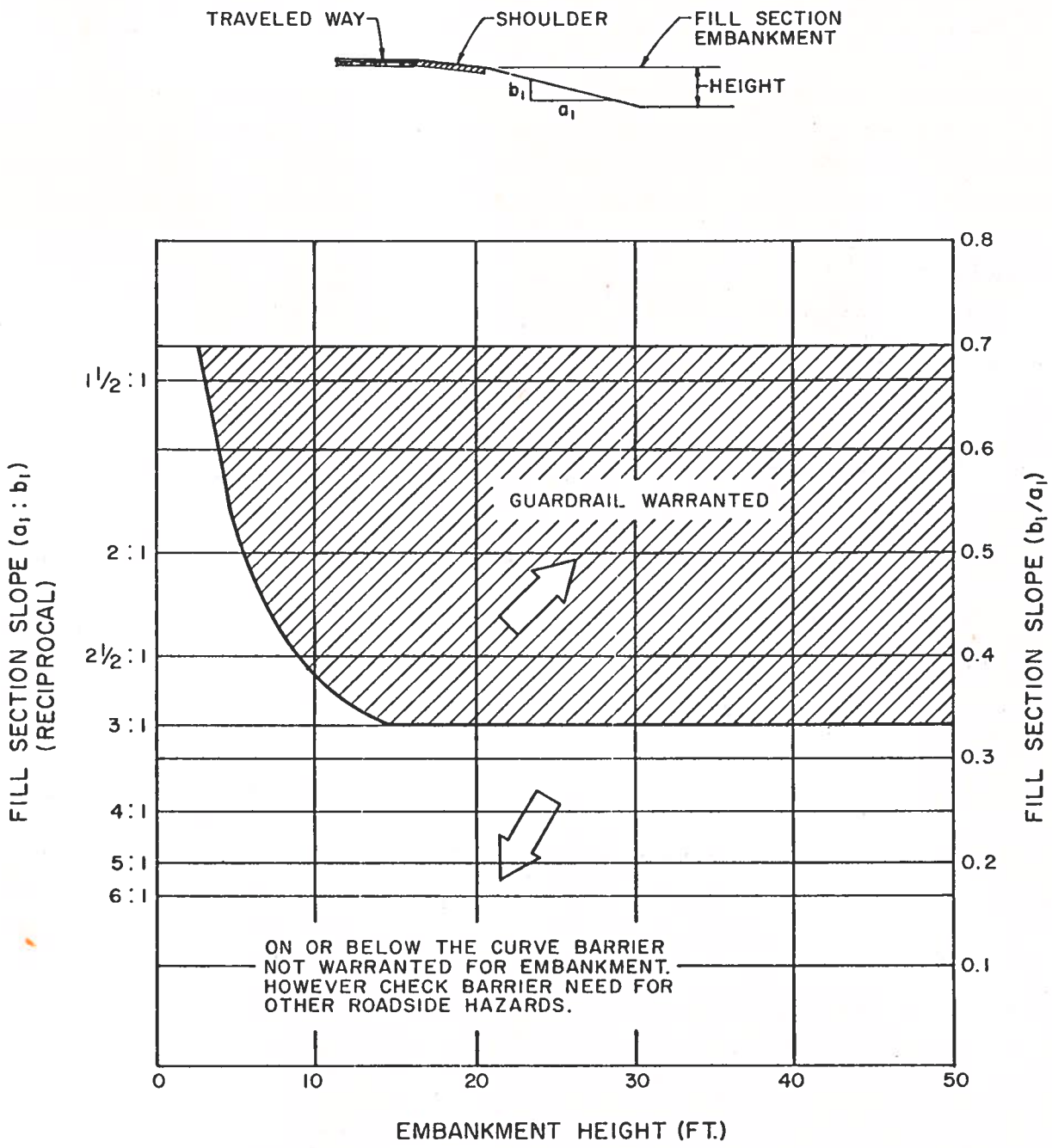
Rounding at the shoulder or the hinge point and the toe of an embankment can reduce its hazard potential. Rounded slopes reduce the chances of an errant vehicle becoming airborne, thereby reducing the hazard of the encroachment and

	BARRIER WARRANTING FACTOR	BARRIER CANDIDATE	
		LONGITUDINAL	VEHICLE IMPACT ATTENUATOR
1. LATERAL DROP-OFF			
a. BRIDGE **	A	X	
b. ABRUPT EMBANKMENT	B, C	X	
c. SLOPED EMBANKMENT	D	X	
2. ROADSIDE OBSTACLE			
a. NON-TRAVERSABLE HAZARD			
(1) ROUGH ROCK CUT	B	X	
(2) LARGE BOULDERS	B	X	
(3) WATER (PERMANENT BODIES)	B, C	X	
(4) LINE OF TREES	B, E	X	
(5) GORES	F		X
(6) SPACE BETWEEN TWIN BRIDGES	G	X	X***
b. FIXED OBJECT			
(1) BRIDGE PARAPET; BRIDGE RAIL END	H	X	X
(2) SIGN SUPPORT	B, E	X	X
(3) BRIDGE PIERS; ABUTMENTS	B	X	X
(4) RETAINING WALLS, CULVERT HEADWALLS	B	X	X
(5) TREES	B, E	X	X
(6) WOOD POLES; POSTS	B, E	X	X
(7) TOWER LIGHTING STRUCTURES	B, E	X	X
3. OPPOSING TRAFFIC	I	X	

* A - ALL BRIDGES
B - DISTANCE FROM PAVEMENT
C - DEPTH OF DROP (WATER)
D - HEIGHT AND SLOPE
E - SIZE
F - ELEVATED EXIT RAMP
G - ADVERSE ACCIDENT EXPERIENCE
H - BRIDGE WIDTH, TRAFFIC REDIRECTION
I - MEDIAN WIDTH, TRAFFIC VOLUME
** - BRIDGE APPROACH BARRIER AND BRIDGE RAIL SHOULD BE AN INTEGRATED BARRIER SYSTEM
*** - SAFETY NET INSTALLATION

TRAFFIC BARRIER WARRANTING CONSIDERATIONS

TABLE 13-1



WARRANTS FOR FILL SECTION EMBANKMENTS

FIGURE 13-A

affording the driver more control over the vehicle. Figure 13-B illustrates shoulder rounding parameters and it contains suggested equations for determining the rounded profile. The equations can be used to determine the lateral extent of rounding needed, d_x, and the elevation, y, of the rounded portion as a function of the lateral distance, x. Rounding may be determined for any combination of shoulder slope, side slope, and vehicle encroachment conditions. The encroachment angle of 15 degrees is used, based upon study data of errant vehicles leaving the highway.

Table 13-2 presents slope rounding requirements for shoulder slopes of -5% to +8% in combination with various embankment slopes and vehicle speeds.

In Figure 13-C, values of the distance (d_x) are given as a function of operating speed and the algebraic difference of the shoulder slope (e) and embankment slope (b/a).

2. Roadside Obstacles

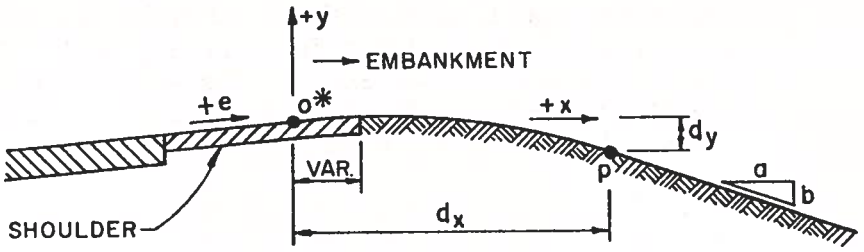
Roadside obstacles are classified as nontraversable hazards and fixed objects. These highway hazards account for over 30 percent of all highway fatalities each year and their removal should be the first alternative considered. For example, sign supports or culvert headwalls should be relocated. If it is not feasible or possible to remove or relocate a hazard, then a barrier may be necessary. However, a barrier should be installed only if it is clear that the barrier offers the least hazard potential.

Barrier warrants for roadside obstacles are a function of the nature of the obstacle and its distance from the edge of the traveled way. Figure 13-D shows criteria for determining the clear zone on fill and cut sections for 3 different vehicle operating speeds with ADTs in excess of 6000 vehicles. Paragraph 13-02.5.4 provides a method for adjusting the clear zone widths for lower volumes and different vehicle speeds. Clear zone is defined as the roadside border area, starting at the edge of the traveled way (edge of pavement), available for safe use by errant vehicles. Nontraversable hazards or fixed objects should be removed, relocated, or shielded by a barrier if they are within the indicated minimum clear zone widths.

The procedure for use of Figure 13-D is as follows:

a. Locate on the appropriate curve, based on given operating speed, the point whose ordinate coincides with the slope of the embankment.

✳ NOTE:
BEGINNING OF ROUNDING MAY BE WITHIN SHOULDER WIDTH, IF DESIRED, PROVIDED SLOPE OF SHOULDER DOES NOT BECOME EXCESSIVE.



ROADSIDE ELEVATION

EQUATIONS

$$y = x\left[e - \frac{6.9x}{V^2\sin^2\theta}\right],$$

$$0 \le x \le d_x$$

$$d_x = \frac{V^2\sin^2\theta}{13.8}\left[e - \frac{b}{a}\right]$$

$$d_y = d_x\left[e - \frac{6.9d_x}{V^2\sin^2\theta}\right]$$

WHERE, e = SHOULDER SLOPE (FT/FT), POSITIVE IF SLOPING UPWARD;

$\frac{b}{a}$ = EMBANKMENT SLOPE (FT/FT), NEGATIVE IF SLOPING DOWNWARD;

V = VEHICLE VELOCITY (FT/SEC); 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.

ROUNDING PARAMETERS AND EQUATIONS FIGURE 13-B

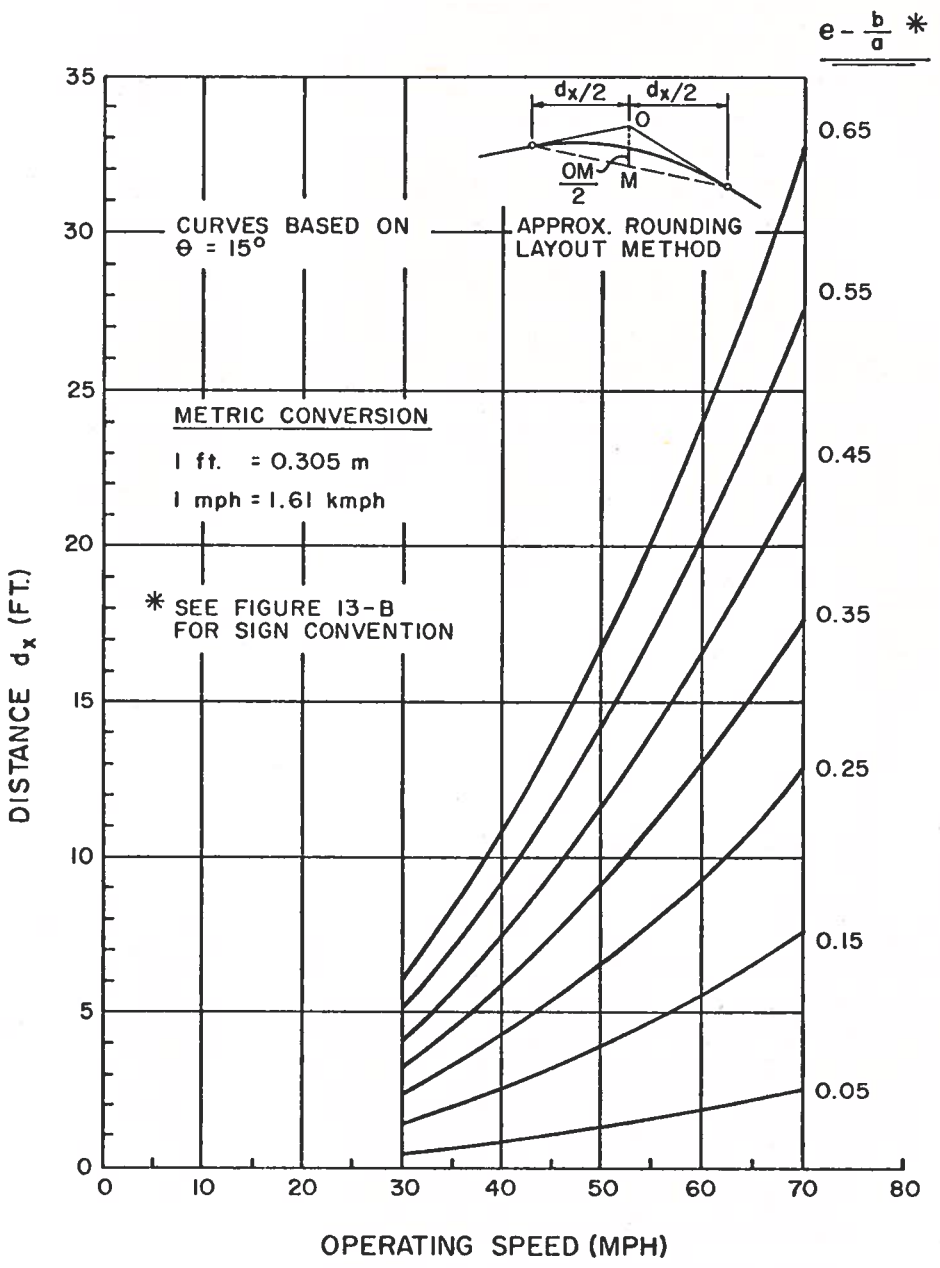
HORIZONTAL LENGTH OF ROUNDING - d_x (FT)

SHOULDER SLOPE, e	EMBANKMENT SLOPE, b/a	DESIGN SPEED (MPH)				
		30	40	50	60	70
+8%	1:3	4	8	12	16	22
	1:4	4	6	10	14	18
	1:6	4	6	8	10	14
	1:8	2	4	6	8	12
	1:10	2	4	6	8	10
+6%	1:3	4	8	12	16	22
	1:4	4	6	10	12	16
	1:6	4	4	6	10	12
	1:8	2	4	6	8	10
	1:10	2	4	6	8	10
+5%	1:3	4	8	12	16	20
	1:4	4	6	8	12	16
	1:6	4	4	6	10	12
	1:8	2	4	6	8	10
	1:10	2	4	4	6	8
+4%	1:3	4	8	10	16	20
	1:4	4	6	8	12	16
	1:6	2	4	6	8	12
	1:8	2	4	6	8	10
	1:10	2	4	4	6	8
+3%	1:3	4	8	10	14	20
	1:4	4	6	8	12	16
	1:6	2	4	6	8	12
	1:8	2	4	6	6	8
	1:10	2	4	4	6	8
+2%	1:3	4	6	10	14	20
	1:4	4	6	8	12	14
	1:6	2	4	6	8	10
	1:8	2	4	4	6	8
	1:10	2	4	4	6	8
-2%	1:3	4	6	10	14	18
	1:4	4	4	6	10	12
	1:6	2	4	4	6	8
	1:8	2	2	4	4	6
	1:10	2	2	4	4	6
-5%	1:3	4	6	8	12	16
	1:4	2	4	6	8	12
	1:6	2	2	4	6	6
	1:8	2	2	2	4	4
	1:10	2	2	2	2	4

SHOULDER ROUNDING

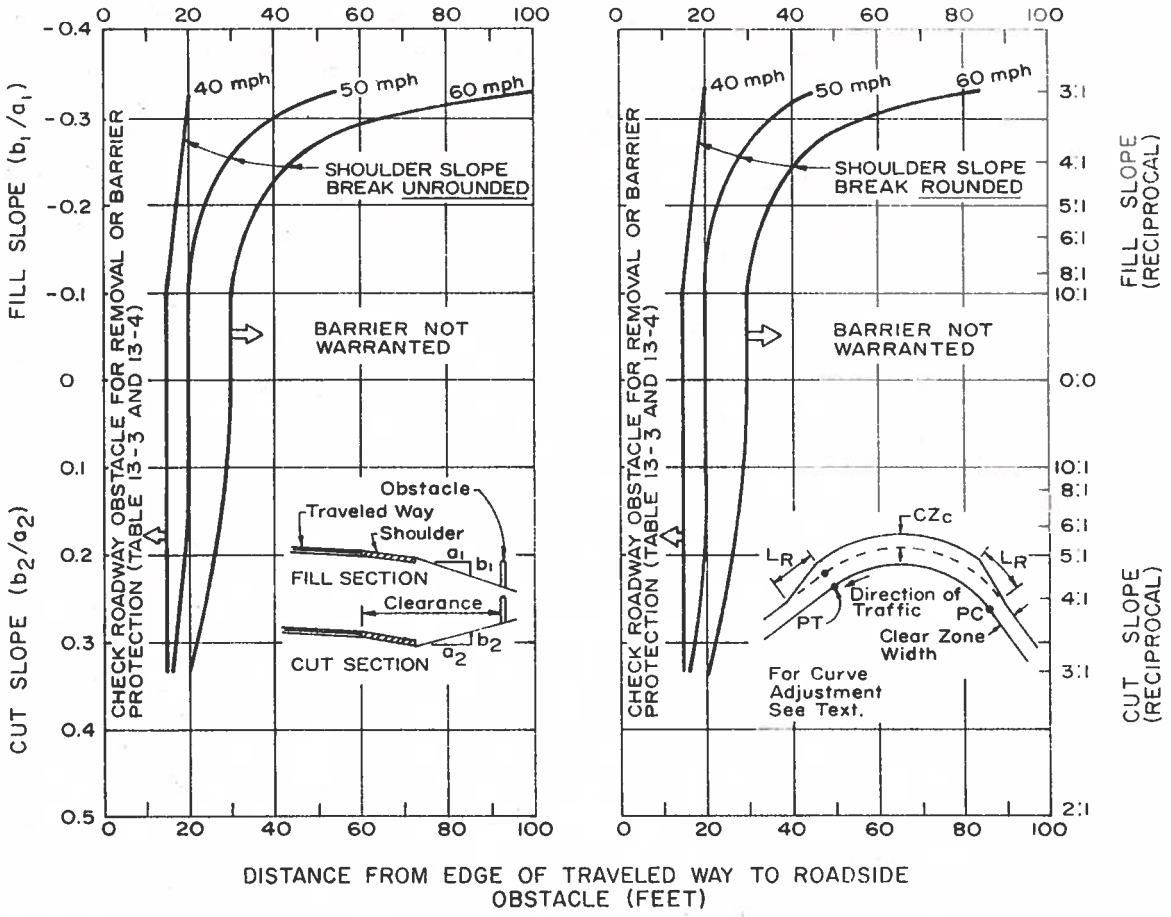
(computed for various shldr. slopes)

TABLE 13-2



LATERAL ROUNDING DISTANCE, d_x,
VERSUS OPERATING SPEED

FIGURE 13-C



METRIC CONVERSION

1 mph = 1.61 kmph

1 ft. = 0.305 m

CLEAR ZONE WIDTH, SPEED AND SLOPE CRITERIA

Quick Check

FIGURE 13-D

r5/15/86

b. Locate in the abscissa the minimum clear zone width from the traveled way.

c. If the obstacle or hazard is located within this clear zone width, it must be removed or relocated. Otherwise, a barrier may be warranted.

The clear zone width should be increased on the outside of curves by the amount $\triangle$DCZc in which:

$$\triangle CZc = R\left(1-\cos\frac{L_R D^o}{100}\right)$$

$\triangle CZc$ = increase in clear zone for curve - feet.

D^o = degree of curve, 100 foot arc definition

R = radius of curve, feet.

L_R = runout path length, from Table 13-7, feet.

The distance $\triangle CZc$ given by this equation is for slopes of 10:1 or flatter. For steeper slopes, this distance should be increased by the ratio of clear zone width on steeper side slope divided by clear zone width on zero side slope.

The additional clear zone width $\triangle CZc$ should be applied at a distance L_R into the curve. The increase should be tapered on the curve at the entrance end and on the tangent at the exit end of the curve.

The clear zone criteria represents the present state of knowledge and it underlines the fact that flat, unobstructed roadsides are highly desirable. This clear zone width should be provided whenever possible.

Typical nontraversable hazards and appropriate barriers warrants are shown in Table 13-3. Any nontraversable hazard that warrants shielding by a barrier should be removed. If this is not practical, a barrier should be provided.

Another common hazard on non-freeway facilities is a driveway or roadway which crosses a main roadway. The slope of the driveway or crossing roadway should be 10:1 or flatter. Also, sloping inlet and outlet culvert grates should be used.

Typical fixed objects and the warranting criteria are given in Table 13-4. Fixed objects within a clear zone that warrant barrier protection by this Table should be removed. If

NONTRAVERSABLE HAZARD WITHIN CLEAR ZONE AS DETERMINED BY FIGURE 13-D	TRAFFIC BARRIER REQUIRED	
	YES[1]	NO
ROUGH ROCK CUTS	X	
LARGE BOULDERS	X	
STREAMS OR PERMANENT BODIES OF WATER LESS THAN 2 FEET IN DEPTH		X
STREAMS OR PERMANENT BODIES OF WATER MORE THAN 2 FEET IN DEPTH	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

[1] ALL ROADSIDE OBSTACLES WITHIN THE CLEAR ZONE SHOULD BE REMOVED IF POSSIBLE, OTHERWISE PROVIDE BARRIER PROTECTION.

METRIC CONVERSION

1 FT. = 0.3048 M.

WARRANTS FOR NONTRAVERSABLE HAZARDS

TABLE 13-3

FIXED OBJECTS WITHIN CLEAR ZONES AS DETERMINED BY FIGURE 13-D	TRAFFIC BARRIER REQUIRED	
	YES	NO
SIGN, TRAFFIC SIGNAL, AND LUMINAIRE SUPPORTS [2]		
(a) BREAKAWAY OR YIELDING DESIGN WITH LINEAR IMPULSE:		
(1) LESS THAN 1,100 LB.-SEC.		X
(2) GREATER THAN 1,100 LB.-SEC.	X	
(b) CONCRETE BASE EXTENDING 6 INCHES OR MORE ABOVE GROUND	X	
FIXED SIGN BRIDGE SUPPORTS	X	
BRIDGE PIERS AND ABUTMENTS AT UNDERPASSES	X	
RETAINING WALLS AND CULVERTS	X	
TREES WITH DIAMETER GREATER THAN 6 INCHES	X[3]	
WOOD POLES OR POSTS WITH AREA GREATER THAN 50 SQ. INCHES	X[3]	

1 FIXED OBJECT SHOULD BE REMOVED OR RELOCATED SO THAT A BARRIER IS UNNECESSARY IF PRACTICAL

2 BREAKAWAY OR YIELDING DESIGN IS DESIRABLE REGARDLESS OF DISTANCE FROM TRAVELED WAY

3 A JUDGMENT DECISION (SEE DISCUSSION IN TEXT)

METRIC CONVERSIONS

1 LB.-SEC. = 4.45 N-SEC.

1 IN. = 0.0254 M

WARRANTS FOR FIXED OBJECTS

TABLE 13-4

removal or relocation is not practical or feasible, the object should then be shielded by a barrier.

3. Bridge Rail Ends, Transitions, and End Treatments

Most bridge rail approach barrier systems are some type of roadside barrier. Figure 13-E summarizes the warrants for an approach barrier to a bridge. These criteria are again based on the clear zone requirements for fixed hazards (Figure 13-D) since the unprotected end of a bridge rail is considered a fixed object hazard.

If an approach barrier is warranted based on Figure 13-E, adequate transition section between the approach barrier and the bridge rail is warranted. If the end of the approach barrier terminates within the clear zone, a crashworthy end treatment is also warranted.

A transition section is warranted when there is a significant change in the lateral strength or lateral stiffness of a roadside barrier.

4. Bystanders, Pedestrians and Cyclists

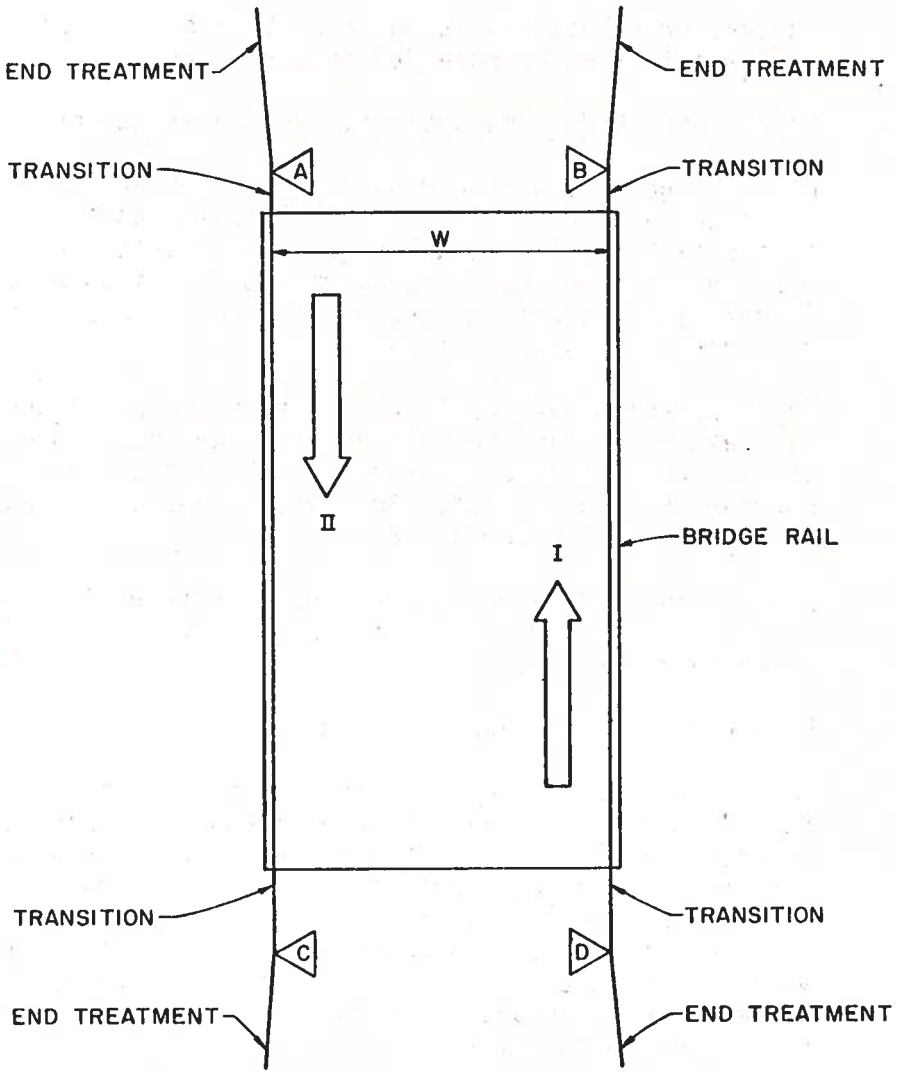
Conventional criteria presented earlier in this section cannot be used to establish barrier needs concerning the "bystander" problem. Even when the boundaries of a school, business and/or residences are beyond the clear zone, a barrier may be needed. If there is any reasonable probability of an errant vehicle encroaching any of these, a barrier would be warranted. In these cases, the barrier should be placed near the boundary of the facility to minimize the hazard to the motorist.

Pedestrians and cyclists should desirably be separated from vehicular traffic by physical means whenever possible. On low speed streets, a barrier curb will be sufficient. At speeds in excess of 45 mph a roadside barrier should be installed. Proper consideration must be given to the deflection characteristics in the selection and lateral placement of the barrier.

13-02.3 Standard Sections of Roadside Barriers

1. General

The standard sections of guardrails used in the State are shown in the following publications:



TRAFFIC DIRECTION	DISTANCE BETWEEN BRIDGE RAILS-W (FT.)	APPROACH BARRIER REQUIRED AT *
I AND II	≤ 2 X CLEAR ZONE WIDTH (SEE FIGURE 13-D)	A,B,C,D
I AND II	> 2 X CLEAR ZONE WIDTH (SEE FIGURE 13-D)	A,D
II ONLY	ALL WIDTHS	A,B
I ONLY	ALL WIDTHS	C,D

* CHECK ROADSIDE FOR OTHER WARRANTING FEATURES.

BRIDGE APPROACH BARRIER CRITERIA

FIGURE 13-E

a. "Standard Details for Public Works Construction" for the Departments of Public Works of the County of Kauai; City and County of Honolulu; County of Maui; and the County of Hawaii; dated September 1984.

b. "Standard Plans", State of Hawaii, Department of Transportation, Highways Division.

The type of guardrail used is the W-Beam type on concrete or metal posts.

Cable guardrails are not acceptable.

Figure 13-F shows the different elements of a longitudinal barrier system.

2. Transitions

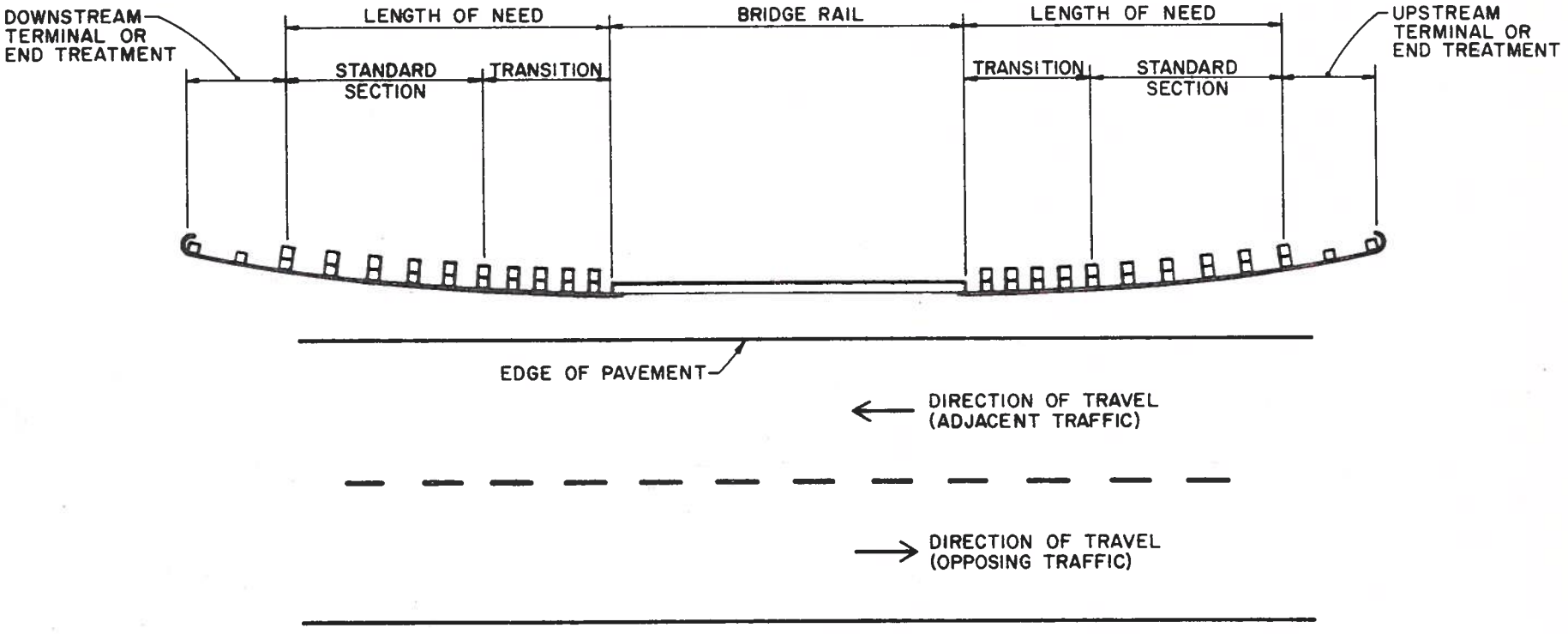
Transition sections are necessary to provide continuity of protection when two different roadside barrier types join, when a roadside barrier joins another barrier system (such as a bridge rail), or when a roadside barrier is attached to a rigid object, such as a bridge pier. The most common use of transitions occurs between approach guardrails and bridge rail ends.

Typical transitions at bridge rails or to concrete barriers are shown in the State Department of Transportation Standard Detail Plans.

3. End Treatment

An untreated end of a roadside barrier is extremely hazardous if impacted, since the beam of the system tends to penetrate the passenger compartment and will generally stop the vehicle abruptly. A crashworthy end treatment is therefore recommended if the barrier terminates within the "clear zone". Both an upstream terminal and downstream terminal must be considered. The length of need for upstream terminal is dependent on the clear zone width for the adjacent traffic, and the length of need for the downstream terminal is dependent on the clear zone width for the opposing traffic.

To be crashworthy, the end treatment should not spear, vault, or roll the vehicle for head-on or "nose" impacts. For impacts between the end and the standard section, the end treatment should have the same redirectional characteristics as the standard roadside barrier, which means that the end



DEFINITION OF ROADSIDE BARRIER ELEMENTS

FIGURE 13-F

must be properly anchored. The end treatment must be capable of developing the full tensile strength of the standard rail element, whether a crashworthy end treatment is warranted or not.

State Department of Transportation Standard Plans show end treatments in current use in Hawaii.

13-02.4 Maintenance and Selection

1. Maintenance

The maintenance of the roadside barrier system should be considered before selecting the system. Table 13-5 contains a number of maintenance factors which should be considered.

Collision maintenance concerns the activities required as a result of vehicle impacts. Such activities should play an important role in the selection of a barrier system since the majority of maintenance costs are usually due to collision repairs.

In Hawaii, the environmental conditions should be given serious consideration due to the high corrosion potential of metal exposed to salt spray.

CATEGORY	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 CREW WORKING KNOWLEDGE
B. ROUTINE MAINTENANCE	1. CLEANING AND PAINTING 2. MOWING AND CLEARING VEGETATION 3. CHECKING AND TIGHTENING BLOCKS AND FASTENERS
C. ENVIRONMENTAL CONDITIONS	1. WEATHERING OR CORROSION DUE TO ENVIRONMENT
D. MATERIAL AND STORAGE REQUIREMENTS	1. DEPENDENCE ON A NUMBER OF PARTS 2. AVAILABILITY OF PARTS 3. STORAGE FACILITIES REQUIRED

MAINTENANCE FACTORS INFLUENCING ROADSIDE BARRIER SELECTION

TABLE 13-5

Material and storage requirements should consider the standardization of the hardware used in the barrier systems.

2. Selection Guides

Once it has been determined that a barrier system is warranted, a selection must be made. Table 13-6 presents 8 items which should be considered in the barrier system selection process. Important among the items presented are the deflection, strength and safety requirements.

13-02.5 Placement

1. General

Major factors to consider in the lateral placement of a roadside barrier include the following:

a. Uniform clearance and distance between barrier and hazard being shielded.

b. Effects of terrain between edge of traveled way and the barrier on the errant vehicle's trajectory.

c. Probability of impact with barrier as a function of its distance off the traveled way.

d. Flare rate and length of need of transitions and approach barriers.

e. Slow moving vehicles on the shoulder of the roadway.

2. Uniform Clearance and Distance Between Barrier and Hazard

A highly desirable characteristic of any roadway is that it has shoulders of constant width, whether it is in cut, fill, or on a structure. Uniform clearance to bridge rails or parapets, retaining walls, abutments, and roadside barriers is also desirable. Such an alignment of these elements enhances highway safety by reducing driver reaction and concerns for those objects and by reducing the probability of vehicle snagging.

To ensure that safe usage of the full shoulder width is possible, longitudinal barriers should be installed with the face of the rail a minimum of 2 feet (0.62 m) from the edge of shoulder. However, if sufficient safe traversable roadside is available, the longitudinal barrier should be placed as far away from the shoulder as feasible.

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

SELECTION CRITERIA FOR ROADSIDE BARRIERS

TABLE 13-6

Where a roadside barrier is used to shield an isolated hazard, adherence to the uniform clearance criteria is not essential. It is more important in such cases that the barrier be located as far from the traveled way as conditions permit. However, where there is a series of "isolated" hazards along the roadway, gaps less than 200 feet (61 m) between barrier installations should be avoided. In such cases, the barrier should be continued at a constant distance from the traveled way until all hazards are shielded.

The amount a barrier will deflect upon impact is a critical factor in its placement if the hazard being shielded is a rigid object. The barrier-to-hazard distance should be sufficient to avoid snagging by the vehicle on the rigid object. Generally, a barrier-to-hazard distance of 3.5 feet to 4 feet (1.07 m to 1.22 m) is adequate for the W-beam guardrail systems used in Hawaii.

If the hazard is a steep dropoff or embankment, the barrier-to-hazard distance should be sufficient to prevent the wheels from dropping and to provide adequate lateral soil resistance for the posts during impact.

Figure 13-G illustrates the above mentioned roadside hazards of concern.

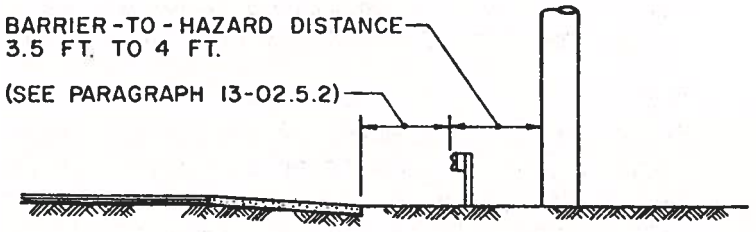
In some cases, the available space between barrier and hazard may not be adequate. In such cases, the barrier should be stiffened in the area of the hazard.

3. Terrain Effects

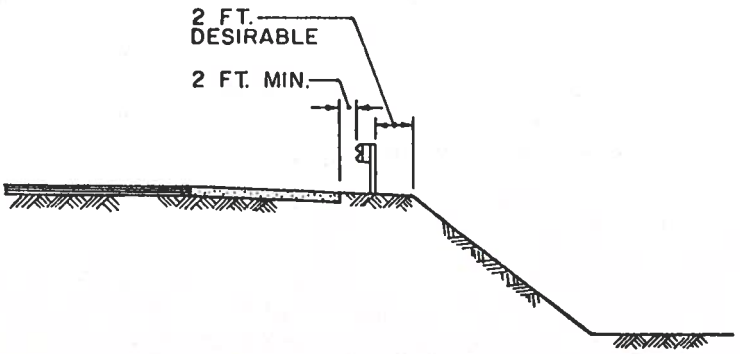
Terrain conditions between the traveled way and the barrier can have significant effects on the barrier's impact performance. Curbs and sloped roadsides are two prominent features which deserve special attention. A vehicle which traverses one of these features prior to impact may go over the barrier or submarine under it or snag on its support posts.

a. Curbs

In general, it has been found that curbs offer no safety benefit on high speed roadways from the standpoint of vehicle behavior following impact. For this reason, a curb should be used alone, or when used in conjunction with a traffic barrier, should be placed behind the barrier.



RIGID OBJECT PROTECTION



EMBANKMENT PROTECTION

METRIC CONVERSION
1 FT. = 0.305 M.

BARRIER-TO-HAZARD DISTANCE FOR ROADSIDE PROTECTION

FIGURE 13-G

If special conditions require the use of curbs, the face of the curb should be no closer to the traveled way than the face of the rail. If the top of the rail is approximately 27 inches (0.69 m) above the top of the curb, a rubrail should be placed between 15 to 20 inches (0.38 to 0.5 m) above the top of pavement.

b. Roadside Slopes

As a general rule, a roadside barrier should not be placed on an embankment if the slope of the embankment is steeper than 6:1. In addition, if the difference between the shoulder slope rate and side slope rate is greater than 0.10, a barrier should not be placed on an embankment. Figure 13-H illustrates this criterion.

However, where there are rigid objects in the median or along the roadside where the side slopes are 6:1 or flatter, a barrier may be placed on the embankment. The hinge point at the shoulder should be adequately rounded or the barrier should be located more than 12 feet (3.66 m) from the hinge point.

4. Flare Rate and Length of Need

Figure 13-I illustrates the variable of interest in the lay-out of an approach barrier to shield an area of concern or hazard. Length of need is equal to the length of the area of concern parallel to the roadway, plus the length of the approach barrier on the upstream side (and downstream side if needed).

Ends of roadside barriers should be flared to attain the following conditions:

a. To locate the barrier and its terminal as far from the traveled way as is feasible.

b. To redirect an errant vehicle without serious injuries to its occupants.

c. To minimize a driver's reaction to a hazard near the traveled way.

Table 13-7 contains the runout length (L_R), flare rate (a:b), and shy line offset (L_S) as a function of ADT (Average Daily Traffic) and design speed. Shy line offset is defined as a

BARRIER SHOULD NOT BE PLACED ON EMBANKMENT IF

$\left|e - \frac{a}{b}\right| > 0.167$ OR IF $\frac{a}{b} > -0.167$

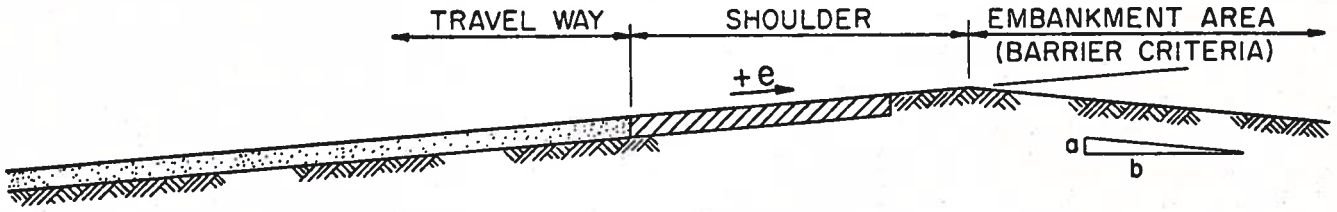
WHERE

e = SHOULDER SLOPE RATE, POSITIVE IF SLOPING UPWARD

$\frac{a}{b}$ = EMBANKMENT SLOPE RATE, POSITIVE IF SLOPING DOWNWARD

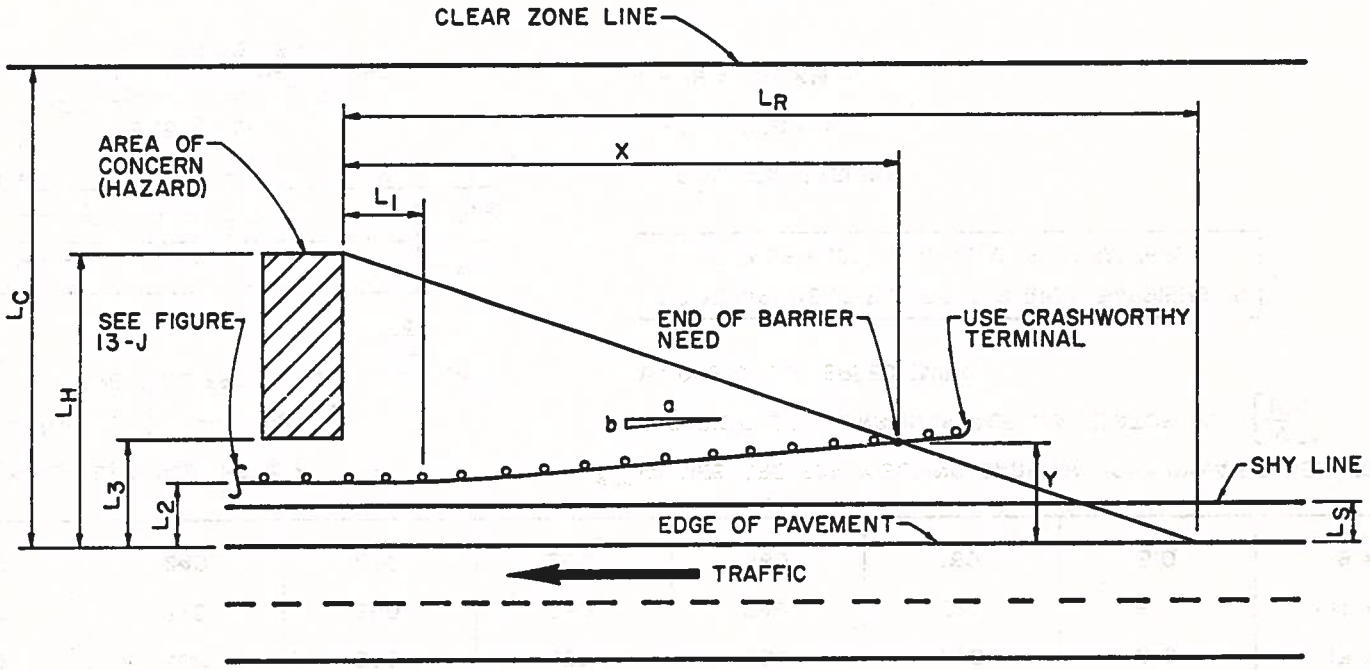
NOTE:

SEE TEXT FOR EXCEPTIONS TO CRITERION



ROADSIDE BARRIER LATERAL PLACEMENT CRITERION

FIGURE 13-H



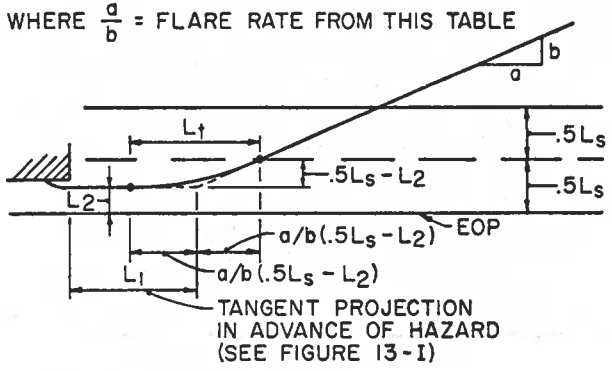
APPROACH BARRIER LAYOUT VARIABLES

FIGURE 13-I

OPERATING SPEED (MPH)	DESIGN TRAFFIC VOLUME (ADT)						
	OVER 6000	2000-6000	800-2000	250-800	UNDER 250		
	RUNOUT LENGTH L_R (FT.)	RUNOUT LENGTH L_R (FT.)	RUNOUT LENGTH L_R (FT.)	RUNOUT LENGTH L_R (FT.)	RUNOUT LENGTH L_R (FT.)	SHY LINE OFFSET (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. = 0.305 M

1 MPH = 1.61 KM/HR

DESIGN PARAMETERS FOR ROADSIDE BARRIER LAYOUT

TABLE 13-7

distance beyond which a roadside object will not be perceived as a threat by a driver. The roadside barrier should be placed beyond the shy line offset.

For highways with operating speeds between 40 and 60 mph, the clear zone width (L_C) can be determined from Figure 13-D. The widths obtained from Figure 13-D are considered appropriate for high traffic volume highways. (Say a design average daily traffic volume of over 6000 vehicles.) The clear zone widths (L_C) obtained from Figure 13-D may be adjusted for lower traffic volumes or for speeds of 30 and 70 mph by proportioning the L_C value obtained by a ratio of L_R value obtained from Table 13-7. This adjusted clear zone width will be useful in the design of roadside for low volume, low speed highways.

To illustrate the procedure to adjust the clear zone widths (L_C)for different speeds and traffic volumes, the following examples are given:

a. Find the clear zone width, L_C, for a rural freeway designed for over 6,000 vpd (ADT) and a design speed of 70 mph. Assume a roadside slope of 10:1.

From Figure 13-D
L_C (60 mph, 10:1 slope)=30 feet

From Table 13-7
L_R (60 mph, over 6,000 gpd)=400 feet
L_R (70 mph, over 6,000 vpd)=480 feet, therefore,
L_C (70 mph, 10:1 slope)= $\frac{480}{400}$ x 30=36 feet

b. Find the clear zone width, L_C, for a low volume highway (500 vpd), and a design speed of 40 mph. Assume a 3:1 fill slope.

From Figure 13-D
L_C (40 mph, 3:1 slope)=20 feet

From Table 13-7
L_R (40 mph, over 6,000 vpd)=240 feet
L_R (40 mph, 500 vpd)=180 feet; therefore,
L_C (40 mph, 3:1 slope, 500 vpd)= 20 x $\frac{180}{240}$ =15 feet

To determine the position (see Figure 13-I) of the end of need, the following equations apply:

$$X = \frac{L_H + \frac{b}{a}(L_1) - L_2}{\frac{b}{a} + \frac{L_H}{L_R}} \qquad \text{Equation (1)}$$

$$Y = L_H - \frac{L_H}{L_R}(X) \qquad \text{Equation (2)}$$

Where,

L_H = distance from edge of traveled way (edge of pavement), to the lateral extent of the hazard;

$\frac{a}{b}$ = slope of flare (see Figure 13-I);

L_1 = length of tangent section of barrier upstream from hazard (see diagram, Table 13-7);

L_2 = distance from edge of pavement to barrier;

L_R = runout length (see Figure 13-I).

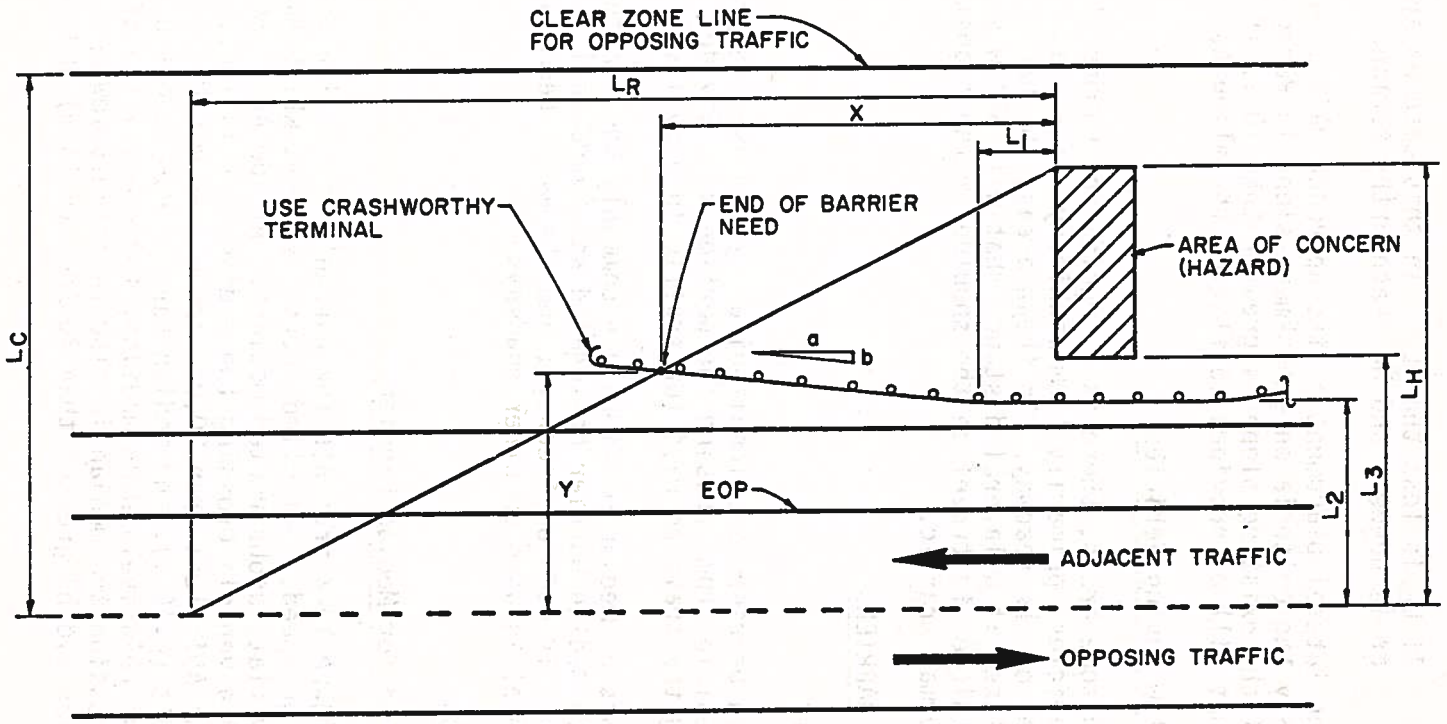
It should be noted that L_H should never exceed the clear zone distance obtained from Figure 13-D, and the distance (L_3-L_2) should satisfy the criteria for barrier-to-hazard distance.

Coordinates X and Y will locate the end of need for the approach barrier; however, to terminate the barrier properly, some type of crashworthy end treatment should be used.

Figure 13-J illustrates the layout variables for an approach barrier for opposing traffic. The length of need and the end of the barrier are determined by use of Equations (1) and (2) just discussed. However, note that all of the lateral dimensions are with respect to the edge of the traveled way of the opposing traffic. If there is a 2 way divided highway, the edge of the traveled way for the opposing traffic would be the edge of pavement (EOP) on the median side.

There are 3 ranges of L_C which deserve special attention for an approach barrier for opposing traffic:

a. If L_3 is less than L_C, and L_C is less than L_H. In this case use L_H equal to L_C.



APPROACH BARRIER LAYOUT
FOR OPPOSING TRAFFIC

FIGURE 13-J

b. If L_2 is less than L_c, and L_c is less than L_3. In this case no approach barrier is needed, but a crashworthy terminal is needed.

c. If L_c is less than L_2. In this case no approach barrier is needed, and no crashworthy terminal is needed.

The lateral placement of the approach rail should also satisfy the criteria on embankment slope in Paragraph 13-02.5.3. If the existing slope is greater than 10:1, it is suggested that fill be provided to flatten the slope to 10:1.

5. Slow Moving Vehicles

In some areas, there is a significant number of slow moving vehicles, primarily farm machinery, that travel on the shoulder of the roadway. In these areas, the barrier should be placed at a lateral distance that will allow slow moving vehicles to travel on the shoulder without obstructing the normal traffic.

13-03 MEDIAN BARRIERS

13-03.1 General

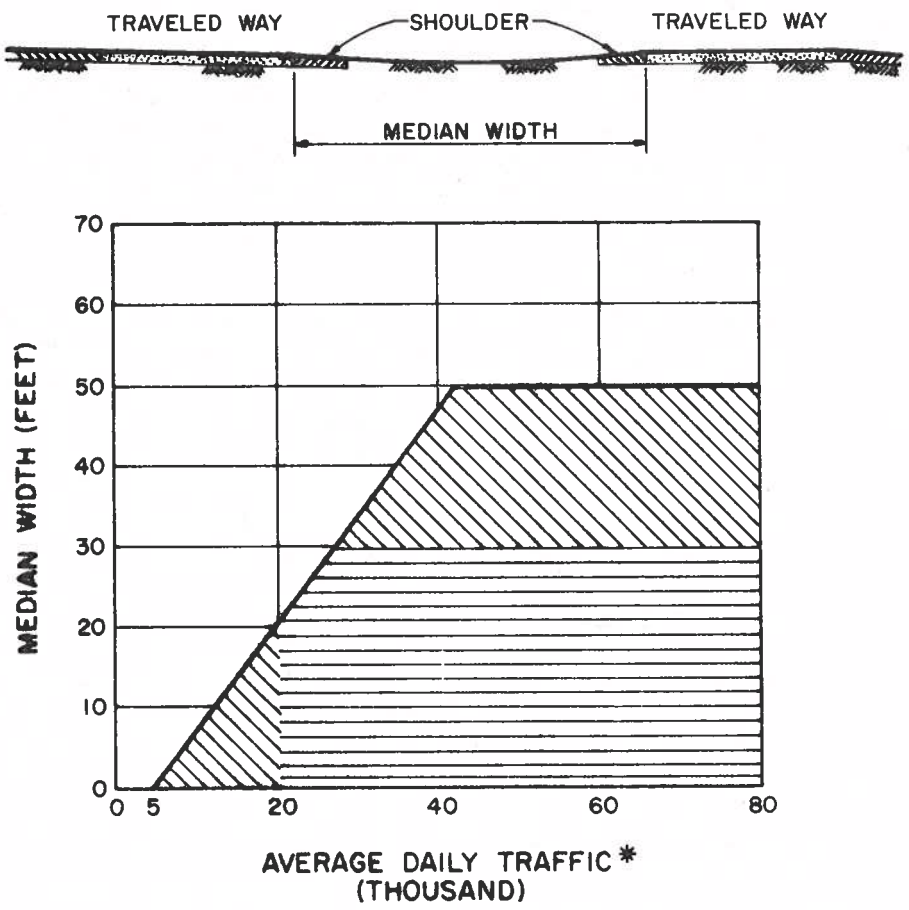
A median barrier is a longitudinal system used to prevent an errant vehicle from crossing the portion of a divided highway separating the traveled ways for traffic in opposite directions.

Two types of median barriers are commonly used in the State. The concrete median barrier designated as "Guard Rail Type 4 (Rigid Barrier)" is used on freeways; and the metal beam barrier (single or double) is used on other roadways.

13-03.2 Warrants

1. Need for Median Barrier

Figure 13-K presents the warrants for median barriers on high speed, controlled access roadways which have relatively flat, unobstructed medians. At low ADTs, the probability of a vehicle crossing the median is relatively low. Thus, for ADTs less than 20,000 and median widths less than 20 feet (6.10 m), the median barrier use is optional. Likewise, for relatively wide medians, the probability of a vehicle crossing the median is also relatively low. Thus, for median widths greater than 30 feet (9.15 m) and ADTs below



MEDIAN BARRIER WARRANTS

FIGURE 13-K

the warrant curve, median barrier is optional. Medians wider than 50 feet (15.25 m) do not warrant a barrier unless there is an adverse history of across-the-median accidents. It should be noted that after a warranted median barrier is installed, accident severity will decrease; however, accident frequency will generally increase since the space available for return-to-the road maneuvers is decreased.

Careful consideration should be given to the installation of median barriers on multi-lane expressways or other roadways with partial control of access. Problems are created at each intersection or median cross-over since the median barrier must be terminated at these points.

2. Transitions

Median barrier transition sections are warranted when it becomes necessary to connect median barriers of differing lateral stiffness. In general, a median barrier transition section is needed when there is a significant change in the lateral strength of the barrier. Another example is median barriers which must also shield fixed objects in a narrow median. In such cases, the median barrier is usually flared so that it encompasses the rigid object.

3. End Treatment

An untreated median barrier terminal is essentially a fixed-object hazard to the motorist. Therefore, a crashworthy end treatment is warranted if the median barrier is terminated within the clear distance.

13-03.3 Median Barrier Systems

Median barriers may be classed into 3 types: as flexible, semi-rigid and rigid barriers.

The flexible barrier systems undergo considerable deflection upon impact and are generally more forgiving than the other 2 types of barriers. They are generally of the beam rail on steel weak post.

The semi-rigid barrier systems include box beam, and blocked out "W" metal beam barriers.

The rigid barrier is the concrete barrier.

13-03.4 Median Barrier Transitions

Median barrier transition sections are needed between adjoining median barriers of significant differences in lateral stiffness; between a median barrier and another type of barrier, such as a bridge rail; or when a median barrier must be stiffened to shield fixed objects in the median such as bridge piers.

Refer to State of Hawaii, Department of Transportation Standard Plans for details of median barrier transitions.

13-03.5 End Treatment of Median Barriers

An untreated end of a median barrier is extremely hazardous. Impact with the untreated end of a metal beam type system may result in the beam penetrating the passenger compartment as well as an abrupt stop. Impact with the untreated end of a concrete median barrier (CMB) system will result in intolerable impact forces. A crashworthy end treatment for a median barrier is essential if the barrier is terminated within the clear distance of travel from either direction.

To be crashworthy, the end treatment should not spear, vault, or roll the vehicle for head-on or "nose" impacts. For impacts between the end and the standard section, the end treatment should have the same redirectional characteristics as the standard median barrier, which means that the end must be properly anchored. The end treatment must thus be capable of developing the full tensile strength of the standard rail element, whether a crashworthy end treatment is warranted or not.

13-03.6 Selection of Median Barrier Type

Table 13-8 suggests median barrier types in relation to median width.

Table 13-8

Median Barrier as Related to Median Width

Median Width	Suggested Barrier
Up to 18 feet	Rigid or Semi-Rigid
18 to 30 feet	Rigid, Semi-Rigid
30 to 50 feet	Semi-Rigid or Flexible

Some consideration should be given to the maintenance of the median barrier system. On high volume, high speed highways and freeways, the rigid concrete median barrier appears to require the lowest maintenance per hit when compared to any of the other systems.

Single cable barriers are unacceptable.

13-03.7 Placement of Median Barriers

Major factors to consider in the lateral placement of a median barrier are the effects of the terrain between the edge of the traveled way and the barrier on the errant vehicle's trajectory, and the flare rate of transition sections. Another factor of concern is rigid objects in the median.

1. Terrain Effects

   Terrain conditions between the traveled way and the barrier can have significant effects on the barrier's impact performance. Curbs and sloped medians (including superelevated sections) are two prominent features which deserve special attention. A vehicle which traverses one of these features prior to impact may go over the barrier or submarine under the barrier or snag on its support posts.

2. Curbs

   In general, it has been found that curbs offer no safety benefits on high speed roadways from the standpoint of vehicle behavior following impact. A curb should never be used for purposes of redirecting an errant vehicle. If special conditions require the use of a curb, the face of the curb should be placed behind the face of the barrier.

3. Sloped Medians

   The most desirable median is one that is relatively flat (flatter than 10:1) and free of rigid objects. If warranted, the barrier can then be placed at the center of the median.

13-04 CRASH CUSHIONS

13-04.1 General

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. The relatively low cost and potentially high safety payoff offered by crash cushions justifies emphasis on their installation.

The designer should obtain the manufacturer's brochures for layout details, design guides and installation of the various crash cushions to aide him in the selection and design of crash cushions.

13-04.2 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. A crash cushion should, also, be considered for shielding rigid objects such as bridge piers, overhead sign supports, abutments, and retaining wall ends. It may also be used to shield roadside and median barrier terminals, especially concrete median barriers.

A crash cushion or a vehicle arresting device may also 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 going beyond the intersection.

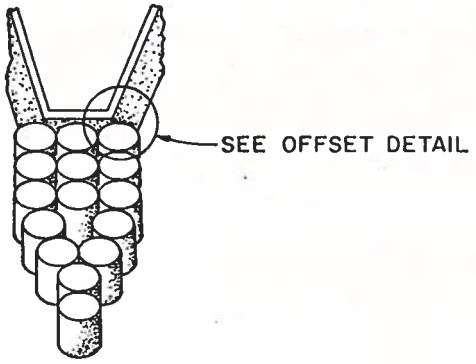
13-04.3 Types of Crash Cushions

There are several types of operational crash cushions. All of them are patented, except for the steel drum system. Figure 13-L shows 3 of the more common types of crash cushions.

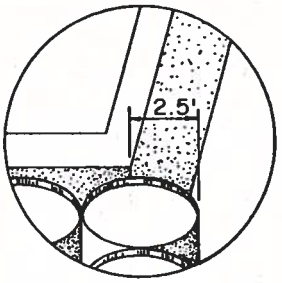
13-04.4 Site Considerations

1. New Roadways

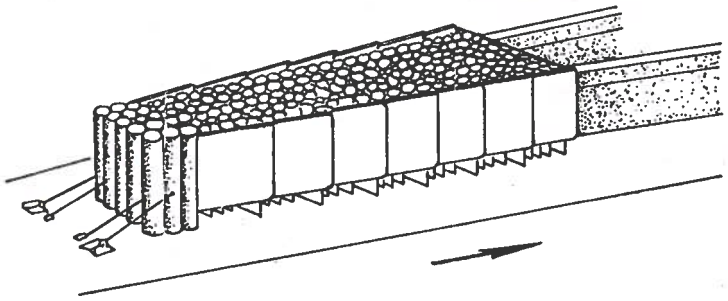
It is recommended that space be reserved on all new construction for potential crash cushion installations. These recommendations are presented in Table 13-9 and shown on Figure 13-M. Under the "minimum" column, the "restricted conditions" represent the absolute minimums and should only be considered where there are extremely tight geometric controls. The "unrestricted conditions" represent the minimum for all projects except for those sites where it can



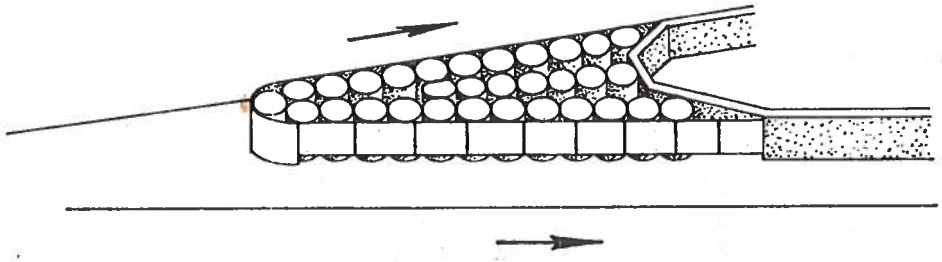
SAND FILLED BARRELS



OFFSET DETAIL



WATER FILLED CELLS



STEEL DRUM

IMPACT ATTENUATORS

FIGURE 13-L

DESIGN SPEED ON MAINLINE (M.P.H.)	DIMENSIONS FOR CRASH CUSHION RESERVE AREA (FEET)								
	MINIMUM						PREFERRED		
	RESTRICTED CONDITIONS			UNRESTRICTED CONDITIONS					
	N	L	F	N	L	F	N	L	F
30	6	8	2	8	11	3	12	17	4
50	6	17	2	8	25	3	12	33	4
70	6	28	2	8	45	3	12	55	4
80	6	35	2	8	55	3	12	70	4

METRIC CONVERSIONS

1 FT. = 0.305 M

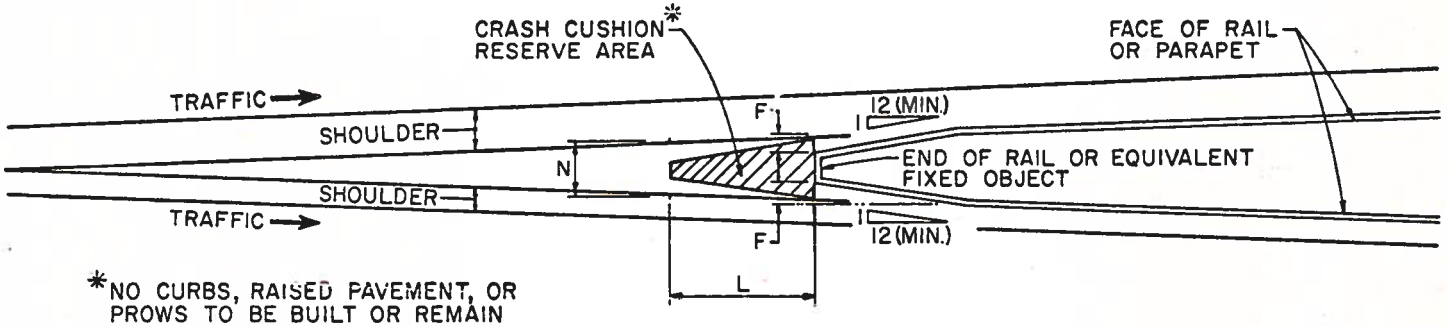
1 MPH = 1.609 KM/HR

NOTE:

SEE FIGURE 13-M FOR GORE AREA LAYOUT.

RESERVE AREA FOR GORES

TABLE 13-9



RESERVE AREA FOR GORES

FIGURE 13-M

DESIGN SPEED ON MAINLINE (M.P.H.)	DIMENSIONS FOR CRASH CUSHION RESERVE AREA (FEET)								
	MINIMUM						PREFERRED		
	RESTRICTED CONDITIONS			UNRESTRICTED CONDITIONS					
	N	L	F	N	L	F	N	L	F
30	6	8	2	8	11	3	12	17	4
50	6	17	2	8	25	3	12	33	4
70	6	28	2	8	45	3	12	55	4
80	6	35	2	8	55	3	12	70	4

METRIC CONVERSIONS

1 FT. = 0.305 M

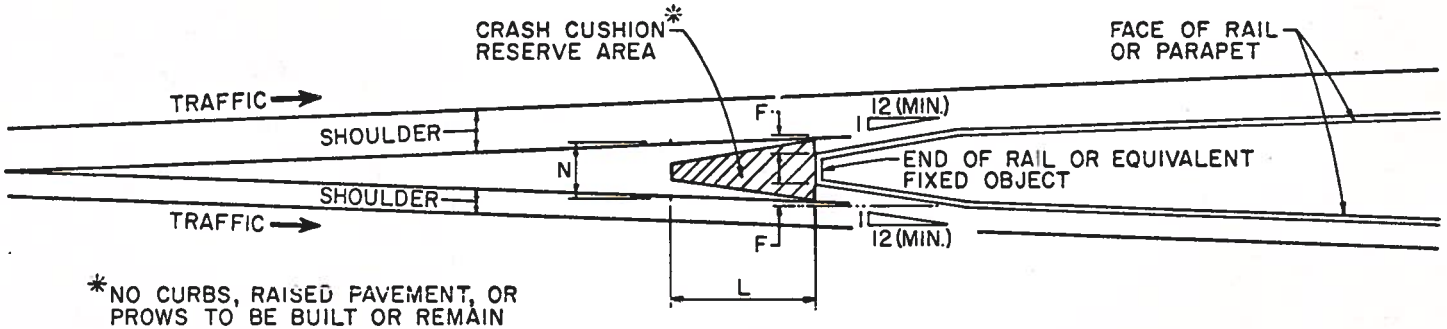
1 MPH = 1.609 KM/HR

NOTE:

SEE FIGURE 13-M FOR GORE AREA LAYOUT.

RESERVE AREA FOR GORES

TABLE 13-9



*NO CURBS, RAISED PAVEMENT, OR PROWS TO BE BUILT OR REMAIN IN THE AREA SURROUNDING OR OCCUPIED BY THE CRASH CUSHION.

RESERVE AREA FOR GORES

FIGURE 13-M

be shown that the increased cost for accommodating these dimensions, as opposed to those for restricted conditions, will be unreasonable. The "preferred" values should be considered optimum. There is no intention to imply that if space is provided in accordance with these dimensions that the space will be fully occupied by a crash cushion device. The reason for proposing these dimensions is so that if experience shows that devices should be designed for greater ranges of vehicle weights and/or for lower deceleration forces, there will be space available for installation of such devices in the future. In the meantime, the unoccupied reserve crash cushion space will provide valuable additional recovery area.

It is suggested that considerations be given, at the design stage of new projects, to the use of crash cushions at other locations also. These would be hazardous sites that could not be avoided in the project. Examples are bridge piers, overhead sign supports, and other non-avoidable rigid objects. These sites could be designed to facilitate the installation of a crash cushion.

2. Existing Roadways

The selection of a crash cushion for some existing roadways may be dictated by site conditions. The following factors should be considered in the selection.

a. Dimensions of Object or Hazard to be Shielded

The width of the object or hazard is an important factor. Certain crash cushions are limited in the width that can be shielded.

b. Structural Details of Object to be Shielded

Several of the operational crash cushions require a backup structure that is capable of withstanding the impact forces. If the object being shielded is structurally inadequate, provisions will have to be made to support or restrain the barrier during impact if these systems are used. Site preparations on existing structures can be extensive. This usually involves removing a concrete gore nose. In some instances, bridge railing ends are revised and a concrete wall or backstop is built. A backstop or secondary barrier is suggested for inertial crash cushions where the vehicle could penetrate the cushion and plunge down a slope.

c. Space Available for Crash Cushion

Two dimensions must be considered with regard to the available space. These are the length ("L" in Figure 13-M) over which the barrier can be placed, and the distance on either side of the barrier to the hazard ("F" in Figure 13-M). When length is an important design consideration, the space efficiency of a barrier throughout the design vehicle speed and weight range will be important.

d. Physical Conditions of the Available Space

The following site conditions should be considered:

(1) the presence of a curb which could affect the performance of the crash cushion;

(2) the existing surface material and condition thereof;

(3) the longitudinal and transverse slope of the crash cushion area;

(4) high wind and/or abnormal vibration conditions; and

(5) the existence of construction or expansion joints in the crash cushion area.

It is desirable that the crash cushion systems be placed on concrete or asphalt surfaces.

13-04.5 Selection Guideline for Crash Cushions

The number and complexity of factors which enter the selection process for crash cushions preclude the development of a simple selection procedure. As has been alluded to in the previous paragraphs, each operational system has its own unique physical and functional characteristics. In some cases, one crash cushion will stand out as the most appropriate, while in other cases 2 or more crash cushions may be considered essentially equal in performance. Listed and discussed below are factors which should be evaluated before making a selection. It is suggested that these factors be evaluated in the order given, although they are not necessarily listed in order of importance.

It is assumed, at this point, that a crash cushion is warranted and that a selection must be made.

1. Site Considerations

The first item to evaluate is the site conditions. Factors to evaluate include dimension of object being shielded, structural characteristics of object being shielded, available space for cushion, and physical condition of site. In many cases, the site conditions will establish the type of barrier needed.

2. Structural and Safety Characteristics of Candidate Systems

If more than one system can be used, the designer should carefully evaluate the structural and safety characteristics of each candidate system. These include factors such as impact decelerations, redirection capabilities, anchorage requirements, debris produced by impact, and backup structure requirements. It is desirable that the most crashworthy system be installed.

3. Maintenance Characteristics and Aesthetic Appeal

Not too infrequently the most appropriate barrier will still not be evident after evaluating items 1 and 2 above. The maintenance characteristics of each barrier may therefore play an important role in the selection.

4. Costs

Limited cost data has indicated that some crash cushions are more expensive than others. This variation in cost can readily be seen in the components which go to make up the systems and the installation effort required. The designer should, if other design factors for the site under consideration have not indicated the one best crash cushion, employ engineering economics so as to arrive at the least expensive system over the estimated life of the system. The economic factors to be considered are initial cost, maintenance costs, vehicle damage and occupant injury costs, time value of money, life of the system, and salvage value.

13-04.6 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 (0.10 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, when placed on elevated gores, may walk or crack due to vibration of the structure. Manufacturers should assure the adequacy of their design in this regard.

SECTION 14.

OTHER DESIGN CONSIDERATIONS

14-01 GENERAL

This section covers a variety of subjects which relate to and impact on the design of highway systems in the State.

14-02 EMERGENCY ESCAPE RAMPS

criteria from FHWA – 80 mph speed revised to fit Hawaii's condition

14-02.1 General

Where long, descending grades exist or where topographic and location controls require such grades on new alignment, the design and construction of an emergency escape ramp at an appropriate location is desirable for the purposes of slowing and stopping an out-of-control vehicle away from the main traffic stream. An out-of-control vehicle generally is the result of an operator losing control of the vehicle because of a loss of brakes either through overheating or mechanical failure, or failure to downshift at the appropriate time. Specific guidelines for the design of escape ramps are lacking at this time. However, considerable experience with ramps constructed on existing highways has led to the design and installation of effective ramps. Reports and studies of the existing ramps indicate that their operational characteristics are providing acceptable deceleration rates and affording good driver control of the vehicle on the ramp.

There are several resistance forces which act on every vehicle to affect the vehicle's speed. These include: engine, braking, and tractive resistance forces. Engine and braking resistance forces can be ignored in the design of escape ramps since they should be designed for the worst case; that is, the vehicle is out of gear and the brake system has failed. Tractive resistance contains 4 subclasses: inertial, air, rolling, and gradient. Inertial and negative gradient resistance forces act to maintain motion of the vehicle while rolling, while positive gradient, and air resistance forces act to retard its motion. Figure 14-A illustrates the action of the various resistance forces on a vehicle.

Inertial resistance can be described as a force which will resist movement in a vehicle at rest, or will keep a vehicle in motion unless the vehicle is acted upon by some external force. Inertial

F_a – AIR RESISTANCE

F_i – INERTIAL RESISTANCE

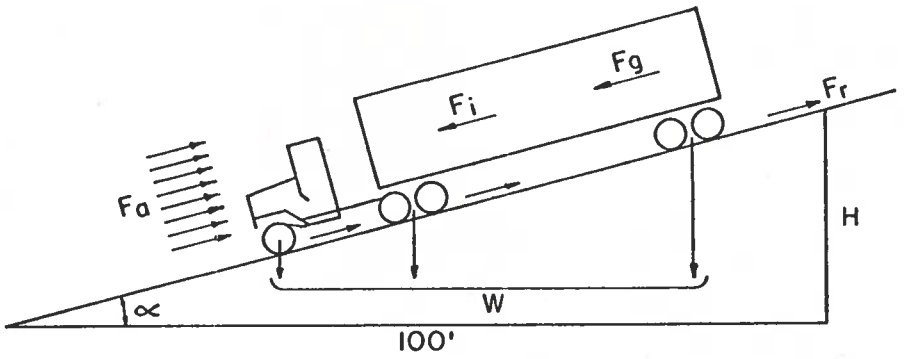
F_g – GRADIENT RESISTANCE

F_r – ROLLING RESISTANCE

W – GROSS VEHICLE WEIGHT

H – HEIGHT

$\propto$ – SLOPE ANGLE



FORCES ACTING ON VEHICLE IN MOTION

FIGURE 14-A

resistance must be overcome to either increase or decrease the speed of a vehicle. Rolling and positive gradient resistance forces are available to overcome the inertial resistance. Rolling resistance is a general term used to describe the resistance at the area of contact between a vehicle's tires and the road surface to motion and is only applicable when a vehicle is in motion. It is influenced by the type and displacement characteristics of the surfacing material of the roadway. Each surfacing material has a coefficient expressed in pounds per 1,000 pounds of Gross Vehicle Weight (Table 14-1) which determines the amount of rolling resistance of a vehicle. The values shown in Table 14-1 for rolling resistance have been obtained from various sources throughout the country and are a best available estimate.

Table 14-1

Rolling Resistance for Surfacing Material

Surfacing Material	Rolling Resistance #/1000#GWV	Equivalent Grade % 1/
Portland Cement Concrete	10	1.0
Asphalt Concrete	12	1.2
Gravel Compacted	15	1.5
Earth, Sandy, Loose	37	3.7
Crushed Aggregate, Loose	50	5.0
Gravel, Loose	100	10.0
Sand	150	15.0
Pea Gravel	250	25.0

1/Rolling Resistance expressed as equivalent gradient.

Gradient resistance is due to the effect of gravity and is expressed as the force required to move the vehicle through a vertical distance. For gradient resistance to provide a beneficial force on an escape ramp, the vehicle must be moving upgrade against gravity. In the case where the vehicle is descending a grade, gradient resistance is negative, thereby reducing the available forces to slow and stop the vehicle. It is influenced by the total weight of the vehicle and the magnitude of the grade. For each percent of grade, the gradient resistance is 20 pounds per ton whether the grade is positive or negative.

The remaining component of tractive resistance is air resistance, the force resulting from the retarding drag on the various surfaces. Air causes a significant resistance at speeds above 50 miles per hour and is negligible under 20 miles per hour. The effect of air resistance has been neglected in determining the length of the arrester bed in the example below, thus introducing a small safety factor.

14-02.2 Types

There are 4 basic types of emergency escape ramps currently in use. These are the sandpile, descending grade, horizontal grade, and ascending grade. The 4 types are illustrated in Figure 14-B.

Sandpiles, composed of loose, dry sand, are usually no more than 200 feet in length. The influence of gravity is small in this installation because of the slight grade of the pile. The increase in rolling resistance is supplied by the loose sand.

The horizontal grade and descending grade type ramps can be rather lengthy because the gravitational effect is not acting to help reduce the speed of the vehicle. For the horizontal grade type ramp, the force of gravity is zero, and for the descending grade type ramp, the force of gravity acts in the direction of vehicle movement. The increase in rolling resistance is supplied by an arresting bed composed of loose aggregate.

The ascending grade type ramp utilizes both the arresting bed and the effect of gravity, in general reducing the length of ramp necessary to stop the vehicle. The loose material in the arresting bed increases the rolling resistance, as in the other types of ramps, while the force of gravity acts opposite to the vehicle movement (downgrade). The loose bedding material also serves to hold the vehicle in place on the ramp grade after it has come to a safe stop.

Each one of the ramp types is applicable to a particular situation where an emergency escape ramp is desirable and must be compatible with established location and topographic controls at possible sites.

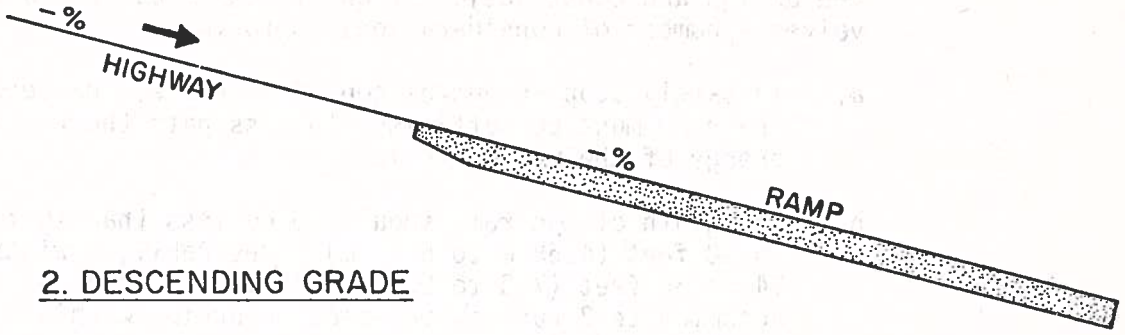
14-02.3 Design Considerations

1. General

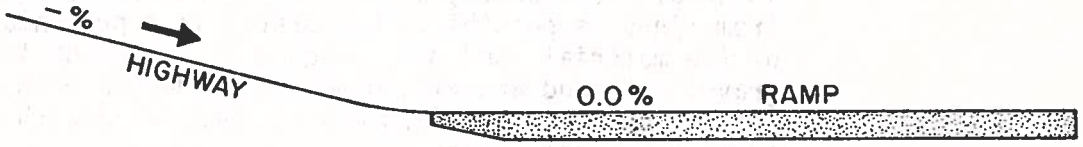
The combination of the above external resistance and numerous internal resistance forces not discussed, act to limit the maximum speed of an out-of-control vehicle. Speeds in



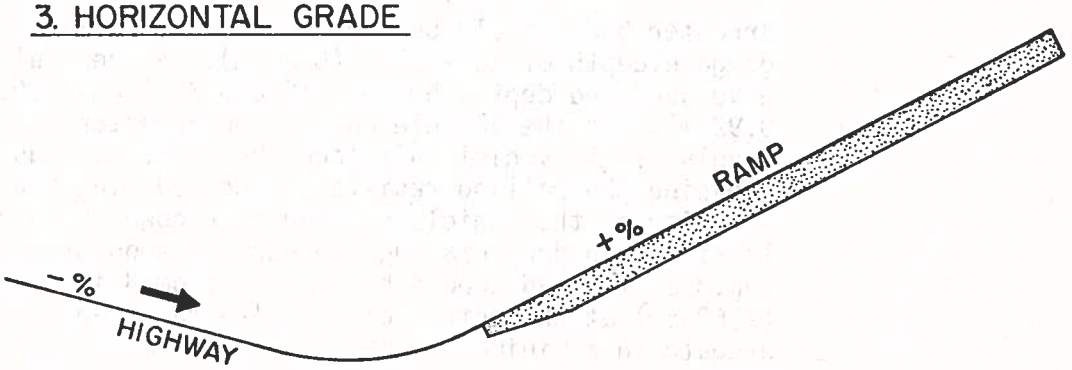
1. SANDPILE



2. DESCENDING GRADE



3. HORIZONTAL GRADE



4. ASCENDING GRADE

NOTE:
PROFILE IS ALONG THE BASELINE OF THE RAMP.

BASIC DESIGNS FOR TRUCK ESCAPE RAMPS

FIGURE 14-B

excess of 80 to 90 miles per hour will rarely, if ever, be attained. Therefore, an escape ramp should be designed for a minimum entering speed of 50 miles per hour with a 70 mile per hour design preferred. For the escape ramp to be effective, it must be able to stop the largest vehicle expected to use the ramp. This vehicle would generally be a WB-40 or a WB-50.

2. Design Considerations

The design and construction of an effective escape ramp involves a number of considerations as follows:

a. To safely stop an out-of-control vehicle, the length of the ramp must be sufficient to dissipate the kinetic energy of the moving vehicle.

b. The width of the ramp should be no less than 15 feet to 18 feet (4.58 m to 5.49 m). Desirably, a width of 24 to 30 feet (7.3 to 9.15 m) should be provided to accommodate 2 runaway vehicles occurring within a short period of time.

c. The surfacing material used in the arrester bed should be clean, not easily compacted, and have a high coefficient of rolling resistance. When aggregate is used, it should be rounded, uniformly graded, and as free from fines as possible. Pea gravel is representative of the material used most frequently, although loose gravel and sand are also used. A gradation with a top size of 1-1/2 inches (3.8 cm) has been used with success in several States.

d. Arrester beds should be constructed with a minimum aggregate depth of 12 inches (0.31 m). Successful ramps have used bed depths between 12 and 36 inches (0.31 and 0.92 m). As the vehicle enters the arrester bed, the wheels of the vehicle displace the surface, thus increasing the rolling resistance and reducing the possibility of the vehicle rolling back down the grade. To assist in decelerating the vehicle smoothly, the depth of the bed should be tapered from 3 inches (7.62 cm) at the entry point to the full depth of aggregate in a minimum of 100 feet (30.5 m).

e. The entrance to the ramp must be designed so that a vehicle traveling at a high rate of speed can enter safely. The main roadway surface should be provided

to a point at or beyond the exit gore so that both front wheels of the out-of-control vehicle will enter the arresting bed simultaneously, and the operator will have preparation time before actual deceleration begins.

f. Access to the ramp must be made obvious by exit signing so as to preclude the possibility of it being missed by the operator of an out-of-control vehicle. Advance signing is required to inform an operator of the existence of an escape ramp and to prepare the operator well in advance of the decision point so as to allow the operator enough time to decide whether or not to use the escape ramp. A regulatory sign near the entrance should be used to discourage other motorists from entering the ramp (see the Manual of Uniform Traffic Control Devices (MUTCD) for sign designs). Illumination of the approach and ramp is desirable.

g. A surfaced service road located adjacent to the arrester bed for maintenance is optional. The width of this lane should be 10 feet (3.05 m) minimum.

h. Escape ramps should exit to the right of the main roadway. The alignment of the escape ramps should be at 5 degrees from tangent, or of very flat curvature to relieve the operator of potential vehicle control problems.

i. Escape ramps generally may be built at any feasible location where the main road alignment is tangent. They should be built in advance of main line curvature that cannot be negotiated safely by an out-of-control vehicle and in advance of populated areas.

3. Placement

Each grade has its own characteristics. Highway alignment, gradient, length, and descent speed contribute to the potential for out-of-control vehicles. An escape ramp should be provided as soon as a need is established. Unnecessary escape ramps should be avoided. For example, if an escape ramp is provided before a sharp horizontal curve, a second ramp would not be required just beyond the curve that created the need for the initial ramp, except as a safety valve in cases where there have been unusual numbers of out-of-control vehicle incidents on the grade. Turnouts or pull-off areas at the summit can be used for equipment inspection to help reduce the number of out-of-control vehicle incidents on the grade.

4. Ascending Grade Ramp

The most commonly used escape ramp is the ascending grade type with an arrester bed. Ramp installations of this type use gradient resistance to advantage, supplementing the effects of the aggregate on the arrester bed.

As a vehicle rolls upgrade, it loses momentum and will eventually stop because of the effect of gravity. To determine the distance required to bring the vehicle to a stop with consideration of the rolling resistance and grade resistance, the following equation may be used:

$$L = \frac{1.075\ (\frac{W}{32.2} + kn)\ \ (V_i^2 - V_f^2)}{W(R+G)-TE} *$$

*A. Taragin, "Grade Length and Speed", Highway Research Board Proceeding, 1945.

Since the mass equivalent constant for the gear in which the vehicle is operating (kn), final velocity (V_f) and Tractive Effort (T.E.) are zero because the vehicle is assumed to be free rolling and is brought to a stop on the escape ramp, the equation simplifies to:

$$L = \frac{V^2}{30(R+G)}$$ where;

L = Distance to stop, i.e., length of arrester bed;

V = Entering velocity in miles per hour;

G = Percent grade divided by 100; and

R = Rolling resistance expressed as equivalent percent gradient divided by 100 (Table 14-1).

For example, assume that topographic conditions at a site selected for an emergency escape ramp limit the ramp to an upgrade of 10 percent (G = +0.10). The arrester bed is to be constructed with loose gravel. Using Table 14-1, R is determined to be 0.10. The ramp is to be designed for an entering speed of 90 miles per hour. The length required is determined from $L=V^2/30(R+G)$. In this case, the length of the arresting bed is 1,350 feet (412 m).

When conditions dictate a design where the arrester bed must be constructed using more than one grade along its length, such as shown in Figure 14-C, the velocity of the vehicle at the end of the first grade is determined knowing entering velocity, length of grade, gradient, and resistance of the material in the bed using the equation: $V_f^2 = V_i^2 - 30L(R+G)$. The velocity of the vehicle is determined at each change of grade on the ramp until sufficient length is provided to stop the out-of-control vehicle.

5. Typical Escape Ramp

Figure 14-C shows a plan and profile of an emergency escape ramp with typical appurtenances.

14-02.4 Maintenance

After each use, aggregate beds should be smoothed and the aggregate loosened as necessary. In addition, the bedding material should be cleaned of contaminants and loosened periodically to retain the retarding characteristics of the bedding material and maintain free drainage. Maintenance of the appurtenances should be accomplished as necessary.

14-02.5 Substandard Location

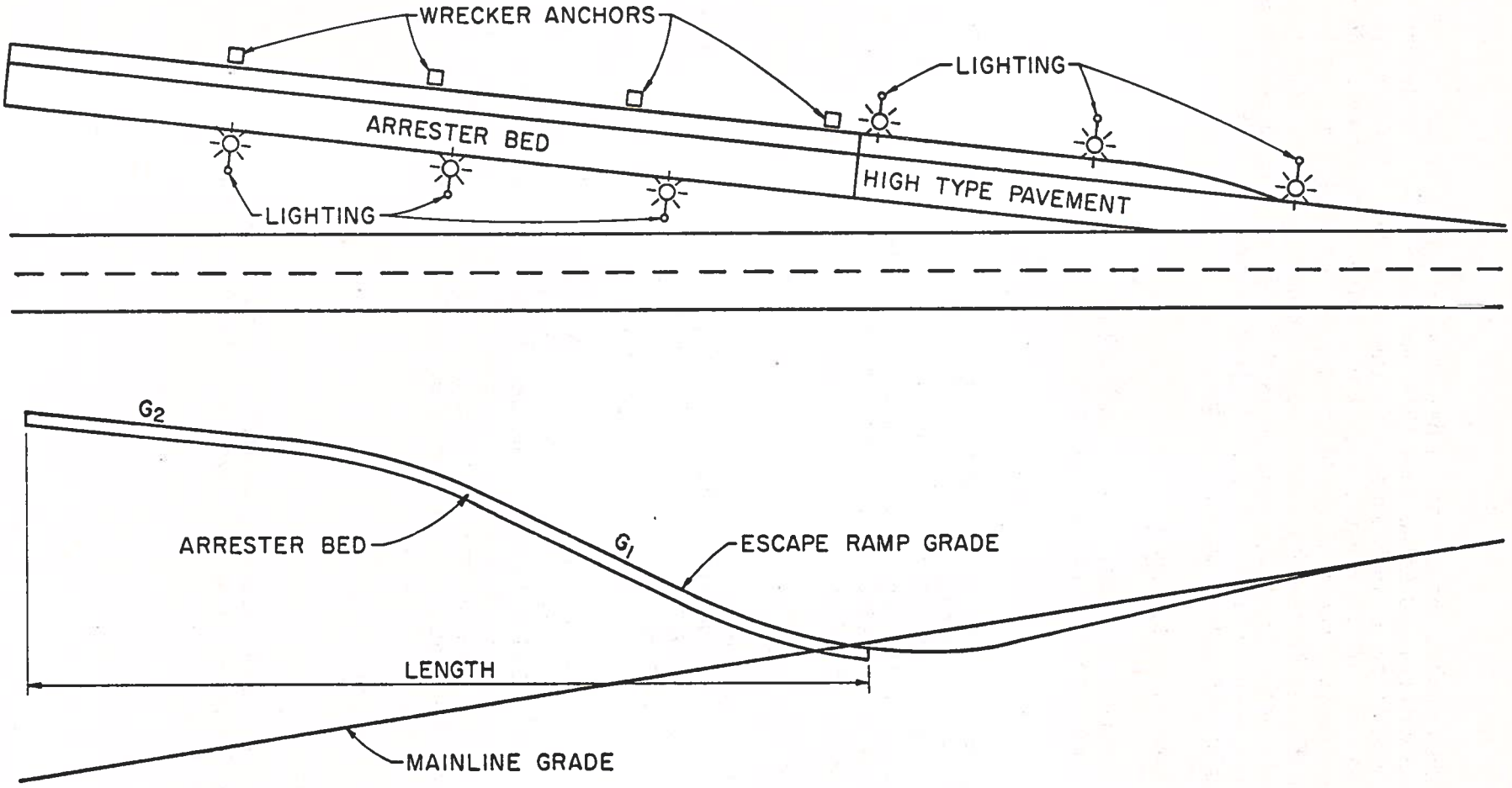
When the only feasible location for an escape ramp will not provide sufficient length, width and grade to completely stop an out-of-control vehicle, it should be supplemented with an acceptable positive attenuation device to prevent an out-of-control vehicle from leaving the end of the ramp.

Where full length ramp is to be provided with full deceleration capability for the design speed, a "last chance" device should be considered when the consequences of leaving the end of the ramp are serious. A mound of the bedding material has been used at the end of ramps in several instances as the "last chance" device. The mounds are between 2 and 5 feet (0.61 and 1.53 m) high with 1-1/2:1 slopes.

14-03 AUXILIARY LANES

14-03.1 General

Auxiliary lanes discussed in this section are truck climbing lanes for both 2 lane highways and multi-lane highways, and passing lanes on 2 lane highways.



TYPICAL EMERGENCY ESCAPE RAMP

FIGURE 14-C

14-03.2 Climbing Lanes

1. General

Climbing lanes are more prevalent on 2 lane highways than on multi-lane highways, for the reason that 2 lane highways are adversely affected by heavily loaded vehicles operating at low speeds on sustained grades. Multi-lane facilities more frequently have sufficient capacity to handle their traffic load, including the normal percentage of trucks, without being severely congested.

Freedom and safety of operation on 2 lane highways, besides being governed by the extent and frequency of passing sections, are adversely affected by heavily loaded vehicular traffic operating on grades of sufficient length to result in speeds that could impede following vehicles. A highway section with a climbing lane is not considered a 3 lane highway, but a 2 lane highway with an extra lane for vehicles moving slowly uphill so that other vehicles using the normal lane to the right of the centerline are not retarded. These vehicles pass the slower vehicles moving upgrade, but not in the lane for opposing traffic, as on the customary 2 lane road. A separate climbing lane exclusively for a slow-moving vehicle is preferred to the addition of an extra lane carrying mixed traffic. Climbing lanes are designed for each direction independently of the other. Depending on the alignment and profile conditions, they may not overlap, or they may overlap as in the case where there is a crest with a long grade on each side.

2. Justification

It is desirable to provide a climbing lane, as an extra lane on the upgrade edge of a 2 lane highway, where the critical length of grade is exceeded, i.e., where the length of grade causes a reduction of 10 mph or more in the speed of loaded vehicles--provided the volume of traffic and percentage of vehicles with high weight/horsepower ratios justify the added cost. Refer to Figure 4-M, Section 4, BASIC GEOMETRIC DESIGN ELEMENTS, to determine the critical length of grade for design.

For detail discussion of the effect of trucks on the capacity of the highway, refer to the Highway Capacity Manual (HCM).

3. Location

The point where an added lane should begin depends on the speeds at which trucks will approach the grade and on the extent of sight distance restrictions on this approach. Where there are no sight distance restrictions or other conditions causing low speed approaches, the extra lane may be introduced on the grade some distance beyond the beginning of the upgrade because the speed of a truck will not be reduced to the point where it creates intolerable conditions for the following drivers until it has traveled a certain distance up the grade. The climbing lane should begin at the point at which the 10 mph speed reduction would occur, i.e., the distance from the bottom of the grade to the point where truck speeds fall 10 mph below the operating speed. Where there are restrictions, or upgrade approaches or other conditions which would cause low speed for approaching trucks, the auxiliary lane should be introduced near the foot of the grade. Figure 14-D shows a typical auxiliary lane layout.

The ideal design would be to extend the climbing lane to a point beyond the crest, where a typical truck could attain a speed that is within 10 mph of the speed of the other vehicles with the desirable speed being at least 35 mph, approximately at Level of Service 'D'. Even this may not be practical in many instances because of the unduly long distance required for trucks to accelerate to the desired speed. For such a condition, a practical point to end the added lane is where the truck can return to the normal lane without undue hazard--in particular, where the sight distance becomes sufficient to permit passing with safety where there is no oncoming traffic or, preferably, at least 200 feet (61 m) beyond this point. In addition, a corresponding length of taper should be provided to permit the truck to return to the normal lane.

4. Geometrics

A climbing lane should desirably be as wide as the through lanes. It should be so constructed that it can immediately be recognized as an extra lane in one direction. The centerline of the normal 2 lane pavement should be clearly marked in the required manner, as for any 2 lane road, including yellow barrier lines for no-passing zones. Adequate signs at the beginning of the upgrade such as "Slower Traffic Keep Right" or "Trucks Use Right Lane" may be used to direct slow-moving vehicles into the climbing lane.

SIGNING AND PAVEMENT MARKINGS SHOULD BE IN ACCORDANCE WITH THE "MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES FOR STREETS AND HIGHWAYS".

LEGEND

— — — TRAFFIC LANE STRIPE

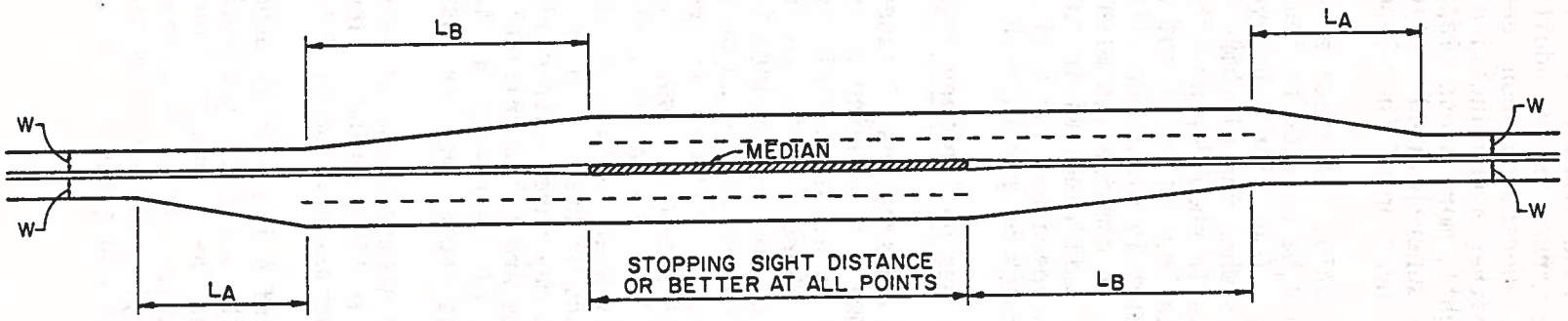
L_A LENGTH OF APPROACH TRANSITION - 350' MAX., 100' MIN. DEPENDING UPON APPROACH SPEED OF SLOWER VEHICLES.

L_B = SW (45 MPH OR GREATER)

$L_B = \frac{WS^2}{60}$ (40 MPH OR LESS)

S = SPEED (MPH)

W = LANE WIDTH (FT.)



SCHEMATIC DIAGRAM

PAVEMENT WIDENING FOR AUXILIARY LANE, TWO-LANE HIGHWAY

FIGURE 14-D

The cross slope of a climbing lane is usually handled in the same manner as the addition of a lane to a multi-lane highway. Depending upon agency practice, this design results in either a continuation of the cross slope or a lane with slightly more cross slope than the adjacent through lane. On a superelevated section, the cross slope is generally a continuation of the slope used on the through lane.

Desirably, the shoulder on the outer edge of a climbing lane should be as wide as the shoulder on the normal 2 lane section. Conditions, however, may dictate otherwise, particularly when the climbing lane is added to an existing highway. A usable shoulder of 4 feet (1.22 m) width or greater is acceptable. Although not wide enough for a stalled vehicle to completely clear the climbing lane, a 4 foot (1.22 m) shoulder, in combination with the climbing lane, generally provide sufficient width for both the stalled vehicle and a slow-speed passing vehicle without need for the latter to encroach on the through lane.

14-03.3 Passing Lanes on Two Lane Highways

Passing lane sections, 4 lanes in width, are constructed on 2 lane highways to provide the desired frequency of safe passing zones, or to eliminate interference from low speed heavy vehicles, or both. Where a sufficient number and length of safe passing sections cannot be obtained in the design of horizontal and vertical alignment alone, an occasional section of 4 lanes may be introduced to provide more sections and length safe for passing. The layout shown in Figure 14-D may be used for passing sections. Such sections are particularly advantageous in rolling terrain, especially where alignment is winding or profile includes critical lengths of grade. In rolling terrain, a highway on tangent alignment may have restricted passing conditions, even though the grades are below critical length. Use of a 4 lane section over some of the crests provides added passing sections in both directions where they are most needed.

Four lane sections should be sufficiently long to permit several vehicles in line behind a slow moving truck to pass before reaching the normal section of 2 lane highway.

Sections of 4 lanes introduced explicitly for passing purposes need not be divided because there is no separation of opposing traffic on the 2 lane portions of the highway. The use of a median, however, is advantageous and should be considered on highways carrying 500 vph or more, particularly on highways to be ultimately converted to 4 lanes divided.

The introduction of a 4 lane section on a 2 lane highway does not necessarily involve much additional grading. A full shoulder width is not as necessary because likely volumes are much less than 4 lane capacity; the vehicles likely to stop are few; and there is little difficulty in passing a vehicle with only 2 wheels on the shoulder. A shoulder width of 4 to 6 feet (1.22 to 1.83 m) may be adequate. Thus, if the normal shoulder width on the 2 lane highway is 10 feet (3.05 m), a 6 to 8 foot (1.83 to 2.44 m) widening of the roadbed on each side is all that may be required. The additional lanes should be at least 10 feet (3.05 m) and preferably 11 or 12 feet (3.35 or 3.66 m) wide, and in no case should the traveled way be less than 40 feet (12.20 m) wide.

14-03.4 Climbing Lanes on Freeways and Multi-Lane Highways

1. General

Climbing lanes have not been used as extensively on freeways and multi-lane highways, perhaps for the reason that multi-lane facilities more frequently have sufficient capacity to handle their traffic load, including the normal percentage of trucks with high weight/horsepower ratios without becoming congested. Climbing lanes generally are not as easily justified on multi-lane facilities as on 2 lane highways because on 2 lane facilities, vehicles following slow moving trucks on upgrades are frequently prevented by opposing traffic from using the adjacent traffic lane for passing, whereas such a barrier to its use is not present on multi-lane facilities. Furthermore, a climbing lane on a 2 lane, 2 way road has some use at all times, even during non-peak hours, whereas on a multi-lane facility, a climbing lane has limited use except during periods of heavy flow. At other times, a slow moving truck in the normal right lane does not impede the following vehicles that can readily move left to the abutting lane and proceed without difficulty or hazard, although there is evidence that slow moving vehicles on through traffic lanes cause accidents.

Although primarily a rural condition, there are instances where climbing lanes will be necessary in urban areas. Climbing lanes are particularly important for freedom of operation on urban freeways where traffic volumes are high in relation to capacity. On older urban freeways and arterial streets with appreciable grades and no climbing lanes, it is a common occurrence for heavy traffic, which may otherwise move well, to queue up on grades.

2. Justification

As in the case for 2 lane highways discussed earlier in this section, the principal determinants of need for climbing lanes are critical lengths of grade, effects of trucks on grades in terms of equivalent passenger cars, service volumes for the desired level of service, and volume for the next poorer level of service.

Generally, climbing lanes should not be considered unless the upgrade directional traffic volume is equal to or greater than the Level of Service 'D' service volume for 2 of the lanes of a 4 lane highway with the normal percentage of trucks. In most cases when the service volume, including trucks, is greater than 1,700 vph and the length of the grade and the percentage of trucks are sufficient to consider climbing lanes, the volume in terms of equivalent passenger cars is likely to approach, or even exceed, the capacity. An increase in the number of lanes throughout the highway section would represent a better investment than the provision of climbing lanes.

A climbing lane is also generally not warranted on 4 lane highways for a directional service volume below 1,000, regardless of the percentage of trucks. Although a truck driver will occasionally pass another truck under such conditions, the inconvenience and hazard with this low volume are not sufficient to justify the cost of a climbing lane in the absence of appropriate criteria.

3. Location

Climbing lanes on multi-lane highways are usually placed on the outer or right hand side of the roadway. The principles for cross slopes, for locating terminal points, and for designing terminal areas or tapers for climbing lanes are discussed earlier in this section in conjunction with 2 lane highways. These principles are equally applicable to climbing lanes on multi-lane arterial highways, the prime factor being that of determining the location of the uphill terminus of the climbing lane at the point where a satisfactory speed is attained by trucks, preferably about 10 mph below the operating speed of the highway. Passing sight distance need not be considered.

Auxiliary lanes for freeways are covered in Section 10, FREEWAYS.

14-04 PEDESTRIAN GRADE SEPARATIONS (meet to Handicap Stds)

14-04.1 General

Pedestrian separations should be provided where pedestrian volume, traffic volume, and other conditions favor their use. Location and design of pedestrian subways or overcrossings require individual study. They may be warranted where there are heavy peak pedestrian movements, such as at central business districts, factories, schools, or athletic fields, in combination with moderate to heavy vehicular traffic, or where abnormal hazard or inconvenience to pedestrians would otherwise result. Pedestrian separations, usually overpasses, may be necessary at freeways or expressways where cross streets are terminated.

Pedestrian subways or overcrossing structures at arterial streets are not likely to be used unless it is obvious to the pedestrian that it is easier to use such a facility than to cross the traveled way. Generally, pedestrians are more reluctant to use undercrossings than overcrossings. The usual pedestrian undercrossing nuisance hazard is minimized by locating the underpass in line with the approach sidewalk and ramping the sidewalk gently to permit continuous vision through the underpass from the sidewalk. Lighting is necessary on longer undercrossings.

Structures that necessitate pedestrians walking up and down stairs are not likely to be used unless strict control measures are used. To make such facilities more inviting, the differences in walk levels should be minimized.

14-04.2 Design Considerations

Pedestrian ramps should be provided at all pedestrian separation structures. When warranted and possible, a stairway can be provided in addition to a ramp. Only when it would be infeasible for mobility-limited persons to reach the pedestrian separation structures due to unusual topographical or architectural obstacles, or when alternate safe and reasonably convenient means are provided to cross the highway should ramps be deleted.

Ramps shall meet the following design criteria:

1. Slope shall not exceed 8.33 percent, or 1 foot (0.31 m) rise in 12 feet (3.66 m);

2. Level platforms shall be provided at 30 foot (9.15 m) intervals, and wherever the ramp makes an abrupt turn;

3. Surface shall be non-slip type;

4. A straight, clear area of at least 6 feet (1.83 m) shall be provided at the bottom of the ramp;

5. Handrails shall be provided at least on one side of the ramp. Handrails shall be placed 32 inches (0.81 m) from the floor of the ramp and extend 1 foot (0.31 m) beyond the top and bottom of the ramp.

Stairways, when provided, should give the appearance of not being steep. A landing should be provided for every 5 or 6 foot (1.53 or 1.83 m) change in elevation.

The vertical clearance over roadways at pedestrian overpasses should be 2 feet (0.62 m) higher than the minimum vertical clearance for vehicular structures over the same roadway. The lighter pedestrian structures are less resistant to impact if struck by an overheight vehicle.

Pedestrian subways should have a vertical walking clearance of 8 feet (2.44 m) and in some cases 10 feet (3.05 m). Light fixtures should not encroach within this clear space. This clearance results in a minimum difference in elevation between the roadway and the walkway beneath of 10 to 12 feet (3.05 to 3.66 m) or more.

Walkways for pedestrian separations should have a minimum width of 8 feet (2.44 m) between handrails. Greater widths may be required where there are exceptionally high volumes of pedestrian traffic, such as in the downtown areas of larger cities and around sport stadiums.

At present, it is not possible to establish absolute warrants as to when or where barriers should be installed to discourage the throwing of objects from structures. The general need for economy in design and desire to preserve the clear lines of a structure unencumbered by screens must be carefully balanced against the absolute requirement that the highway traveler and overpass pedestrian be provided maximum safety.

Locations where screens definitely should be considered at the time the overpass is constructed are as follows:

1. On an overpass near a school, a playground, or elsewhere where it would be expected that the overpass would be frequently used by children unaccompanied by adults;

2. On all overpasses in large urban areas used exclusively by pedestrians and not easily kept under surveillance by police;

3. On overpasses where experience on nearby structures indicates a need for screens.

14-05 LIGHTING

Lighting affects the safety of a highway or street and the ease and comfort of operation thereon. Statistics indicate that the nighttime accident rate is higher than that during daylight hours which, to a large degree, may be attributed to impaired visibility. There is evidence that in urban and suburban areas, where there are concentrations of pedestrians and roadside intersectional interferences, fixed-source lighting tends to reduce accidents. Lighting of rural highways may be desirable, but the need for it is much less than on streets and highways in urban areas. The general consensus is that lighting of rural highways seldom is justified except on certain critical portions, such as interchanges, intersections, narrow or long bridges, tunnels, sharp curves, and areas where roadside interference is a factor. Most modern rural highways should be designed with an open cross section and horizontal and vertical alignment of a fairly high type. Accordingly, they offer an opportunity for near maximum use of vehicle headlights and resultant diminishing justification for fixed highway lighting throughout.

On freeways where there are no pedestrians, roadside entrances, or other intersections at grade, and where rights-of-way are relatively wide, the justification for lighting differs from that of noncontrolled streets and highways.

Whether or not rural intersections at grade should be lighted depends on the layout and the traffic volumes involved. Intersections that normally do not require channelization are frequently left unlighted. On the other hand, intersections with substantial channelization, particularly multiple-road layouts and those designed on a broad scale, often are lighted. It is especially desirable to illuminate large-scale channelized intersections. There is need to obtain a reduction in the speed of vehicles approaching some intersections. The indication of this need should be definite and visible at a distance from the intersection that is beyond the range of headlights. Illumination of the intersection with fixed-source lighting accomplishes this.

At interchanges it also is desirable, and sometimes necessary, to provide fixed-source lighting. Drivers should be able to see

not only the road ahead, but also the entire turning roadway area to discern properly the paths to be followed. They should also see all other vehicles that may influence their own behavior. Without lighting, there may be noticeable decrease in the usefulness of the interchange at night, when there would be more cars slowing down and moving with uncertainty than during daylight hours. Consideration should be given to making visible at night by roadway lighting (or reflectorizing devices) the parts of grade separation structures that may be hazardous--such as curbs, piers and abutments. The greater the volume of traffic, particularly turning traffic, the more important the fixed-source lighting at interchanges. Illumination should also be considered on those sections of major highways where turning movements to and from roadside developments are definite hazards.

Tunnels nearly always are lighted and so are bridges of substantial length in urban and suburban areas. It is questionable whether the cost of lighting long bridges in rural areas is justified or desirable.

Lighting standards (poles) should be placed outside the clear roadside recovery areas whenever practical. On freeways, poles that are exposed to traffic should be located at least 15 feet (4.57 m), and desirably 20 feet (6.10 m) or more from the edge of the traveled way and not less than 5 feet (1.53 m) from the edge of the paved shoulder. Where poles are located within the clear roadside recovery area regardless of distances from the traveled way, they should be designed to have a suitable impact attenuation feature; normally a breakaway design is used. Breakaway poles should not be used on streets in densely developed areas, particularly with sidewalks. Energy efficient lighting systems should always be considered in the design of fixed-source lighting to reduce energy costs.

The designer should refer to the following current references for highway lighting:

1. "American Standards Practice for Roadway Lighting", Illuminating Engineering Society.

2. "An Informational Guide for Roadway Lighting", AASHTO.

3. Lighting of Tunnels", Illuminating Engineering Society.

4. "IES Lighting Handbook", Illuminating Engineering Society.

14-06 DRAINAGE

Highway drainage facilities provide for carrying water across the right-of-way and for removal of storm water from the road itself. These facilities include bridges, culverts, channels, curbs, gutters and various types of drains. Hydraulic capacities and locations of such structures should be adequate to minimize damage to upstream property and to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements, and available funds.

Hydraulic requirements for stream crossings and flood plain encroachments frequently affect highway alignment and profile. The probable effects of a highway encroachment on flood hazards to other property and the risk of flood damage to the highway should be evaluated when a flood plain location is under consideration. Water surface elevations for floods of various return periods will influence decisions regarding the highway profile where an encroachment is necessary. Highway profiles at stream crossings will often be determined by high-water elevations of historical floods, headwater elevations above culverts, and backwater from bridges. All stream crossings and other highway encroachments on flood plains should be located and aligned to preserve the natural flood flow distribution and direction to the extent practicable. Stream stability and the stream environment are also important and complex considerations in highway location and design.

Surface channels are used to intercept and remove surface runoff from roadways, wherever practicable. They should have adequate capacity for the design runoff, and should be located and shaped to avoid creating a traffic hazard.

Drainage inlets should be so designed and located as to limit the spread of water on traffic lanes to tolerable widths. Because grates may become blocked by trash accumulation, curb openings or combination inlets with both grate and curb openings are advantageous for urban conditions. Grate inlets and depressions for curb-opening inlets should be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid riding over them and with consideration for bicycle traffic.

4-07 EROSION CONTROL AND LANDSCAPE DEVELOPMENT

Erosion prevention is one of the major factors in design, construction and maintenance of highways. It should be considered early

in the location and design stages. Temporary erosion control plans to be implemented during the construction phase should be prepared and included as a construction requirement. Some degree of erosion control can be incorporated into the geometric design, particularly in the cross section elements. Of course, the most direct application of erosion control occurs in drainage design and in the preparation of specifications for landscaping and slope planting.

Erosion and maintenance are minimized largely by the use of flat side slopes, rounded and blended with natural terrain; serrated cut slopes; drainage channels designed with due regard to width, depth, slopes, alignment and protective treatment; inlets located and spaced with erosion control in mind; prevention of erosion at culvert outlets; proper facilities for groundwater interception; retention basins; dikes, berms, and other protective devices; and protective ground covers and planting.

Landscape development should be in keeping with the character of the highway and its environment. Programs include the following general areas of improvement: (1) preservation of existing vegetation, (2) transplanting of existing vegetation where feasible, (3) planting of new vegetation, (4) selective clearing and thinning, and (5) regeneration of natural plant species and material.

The objectives in planting or the retention and preservation of natural growth on roadsides are closely related. In essence, they are to provide (1) vegetation that will be an aid to aesthetics and safety, (2) vegetation that will aid in lowering construction and maintenance costs, and (3) vegetation that creates interest, usefulness, and beauty for the pleasure and satisfaction of the traveling public.

14-08 UTILITIES

14-08.1 General

All highway and street improvements, whether upgraded within the existing right-of-way or entirely on new right-of-way, generally entail adjustment of utility facilities. Although utilities generally have little effect on the geometric design of the highway or street, full consideration should be given to measures, reflecting sound engineering principles and economic factors, necessary to preserve and protect the integrity and visual quality of the highway or street, its maintenance efficiency, and the safety of traffic.

Utilities that are to cross or otherwise occupy the right-of-way of State Highways and Federal Aid Secondary County Highways should conform to "RULES AND REGULATIONS RELATING TO THE ACCOMMODATION AND INSTALLATION OF UTILITIES ON STATE HIGHWAYS AND FEDERAL AID SECONDARY COUNTY HIGHWAYS" of the Department of Transportation of the State of Hawaii, Highways Division.

To the extent feasible and practicable, utility line crossings of the highway should cross on a line generally normal to the highway alignment. Those utility crossings that are more likely to require future servicing should be encased or installed in tunnels to permit servicing without disrupting the traffic flow.

The horizontal and vertical location of utility lines within the highway right-of-way limits should conform to the clear roadside policies applicable for the system, type of highway or street, and specific conditions for the particular section involved. Safety of the traveling public should be a prime consideration in the location and design of utility facilities on highway and street rights-of-way.

14-09 MAILBOXES

14-09.1 General

Mailboxes should be placed for maximum convenience to the patron, consistent with safety considerations for highway traffic, the carrier, and the patron. Consideration should be given to (1) minimum walking distance within the roadway for the patron, (2) available stopping sight distance in advance of the mailbox site (especially troublesome on older roads), and (3) possible restriction to corner sight distance at intersections and driveway entrances. New installations should, where feasible, be located on the far right side of an intersection with a public road or private driveway entrance.

Boxes should be placed only on the right-hand side of the highway in the direction of travel of the carrier, except on one-way streets where they may be placed on the left-hand side. It is undesirable to require pedestrian travel along the shoulder. However, this may be the preferred solution for distances up to 200 feet when compared to alternatives, such as constructing a turnout in a deep cut, placing a mailbox just beyond a sharp crest vertical curve (poor sight distance), or constructing two or more closely spaced turnouts.

The placing of mailboxes along high-speed, high-volume highways should be avoided if other practical locations are available. Mailboxes should not be located where access is from the lanes of an expressway or where access, stopping, or parking is otherwise prohibited by law or regulation. Where there are frontage roads, the abutting property owners may be served by boxes located along the frontage roads. No mailbox should be at a location that would require a patron to cross the lanes of an expressway to deposit or retrieve mail. Where the U.S. Postal Service deems that service is not warranted on both frontage roads, or where there is a frontage road only on one side, patrons not served directly should be accommodated by mailboxes at a suitable and safe location in the vicinity of the crossroad nearest the patron's property.

Placing a mail stop near an intersection will have an effect on the operation of the intersection. The nature and magnitude of this effect depend on traffic speeds and volumes on each of the intersecting roadways, the number of mailboxes at the stop, extent of traffic control, how the stop is located relative to the traffic control, and the distance the stop is from the intersection.

At intersections where one roadway is given the right of way and the other is stop controlled, a vehicle at a mail stop on a through roadway approach to the intersection may restrict the

view between a vehicle entering the intersection from the right and the through traffic behind the mail stop. A mail stop on the through road on the far side of the crossroad increases the chance the crossroad driver will pull into the path of the vehicle on the through road that is headed for the mail stop. A mail stop in advance of a stop sign creates the potential for a vehicle at the mail stop blocking the view of the stop sign. The least troublesome location for a mail stop at these intersections is adjacent to a crossroad lane leaving the intersection. Nevertheless, there is still a chance that a driver reentering traffic from the mail stop will not see or be seen from a vehicle turning onto the crossroad. Figure 14-E shows possible locations of mail stops at a typical rural intersection. Using the mail stop location dimensions in the figure will minimize the effect a stop will have on an intersection's operation and minimize the hazard to persons using the mail stop.

Mailbox heights are usually set to accommodate the mail carrier. Typically, the bottom of the mailbox is located 42 inches to 48 inches above the mail stop surface.

Mailboxes should be located so that a vehicle stopped at a mailbox is clear of the adjacent traveled way. An exception to this principle may be reasonable on low-volume, low-speed streets and roads. But basically, a vehicle stopped at a mailbox should be clear of the traveled ways and the higher the traffic volume or speed, the greater the clearance should be.

Most vehicles stopped at a mailbox will be clear of the traveled way when the mailbox is placed outside an 8-foot wide usable shoulder or turnout. This position is recommended for most rural highways. For high-volume, high-speed highways, it is recommended that 10-feet-wide turnouts should be provided where the shoulder is not at least 10-feet-wide or 12-feet-wide for some conditions. Where conditions justify, 12-feet wide turnouts should be provided. However, it may not be reasonable to require even an 8-foot shoulder or turnout on very low-volume, low-speed roads or streets. To provide space outside the all-weather surface for opening the mailbox door, it is recommended that the roadside face of a mailbox be set 8 to 12 inches outside the all-weather surface of the shoulder or turnout. Suggested guidelines for the placement of mailboxes are shown in Table 14-2. These are based on experience and engineering judgment.

When a mailbox is installed in the vicinity of an existing guardrail, it should, wherever practical, be placed behind the guardrail.

Through Road Speed M.P.H.	D_c (feet)		
	$\frac{V_c}{1.5n - .5} \leq 50$	$50 < \frac{V_c}{1.5n - .5} \leq 400$	$\frac{V_c}{1.5n - .5} > 400$
40	70	100	100
⩾55	150	150	200

Through Road Speed M.P.H.	D_a (feet)	
	$nV_cV_m \leq 4000$	$nV_cV_m > 4000$
40	70	200
⩾55	70	300

D (feet)	
Preferred	Minimum
150	90

D_c (feet)	
Preferred	Minimum
90	60

V_c = Average Daily Traffic on Cross Road Vehicles Per Day

V_m = Average Daily Traffic on Through Road Vehicles Per Day

n = Number of Mailboxes at Mail Stop

SUGGESTED MINIMUM CLEARANCE DISTANCES NEAREST MAILBOX IN MAIL STOP AT INTERSECTIONS

FIGURE 14-E

TABLE 14-2

SUGGESTED GUIDELINES FOR LATERAL PLACEMENT OF MAILBOXES

Highway Type and Traffic Conditions	Width of All-Weather Surface of Turnout or Available Shoulder at Mailbox—Feet		Distance Roadside Face of Mailbox is to be Offset Behind Edge of Turnout or Usable Shoulder—Inches	
	Preferred	Minimum	Preferred	Minimum
Rural highway ADT over 10,000 vpd	> 12	12	8 to 12	0
Rural highway ADT = 1,500 to 10,000 vpd	12	10		
Rural highway ADT = 100 to 1,500 vpd	10	8		
Rural road ADT under 100 vpd	8	6		
Rural road ADT under 50 vpd Speed = 40 mph or less	6	2		
Residential street without curb or all-weather shoulder	6	0		8*
Curbed residential street	Not applicable		8 to 12 Behind Traffic Face of Curb	6 Behind Traffic Face of Curb

ADT = Average Daily Traffic vpd = Vehicles Per Day

*If a turnout is provided, this may be reduced to zero.

14-09.2 Design Considerations

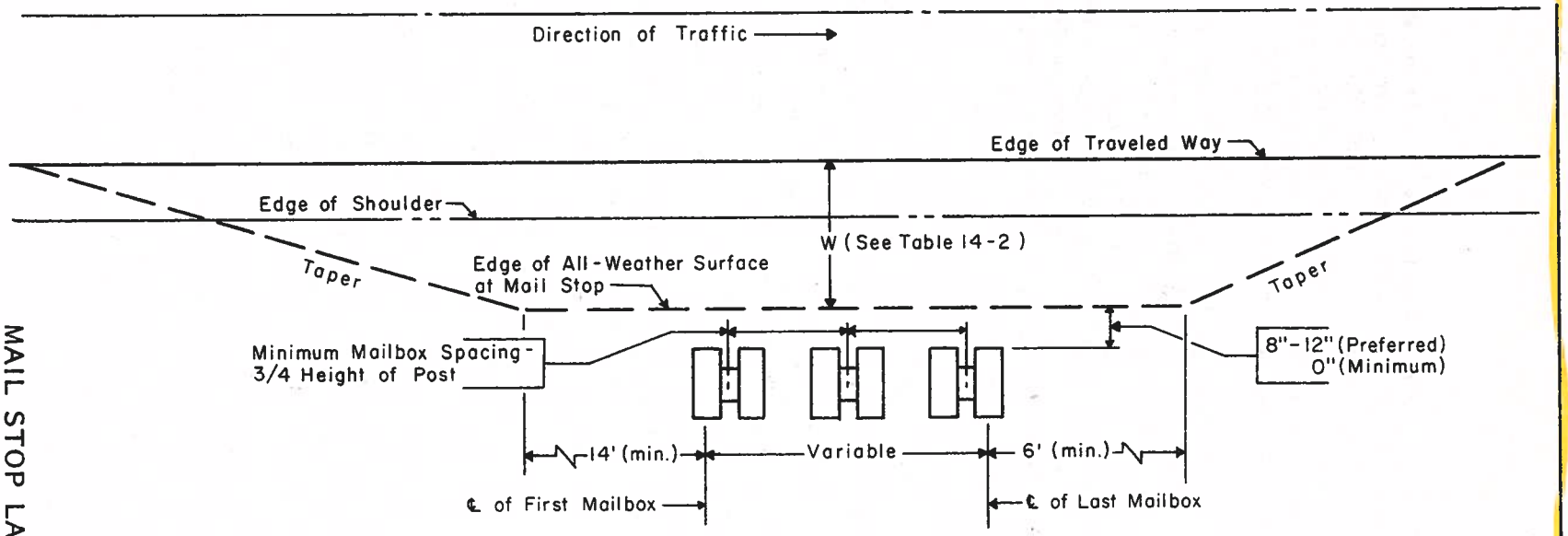
Shoulder or turnout widths suitable to safely accommodate vehicles stopped at mailboxes are discussed in the "Mail Stop and Mailbox Location" section, and are shown in Table 14-2.

The surface over which a vehicle is maneuvered to and from a mailbox must be sufficiently stable to support passenger cars stopping regularly during all-weather conditions. Where the available shoulder surface strength or width are not sufficient for the purpose, they should be modified to provide a suitable all-weather mail stop turnout. In most instances, adequate surface stabilization can be obtained by the addition of select materials to the in-place soils. A mail stop for grouped mailboxes may require greater stabilization or possibly a surface treatment course to accommodate multiple patron use. Special measures may also be needed where highway traffic conditions encourage hard braking or high acceleration of vehicles in a mail stop.

Slowing a vehicle in traffic, as drivers are usually required to do when entering a mail stop, increases the risk of an accident. The ideal way to minimize this risk is to provide a speed change lane. A wide, surface-treated shoulder can be used for this purpose. Unfortunately, at most mail stop locations, suitable shoulders are not available and it would be far too expensive to provide shoulders or turnouts that would allow all speed change to be accomplished outside the traveled ways. Figure 14-F shows mail stop layouts considered appropriate for different traffic conditions. Figure 14-F shows the minimum space needed for maneuvering to a parallel position out of traffic and for returning to traffic. The typical driver would probably slow to about 10 miles per hour before starting into this turnout, making it unsuitable for high-speed highways where driver expectancy does not include such slow moving traffic. Use of this layout should be limited to low speed as well as local and collector roads carrying under 400 vehicles per day.

Before entering an 8-foot-wide turnout similar to that shown in Figure 14-F, a driver would probably slow to about 30 miles per hour and would slow considerably more, possibly to as slow as 10 miles per hour, before clearing the traveled way. While this is not an ideal exit maneuver, for the few stops generated by a single mailbox, it probably would not create an unacceptable hazard on most rural highways.

Increasing the width of the turnout to 12 feet and maintaining the taper rate suggested in Figure 14-F might induce a driver using the turnout to enter it at about 45 miles per hour and to clear the traveled way at about 25 miles per hour. While this is still not ideal, it should be quite acceptable for all but very few sites.



TAPER

Traffic	Approach Taper	Trailing Taper
Local and Collector Road < 400 vpd	4:1	2½:1
High Speed	20:1	12:1

FIGURE 14-F MAIL STOP LAYOUT

These very few sites might be found on highways operating at high speeds and carrying over 3,000 to 4,000 vehicles per day with a high percentage of vehicles on long trips. For these conditions, consideration should be given to providing shoulders or turnouts at unavoidable mail stops that will provide for greater speed change opportunity outside the traffic stream.

The tapers shown in Figure 14-F represents theoretical layouts. It may be more practical to square the ends of the turnout or to provide a stepped layout with the full width of the shoulder strengthened, if required, for the length of the turnout and the shoulder widened to the full width of the turnout for the entire length of the portion of the turnout outside the shoulder width. It may also be simpler to construct a continuous turnout-width shoulder rather than individual turnouts where mail stops are closely spaced.

14-09.3 Mailbox Support and Attachment Design

All exposed mailboxes should be firmly attached to supports that yield or break away safely if struck by a vehicle. There are presently no established performance criteria for mailbox supports. Nevertheless, there are some criteria that can be inferred from observing mailbox installations and crashes involving them. These criteria can be summarized as follows:

- Mailbox supports should, with a minor qualification, be no more substantial than required to resist service loads and reasonably minimize vandalism. Nominal 4-inch x 4-inch or 4½-inch diameter wood posts or 1½-inch to 2-inch diameter standard steel or aluminum pipe posts, embedded no more than 24 inches into the ground, are the maximum strength supports that should be considered. Lower strength supports, such as lightweight flanged channel steel posts, have given satisfactory service in most environments. A metal post should not be fitted with an anchor plate. However, an anti-twist device that extends no more than 10 inches below the ground surface is acceptable. The qualification to this criterion minimizing post strength is that for supports, such as those made of wood that break rather than bend under impact, the support needs sufficient strength to accelerate the box to a speed approaching that of the impacting vehicle so that the chances of the box penetrating the vehicle's windshield are minimized. Test results indicate that the 4-inch x 4-inch and 4½-inch diameter dimensions for wood supports should be both minimum and maximum post dimensions.

- Mailbox to post attachments should prevent mailboxes from separating from their supports under vehicle impacts. The lighter the mailbox, the easier it will be to meet this criterion or, conversely, given sufficient post attachment strength, the less sensitive the safety of an installation will be to the weight of the mailbox. (See reference.)

- Multiple mailbox installations must meet the same criteria as do single mailbox installations. This requirement precludes the use of a heavy horizontal support member (See reference.) The use of a series of such installations or of individually supported boxes is acceptable. However, vehicle rollover occurred when a row of eight mailboxes individually supported with 2-pound per foot steel channel post supports was crash tested with a small car, impacting off center at high speed. Film from this test and results from other tests suggest that the reason for this performance was a ramping caused by the closely spaced mailboxes piling up. To avoid this problem, it is recommended that mailbox supports be separated a distance at least equal to three-fourths of their heights above ground.

  The Neighborhood Delivery and Collection Box Units (NDCBU) are a specialized type of multiple mailbox installations that should be located at sites that provide adequate safety to errant motorists and safe access by postal patrons and carriers. The NDCBU is a cluster of 8 to 16 locked boxes mounted on a pedestal or within a framework, the combination of which generally weighs between 100 and 200 pounds. While NDCBUs usually serve a limited number of single family residences in urban areas, their use has been observed in rural areas. A crash test of one of these units at 60 miles per hour showed that it failed to meet safety requirements. Therefore, NDCBUs should be located outside the clear area needed for the safe recovery of errant vehicles. Postmasters and their designers responsible for the location of NDCBUs should be instructed to contact local government authorities, including the appropriate highway officials (State and county) prior to installation to ensure safe location of NDCBUs.

The Designers should refer to "A Guide for Erecting Mailboxes on Highways", AASHTO, May 24, 1984 for reference.

14-10 BRIDGES TO REMAIN IN PLACE

Because highway geometric and roadway improvements encourage higher speeds and attract larger vehicles, existing structures also should be improved correspondingly, but because of their high replacement cost, reasonably adequate bridges and culverts that meet tolerable criteria may be retained. Some of the nontechnical factors that should be considered are the esthetic value and the historical significance attached to famous structures, covered bridges, and stone arches.

Where an existing local or collector road is to be reconstructed, an existing bridge that fits the proposed alignment and profile may remain in place when its structural capacity in terms of design loading and clear roadway width are at least equal to the values given in Tables 14-3 and 14-4 for the applicable traffic volume on local or collector road or street.

The values in Table 14-3 and 14-4 do not apply to structures with total length greater than 100 ft. These structures should be analyzed individually, taking into consideration the clear width provided, traffic volumes, remaining life of the structure, pedestrian volumes, snow storage, design speed, accident record, and other pertinent factors.

Table 14-3

Minimum structural capacities and minimum roadway widths for bridges to remain in place for local road or street

Traffic	Design Loading Structural Capacity Minimum	Roadway Clear Width (ft)[b] Minimum[c]
[a]ADT-0-50	H-10	20[d]
[a]ADT 51-250	H-15	20
[a]ADT 250+	H-15	22
DHV 100-200	H-15	22
DHV 200-400	H-15	24
DHV over 400	H-15	28

[a]Current ADT.

[b]Clear width between curbs or rails, whichever is the lesser.

[c]Minimum clear widths that are 2 ft narrower may be used on roads with few trucks. In no case shall the minimum clear width be less than the approach traveled way width.

[d]For one-lane bridges use 18 ft.

Table 14-4

Minimum structural capacities and minimum roadway widths for bridges to remain in place for collector road or street

Current Traffic ADT	Traffic DHV	Design Loading Structural Capacity	Roadway Clear Width (ft)[a]
Under 400	—	HS-15	22
400 and Over	—	HS-15	22
—	100-200	HS-15	22
—	200-400	HS-15	24
—	Over 400	HS-15	28

[a]Clear width between curbs or rails, whichever is the lesser, is considered to be at least the same as the approach traveled way width.

SECTION 15.

MAINTENANCE OF TRAFFIC THROUGH CONSTRUCTION AREAS

15-01 GENERAL

Problems of traffic control occur when traffic must be moved through or around road or street construction, maintenance operations and utility work.

In accordance with the Hawaii Administrative Rules Title 19, Subtitle 5, Chapter 129 - Use of Traffic Control Devices at Work Sites on or Adjacent to Public Streets and Highways, the use of traffic control devices is mandated whenever the work is being performed on, adjacent to, above, below or near any street or highway in the State.

Maintenance of traffic during construction must be carefully planned and executed. Although it is often better to provide detours, frequently it will be necessary to maintain the flow of traffic through the construction area. Sometimes it will also be necessary to encroach upon the through-traffic lanes or shift lanes entirely in order that the construction can be undertaken. When this is necessary, it is essential that in the development of construction plans, designs for traffic control should produce as minimal an effect on traffic operations as is possible by minimizing the frequency of time of encroachment on normal traffic flow. The development of traffic control plans is an essential part of the overall project design and may affect the design of the facility itself. The traffic control plan depends on the nature and scope of the improvement, volume of traffic, highway or street pattern, and capacities of available highways or streets. A well thought-out and carefully developed plan for the movement of traffic through a work zone will contribute significantly to the safe and expeditious flow of traffic as well as the safety of the construction forces. It is desirable that these plans have some built-in flexibility to accommodate unforeseen changes in work schedule, delays, or traffic patterns.

15-02 TRAFFIC CONTROL PLAN

15-02.1 Lane Closure Plan

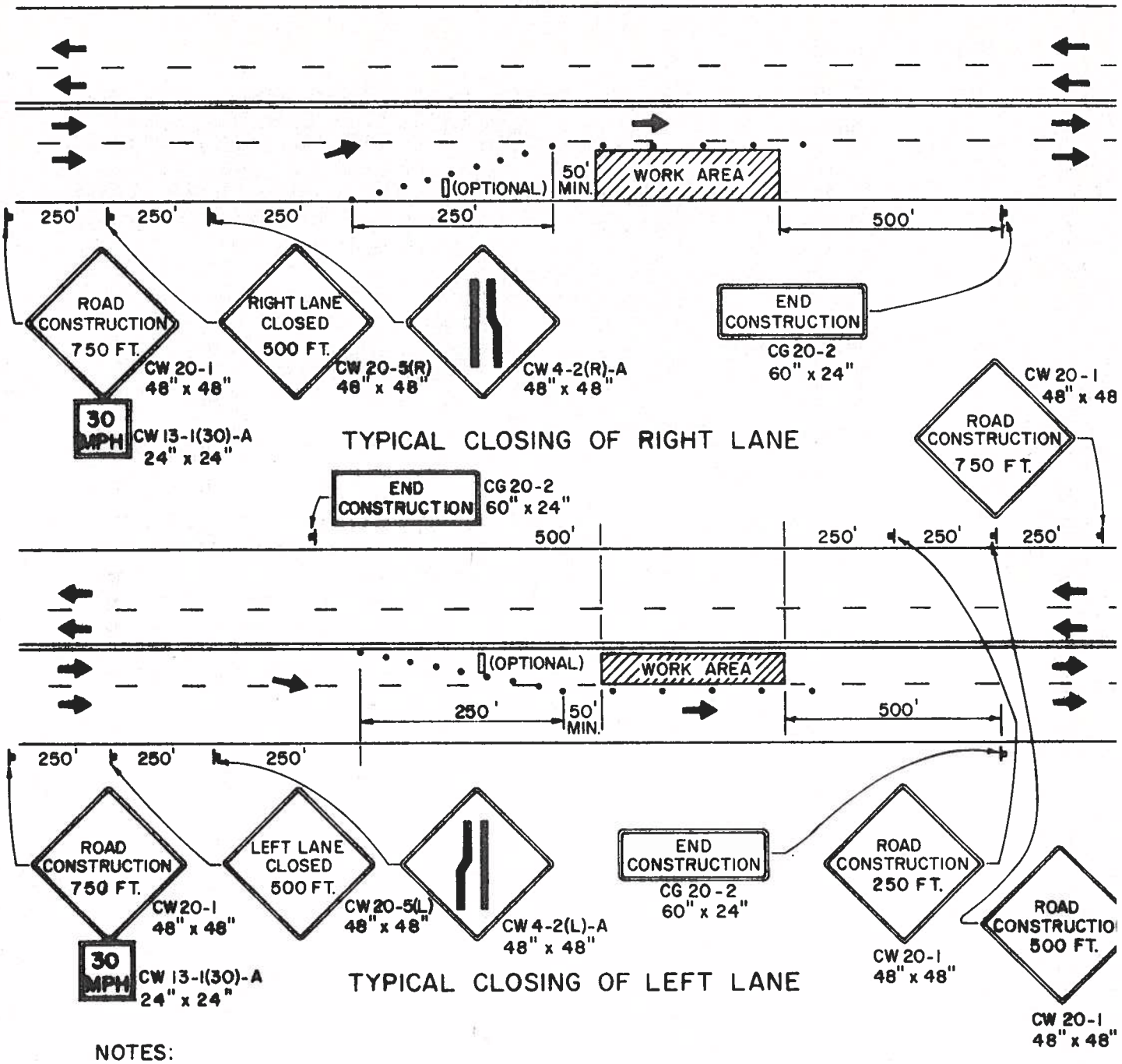
The goal of any traffic control plan should be to safely route traffic at a controlled speed through or around construction

areas with geometrics and traffic control devices as nearly as possible comparable to those for normal operating situations, while providing room for the contractor to work effectively. Too frequently, this goal is not achieved, the result being that accidents occur because unsuspecting motorists are surprised and trapped by inadequate geometrics or traffic controls. The solution is largely one of providing the driver with adequate advance warning and sufficient follow-up information. Standards for the use and application of signs and other traffic control devices when highway construction and maintenance occur are set forth in Part VI of the Manual on Uniform Traffic Control Devices (MUTCD). It cannot be emphasized to strongly that the principles must be applied and a plan developed for the particular type of work performed. There is no standard application in the MUTCD covering every kind of construction activity. In addition, reference is made to FHWA publication The Work Zone Traffic Control--Standards and Guidelines Handbook for use by the designer.

Adequate advance notice must be given to drivers to prepare them for the changed operating conditions in construction areas. The size of and the distance that such signing should be located in advance of construction operations varies with the speed on the affected facility. Various configurations are illustrated in the MUTCD and should be used in developing traffic control plans. Figure 15-A shows a sample lane closure plan for a 4-lane highway based upon a speed limit of 30 mph.

The stopping of traffic by a flagman or other means should be avoided wherever possible. Stopping vehicles is usually difficult and hazardous and it presents a constant opportunity for rear-end collisions. Designs that provide for constant movement around an obstruction in the roadway, even if it is slow, are more acceptable and are less irritating to drivers than requiring them to stop.

The roadway near falsework openings should be well lighted and delineated. Channelization of traffic should be accomplished by the use of signing on yielding supports, pavement markings, and barricades.



NOTES:

1. DELINEATORS: CONES OR DELINEATORS SHOULD BE PLACED AT 30' SPACING ON TAPER AND 30' SPACING ON TANGENT.
2. DIVIDED ROADWAY: SUPPLEMENTAL ADVANCE WARNING SIGNS SHALL BE POSTED IN THE MEDIAN, (INCLUDING PAINTED MEDIAN). SIGNS ARE NOT REQUIRED ON OPPOSITE APPROACH.

LEGEND:

- CONE OR DELINEATOI
- DIRECTION OF TRAF
- FLASHING ARROW SI
- SIGN

SAMPLE TRAFFIC CONTROL PLAN

FIGURE 15-A
r 5/15/86

15-02.2 Detour Plan

Traffic operation controls for a construction detour are speed, capacity, travel distance, and safety. The speed for a detour may be less than that on the facility being improved, but should be high enough so as not to affect the capacity. When an existing highway or street is used as a detour, higher volumes result and it may be necessary to increase the capacity of such a route in advance. This capacity increase generally is done by eliminating troublesome turning movements, rerouting transit vehicles and trucks, adopting and enforcing a loading-unloading ban during peak hours, eliminating or adjusting certain transit stops, coordinating signal timing, and sometimes physically widening the traveled way. A detour plan is tested by comparing the traffic volumes expected to use the rearranged plan to the calculated capacity of the detour system.

15-03 IMPLEMENTATION

The implementation of the traffic control plan at the job site is usually the responsibility of the contractor. The contractor should also revise the traffic control plan as necessary to accommodate existing field conditions, especially at intersections or at driveway entrances. In addition, the contractor should maintain the lane closure or detour route to assure that all signs, traffic cones, delineators and other traffic control devices are in their proper locations and visible to traffic.

SECTION 16.

BICYCLE FACILITIES

(basically followed Northwestern Criteria)

16-01 GENERAL

The bicycle is another mode of transportation, and, like automobile transportation, it is necessary to follow a transportation planning process in order to warrant, coordinate, design and produce a facility that will best serve the public.

A Statewide bike plan, entitled BIKE PLAN HAWAII, A STATE OF HAWAII MASTER PLAN, was completed in 1977. The report includes background data and criteria for the establishment of some bikeways in the State.

FHWA's A BIKEWAY CRITERIA DIGEST: THE ABCD'S OF BIKEWAY, FHWA-TS-77-201 includes procedures and guidelines for planning and designing bicycle facilities.

AASHTO's GUIDE FOR BICYCLE ROUTES, 1974.

County of Hawaii, BIKEWAY PLAN, 1977.

CALTRANS' PLANNING AND DESIGN CRITERIA FOR BIKEWAYS IN CALIFORNIA, June 30, 1978.

16-02 DEFINITIONS AND TERMS

16-02.1 Bikeway Classification

Bikeways are classified into 3 classes as follows:

Class I: A completely separated right-of-way designated for the exclusive use of bicycles. Crossflows by pedestrians and motorists are minimized.

Class II: A restricted right-of-way designated for the exclusive or semi-exclusive use of bicycles. Through travel by motor vehicles or pedestrians is not allowed; however, vehicle parking may be allowed. Crossflows by motorists (for example, to gain access to driveways or parking facilities) are allowed; pedestrian

crossflows (for example, to gain access to parked vehicles or bus stops or associated land use) are allowed.

Class III: A shared right-of-way designated as such by signs placed on vertical posts or stenciled on the pavement. Any bikeway that shares its through-traffic right-of-way with either or both moving (not parking) motor vehicles and pedestrians is considered a Class III bikeway.

16-02.2 Definitions

1. Bikeway

Any facility that explicitly provides for bicycle travel. It can be any road, street, path, or way that, in some manner, is specifically designated as being open to bicycle travel. The term is applicable whether or not the facility is reserved for the exclusive use of bicycles or is shared with some other type of vehicle or with pedestrians.

2. Bike Path

A completely separate right-of-way for the exclusive or semi-exclusive use of bicycles. Where such a facility forms part of a roadway, it is separated from the roadway by a significant amount of open space and/or a major physical barrier (such as trees or a considerable change in ground elevation). In general, bike paths function as Class I facilities.

3. Bike Lane

A portion of the roadway that has been designated for the preferential or exclusive use of bicycles. Through travel by motor vehicles is not allowed. While there are many variations on the general concept, there are only two different types of bike lanes--protected and unprotected. A protected bike lane is separated from adjacent vehicular traffic by a positive physical barrier such as concrete bumper stops, placement of the lane between parked cars and the curb, a median buffer landscape area, or other similar means. An unprotected bike lane, on the other hand, is delineated by a painted white line or a row of pavement markers. Most bike lanes serve as Class II bikeways.

4. Bike Route

A street or system of streets that meets certain minimum standards and is officially designated and marked as a "bike route". Bicycles share the roadways with either moving motor vehicles, pedestrians, or both. In general, bike routes function as Class III bikeways.

16-03 PHYSICAL CHARACTERISTICS OF BICYCLES

The dimensions of bicycles and riders are important towards establishing guidelines for bikeway design.

16-03.1 Bicycle and Rider Dimensions

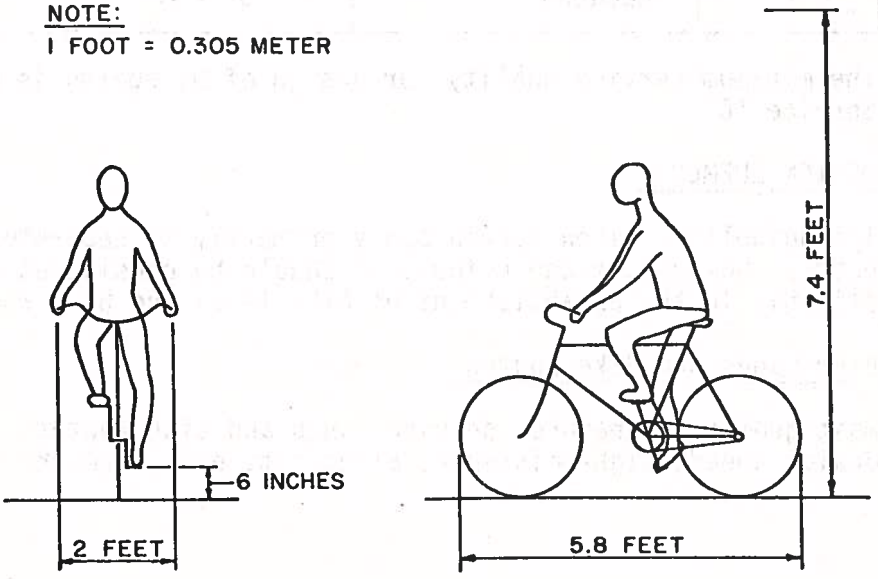
Table 16-1 shows the critical dimensions of bicycle and rider.

TABLE 16-1

BICYCLE AND RIDER DIMENSIONS

TYPICAL BICYCLE AND RIDER DIMENSIONS	
CHARACTERISTICS	DIMENSION (FEET)
WIDTH	2.00 (HANDLEBAR WIDTH)
LENGTH	5.75
HEIGHT	7.40 MINIMUM
VERTICAL PEDAL CLEARANCE	0.50

NOTE:
1 FOOT = 0.305 METER



16-03.2 Maneuvering Area and Lateral Clearance

In addition to physical space taken up by the bicycle and rider, the bicyclist requires a certain amount of clearance and space above and on each side to be safe and comfortable while operating the bicycle.

Figure 16-A shows clearance requirements for maneuvering and at lateral obstructions. Vertical clearance should be at least 8.2 feet (2.50 m), and desirably 10 feet (3.05 m).

16-04 LEVEL OF SERVICE AND LANE WIDTHS

The following Level of Service Criteria shown in Table 16-2 has been established for bicycle facilities and is used in establishing bicycle facility width requirements:

Table 16-2

Level of Service/Lane Width Criteria

Level	Type of Flow	Speed (mph)	Lane Width (Inches)
A	Free, Low Volume	12	50
B	Stable	10.5-11	48
C	Stable	9.5-10.5	42
D	Depressed	8 - 9.5	36
E	Reduced	6 - 8	30

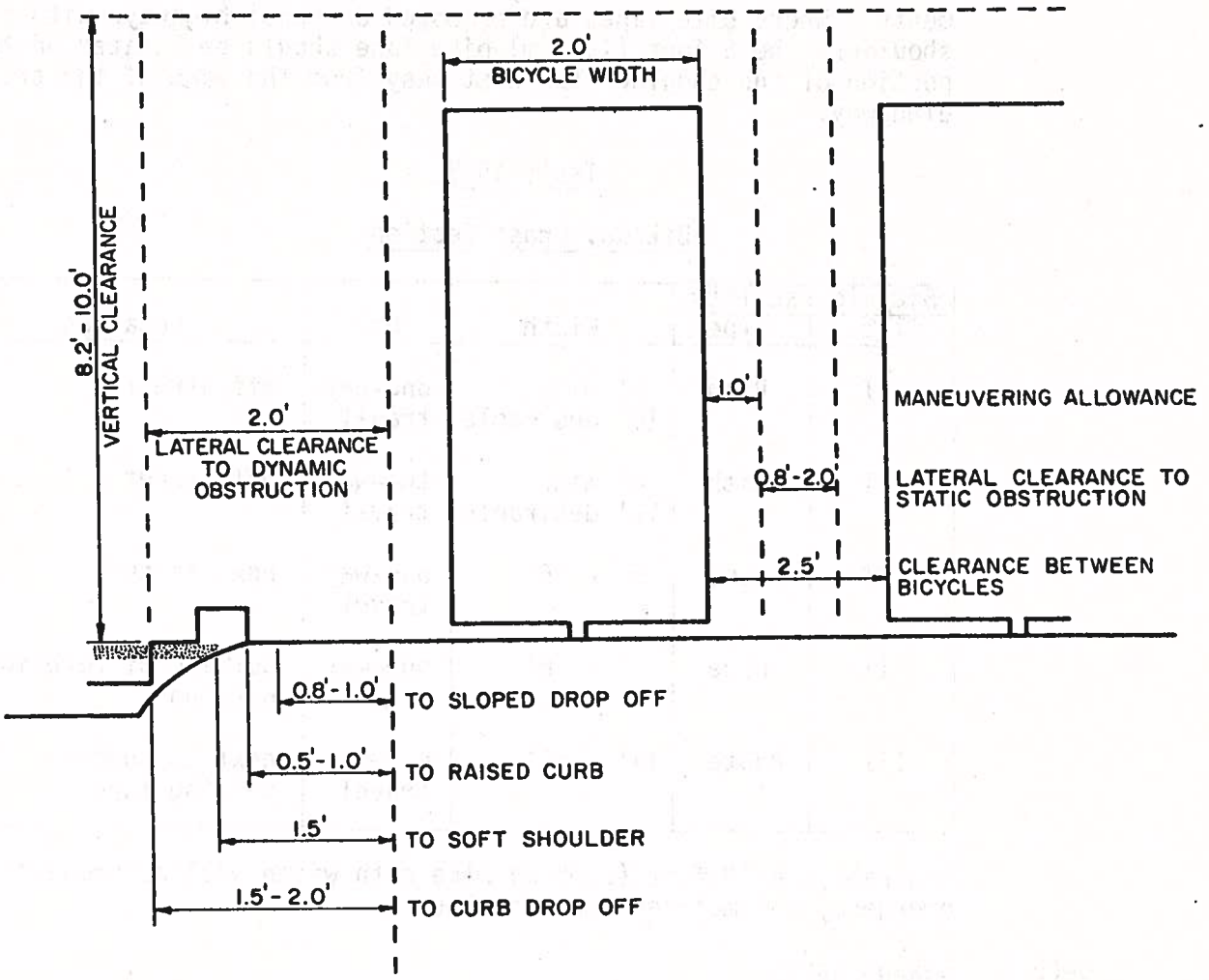
The minimum service quality for design of bikepaths is Level of Service 'C'.

16-05 DESIGN ELEMENTS

The guidelines given herein apply primarily to separated bicycle paths. However, these guidelines should be considered where applicable in the establishment of bike lanes and bike routes.

16-05.1 Bike Lanes and Bike Routes

Most geometric features on bike lanes and bike routes, including design speed, sight distance, alignment, etc., are the same as



NOTE:
1 FOOT = 0.305 METER

MINIMUM AND DESIRABLE
CLEARANCE REQUIREMENTS

FIGURE 16-A

the highway of which these facilities are a part, and thus are usually adequate.

16-05.2 Cross Section

Table 16-3 presents a summary of bikeway cross sectional requirements. Where bike lanes are proposed on rural highways with paved shoulder, the 5 foot (1.53 m) bike lane should be located on the portion of the shoulder furthest away from the edge of the traveled way.

Table 16-3

Bikeway Cross Sections

Bicycle Facility		Width	Use	Location
Class	Type			
I	Path	6' min. 10' desirable	one-way travel	off-street
I	Path	8' min. 12' desirable	two-way travel	off-street
II	Lane	5' - 6'	one-way travel	next to curb
II	Lane	5' - 6'	one-way travel	outside of curbside parking
III	Route	14' - 17'	one-way travel	next to curb or parking lane

Desirably, a 10 foot (3.05 m) bike path width will accommodate emergency and maintenance vehicles.

16-05.3 Design Speed

The design speed of a bicycle facility is dependent upon the type of use it is intended for and the type of bicycle users that will utilize it.

The avid bicyclist or the commuter bicyclist can achieve speeds of 18 to 25 mph, whereas the general bicycling public may only achieve speeds of 10 to 15 mph.

The following is the recommended guideline for bicycle facility design speeds:

Conditions	Desirable Design Speed (mph)
Urban Area (Bike Lane or Bike Route)	15 to 20
Open Country, Level or Undulating: Bike Paths, Urban or Rural	20
Long Downgrades	30
One-Way Climbing Grade +3 Percent or Steeper	15

16-05.4 Stopping Sight Distance

The stopping sight distance is the length of route ahead visible to the bicyclist including intersections and driveways.

Sufficient sight distances must be provided for a bicyclist to stop or take evasive action. An adequate lateral view must also be available at intersections and at driveway connections. Passing sight distance is not considered in bikeway design due to relatively low speeds of bicyclists. Table 16-4 presents the design stopping sight distances for bicycles.

Table 16-4

Design Stopping Sight Distances for Bicycles

Design Speed	Stopping Distances for Downhill Gradients Of:			
	0% feet(m)	5% feet(m)	10% feet(m)	15% feet(m)
10 MPH	50(15.3)	50(15.3)	60(18.6)	70(21.4)
15 MPH	85(25.0	90(27.5)	100(30.5)	130(39.6)
20 MPH	130(39.6)	140(92.6)	160(48.7)	200(61.0)
25 MPH	175(53.4)	200(61.0)	230(70.0)	300(91.9)
30 MPH	230(70.0)	260(79.1)	310(94.4)	400(122)

Stopping sight distances for crest vertical curves are given in Figure 16-B, while Figure 16-C gives formulation for horizontal curves.

16-05.6 Grades

A level bikeway is preferred by nearly all bicyclists; however, because existing terrain, highway alignment or available right-of-way may not be level, most bikeways will have grades. The amount of usage the bikeway receives can depend upon the grades on the bikeway. Therefore, the grades should be kept at a minimum. Bikeway grades should not exceed 10 percent. Figure 16-D shows desirable and acceptable standards for bikeway grades.

16-05.7 Horizontal Curves

The minimum radius of curvature for horizontal curves must be consistent with design speed, safety, grade profile, type of facility, topography and construction cost. Table 16-5 shows bikeway design curvature for various design speeds.

Table 16-5

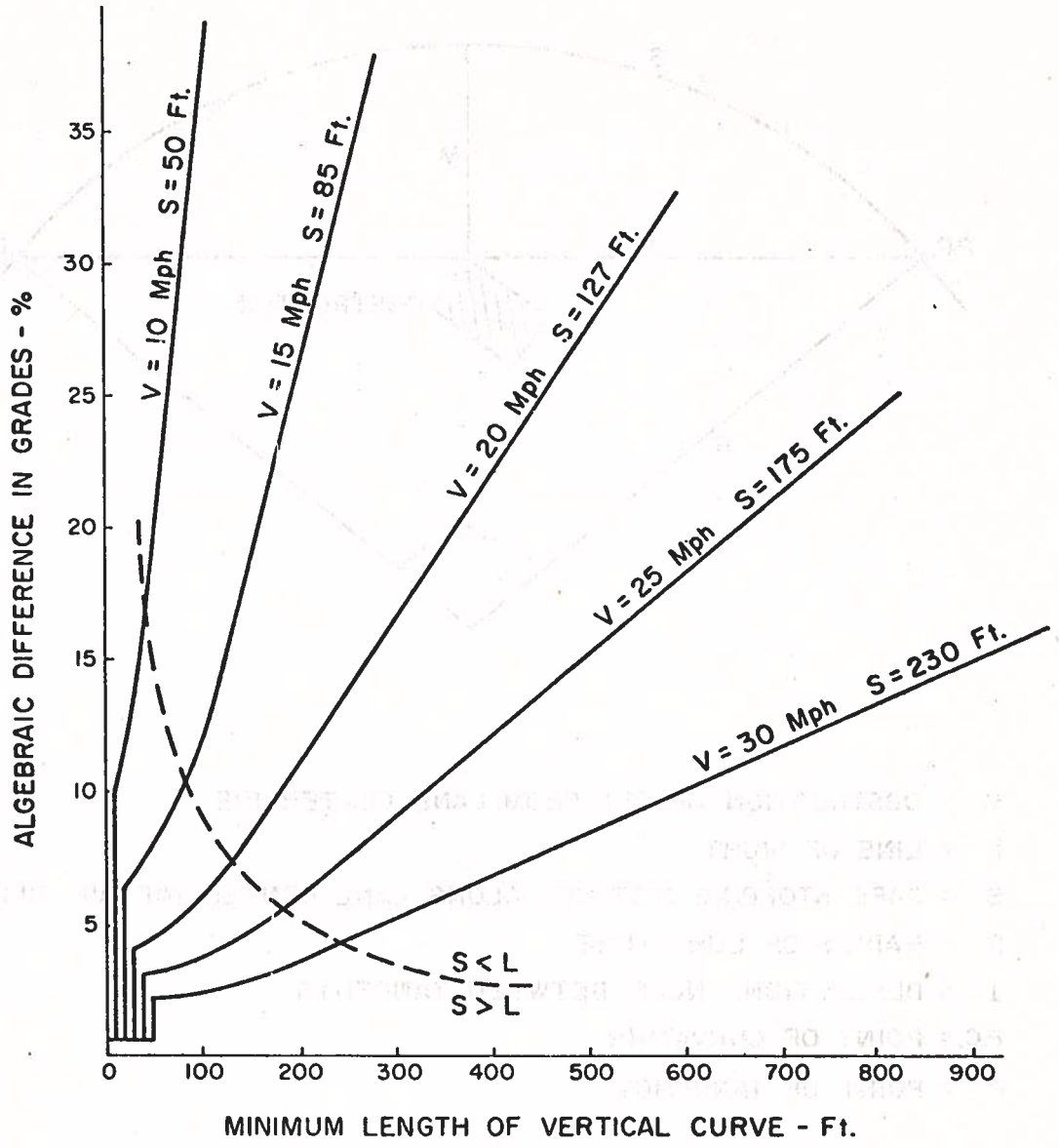
Bikeway Minimum Curve Radii

Design Speed mph	Minimum Radius* Feet (Meters)
10	15 (4.6)
15	35 (10.7)
20	65 (19.8)
25	100 (30.5)
30	140 (42.7)

* Assumes 2% Superelevation

Bicycle facilities utilizing roadways usually have curvature dictated by the street system. Usually, the street curve radius will be adequate for bicycles.

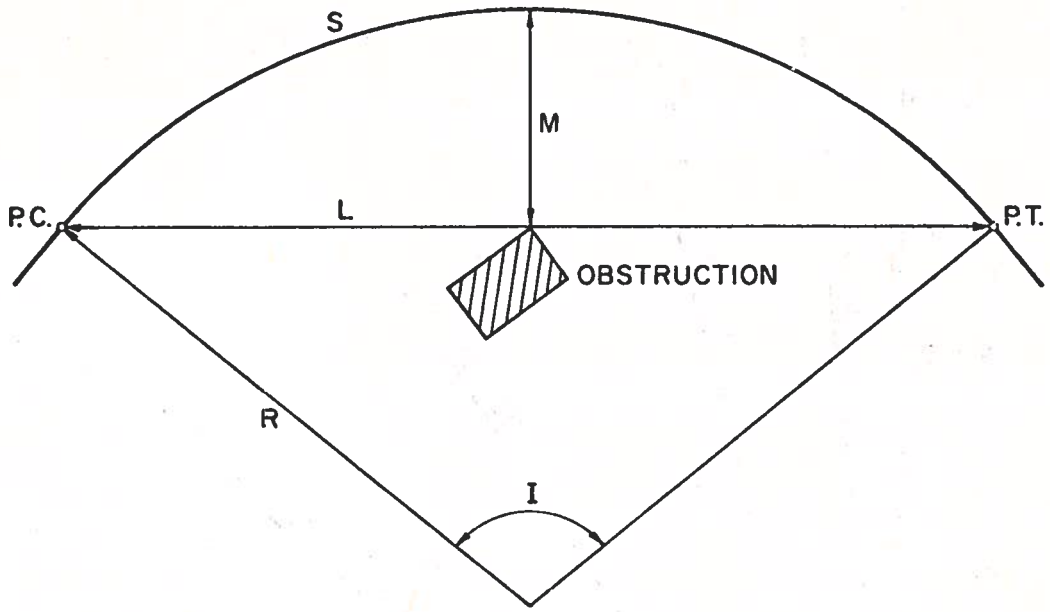
On 2 way bike paths, or bikeways shared with pedestrians, curve widening should be considered on short radius curves of less than 100 feet (30.50 m) in order to minimize the potential for possible conflicts by opposing bicyclists or between pedestrians and bicyclists. Guidelines for curve widening are shown in Figure 16-E.



WHERE: S = STOPPING SIGHT DISTANCE
L = MINIMUM VERTICAL CURVE LENGTH
V = DESIGN SPEED

STOPPING SIGHT DISTANCE FOR CREST VERTICAL CURVES FOR BICYCLE FACILITIES

FIGURE 16-B



M = OBSTRUCTION OFFSET FROM LANE CENTERLINE

L = LINE OF SIGHT

S = SAFE STOPPING DISTANCE ALONG LANE CENTERLINE (ARC DISTANCE)

R = RADIUS OF CURVATURE

I = DEFLECTION ANGLE BETWEEN TANGENTS

P.C. = POINT OF CURVATURE

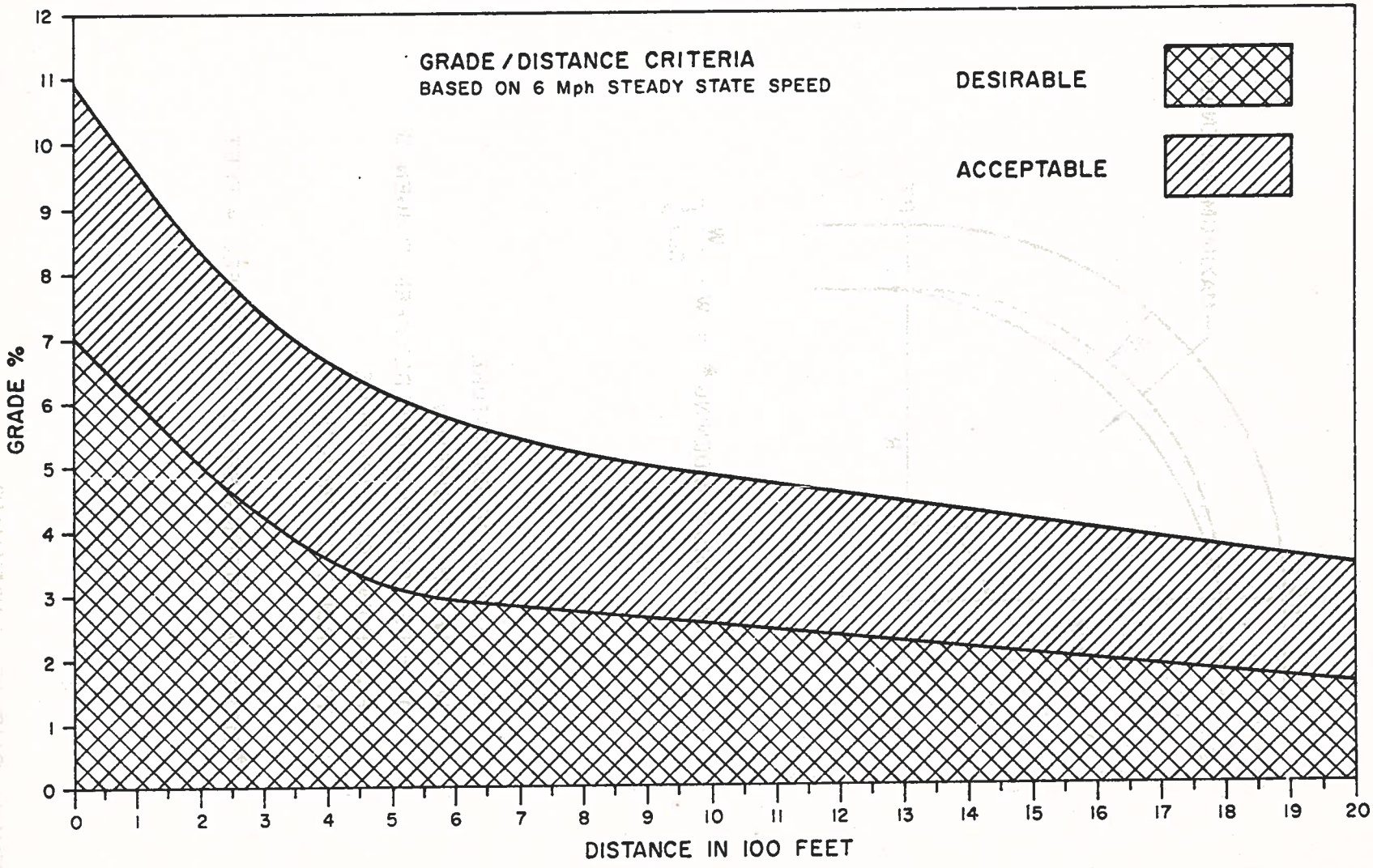
P.T. = POINT OF TANGENCY

$$M = R(1 - \cos I/2I)$$

$$L = 2R \sin I/2I$$

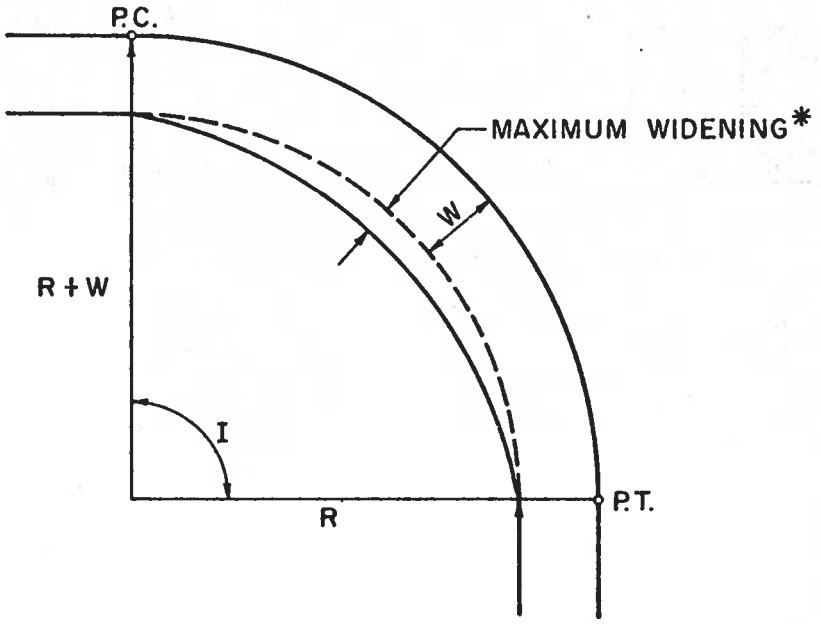
HORIZONTAL SIGHT DISTANCE

FIGURE 16-C



BIKEWAY GRADE CRITERIA

FIGURE 16-D



$$\text{MAXIMUM WIDENING}^{*} = W - \frac{W}{\cos\frac{I}{2}}$$

R = RADIUS OF CURVATURE
W = WIDTH OF BIKEWAY
I = DEFLECTION ANGLE BETWEEN TANGENTS
P.C. = POINT OF CURVATURE
P.T. = POINT OF TANGENCY

*MAXIMUM WIDENING SHALL NOT EXCEED 4 FEET

BIKEWAY CURVE WIDENING

FIGURE 16-E

16-05.8 Intersections

Intersections are, by nature, places of more intense activity and conflict than other points on a street network. Safe movement of bicycles through them is a topic of concern for bikeway designers. Satisfactory solutions to this problem have been elusive, with each location presenting individual and unique traffic patterns and problems.

16-05.9 Grade Separations

Bikeway grade separations are an effective means of eliminating bicycle/motor vehicle conflicts and often the only way of providing bikeway linkage across barriers to bicycle travel such as freeways, rivers, etc. Design of grade separations is predominantly determined by user needs, site conditions and available funds.

16-05.10 Drainage

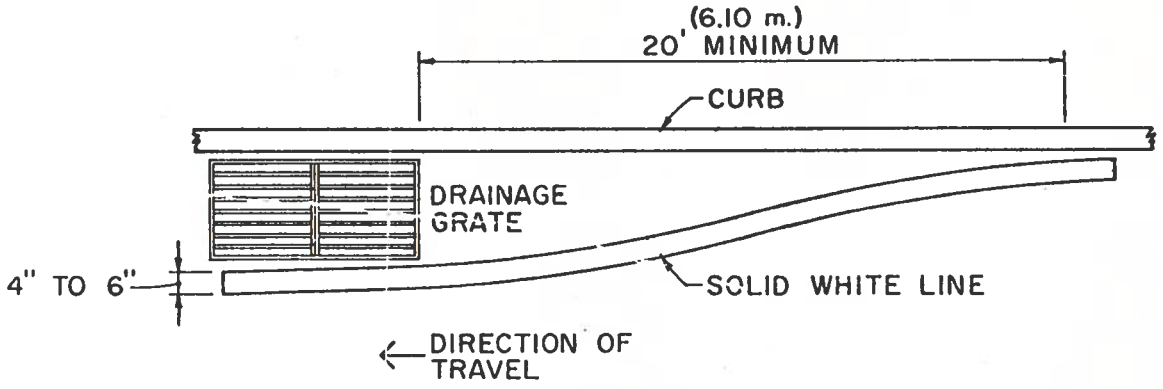
To provide an all weather Class I bikeway, adequate drainage design must be prepared, similar to highway drainage design. A minimum cross slope of 0.02 foot/foot should be provided throughout the facility.

Existing street or highway drainage design on Class II or III bikeways can, in most cases, be accepted as being hydraulically adequate. However, drainage grates along selected corridors frequently present cyclist safety problems. Grate with bars running parallel with the direction of travel present the greatest potential for wheel entrapment to the cyclist. While this grate is hydraulically efficient, it often traps the cycle wheel between the bars, resulting in serious accidents. Grates of the type within the bikeway should be altered or replaced with a grate which bicycles can safely traverse. The replacement design should accept surface water and debris almost as efficiently as paralleled bar grates.

When grate replacement is not feasible due to prohibitive cost, excessive grades or other hydraulic requirements, then the presence of these grates should be distinctly marked with warning stripes (Figure 16-F). Provisions should then be made for the bikeway to bypass the grate without encroachment into automobile traffic lanes.

16-06 SIGNING AND STRIPING

Traffic control devices are the primary elements of communication directed towards the bikeway user. Basic principles governing the



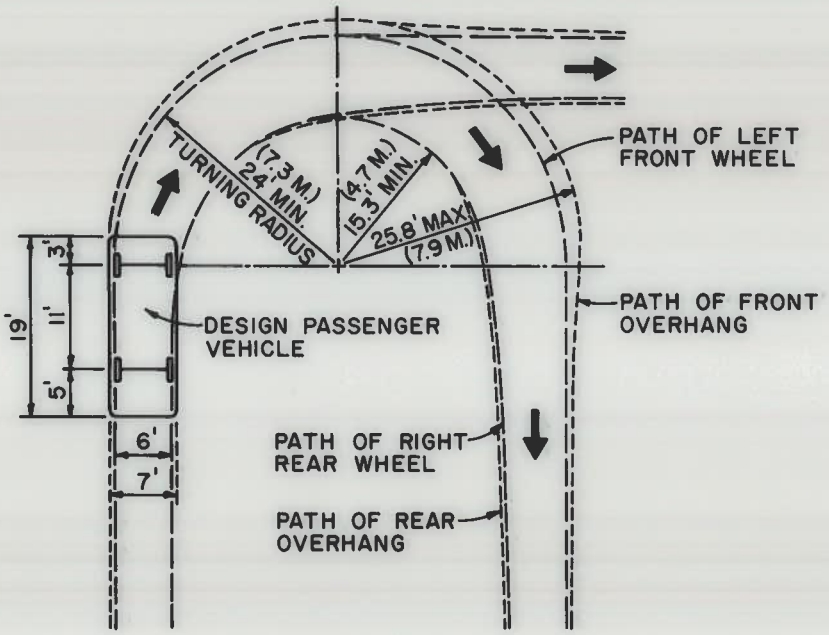
NOTE:

LENGTH OF TRANSITION VARIABLE:
BASED UPON SPEED OF BICYCLES
GRADE, SURFACE, AND WIDTH OF GRATE.

BIKEWAY HAZARD MARKING

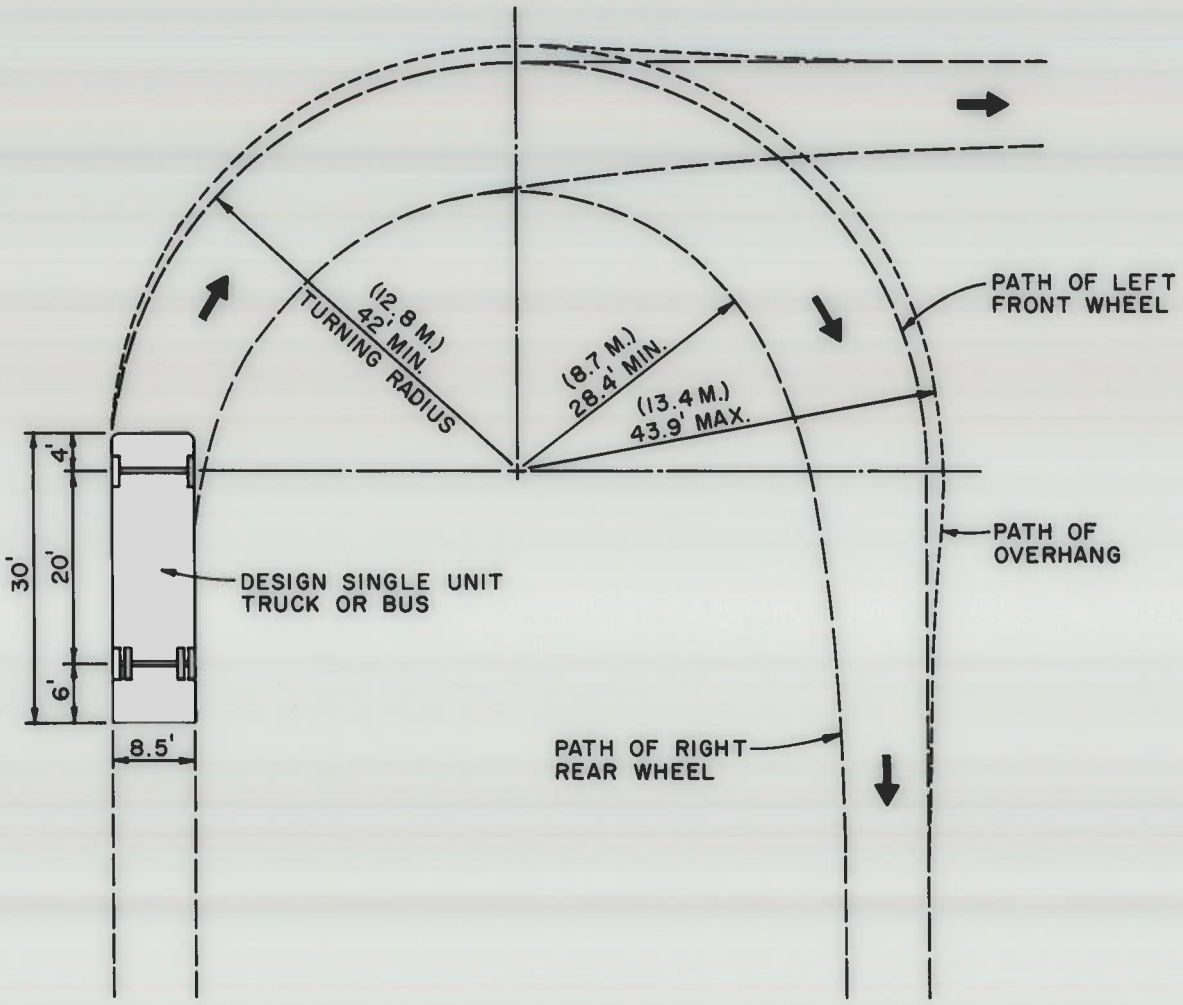
FIGURE 16-F

design and usage of traffic control devices for bikeway signing and striping are set forth in Part IX of the Manual on Uniform Traffic Control Devices, MUTCD.



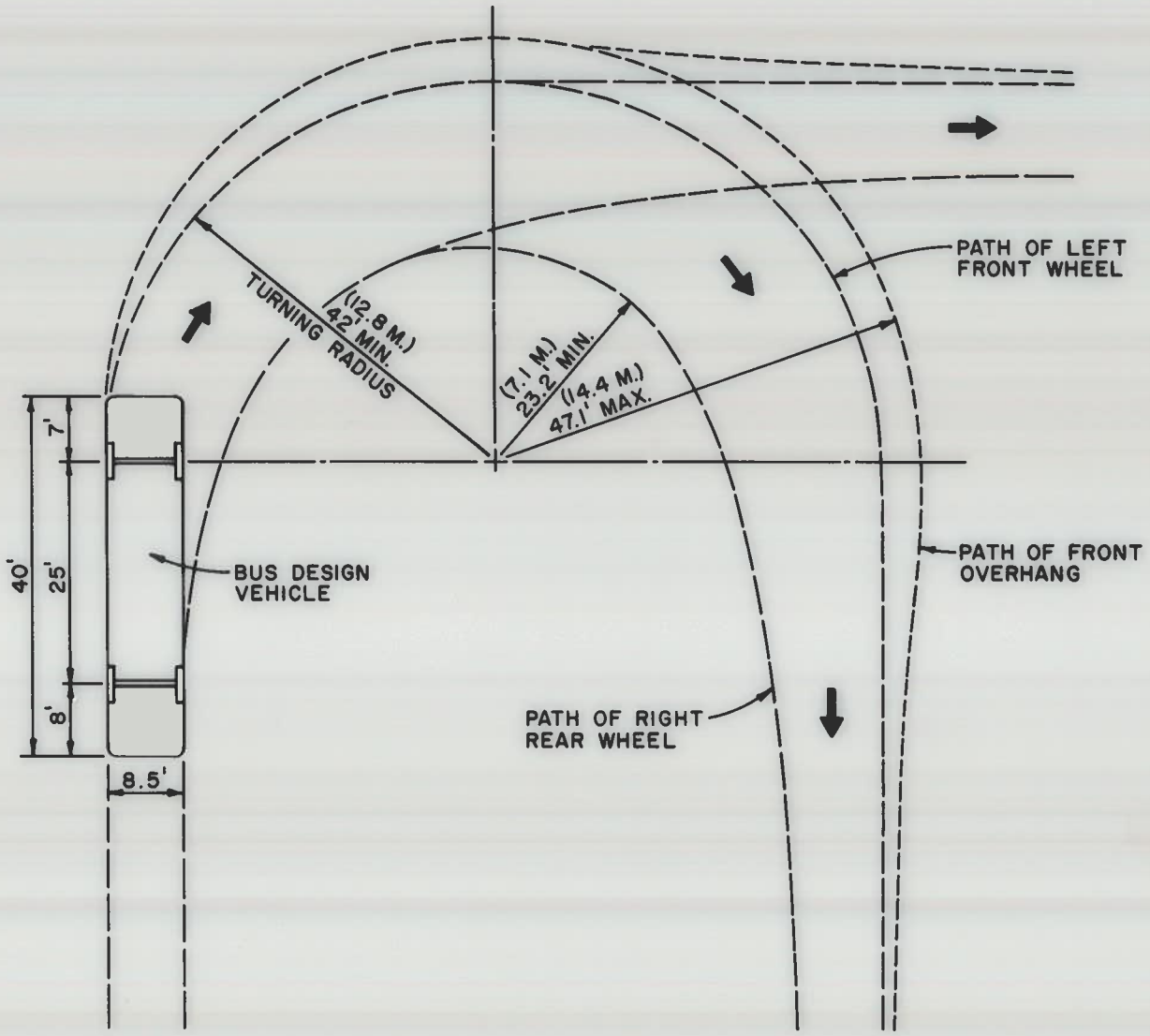
MINIMUM TURNING PATH FOR P DESIGN VEHICLE

SCALE: 1" = 20'



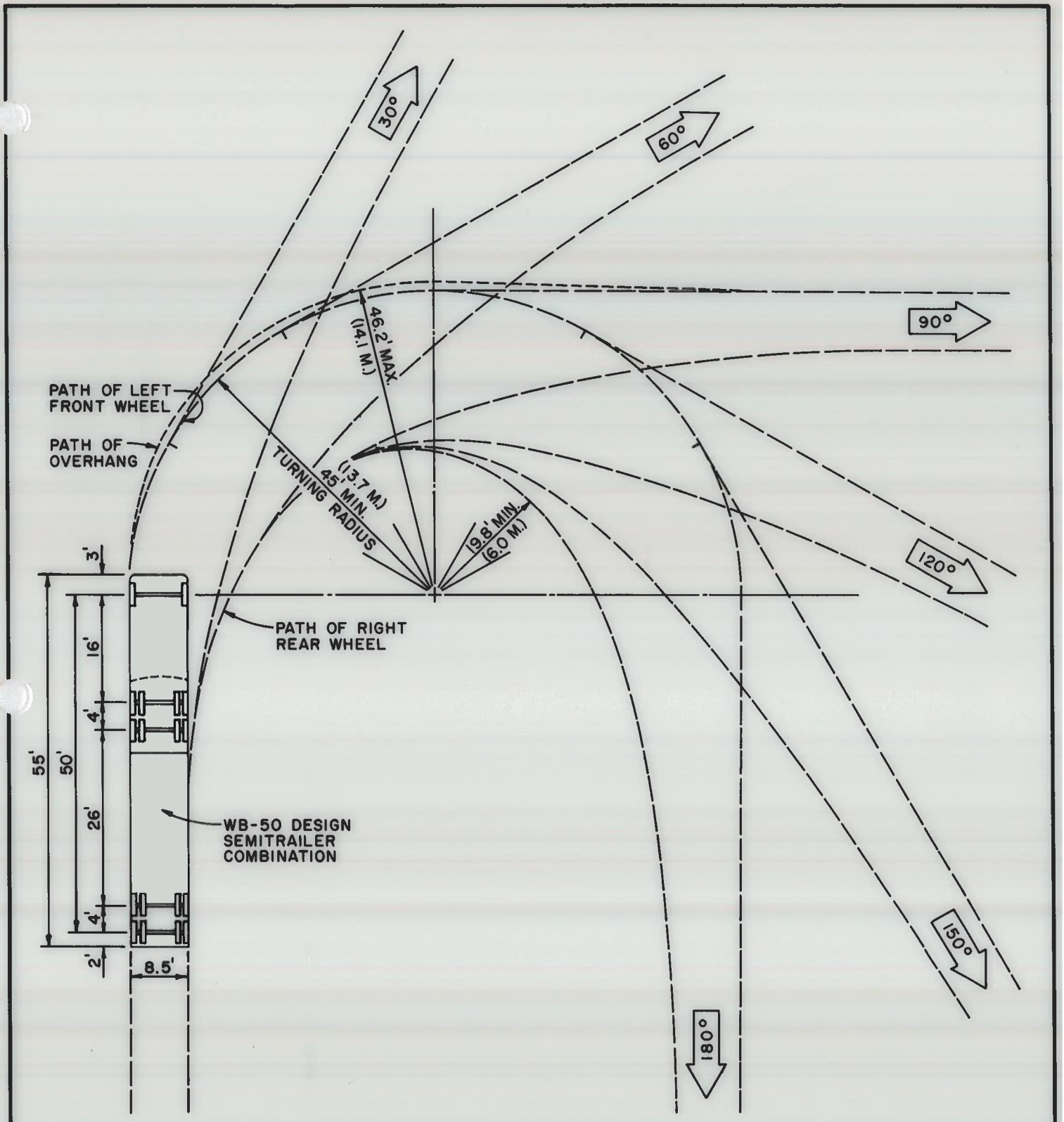
MINIMUM TURNING PATH FOR SU DESIGN VEHICLE

SCALE: 1" = 20'



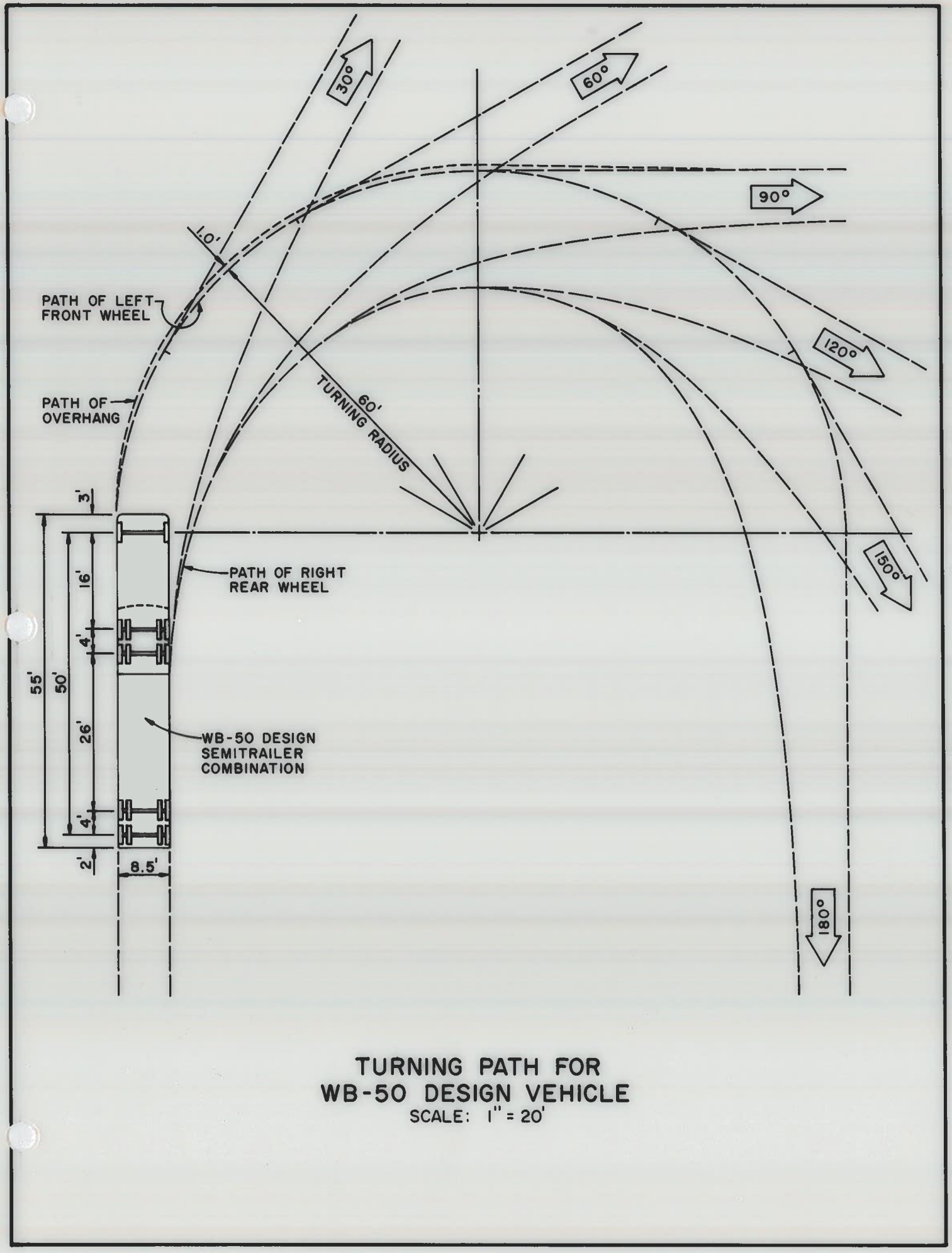
MINIMUM TURNING PATH FOR BUS DESIGN VEHICLE

SCALE: 1" = 20'



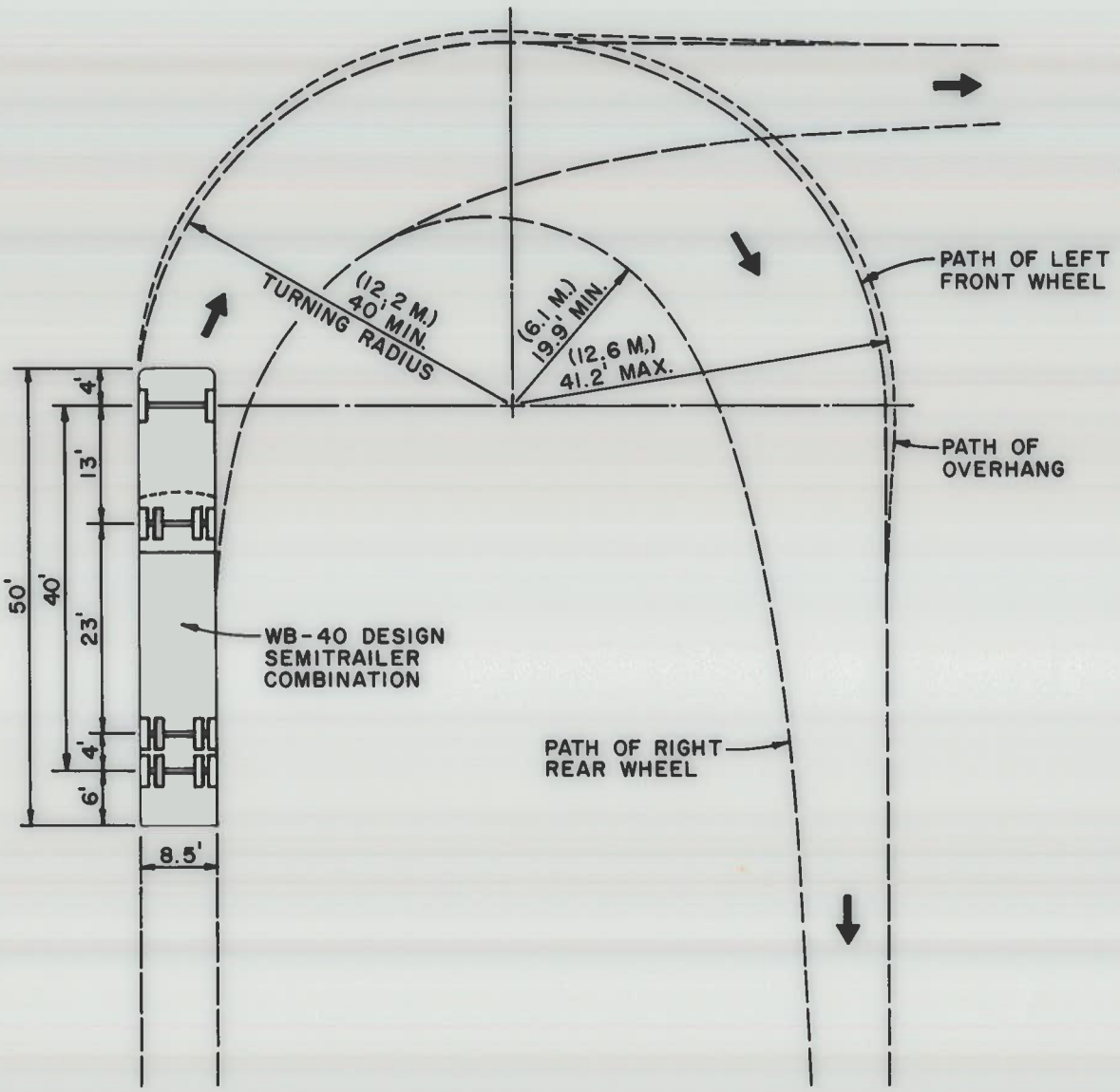
MINIMUM TURNING PATH FOR WB-50 DESIGN VEHICLE

SCALE: 1" = 20'



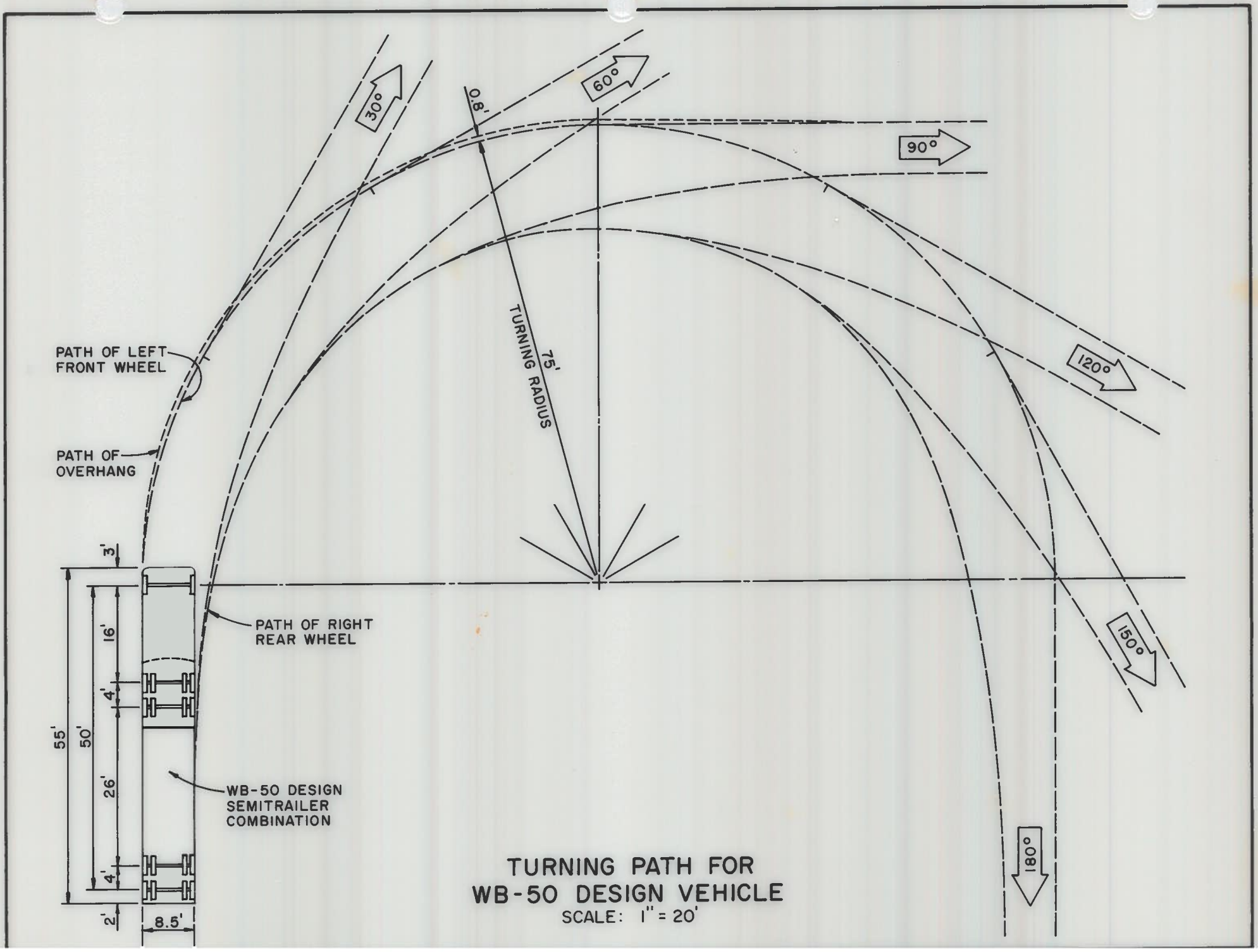
TURNING PATH FOR WB-50 DESIGN VEHICLE

SCALE: 1" = 20'



MINIMUM TURNING PATH FOR WB-40 DESIGN VEHICLE

SCALE: 1" = 20'



TURNING PATH FOR WB-50 DESIGN VEHICLE

SCALE: 1" = 20'

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